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Theme

**DESIGN AND ANALYSIS OF A ROAD BRIDGE FOR
RAILWAY LINE REINSTATEMENT INCLUDING THE
REMOVAL OF THE LEVEL CROSSING AT KERMA
ORAN PK 00+262.**

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I dedicate this work, the fruit of years of perseverance and effort, to those who have stood as the unwavering pillars of my journey...

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ملخص

يندرج هذا العمل، المنجز في إطار مشروع التخرج، في سياق تصميم ودراسة جسر يقع بين النقطتين الكيلومتريتين 00+262 و 00+493، بطول إجمالي يبلغ 231 متراً. يأتي هذا الجسر كجزء من مشروع إعادة تأهيل السكة الحديدية مع إزالة المعبر المستوي بمنطقة الكرمة بولاية وهران.

تناول هذه الأطروحة في مقدمتها دراسة نظرية حول ظاهرة الالتواء في الجسور. ثم عرضاً تقديمياً للمشروع والتصميم العام، ثم الشروع في تصميم الهيكل من خلال اقتراح ثلاثة تصاميم متميزة لتنفيذ المشروع. تم اختيار التصميم الأمثل بناءً على تحليل متعدد المعايير، حيث وقع الاختيار على جسر صندوقي ذي ارتفاع ثابت يتم إنجازه بطريقة التتابع المتتالي. تبع ذلك دراسة تفصيلية للبنية الفوقية والتحتية، شملت دراسة الإجهاد المسبق بشد لاحق، والدراسة الزلزالية، و تسليح العناصر المكونة للجسر. انجزت النمذجة باستخدام برنامج MIDAS CIVIL، وتم الحساب التقني و التحقق من المقاومة وفقاً للمعايير والقواعد سارية المفعول.

الكلمات المفتاحية: خرسانة مسبقة الإجهاد، لاحقة الشد، جسر منحنى، جسر صندوقي، بنية تحتية، طيف الاستجابة.

Résumé

Ce travail, réalisé dans le cadre d'un projet de fin d'études, vise la conception et l'étude d'un pont situé entre les points kilométriques 00+262 et 00+493, sur une longueur totale de 231 mètres. Ce pont s'inscrit dans le cadre d'un projet de rétablissement de la voie ferrée et de suppression du passage à niveau dans la région d'El KERMA, à ORAN.

L'introduction de cette thèse présente une étude bibliographique sur le phénomène de torsion dans les ponts. Après une présentation du projet, la conception structurale a débuté par la proposition de trois variantes distinctes afin de répondre aux besoins du projet. Le meilleur choix a été sélectionné sur la base d'une analyse multicritères ; le pont retenu est un pont voussoir à hauteur constante réalisé par la méthode de construction en encorbellement successifs. Une étude détaillée de la superstructure et de l'infrastructure a ensuite été menée, incluant l'étude de la précontrainte, l'étude sismique ainsi que le dimensionnement des différents équipements constitutifs du pont. La modélisation a été effectuée à l'aide du logiciel MIDAS CIVIL, et les vérifications réglementaires ont été réalisées conformément aux normes et règles en vigueur.

Mots-clés : Béton précontraint, précontrainte par post-tension, pont courbe, pont voussoir, infrastructure, spectre de réponse.

Abstract

This work, carried out as part of a graduation project, aims to design and study a bridge located between kilometer points 00+262 and 00+493, with a total length of 231 meters. This bridge is part of a railway reinstatement project and the removal of a level crossing in the El Kerma area of Oran.

The introduction of this thesis provides a bibliographical study on the phenomenon of torsion in bridges. Following a project presentation, the structural design began by proposing three distinct variants to meet the project's requirements. The optimal design was selected based on a multi-criteria analysis; the chosen structure is a constant-depth segmental bridge constructed using the balanced cantilever method. A detailed study of the superstructure and substructure was then carried out, including prestress design, seismic analysis, and the dimensioning of the various bridge components. Modeling was performed using MIDAS CIVIL software, and design checks were carried out in accordance with applicable standards and codes.

Keywords : Prestressed concrete, post-tensioning, curved bridge, segmental bridge, substructure, response spectrum.

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GENERAL INTRODUCTION

Transport is one of the fundamental pillars of sustainable development and the prosperity of any country. Efficient transport systems and modern networks are therefore a necessity for economic development, social well-being, large-scale production, and environmental preservation.

In Algeria, the transport sector, particularly urban transport, is undergoing a real transformation. A large number of projects have been completed or are in the process of being carried out in order to make this sector more efficient and effective in its contribution to the country's economic development.

In line with the national momentum, the Algerian Ministry of Transport issued a ministerial decree mandating the removal of level crossings across the country due to safety and traffic flow concerns. This decree is the direct reason for the present work.

The aim of our work is to present, below, the design and study of a curved viaduct with a length of 231 meters located in El Kerma, Oran, which was supervised by the design office "SAETI".

This present thesis, which constitutes our final year project, is divided into five main parts:

1. The first part is an introduction to research, dealing with the behavior of bridges under torsion.
2. General presentation of the project as well as all the data necessary for the design, the assessment of loads and surcharges, and the calculation hypotheses.
3. The proposal and sizing of variants, the preparation of a cost estimate, and a multi-criteria analysis.
4. The study of the superstructure as well as the sizing of all the components of the deck, followed by the resulting verifications.
5. The study of the substructure based on the soil characteristics.

Concluding with a general conclusion that closes the dissertation.

CHAPTER I

BIBLIOGRAPHIC
RESEARCH BEHAVIOR OF
BRIDGES UNDER TORSION

I.1. INTRODUCTION

This literature review addresses the behavior of structural members subjected to complex loading, commonly known as torsion. Torsion refers to the twisting of a straight bar when it is subjected to moments (or couples) that tend to produce rotation about its longitudinal axis.

The idealized case of torsional loading is illustrated by a straight bar fixed at one end and subjected to two pairs of equal and opposite forces. The first pair consists of forces P_1 applied near the middle of the bar, while the second pair consists of forces P_2 applied at the end. Each pair forms a couple that induces torsion.

According to statics, the moment of a couple is equal to the product of one of the forces and the perpendicular distance between their lines of action:

$$T_1 = P_1 \cdot d_1 \quad T_2 = P_2 \cdot d_2$$

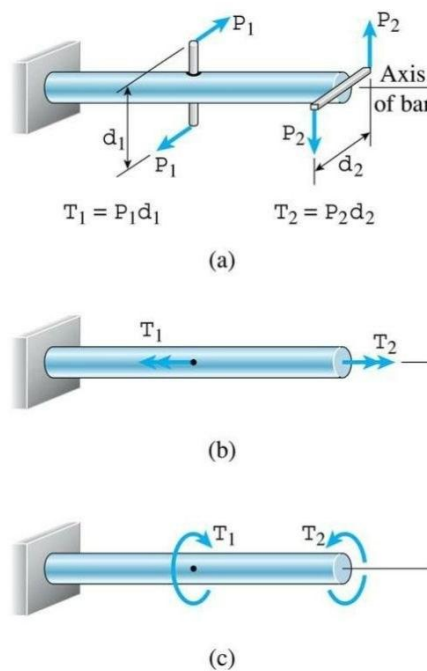


Figure I.1. Circular bar subjected to torsion by the couples T_1 and T_2 .

In this chapter, a structured literature review is presented on torsion and its effects on engineering structures, particularly bridges. Numerical applications involving bridge deck sections are also provided at the end of the chapter.

I.2. TORSION IN BRIDGE STRUCTURES

Torsion is a common phenomenon in bridge structures. However, it is often neglected because it is difficult to evaluate and, in simple bridges (such as multi-girder bridges), its effects are usually not critical. In such cases, torsion is taken into account using simplified methods such as the Guyon-Massonnet method.

For more complex bridges, torsion must be explicitly considered in design. This includes:

- Suspension bridges

- Curved bridges
- Skew bridges
- Bridges subjected to eccentric loading

I.3. CAUSES OF TORSION IN BRIDGES

Torsion in bridge structures may arise from several causes:

1. **Differential vertical movements**, especially under seismic actions (yaw motion).
2. **Wind loads**, which can induce torsional vibrations.
3. **Structural asymmetry** of the load-bearing system.
4. **Geometric curvature** of the bridge (e.g., curved alignments), where self-weight induces torsion in addition to bending and shear.
5. **Eccentric loading**, due to uneven traffic distribution or lane positioning.

I.3.1. Example: Tacoma Narrows Bridge (1940)

The first Tacoma Narrows suspension bridge was opened on July 1, 1940. On November 7, 1940, under a wind speed of approximately 65 km/h, large torsional oscillations developed. At around 10:00 AM, significant twisting motion began, and the bridge collapsed at 11:10 AM.

Although the bridge had been designed to resist wind loads, the aeroelastic flutter were not properly considered, which led to its failure.



Figure I.2. Tacoma Bridge Torsion Mode.

I.3.2. Example: Golden Gate Bridge

On December 1, 1951, a storm caused torsional vibrations in the Golden Gate Bridge, leading to minor damage. Rehabilitation works (1953–1954) included adding bracing elements under the deck, increasing its torsional stiffness.

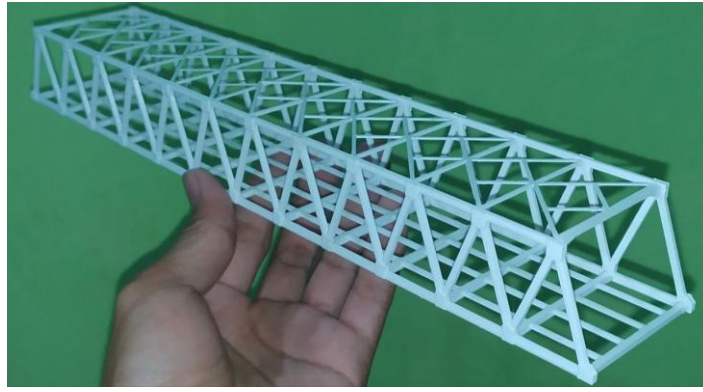


Figure I.3. Model of the closed rectangular deck after renovation.

The original shape of the deck consists of a 152-meter section of the original Golden Gate Bridge deck structure, without the roadway it supports. The lateral lattice girders, connected by the frame that supports the roadway, form an inverted open 'U' profile (Fig. I.4, left). The renovated deck shape (the bridge as it exists today) is shown in (Figure I.4, right). To the original shape (below), bracing elements have been added, so that the cross-section now takes the form of a closed rectangle.

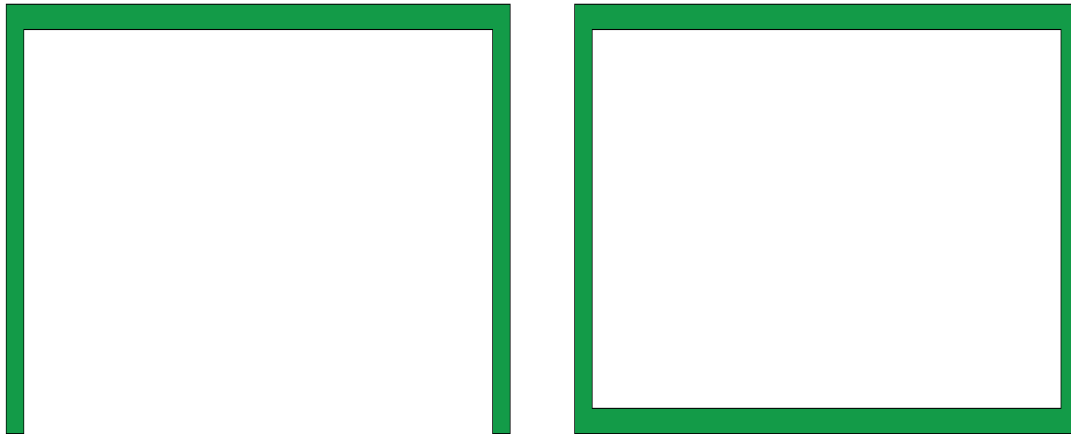


Figure I.4. (Left) Original deck shape, (Right) Renovated deck shape.

This solution was later adopted in other bridges, including the redesigned Tacoma bridge.



Figure I.5. The Tacoma Bridge after renovation.

Over time, engineers have developed new deck configurations. The following figures illustrate examples of deck shapes capable of resisting torsional effects (Figures I.6,7).

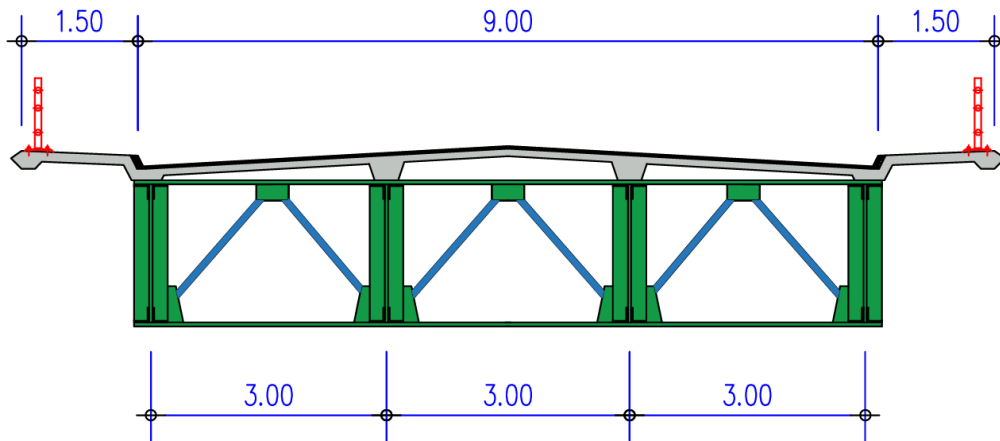


Figure I.6. Cross-section of the Maréchal-Joffre Bridge.

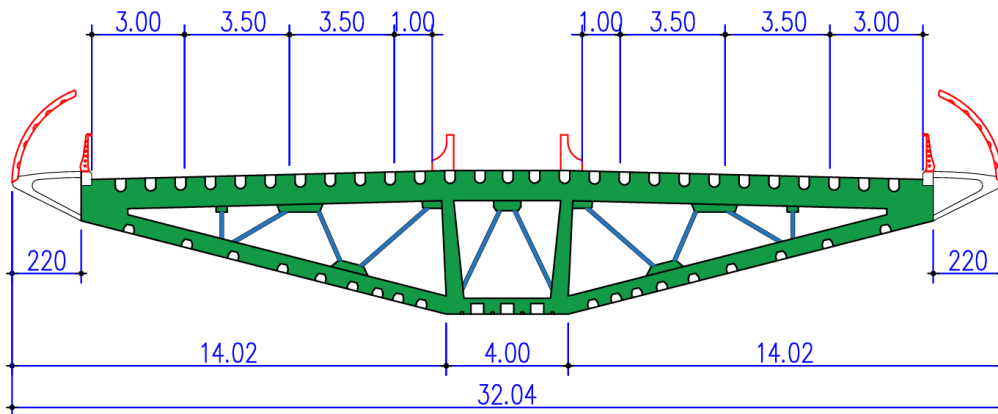


Figure I.7. Cross-section of the Millau Viaduct.

I.4. PURE, FREE, AND UNIFORM TORSION

Pure, free, and uniform torsion refers to a loading condition where the torsional moment remains constant along the member, and no restraints prevent warping of the cross-sections.

In contrast:

- **Non-uniform torsion:** variable torsional moment
- **Restrained torsion:** warping is prevented

This study focuses on:

- Open thin-walled sections
- Closed thin-walled sections

I.4.1. Parameters of Pure Torsion

Pure, free, and uniform torsion is defined by three fundamental parameters, summarized in Table I.1:

- **J** : torsional constant, which represents the resistance of a given cross-section to torsion. The larger the value of J , the greater the torsional stiffness of the member.
- **φ** : angle of distortion, which measures the rotation of one end section of a member of length L relative to the other end, under the effect of torsion.
- **τ_{max}** : maximum shear stress developed in the cross-section due to the applied torsional moment.

Table I.1. Stresses and strains for the three types of sections.

Section Type	Torsional Constant J	Shear Stress τ	Rotation θ
Solid circular	$J = I_p$	$\tau = \frac{T \cdot r}{I_p}$	$\theta = \frac{T \cdot L}{G \cdot I_p}$
Thin-walled open	$J = \frac{1}{3} \sum (s \cdot t^3)$	$\tau = \frac{T \cdot t_{max}}{J}$	$\theta = \frac{T \cdot L}{G \cdot J}$
Thin-walled closed	$J = \frac{4A_m^2}{\oint \frac{ds}{t}}$	$\tau = \frac{T \cdot t_{max}}{2 \cdot t_{min} \cdot A_m}$	$\theta = \frac{T \cdot L}{G \cdot J}$

Where:

- T** : torsional moment
 L : length of the member
 G : shear modulus
 A_m : area enclosed by the median line
 t : thickness
 s : length of segment

For sections combining open and closed parts (e.g. box girder bridges), torsional behavior depends on both contributions.

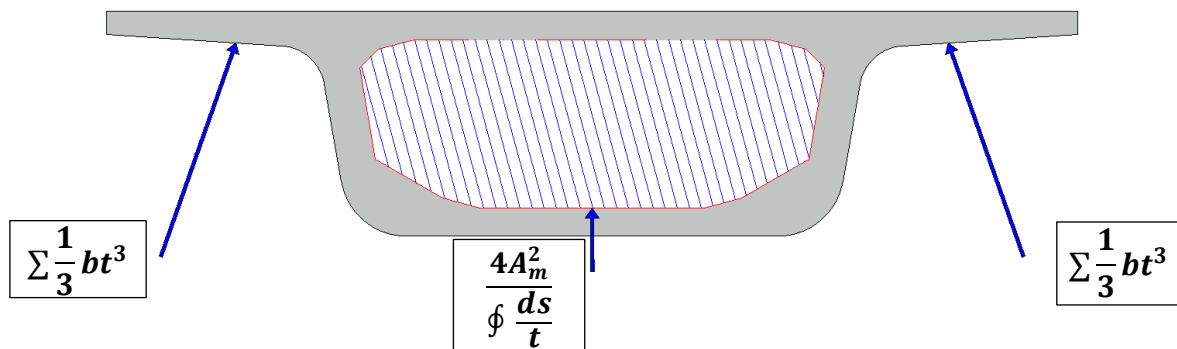


Figure I.8. Section with open and closed compartments of a hollow box girder bridge.

NUMERICAL EXAMPLES

I.4.2. Example 1: Open vs Closed Section

In this application, the torsional resistance of two bridge deck sections is studied. These sections are similar to the configurations of the Golden Gate Bridge before and after its rehabilitation.

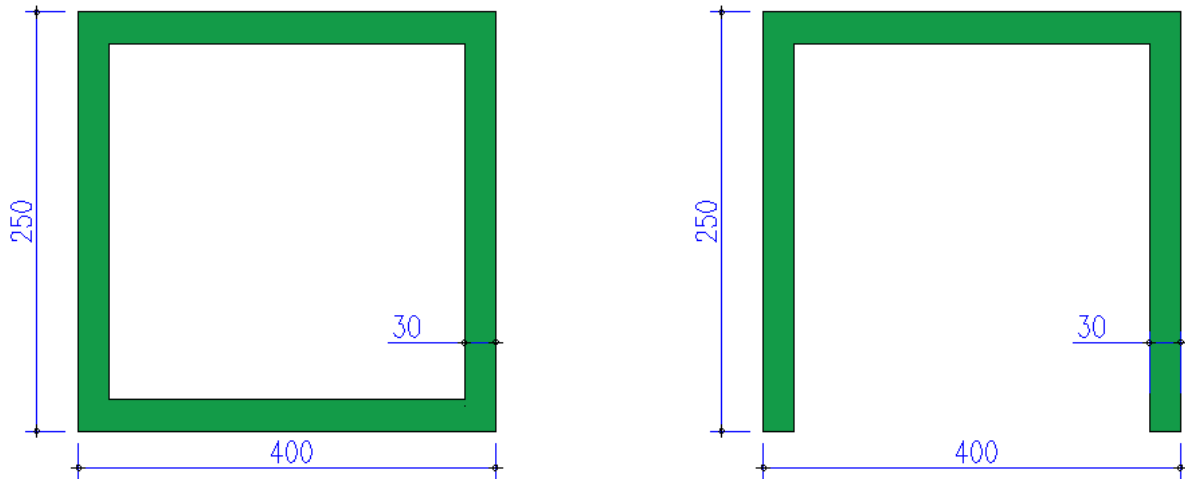


Figure I.9. Comparison of torsional behavior of two sections: open (right), closed (left).

Consider a bridge with a span $L = 4 \text{ m}$, subjected to a torsional moment $T = 3 \text{ MN} \cdot \text{m}$, with a shear modulus $G = 76 \text{ GPa}$.

The deck is assumed to have:

- an open section (Figure I.9 – left),
- a closed section (Figure I.9 – right).

❖ **Open Section**

- Torsional constant: $J_o = \sum \frac{1}{3} b t^3 = 3 \times \frac{1}{3} \times 4 \times (0.3)^3 = 0.162 \text{ m}^4$
- Maximum shear stress: $\tau_o = \frac{T \cdot t_{\max}}{J_o} = \frac{3 \times 0.3}{0.162} = 5.56 \text{ MPa}$
- Angle of twist: $\theta_o = \frac{T \cdot L}{G \cdot J_o} = \frac{3 \times 4}{76000 \times 0.162} = 9.74 \times 10^{-4} \text{ rad} \approx 0.056^\circ$

❖ **Closed Section**

- Torsional constant: $J_f = \frac{4A_m^2}{\oint \frac{ds}{t}} = \frac{4 \times 4^2}{\frac{4 \times 4}{0.3}} = 1.2 \text{ m}^4$
- Maximum shear stress: $\tau_f = \frac{T \cdot t_{\max}}{J_f} = \frac{3 \times 0.3}{1.2} = 0.75 \text{ MPa}$
- Angle of twist: $\theta_f = \frac{T \cdot L}{G \cdot J_f} = \frac{3 \times 4}{76000 \times 1.2} = 1.32 \times 10^{-4} \text{ rad} \approx 0.0075^\circ$

❖ **Comparison**

$$\frac{J_f}{J_o} = \frac{1.2}{0.162} \approx 7.41$$

The closed section has a torsional constant much greater than that of the open section, which indicates a significantly higher resistance to torsion.

In fact: $\tau_f \approx \frac{\tau_o}{7.41} \quad \theta_f \approx \frac{\theta_o}{7.44}$

❖ **Equivalent Height for Open Section**

To achieve the same torsional rigidity as the closed section, the open section must satisfy:

$$J'_o = \frac{1}{3}bt^3 + 2 \times \frac{1}{3}Ht^3$$

$$\frac{1}{3} \times 4 \times (0.3)^3 + 2 \times \frac{1}{3} \times H \times (0.3)^3 = 1.2$$

$$H = 64.67 \text{ m}$$

Thus, the open section would need to increase its height from 4 m to 64.67 m, which represents approximately 16 times its original height, in order to achieve the same torsional rigidity as the closed section.

❖ Conclusion of the Example

The closed section exhibits a much better behavior under torsion compared to the open section.

This example clearly demonstrates, from a numerical standpoint, that the rehabilitation solution applied to the Golden Gate Bridge transforming the deck from an open section to a closed section is highly effective and appropriate in improving torsional performance.

I.4.3. Example 2 – Torsional Resistance of Different Bridge Types

In Algeria, when dealing with curved bridges, designers generally adopt one of the following three structural solutions:

- Box girder bridge
- Composite box girder bridge
- Composite twin-girder bridge

As previously mentioned, torsion must always be taken into account in curved bridges. Therefore, the following application aims to evaluate the torsional resistance of these three bridge types (representing sections of a curved bridge deck), and to perform a comparative study in order to determine the most efficient configuration.

The geometric characteristics of the three variants are illustrated in Figures I.10,11,12.

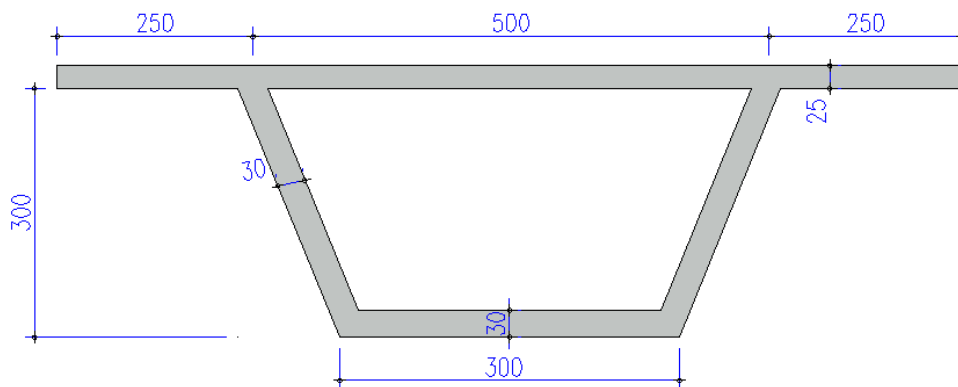


Figure I.10. Cross-section of the box girder bridge.

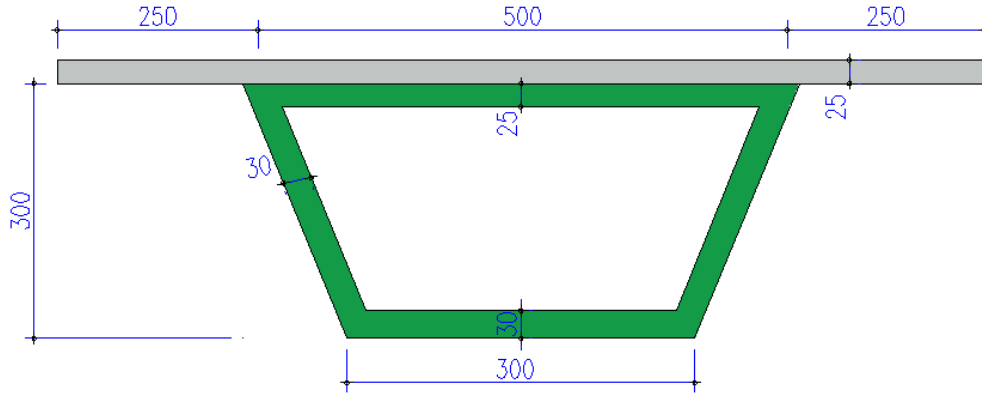


Figure I.11. Cross-section of the composite box girder bridge.

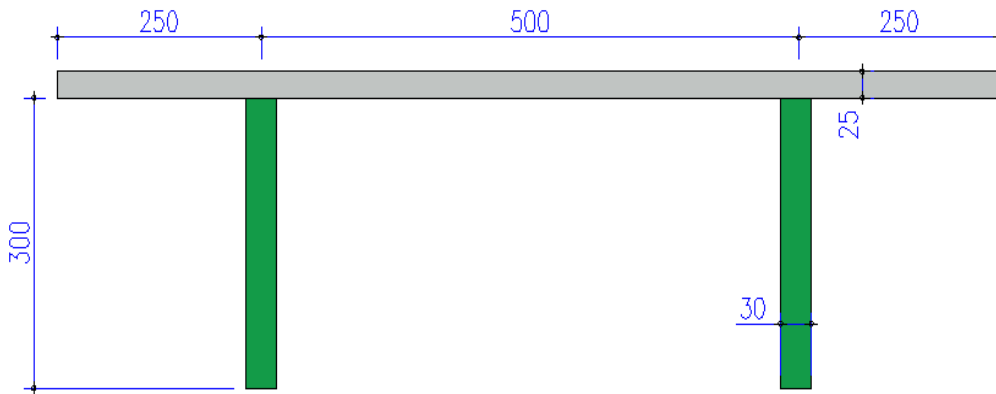


Figure I.12. Cross-section of the composite twin-girder bridge.

The following assumptions are adopted:

$$T = 8 \text{ MN.m} \quad G_{\text{steel}} = 76 \text{ GPa} \quad G_{\text{concrete}} = 12 \text{ GPa}$$

Table I.2. Calculation results for the three types of sections.

Parameter	Box Girder Bridge	Composite Box Girder	Twin-Girder Bridge
$J_1 (\text{m}^4)$	$\frac{4A_m^2}{\oint \frac{ds}{t}} = 11.52$	$\frac{4A_m^2}{\oint \frac{ds}{t}} = 11.835$	$\frac{2}{3}bt^3 = 0.054$
$J_2 (\text{m}^4)$	$\frac{1}{3}bt^3 = 0.026$	$\frac{1}{3}bt^3 = 8.2 \times 10^{-3}$	$\frac{1}{3}bt^3 = 8.2 \times 10^{-3}$
$J (\text{m}^4)$	11.546	11.992	0.0622
$\tau_1 (\text{MPa})$	$\frac{T}{2 \cdot t_{\min} \cdot A_{\min}} = 4.05$	$\frac{T}{2 \cdot t_{\min} \cdot A_{\min}} = 4.13$	$\frac{T \cdot t_{\max}}{J} = 38.59$
$\tau_2 (\text{MPa})$	$\frac{T \cdot t_{\max}}{J} = 0.21$	$\frac{T \cdot t_{\max}}{J} = 0.20$	
$\theta (^{\circ})$	$\frac{T \cdot L}{G \cdot J} = 0.013$	$\frac{T \cdot L}{G \cdot J} = 0.002$	$\frac{T \cdot L}{G \cdot J} = 0.388$

❖ Modification for Composite Section

For the composite bridge, the slab thickness is transformed into an equivalent steel thickness:

$$\text{using TGC vol 12 } e' = \frac{e}{\sqrt[3]{\eta}} \text{ with } \eta = \frac{G_{steel}}{G_{concrete}} = \frac{76}{12} = 6.33$$

$$e' = \frac{0.25}{\sqrt[3]{6.33}} = 0.14 \text{ m}$$

❖ Interpretation of Results

The comparison leads to the following relationships:

$$J_{\text{composite box}} > J_{\text{box}} \gg J_{\text{twin girder}}$$

$$\tau_{\text{composite box}} < \tau_{\text{box}} \ll \tau_{\text{twin girder}}$$

$$\theta_{\text{composite box}} < \theta_{\text{box}} \ll \theta_{\text{twin girder}}$$

❖ Conclusions

The box girder bridge (closed section) exhibits significantly better torsional behavior than the twin-girder bridge (open section).

The composite box girder performs even better than the conventional box girder. This highlights that the type of construction material plays a crucial role in torsional resistance. The use of steel enhances torsional stiffness.

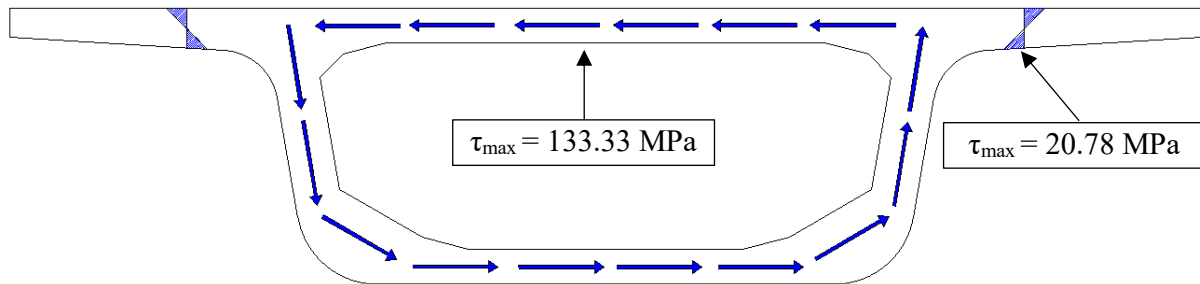


Figure I.13. Representative diagram of shear stress flow for the box girder section.

Figure I.13 indicates that for a segmental bridge, the box girder (closed compartment) carries most of the torsional load, whereas the flanges contribute very little—approximately seven times less.

I.5. CONCLUSION

From this chapter, it can be concluded that the cross-sectional shape of a bridge deck plays a decisive role in its torsional stability. Closed sections (e.g. box girders) perform significantly better under torsion compared to open sections (e.g. T-beams, I-beams, U-beams). Additionally, the construction material has a significant effect on torsional resistance.

Based on the bibliographic research, the behavior of bridges under torsion is a very extensive topic, highlighting the importance of the relationship between torsion and deck geometry.

This modest study serves as an introduction to research in structural engineering.

CHAPTER II

PROJECT PRESENTATION

II.1. INTRODUCTION

To successfully achieve its spanning and service objectives, the design of a bridge must satisfy a number of functional and natural requirements derived from road alignment data and environmental constraints, while reconciling them with the expectations of the project owner. A bridge study can only be undertaken once sufficient information has been collected regarding both the structure and its location. All this data is necessary and indispensable to carry out the study under proper conditions, and must therefore be defined beforehand, which is precisely the subject of this present chapter.

II.2. PRESENTATION OF THE STRUCTURE

II.2.1. Project Objective

The present final year project aims to carry out detailed technical studies for the construction of a road bridge located on National Road 2A (RN2A), between PK00+262 and PK00+493, in the Oran El Kerma region. This bridge is part of the ministerial decision for the suppression of level crossings, which requires the replacement of existing at-grade intersections with dedicated crossing structures in order to improve traffic safety and flow.

The following figure shows an aerial view of the project site.



Figure II.1. Structure location on Google Earth.

II.2.2. Site Location

The structure is located on National Road 2A, between PK0+262 and PK0+493, in the Oran region, running from El Kerma to Senia.

II.2.3. Obstacle to Cross

The structure passes over the following obstacles:

- a railway line at PK00+393
- a two-lane road at PK00+330
- and a two-lane road at PK00+450.

II.3. PROJECT DATA

II.3.1. Functional Data

II.3.1.1. Plan Alignment Characteristics

The geometric characteristics of the horizontal alignment of the main axis of the structure are defined as follows:

- Start of structure at PK 00+262: Straight alignment, $L = 6.12$ m;
- PK 00+268.12 to PK 00+330.13: Circular curve, $R = 120.00$ m, $L = 62.01$ m.
- PK 00+330.13 to end of structure: Straight alignment, $L = 162.87$ m;

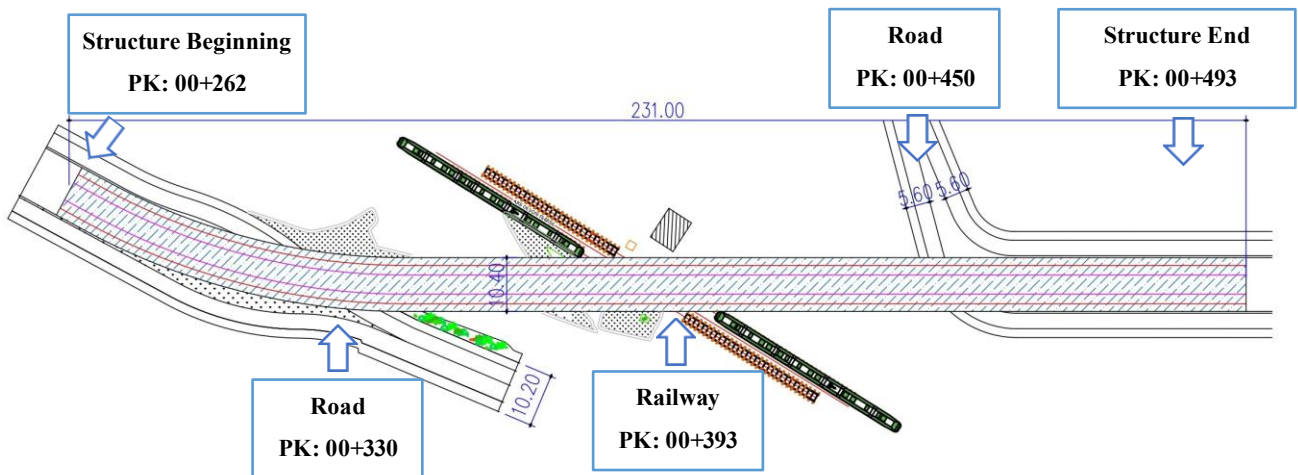


Figure II.2. Plan view of the elevated track.

II.3.1.2. Longitudinal Profile

Longitudinally, the alignment of the structure presents the following characteristics:

- Start of structure at PK 00+262: Circular curve, $R = 2\,000$ m, $L = 216.08$ m
- PK 00 + 478.08 to end of structure : straight alignment $P = -5.60\%$

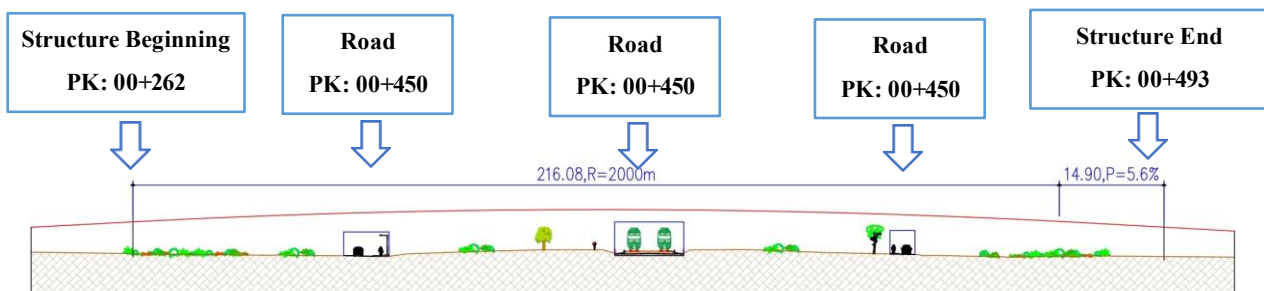


Figure II.3. Longitudinal profile of the elevated track.

II.3.1.3. Cross Section

The cross section of the project is defined as follows:

- Two-lane carriageway: 7.40 m;
- Two sidewalks: 1.50 m each;

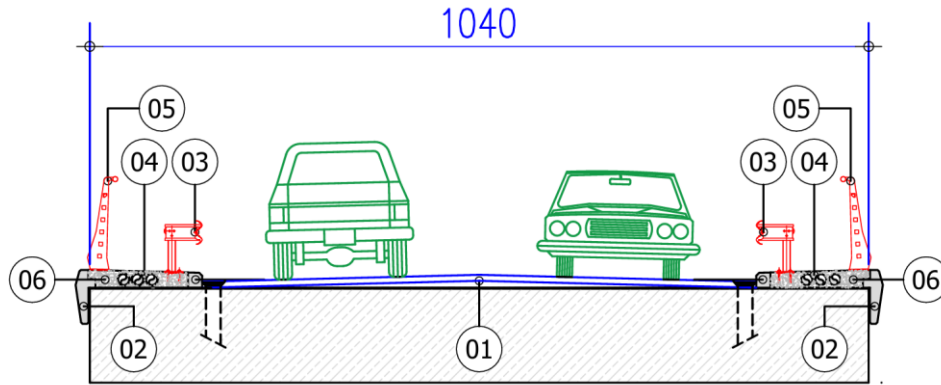


Figure II.4. Cross-section of the elevated track.

- 1 : Asphalt concrete wearing course
- 2 : Precast cornices
- 3 : Safety barrier
- 4 : Electrical conduit bank
- 5 : Guardrail
- 6 : Concrete sidewalk C20/25

II.3.1.4. Clearance

The structure crosses the following obstacles, requiring the minimum clearances below:

- Railway line: minimum clearance = 6.50 m;
- Roadway: minimum clearance = 5.25 m;

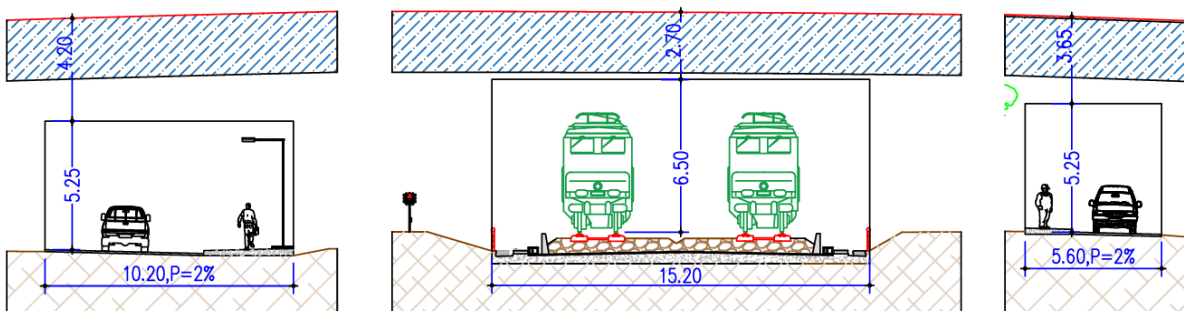


Figure II.5. Representative diagram of the clearance envelope.

II.3.2. Natural Data

II.3.2.1. Topography

According to the analysis of the topographic survey, the site is generally flat.

II.3.2.2. Geotechnical Conditions

The geotechnical investigation aims to determine the nature, arrangement, and thickness of the different soil layers, as well as their physical, mechanical, and chemical properties, in order to adapt the project to the ground conditions, including:

- Foundation system: type and embedment depth;
- Bearing capacity;
- Assessment of potential risks: stability, soil aggressiveness, swelling.

II.3.2.3. Seismicity of the Site

Seismicity is determined according to the Algerian seismic design regulations applicable to engineering structures (RPOA 2008).

- ❖ **Seismic zone classification (Article 3.1.1, RPOA 2008):** According to the seismic zoning map (Figure 3.1 of RPOA), the region of ORAN belongs to Zone IIa;
- ❖ **Bridge classification (Article 2.2, RPOA 2008):** Bridges crossing or running along major substructure links are classified in Group 2 (important bridges);
- ❖ **Zone acceleration coefficient A (Article 3.1.2, RPOA 2008):** Defined according to the seismic zone and bridge importance; according to Table 3.1 of RPOA, $A = 0.20$.

II.3.2.4. Climatology

❖ Temperature:

Warm temperate climate in ORAN; seasonal temperature variation between -15°C and $+35^{\circ}\text{C}$;

❖ Wind:

Wind effects on the structure defined by RCPR as a uniformly distributed load of 2.00 kN/m^2 .

II.4. CONCLUSION

The study of the project site is a crucial step; proper consideration of the environment and the obstacles to be crossed (natural and functional data) is required before starting the design and analysis of the structure itself.

CHAPTER III

DESIGN AND DIMENSIONING

III.1. INTRODUCTION

This chapter addresses the design and preliminary sizing of the bridge, aiming to determine the most suitable structural type that satisfies site constraints, architectural and landscape requirements, while remaining within the economic limits.

To achieve these objectives, a multi-criteria analysis is conducted for each proposed alternative to identify the solution with the optimal characteristics based on:

- The nature of the obstacle to be crossed
- The location of supports
- Construction techniques and available means according to site accessibility
- The aesthetic quality of the structure

III.2. DESIGN APPROACH

To narrow down our choice, we will first provide a brief overview of the typical span ranges of bridges according to construction methods and materials used.

III.2.1. Span Range

Table III.1. Span Range.

Bridge Type		Span Range
Ribbed Slab Bridge	Constant-Height	25 to 30 m
	Variable-Height	35 to 45 m
Precast Beams	Prestressing (PRAD)	10 to 30 m
	Post-Tensioning (VIPP)	20 to 50 m
Composite Bridge	Twin-Girder	40 to 90 m
	Four-Girder	40 to 110 m
	Steel Box Girder	50 to 150 m
Prestressed Bridge Constructed by Balanced Cantilever – Constant Height		40 to 80 m
Prestressed Bridge Constructed by Balanced Cantilever – Variable Height		65 to 200 m
Cable-Stayed Bridge		Over 100 m
Suspension Bridge		Over 100 m

III.2.2. Bridge Length

The length of the bridge depends on site-specific constraints, as well as the design of the approach embankments and the end supports. According to SETRA guidelines, the normal embankment height ranges from 6 to 8 m.

III.2.3. Selection of Design Alternatives

For this project, the alignment follows the route planned for the elimination of the level crossing at ORAN EL KERMA. The deck design is influenced by the possible placement of supports, the spans to be crossed, clearance requirements, and the construction method. The obstacle to be crossed is significant, so traffic flow must not be interrupted during construction.

These site-specific considerations mainly affect the location of supports and the construction procedure to be adopted.

Based on these criteria, the considered alternatives are:

- Prestressed concrete bridge constructed using the balanced cantilever method–
Constant Height
- Composite bridge with an open steel box girder
- Arch bridge with a lower deck

III.3. DESIGN AND SIZING OF VARIANT 1: PRESTRESSED CONCRETE BRIDGE CONSTRUCTED BY BALANCED CANTILEVER–CONSTANT HEIGHT

III.3.1. Preliminary Sizing of the Superstructure

The balanced cantilever construction method is well-suited for medium-span bridges, reaching up to approximately 150 m. It is particularly advantageous in urban areas where maintaining traffic flow on surrounding roads is essential.

This method involves building the bridge deck outward from the piers, either by casting the segments in place or by using precast segments, the latter being the chosen approach for this variant.

III.3.1.1. Longitudinal Design

The choice of spans is determined by the possible placement of supports, a key step in identifying feasible solutions while staying within the typical span range for segmental bridges. For this variant, a five-span bridge is adopted, consisting of three main spans of 56 m each, flanked by two end spans of 36 m, resulting in a total bridge length of 242 m.

III.3.1.2. Segment Division

For a bridge constructed with precast segments using the balanced cantilever method, three types of segments are distinguished:

Table III.2. Segment Division.

Segment Types	Description	Retained value
Pier Segments	For precast bridges, the dimensions of pier segments are often incompatible with the capacity of the transport and erection equipment.	1.6 m
Typical Segments	Typical segment lengths generally range between 2 and 2.6 m.	2.2 m
Closure Segments	They must allow operators to tension the prestressing tendons freely without being too slender, with lengths ranging from 1.5 to 3 m.	1.6 m

Using this segmentation, the bridge will consist of a total of 113 segments distributed as follows:

- 102 typical segments
- 4 pier segments
- 2 abutment segments
- 5 closure segments

III.3.1.3. Transverse Design

Since the deck width is less than 20 m, the most economical solution is a closed-box section. Its preliminary sizing is shown in the table below, based on the SETRA guidelines for prestressed concrete bridges constructed using the balanced cantilever method :

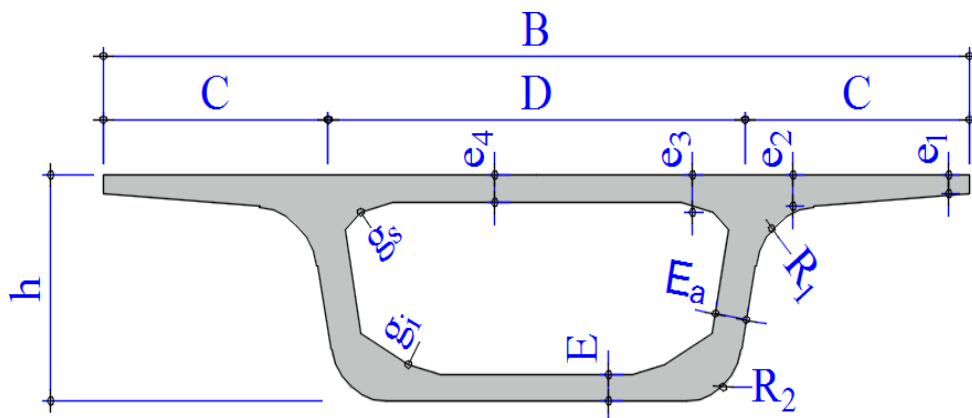


Figure III.1. Transverse cross-section of a cantilever segment.

Figure III.1 illustrates the notations used, while the selected values are listed in the table below :

Table III.3. Cross-Section Dimensions of the Segment.

Element	Formula	Retained value
Segment height h	$L/25 < h < L/20$	2.40 m
Segment width B	$B = 1$	10.40 m
C	$C = B/4$	2.60 m
D	$D = B / 2$	5.20 m
Upper slab thickness e_1	$e_1 > 16 \text{ cm to } 18 \text{ cm}$	25 cm
e_2	$C/8 < e_2 < C/7$	37 cm
e_3	$e_3 = 0.1 + D/25$	40 cm
e_4	$D/30 < e_4 < D/25$ $e_{4 \text{ min}} = 20 \text{ cm}$	30 cm
Web inclination	$10 \leq \beta \leq 30\%$	30%
Web thickness E_a	Typical: $E_a = \frac{L}{275} + 1.25 \frac{B}{L} - 0.125$ Special: $E_a = 0.26 + \frac{L}{500}$	$E_a = 40 \text{ cm}$
Lower slab thickness $E_c = E_p = E$	$E \geq 2\Phi + d + e$ $E \geq 18 \text{ to } 22 \text{ cm}$	$E = 40 \text{ cm}$
Upper gusset angle	$30^\circ < \alpha < 45^\circ$	$\alpha = 30^\circ$
Lower gusset angle	$30^\circ < \alpha < 45^\circ$	$\alpha = 30^\circ$
Radius	$R_1 = 0.8 \text{ m} ; R_2 = 0.7 \text{ m}$	-

III.3.2. Substructure Design

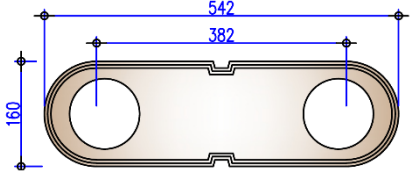
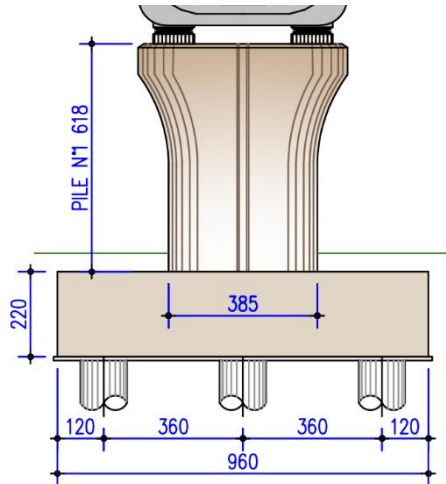
III.3.2.1. Piers

Piers play a crucial role in the structural behavior of the bridge deck. The deck may be fully fixed or simply supported on the piers, allowing all applied loads to be transmitted to the ground through the footing and piles (in the case of deep foundations).

The design of the piers depends on the construction method of the deck, the type and execution method of the foundations, and certain site-specific natural or functional constraints.

The table below summarizes our choice of piers for this variant:

Table III.4. Selection and Dimensions of the Pier for Variant 1.

Element		Value / Description	Figure
Section	Shape:	Rectangular with circular connections	
	Dimensions	$3.82 \times 1.60 \text{ m}^2$ $\Phi = 1.60 \text{ m}$	
Height	P1	6.18 m	
	P2	6.75 m	
	P3	6.27 m	
	P4	6.16 m	
Foundation	Type	deep foundation on 9 piles $\Phi = 1.20 \text{ m}$ $L = 20.00 \text{ m}$	
	Footing Dimensions	$9.60 \times 9.60 \times 2.20$ (m×m×m)	

III.3.2.2. Abutments

The abutment is a fundamental element of any bridge structure, serving a dual role. It acts as a support for the deck, being subjected to similar forces as the piers, and also functions as a retaining wall, resisting earth pressure and surcharge from the embankments.

Based on geotechnical results and site conditions, the use of filled abutments appears to be the most suitable solution for this bridge. The design and construction of this type of abutment are relatively straightforward.

A filled abutment consists of an exterior face forming a front wall fixed on a footing, topped by a parapet wall, and extended by return walls that, in this case, connect on both sides of the bridge with retaining walls over a total length of 49 m.

This type of abutment includes the following elements:

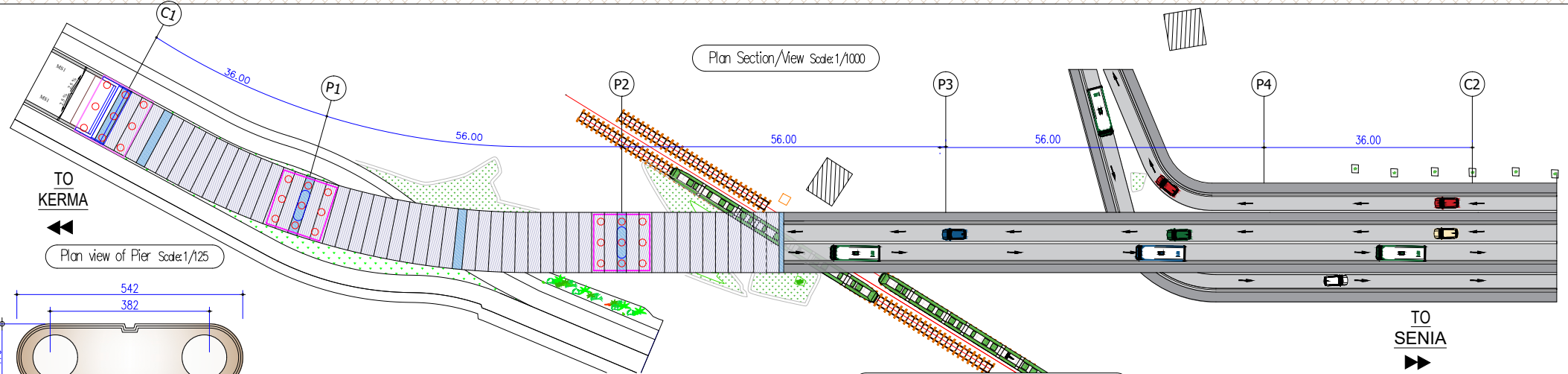
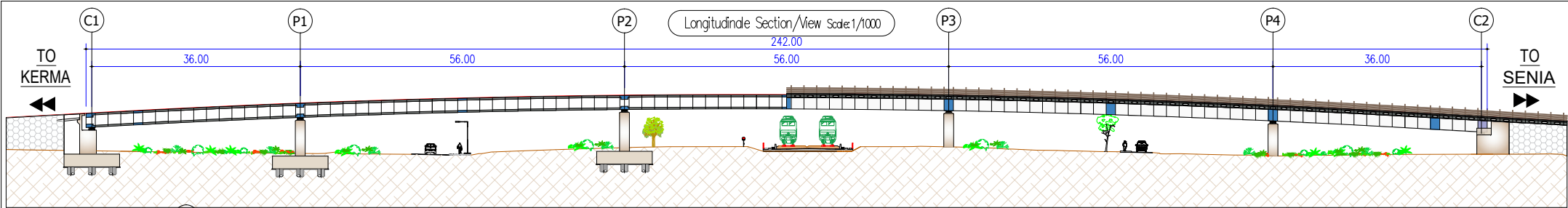
- Front wall supports the bridge deck.
- Return walls retain the embankment fill.
- Parapet wall protects the deck end by retaining earth and securing the pavement joint.

- Transition slab reduces the effect of elevation differences between the roadway and the bridge, ensuring user comfort.

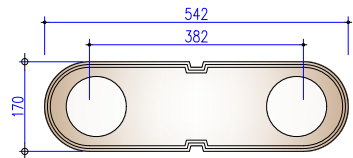
The dimensions of the abutment elements are defined below:

Table III.5. Selection and Dimensions of the Abutment for Variant 1.

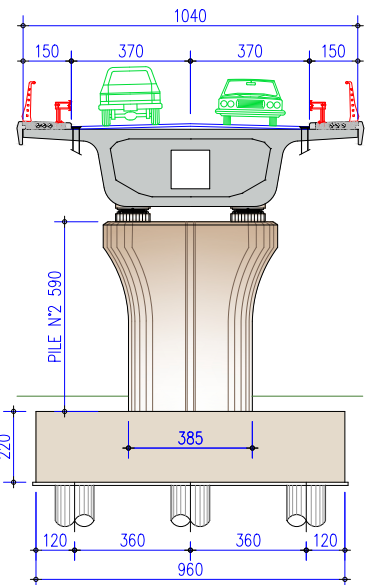
Element		Value / Description	Figure
Front wall	Thickness	2.89 m	
	Length	10.40 m	
	Height	4.15 / 3.80 m	
Parapet wall	Thickness	0.40 m	
	Length	10.40 m	
	Height	2.82 m	
Return wall	Thickness	0.50 m	
Transition slab	Height	0.30 m	
	Width	4.20 m	
Foundation	Type	deep foundation on 9 piles $\Phi = 1.20 \text{ m}$ $L = 20.00 \text{ m}$	
	Footing	$9.60 \times 10.80 \times 2.20$ (m×m×m)	



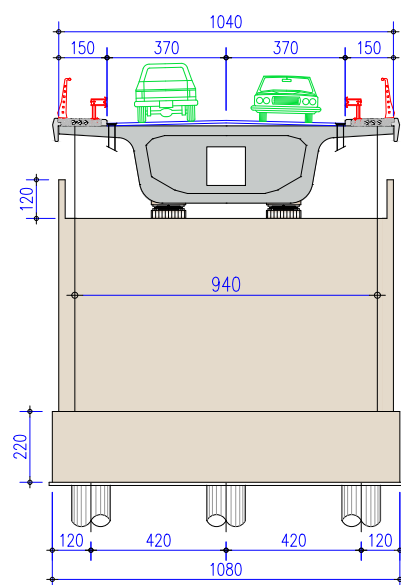
Plan view of Pier Scale:1/125



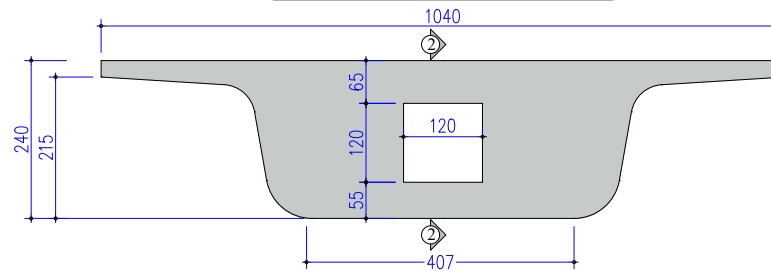
Cross section of Pier Scale:1/250



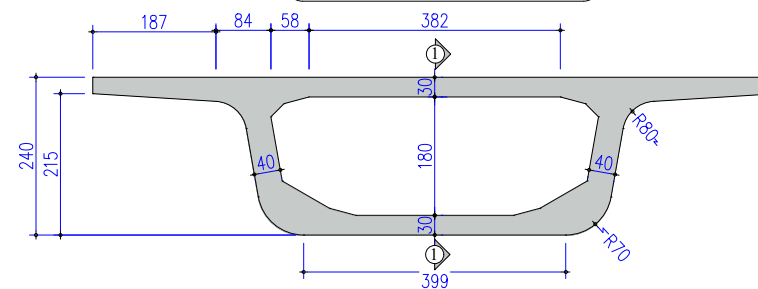
Cross section of Abutment Scale:1/250



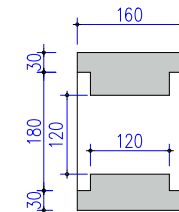
Section of Pier/Abutment segment Scale:1/125



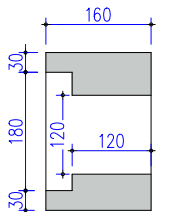
Section of regular segment Scale:1/125



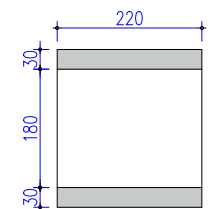
Section 2-2 Scale:1/125



Section 2-2 Scale:1/125



Section 2-2 Scale:1/125



III.3.3. Verification of Variant 1

III.3.3.1. Modelling and Results

- The deck is modelled as a beam element, with its cross-section imported from the drawing software AutoCAD 2026, using its real geometric dimensions.
- Regarding boundary conditions, the structure is: Double-supported at abutment C1, simply supported at the remaining supports
- For structural analysis, We apply a load of G+1.2A(l)

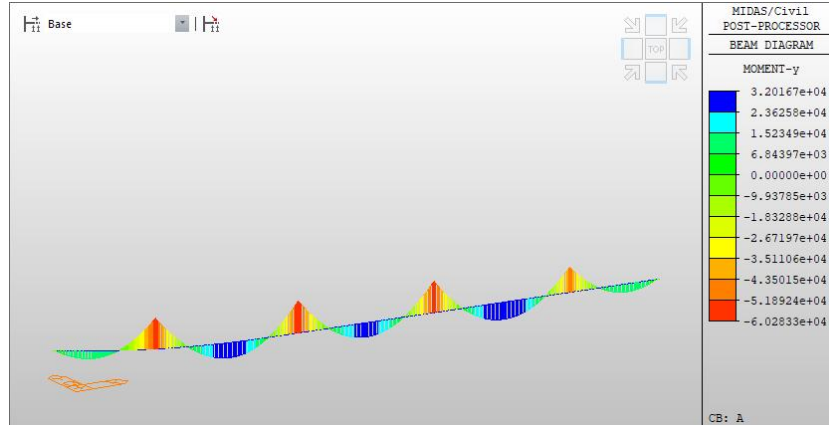


Figure III.2. Bending moment for variant 1.

III.3.3.2. Estimation of Prestressing Tendon Sections.

The prestressing sections are primarily estimated at the supports, based on the assumption that internal prestressing fully counteracts the effects of permanent and variable loads, while passive reinforcement is considered as minimum construction reinforcement.

The prestressing area is evaluated at:

- Mid-span
- Pier (support section)

The prestressing system adopted consists of 13T15 tendons.

❖ Determination of the number of tendons

The prestressing force must satisfy:

$$P \geq \frac{\frac{M_{ext} \cdot v}{I}}{\left(\frac{e \cdot v}{I} + \frac{1}{B}\right)} \text{ (for cantilever tendons)}$$

$$P \geq \frac{\frac{M_{ext} \cdot v'}{I}}{\left(\frac{e \cdot v'}{I} + \frac{1}{B}\right)} \text{ (for continuity tendons)}$$

Where :

e : eccentricity of the prestressing tendon

A_p : prestressing steel area to be determined

M_{ext} : maximum bending moment due to load combination $G + 1.2Q$

v, v' : distances from the centroid to extreme fibers

I : moment of inertia of the section

B : segment area

The initial prestressing force is given by: $P_0 = \sigma_{p0} \cdot A_0$

With :

$\sigma_{p0} = 1488$ MPa (initial stress for 13T15 tendon)

$A_0 = 1820$ mm² (area of one tendon)

Thus : $P_0 = 2.71$ MN

Prestress losses are assumed as: $\Delta P = 25\%$

Results :

At Pier:

$M_{pier} = 60.28$ MN\m

$P > 42.46$ MN $\Rightarrow n > 20.88$

Adopted: 24 tendons of 13T15

At Mid-Span:

$M_{Mid-Span} = 32.02$ MN\m

$P > 23.78$ MN $\Rightarrow n > 11.69$

Adopted: 12 tendons of 13T15

❖ Stress Verification

The stresses in concrete must satisfy the following conditions:

- Lower Fiber Stress $\sigma_{inf} = \frac{P}{B} + \frac{(P \cdot e_0 - M) \cdot v'}{I}$
- Upper Fiber Stress $\sigma_{sup} = \frac{P}{B} - \frac{(P \cdot e_0 - M) \cdot v}{I}$
- Allowable Stresses
 $\bar{\sigma}_{bc} = 0.6 \cdot f_{c28} = 0.6 \times 35 = 21$ MPa
 $\bar{\sigma}_{bt1} = -1.5 \cdot f_{t28} = -1.5 \times 2.7 = -4.05$ MPa (outside cover zone)
 $\bar{\sigma}_{bt2} = -1.0 \cdot f_{t28} = -2.70$ MPa (within cover zone)

❖ Numerical verification

Table III.6. Stress Verification.

Section	M_{max} (MN.m)	v' (m)	v (m)	S (m ²)	I (m ⁴)	d (m)	P (MN)	σ_{sup} (MPa)	σ_{inf} (MPa)	Obs
Pier	60.28	1.36	1.04	13.31	7.33	0.15	48.78	3.90	3.49	O.K
Mid-Span	32.02	1.45	0.95	6.84	5.42	0.15	24.39	6.91	1.37	O.K

III.3.3.3. Conclusion

The computed stresses in both sections remain within the allowable limits for concrete in both compression and tension. Therefore, the selected prestressing tendons (13T15) and their corresponding numbers are adequate and ensure the structural safety of the bridge deck.

III.4. DESIGN AND PRELIMINARY SIZING OF VARIANT 2: OPEN STEEL BOX GIRDER BRIDGE

The simplest open steel box girder decks consist of a reinforced concrete slab supported by a U-shaped steel structure. This structure is formed by longitudinal plates creating the outer U-shape (top flanges, webs, and bottom plate), and transverse elements of two types: diaphragms located at supports and frames placed along the spans. The typical span range for such composite structures is between 50 m and 150 m. Beyond this range, closed steel box girders or orthotropic decks become more economical. This configuration provides high torsional stiffness with low steel weight, making it ideal for curved highway bridges.



Figure III.3. Balambu Steel Box Bridge, Nepal.

III.4.1. Preliminary Sizing of the Superstructure

III.4.1.1. Longitudinal Design

For this variant, an open steel box girder bridge with continuous spans, skew alignment, and constant depth is adopted. The total bridge length is 227 m, consisting of three main spans of 55 m and two end spans of 30 m.

III.4.1.2. Transverse Design

With a relatively wide deck (10.4 m), the common solution is to connect a reinforced concrete slab longitudinally to the steel box girder using shear connectors, and transversely to cross beams spaced at 5 m.

The open steel box girder is composed of a bottom flange, inclined webs, and a top flange connected to the slab, ensuring good torsional stiffness and efficient load distribution.

III.4.2. Design of Structural Elements

III.4.2.1. Bottom Plate

The bottom plate is a steel plate of constant thickness along each segment, usually stiffened by U-shaped ribs, T-sections, or sometimes flat plates.

III.4.2.2. Webs

The webs are plates of constant thickness, generally inclined relative to the bottom plate, and stiffened by flat or T-shaped elements.

III.4.2.3. Frames

Frames are transverse elements, highly perforated and welded to the top flanges, webs, and bottom plate. They prevent excessive transverse deformation of the box girder and are typically spaced between 4 and 7.5 m; in this project, a spacing of 7.5 m is adopted.

III.4.2.4. Slab

In the case of a bridge without cross beams, SETRA guidelines recommend a variable thickness slab.

III.4.2.5. Steel Grade Selection

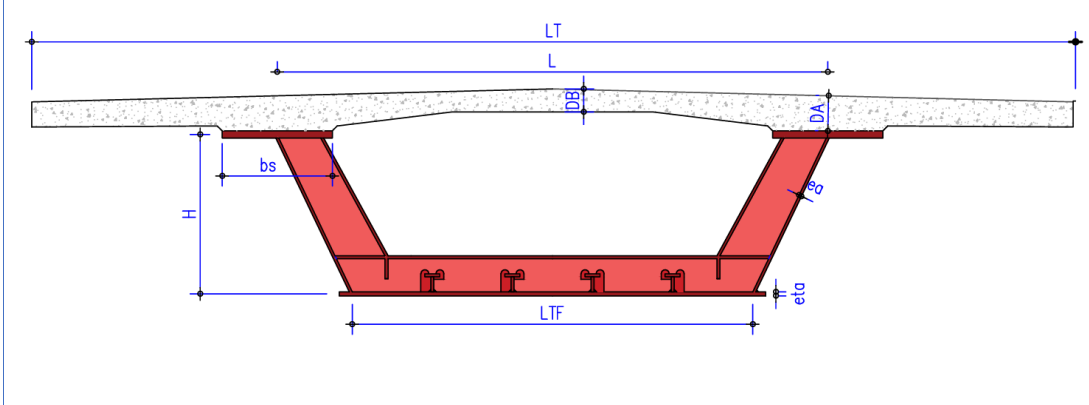
The structural steel used is of grade S355 N, which ensures adequate toughness, defined as the ability of the steel to absorb impact energy without brittle failure.



Figure III.4. Steel box girder example.

The selected dimensions are summarized in the table below.

Table III.7. Sizing of the steel box girder.



Element		Formula	Retained value
Webs	Spacing between webs	$L = 0.50 \text{ to } 0.55 L_T$	5.50 m
	Web height	$H = 1/30 \text{ to } 1/40$	1.60 m
	Web inclination	0 to 50 %	30 %
	Web thickness e_a	16 to 35 mm	20 mm
Top flange	Width	$500 \text{ mm} \leq b_s \leq 1\,200 \text{ mm}$	1 100 mm
	Thickness	$20 \leq t_s \leq 150$	70 mm
Bottom plate	Thickness e_{tf}	25 to 80 mm	40 mm
Slab	Thickness	At webs : $0.13 + \left(\frac{L_T - L}{26}\right)$	$D_A = 35 \text{ cm}$
		At mid-deck : $0.12 + \frac{L}{50}$	$D_B = 23 \text{ cm}$

III.4.2.6. Connection Between the Slab and Steel Beams

In composite steel–concrete structures, the slab is cast directly onto the steel beams. However, the natural bond between concrete and steel is generally insufficient to ensure effective composite action over time. To overcome this limitation, specific mechanical devices known as shear connectors are used.

These connectors enable interaction between the concrete slab and the steel beam, allowing both materials to act together structurally. As a result, the concrete contributes to the bending resistance of the beam, while the steel provides tensile strength. This composite behavior significantly improves the overall performance of the structure.

The main role of the connection is to prevent relative longitudinal slip and uplift of the slab with respect to the steel beam, thereby ensuring full composite action.

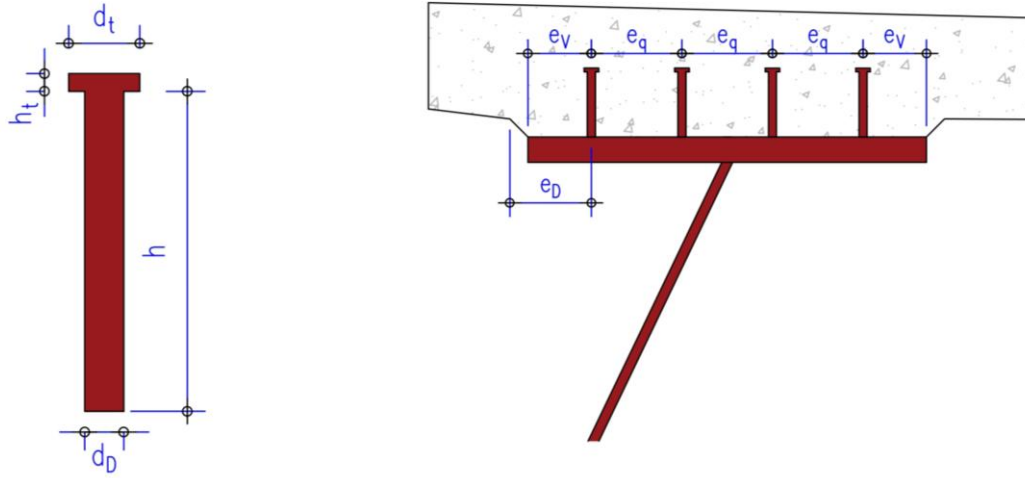


Figure III.5. Sizing and arrangement of studs.

III.4.2.7. Design and Preliminary Sizing

In this project, the slab-to-beam connection is achieved using headed stud shear connectors. A stud diameter of $d_D = 22$ mm is adopted, in accordance with Eurocode 4, which permits stud diameters between 16 mm and 22 mm, provided that the diameter does not exceed 1.5 times the flange thickness of the steel beam to which it is welded.

The remaining geometric characteristics of the studs are selected based on code requirements and are summarized in the table below.

Table III.8. Summary of the stud dimensions.

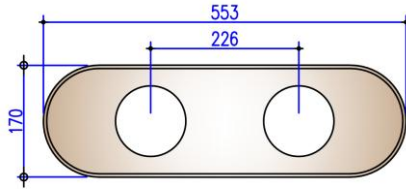
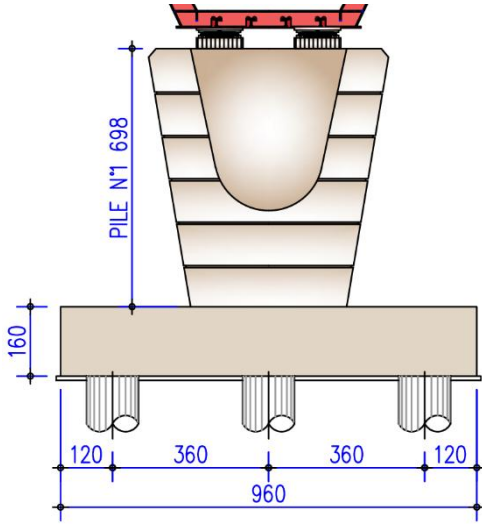
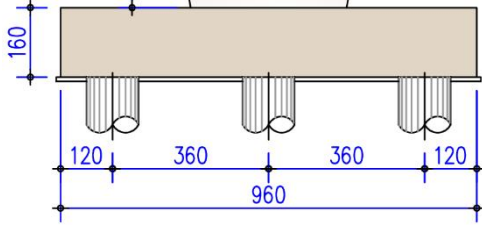
Characteristic	Symbol	Eurocode 4 Requirement	Value (mm)
Stud diameter	d_D	$16 \leq d_D \leq 22$ mm $d_D \leq 1.5 t_f$	22
Stud height	h	$h \geq 3 d_D$	150
Head diameter	d_t	$d_t \geq 1.5 d_D$	50
Head height	h_t	$h_t \geq 0.4 d_D$	10
Distance to slab edge	e_D	$e_D \geq 25$ mm	180
Concrete cover thickness	e_v	$e_v \geq 50$ mm	300
Transverse spacing	e_q	$e_q \geq 2.5 d_D$	300

III.4.3. Preliminary Sizing of the Substructure

III.4.3.1. Preliminary Sizing of Piers (SETRA Guide)

For this variant, and according to SETRA guidelines, wall-type piers are selected. This type satisfies both mechanical efficiency and aesthetic requirements. The dimensions of the piers are presented in the table below:

Table III.9. Selection and Dimensions of the Pier for Variant 2.

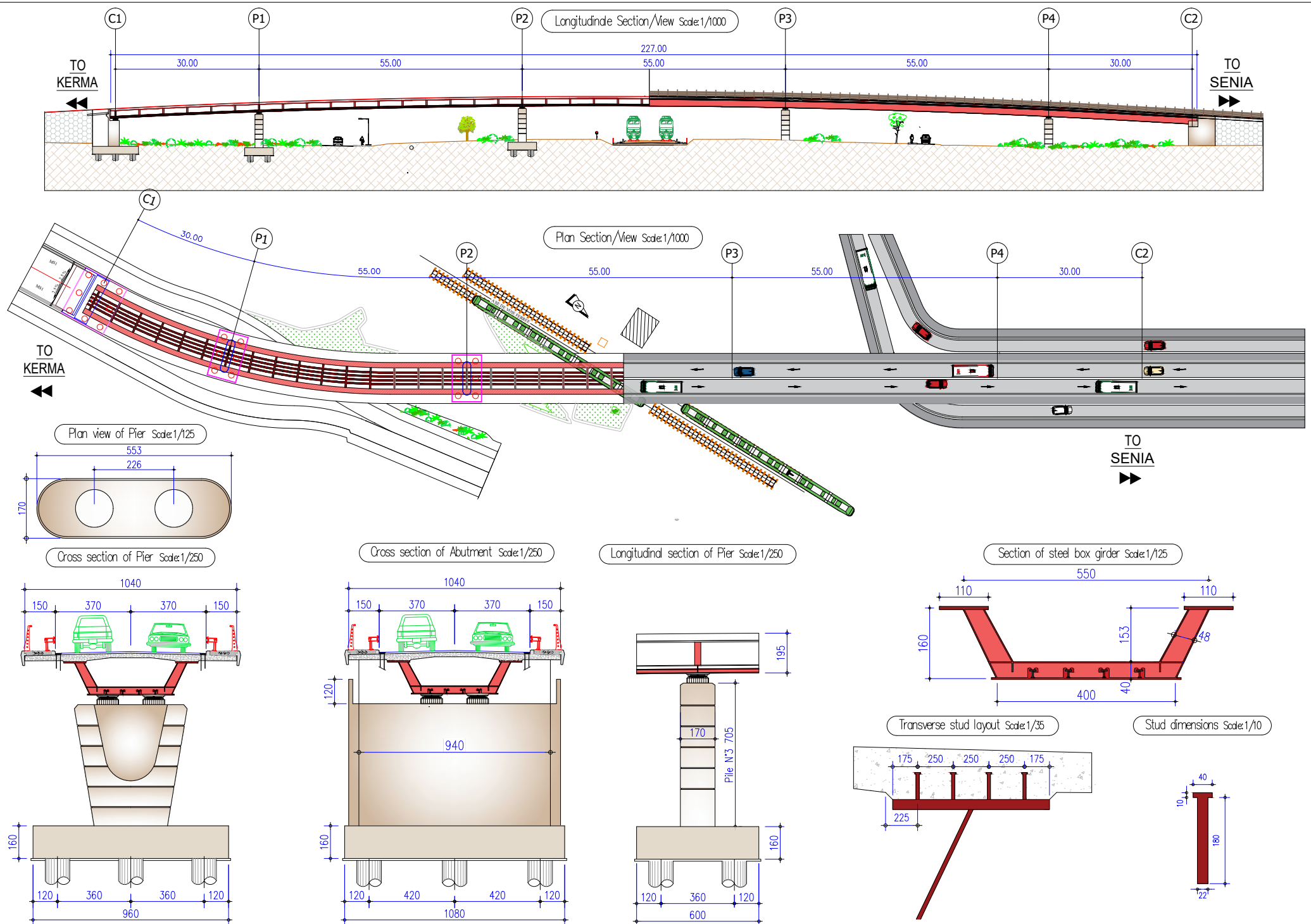
Element		Value / Description	Figure
Section	Shape	Rectangular with circular connections	
	Dimensions	$5.50 \times 1.60 \text{ m}^2$ $\Phi = 1.60 \text{ m}$	
Height	P1	6.98 m	
	P2	7.05 m	
	P3	6.75 m	
	P4	6.34 m	
Foundation	Type	deep foundation on 6 piles $\Phi = 1.20 \text{ m}$ $L = 20.00 \text{ m}$	
	Footing Dimensions	$9.60 \times 6.00 \times 1.60$ (m×m×m)	

III.4.3.2. Preliminary Sizing of Filled Abutments (SETRA Guide)

For the abutments, the same type as Variant 1 (filled abutments) is adopted. Their dimensions are defined according to the document “*Bridge Design and Construction: General Principles – Foundations – Supports – Standard Structures*” and are summarized in the table below.

Table III.10. Selection and Dimensions of the Abutment for Variant 2.

Element		Value / Description	Figure
Front wall	Thickness	2.30 m	
	Length	10.40 m	
	Height	5.40 / 4.45 m	
Parapet wall	Thickness	0.30 m	
	Length	10.40 m	
	Height	2.39 m	
Return wall	Thickness	0.50 m	
Transition slab	Height	0.30 m	
	Width	4.70 m	
Foundation	Type	deep foundation on 9 piles $\Phi = 1.20$ m $L = 20.00$ m	
	Footing	$9.60 \times 10.80 \times 2.20$ (m×m×m)	



III.4.4. Verification of Shear Buckling Resistance of the Web

The web is transversely stiffened at a spacing of 8 m. According to Eurocode 3 (Article 5), the susceptibility of the web to shear buckling must first be assessed by checking its slenderness.

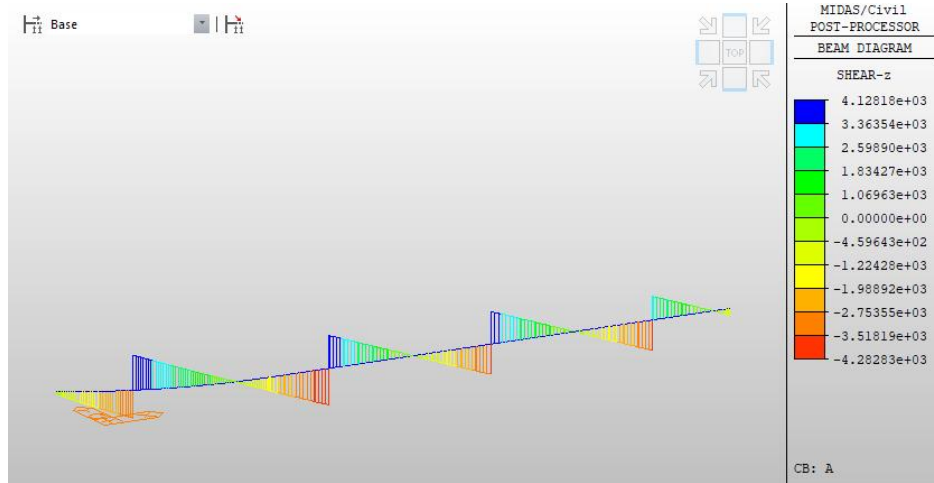


Figure III.6. Maximum shear force variant 02.

The verification consists of comparing the actual slenderness ratio h_w/t_w with a limiting value depending on the material parameter ε , the shear buckling coefficient k_τ , and the factor η .

$$\frac{h_w}{t_w} \leq \frac{31}{\eta} \varepsilon \sqrt{k_\tau}, \varepsilon = \sqrt{\frac{235}{f_y}}$$

$$\alpha = \frac{a}{h_w}, k_\tau = 5.34 + 4 \left(\frac{h_w}{a} \right)^2 (\alpha \geq 1)$$

Numerical Application :

$$\begin{aligned} \varepsilon &= 0.81 \\ \alpha &= \frac{7500}{1545} = 4.85 \geq 1 \\ k_\tau &= 5.51 \\ \frac{h_w}{t_w} &= \frac{1545}{20} = 77.25 \end{aligned}$$

Since : 77.25 > 51.06

The web is susceptible to shear buckling, and a detailed verification is required.

III.4.4.1. Shear Buckling Resistance

- The verification criterion is: $\frac{V_{Ed}}{V_{b,Rd}} \leq 1$
- The design resistance is given by: $V_{b,Rd} = V_{bw,Rd}$
- Critical shear stress: $\tau_{cr} = 0.9 k_\tau E \left(\frac{t_w}{h_w} \right)^2$
- Reduced slenderness: $\bar{\lambda}_w = \sqrt{\frac{f_y}{\sqrt{3} \tau_{cr}}}$

- Reduction factor: $\chi_w = \frac{1.37}{0.7 + \bar{\lambda}_w}$
- Resistance: $V_{bw,Rd} = \min \left(\frac{\chi_w f_y h_w t_w}{\gamma_{M1} \sqrt{3}} ; \frac{\eta f_y h_w t_w}{\gamma_{M1} \sqrt{3}} \right)$

Numerical Application :

$$\tau_{cr} = 174.51 \text{ MPa}$$

$$\bar{\lambda}_w = 1.09$$

$$\chi_w = 0.77$$

$$V_{b,Rd} = 4.88 \text{ MN}$$

$$\frac{V_{Ed}}{V_{b,Rd}} = \frac{4.28}{4.88} = 0.877 \leq 1$$

III.4.4.2. Conclusion

The web is adequately resistant to shear buckling, and the verification is satisfied.

III.4.5. Lateral Torsional Buckling Verification

The top flange is subjected to compression in the span and is therefore potentially vulnerable to lateral-torsional buckling.

This risk is critical during Phase 2 (slab casting), when the concrete is still fresh and composite action is not yet developed. At this stage, the top flange remains laterally unrestrained and must be checked for stability. After hardening, the slab provides restraint and carries the compressive stresses, eliminating the buckling risk. Therefore, the verification is performed only for the construction phase.

III.4.5.1. Section Verification

The local stability of the compressed flange is verified according to TGC Volume 10 to ensure that the section proportions are adequate or to justify the use of an effective width.

- Web-flange interaction: $\frac{h_f}{t_w} \leq 0.4 \frac{E}{f_y}$
- Flange local buckling: $\frac{b_f/2}{t_f} \leq 0.56 \sqrt{\frac{E}{f_y}}$

Numerical Application :

$$\frac{1545}{20} = 77.25 \leq 236.62 \Rightarrow \text{OK}$$

$$\frac{1100/2}{70} = 7.85 \leq 13.62 \Rightarrow \text{OK}$$

The web-flange interaction is satisfactory, but the flange is too slender. An effective width is therefore adopted:

$$b_{eff} = 1.12 \sqrt{\frac{E}{f_y}} \times t_f = 544.81 \text{ mm}$$

III.4.5.2. Lateral-Torsional Buckling Resistance

The resistance is evaluated according to TGC Volume 12, based on the critical stress and buckling length.

The bracing stiffness is determined by applying a unit load $H = 1$ N, leading to a displacement:

$$\nu = 20 \times 10^{-7} \text{ mm/N}$$

Table III.11. Geometric properties of the reduced section.

Parameter	Expression / Source	Value
Steel area	/	2.65
Centroid position	/	505
Moment of inertia	/	509
Compressed height	$h_c = H_f - Z_G$	1 040
Stress ratio	$\psi = \frac{-Z_G}{h_c}$	0.49
Buckling coefficient	$k = \frac{16}{1 + \psi + \sqrt{(1 + \psi)^2 + 0.112(1 - \psi)^2}}$	5.35
Effective compressed height	$h_{c,eff} = 0.86 \sqrt{k} \times \sqrt{\frac{E}{f_y}} \times \frac{h_c}{h_f} \times t_w$	651.34
Eccentricity	$e = \frac{h_c}{2} \cdot \frac{t_w(h_c - h_{c,eff})}{A_a - t_w(h_c - h_{c,eff})}$	15.71
Effective moment of inertia	$I_{eff} = I_a - \frac{h_c^2}{4} \cdot \frac{A_a t_w(h_c - h_{c,eff})}{A_a - t_w(h_c - h_{c,eff})} - \frac{t_w(h_c - h_{c,eff})^3}{12}$	506.74
Elastic section modulus	$W_{c,eff} = \frac{I_{eff}}{h_c + e}$	479.99
Compressed flange area	$A_D = b_{eff} t_w + \frac{h_{c,eff}}{2} t_w$	17.41
Flange inertia	/	0.76
Buckling length	$l_D = \sqrt[4]{\frac{\pi^4 E I_D}{e_v}}$	2 581.61

This gives:

$$l_D = 2\,581.61 \text{ mm}$$

Since this is smaller than the actual spacing (7 500 mm), the governing value is: $l_D = 7\,500\text{ mm}$

And the definition of ψ :
$$\psi = \frac{M_{\min}}{M_{\max}}$$

- ψ is the ratio between the minimum end moment and the maximum end moment of the considered segment
- The signs of moments are taken into account

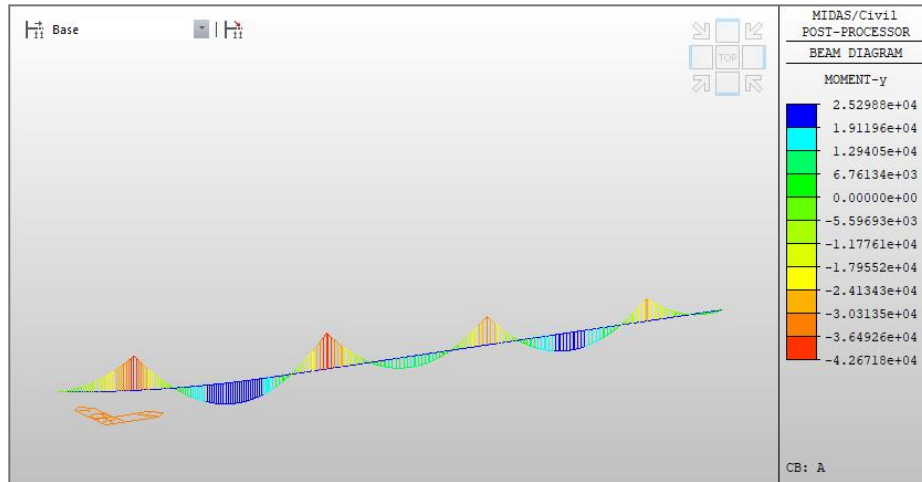


Figure III.7. Maximum bending moment variant 02.

We have : $M_{\min} = 0\text{ kN.m}$, $M_{\max} = -42\,671.81\text{ kN.m}$

Thus : $\psi = 0 \Rightarrow \eta = 1$

Where : $M_{\min} = -42\,671.81\text{ kN.m}$, $M_{\max} = 25\,298.83\text{ kN.m}$

Thus :
$$\psi = \frac{25\,298.83}{-42\,671.81} = -0.59$$

We then take the value $\eta = 1$ and calculate:

The reduced lateral buckling length: $l_k = l_D = 7\,500\text{ mm}$

$$i_D = \sqrt{\frac{I_D}{A_D}} , \lambda_k = \frac{l_k}{i_D} , \sigma_{cr,D} = \frac{\pi^2 E}{\lambda_k^2} , \bar{\lambda}_D = \sqrt{\frac{f_y}{\sigma_{cr,D}}} \\ \Phi_D = 0.5 \left[1 + \alpha_D (\bar{\lambda}_D - 0.2) + \bar{\lambda}_D^2 \right] , \chi_D = \frac{1}{\Phi_D + \sqrt{\Phi_D^2 - \bar{\lambda}_D^2}} , M_D = \sigma_D \times W_{c,eff}$$

Numerical Application :

$$\begin{aligned} i_D &= 208.93\text{ mm} , \lambda_k = 35.90 \\ \sigma_{cr,D} &= 1\,608.16\text{ MPa} , \bar{\lambda}_D = 0.47 \\ \Phi_D &= 0.71 , \chi_D = 0.81 , \sigma_D = 285.78\text{ MPa} , M_D = 137\,169.14\text{ kN.m} \\ M_{Rd} &= \frac{137\,169.14}{1.05} = 130\,637.28\text{ kN.m} , M_{Ed} = 42\,671.81\text{ kN.m} \\ M_{Ed} &< M_{Rd} \end{aligned}$$

The beam is safe against lateral-torsional buckling during construction.

III.5. DESIGN AND SIZING OF VARIANT 3: ARCH BRIDGE WITH LOWER DECK

Arch bridges are among the oldest types of structures used to span obstacles, often large gaps. They were initially built in stone and masonry and have evolved with modern materials, leading to metal arch bridges capable of very long spans (over 400 m).

The proposed bridge is a lower-deck arch bridge, consisting of two vertical steel arches connected by hangers, with two longitudinal steel side girders integrated with bridge cross members supporting a reinforced concrete deck (forming a composite section) and a roadway platform.



Figure III.8. Pearl River Bridge.

III.5.1. Longitudinal Design

Choosing appropriate spans is essential to ensure the bridge fits its site while effectively crossing the obstacle. For this project, three identical spans of 77 m each are selected, supported by two arches and inclined columns/braces, resulting in a total bridge length of 231 m.

III.5.1.1. Geometric Characteristics of the Arch

The arch is defined by the following set of characteristics:

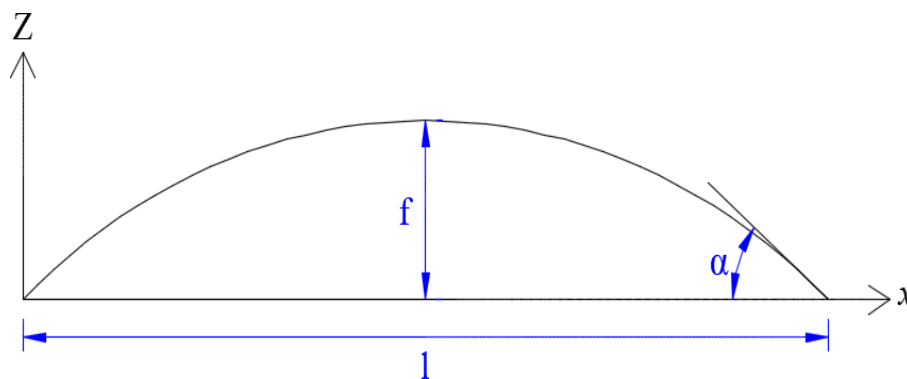


Figure III.9. Geometric characteristics of the project's arch.

❖ **Span l :**

The span of the arch is the horizontal distance between its two spring points or ends. It is determined by the engineer based on the obstacle to be crossed (in this case: railway tracks, traffic lanes, and sidewalks). For this project, a span of $l = 77$ m was chosen to satisfy site constraints.

❖ **Rise f :**

The rise of the arch represents the vertical distance from the line connecting the spring points to the highest point of the midline. It is determined by the slenderness ratio $\lambda_1 = f/l$, which, according to civil engineering guidelines (Volume 12, Steel Bridges), must satisfy:

$$\frac{l}{6} \leq f \leq \frac{l}{5}$$

For a 77 m span, a rise of $f = 14$ m is typically used.

❖ **Geometric equation:**

The arch is assumed to follow a symmetric parabolic shape. Its equation as a function of the horizontal coordinate x is:

$$Z = \frac{4f}{l^2} \cdot x \cdot (l - x)$$

III.5.2. Design of Structural Elements

The arch bridge consists of several load-bearing elements, each playing an essential role in load transfer. Loads applied on the deck are first carried by the slab, which works mainly in bending, then transferred to the cross beams and to the lateral beams. These beams distribute the loads through hangers to the arch, which works primarily in compression with some bending, before transmitting the forces to the supports and foundations.

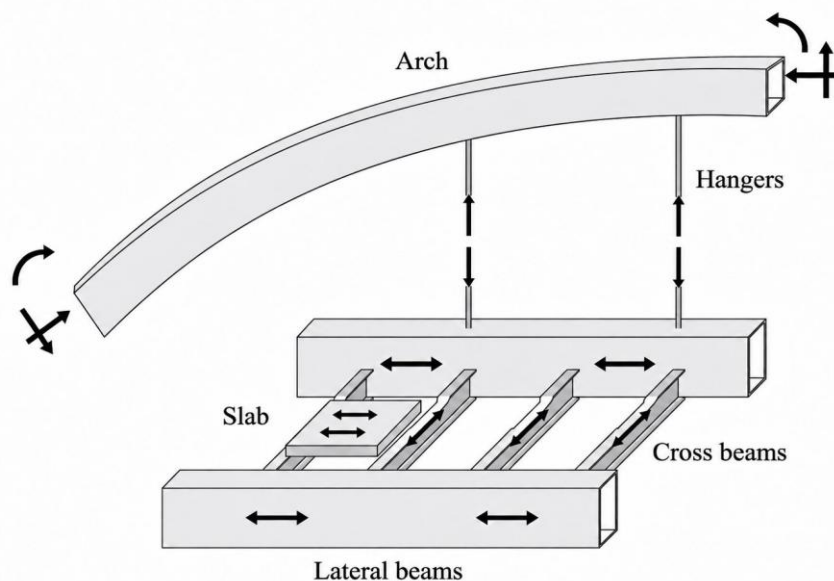


Figure III.10. Figure showing load transmission.

III.5.2.1. Arch

The arches must be designed to primarily resist compression as well as bending forces. Their resistance to buckling and bending depends on the inertia of the cross-section; therefore, sections with high inertia about both axes are preferred, such as tubular or welded box sections. For this project, a welded box section is adopted with dimensions $1200 \times 700 \text{ mm}^2$ and a thickness of 40 mm.

III.5.2.2. Lateral beams

The lateral beams consist of a constant rectangular tubular steel section. Their height is determined according to civil engineering guidelines using the slenderness ratio $\lambda_2 = (h_1 + h_2)/l$, generally between 1/30 and 1/45. With $h_1 = 1200\text{mm}$, we adopt $h_2 = 1150\text{mm}$, leading to a section of $1150 \times 600 \text{ mm}^2$ with a thickness of 30 mm. These beams are designed to resist bending due to deck loads and may also carry horizontal thrust from the arch, resulting in tensile forces.

III.5.2.3. Cross beams

The deck is stiffened by cross beams that transfer slab loads to the lateral beams. It consists of a steel grid of HEB 600 beams, 10.40 m long, spaced every 2.5 m (except the last at 1.0 m), giving a total of 95 beams. They mainly work in simple bending.

III.5.2.4. Deck slab

The roadway platform is supported by a reinforced concrete slab with a thickness of 0.25 m. The connection with the transverse beams is ensured by shear connectors.

III.5.2.5. Hangers

Hangers are vertical tension elements transferring loads from the deck to the arch and may be made of cables or bars, while rolled sections are rarely used. Threaded steel bars are preferred due to simpler assembly, easier prestressing, and a higher modulus of elasticity (about 15% greater than cables). In this project, Macalloy 460 high-strength carbon steel bars ($f_y = 460 \text{ N/mm}^2$) with rolled threads are adopted. Each hanger includes adjustable right/left threaded end fittings and is supplied with the required accessories. The nominal diameter of the hangers is selected from the available range according to the loads they must carry under different loading cases.



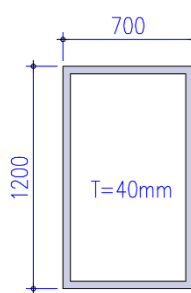
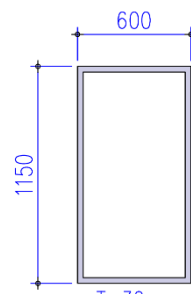
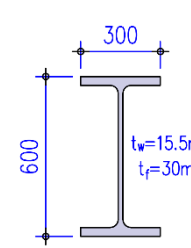
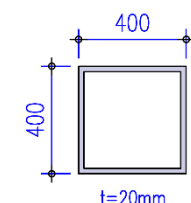
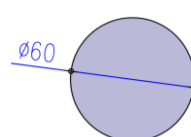
Figure III.11. Macalloy bar system.

In this project, M64 bars with a diameter of 60 mm are adopted.

To ensure a regular longitudinal distribution and adequate wind permeability, hanger spacing should generally exceed 3 m. For an arch span of $L = 77$ m, and for symmetry reasons, a spacing of 5 m is adopted, resulting in 78 hangers connecting each side girder to the arch.

The dimensions of the selected sections are presented in the table below:

Table III.12. Definition of Structural Element Sections.

Element	Dimensions	Figure
Arch / Column	$h = 1\ 200\text{ mm}$	
	$b = 700\text{ mm}$	
	$t_w = 40\text{ mm}$	
Lateral beams	$h = 1\ 150\text{ mm}$	
	$b = 600\text{ mm}$	
	$t_w = 30\text{ mm}$	
Cross beams	$h = 600\text{ mm}$	
	$b = 300\text{ mm}$	
	$t_f = 30\text{ mm}$	
	$t_w = 15.5\text{ mm}$	
Bracing system	$h = 400\text{ mm}$	
	$h = 400\text{ mm}$	
	$t_w = 20\text{ mm}$	
Hangers	$\Phi = 60\text{ mm}$	

III.5.3. Substructure Design

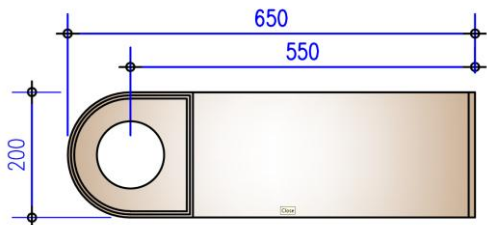
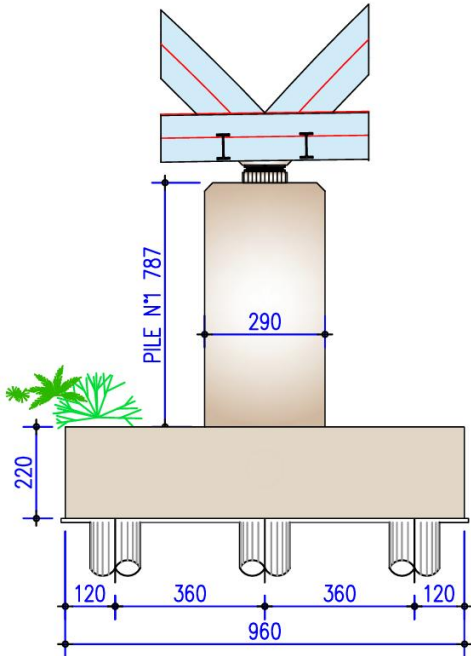
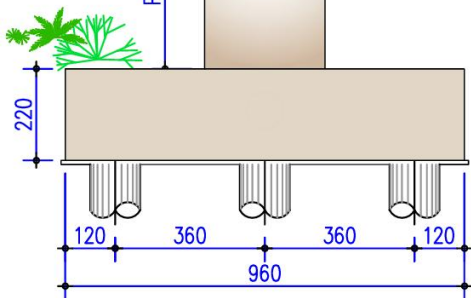
III.5.3.1. Supports Design

The choice of the static system for an arch bridge is critical and is influenced by the construction method. Common systems include double-fixed, double-hinged, or three-hinged arches. In this project, a double-fixed system is adopted at the arch ends on the piers, either cast directly into the supports or using fixed bearing devices. End spans are simply supported on the abutments.

III.5.3.2. Piers

Piers transfer arch forces to the foundations. For this variant, wall-type piers are chosen, with their dimensions summarized in the table below:

Table III.13. Selection and Dimensions of the Pier for Variant 3.

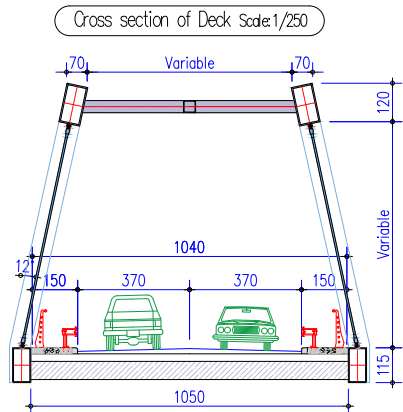
Element		Value / Description	Figure
Section	Shape	Rectangular with circular connections	
	Dimensions	$11.00 \times 1.50 \text{ m}^2$ $\Phi = 1.50 \text{ m}$	
Height	P1	7.87 m	
	P2	7.77 m	
Foundation	Type	deep foundation on 9 piles $\Phi = 1.20 \text{ m}$ $L = 20.00 \text{ m}$	
Footing	Length	13.10 m	
	Width	9.60 m	
	Height	2.20 m	

III.5.3.3. Abutments

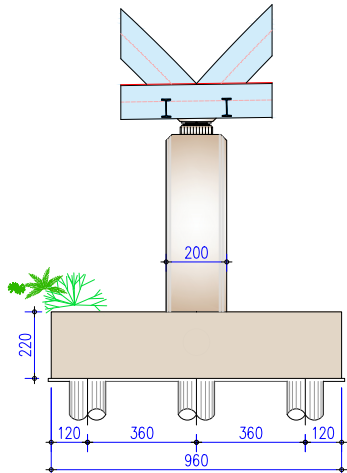
As with the previous variants, filled abutments are chosen. Their dimensions are defined according to the document “Bridge Design and Construction: General Principles – Foundations – Supports – Standard Structures”, as summarized in the table below:

Table III.14. Selection and Dimensions of the Abutment for Variant 3.

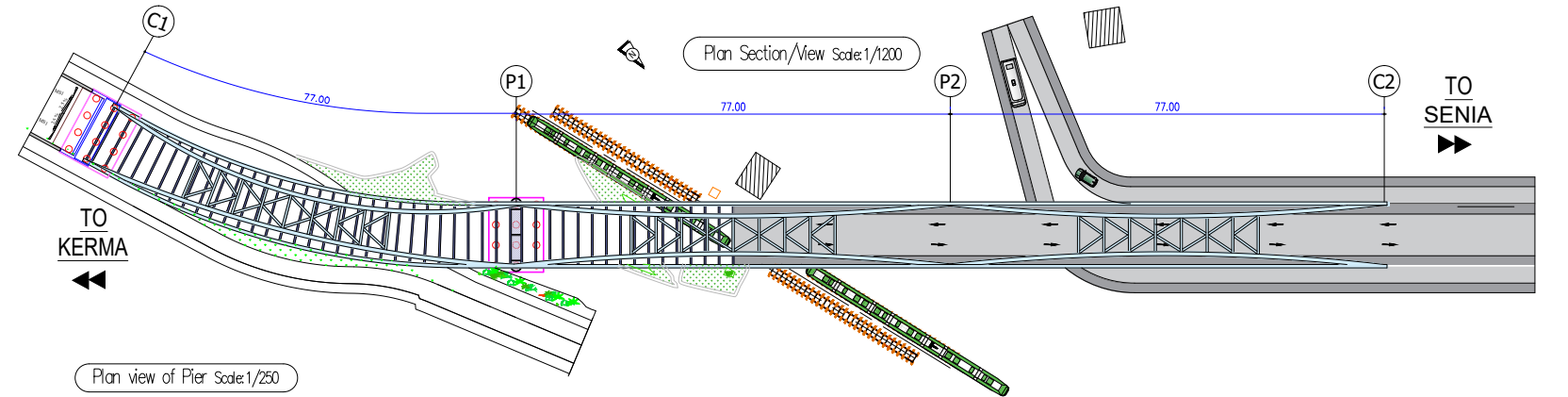
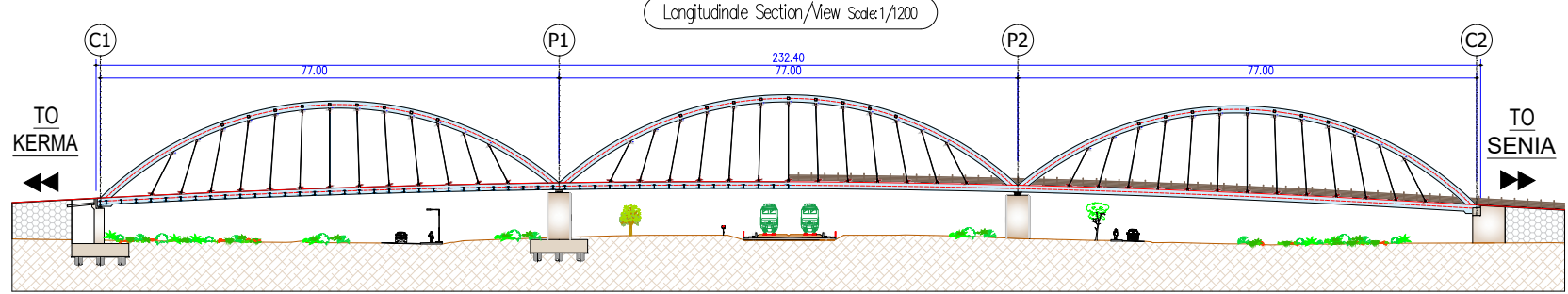
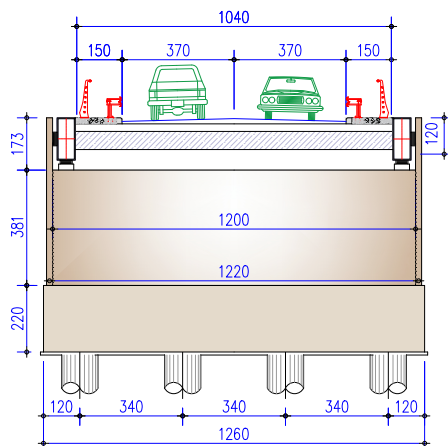
Element		Value / Description	Figure
Front wall	Thickness	1.65 m	
	Length	10.40 m	
	Height	5.91 / 5.13 m	
Parapet wall	Thickness	0.30 m	
	Length	10.40 m	
	Height	1.64 m	
Return wall	Thickness	0.50 m	
Transition slab	Height	0.30 m	
	Width	4.55 m	
Foundation	Type	deep foundation on 9 piles $\Phi = 1.20$ m $L = 20.00$ m	
	Footing	$12.60 \times 9.60 \times 2.20$ (m×m×m)	



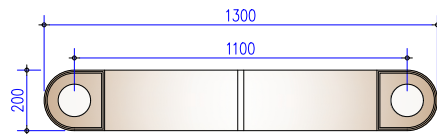
Longitudinal section of Pier Scale:1/250



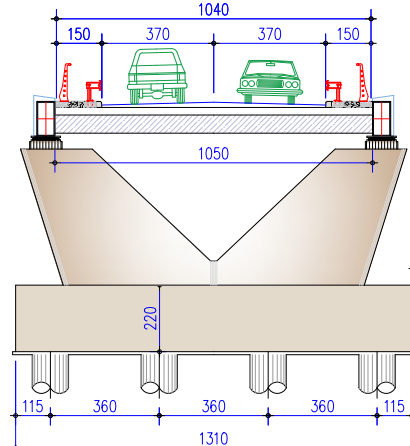
Cross section of Abutment Scale:1/250



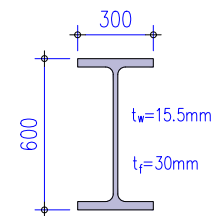
Plan view of Pier Scale:1/250



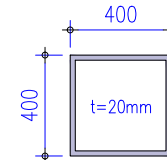
Cross section of Pier Scale:1/250



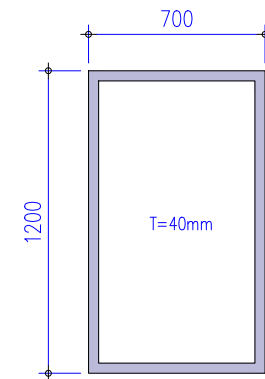
Bracing system Scale:1/30



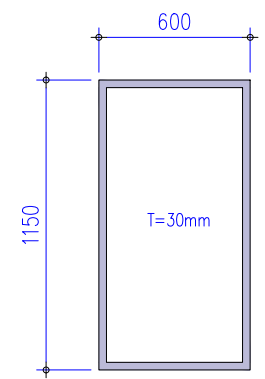
HEB 600 Scale:1/30



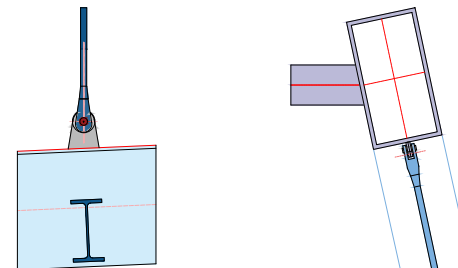
Lateral beam Scale:1/30



Arch sectionr Scale:1/30



Details of Hangers Scale:1/75



III.5.4. Structural Modelling

As with the previous two variants, the modelling of this alternative was carried out using MIDAS Civil 2022 VERSION.

The following modelling assumptions were adopted:

- All load-bearing elements of the structure are modelled as beam elements.
- The hangers are modelled as tension-only bar elements, since they are designed to carry axial tensile forces exclusively.
- Regarding boundary conditions, the structure is: Simply supported at the abutments, Fully fixed at the ends of the arches
- For structural analysis, We apply a load of $G+1.2A(l)$

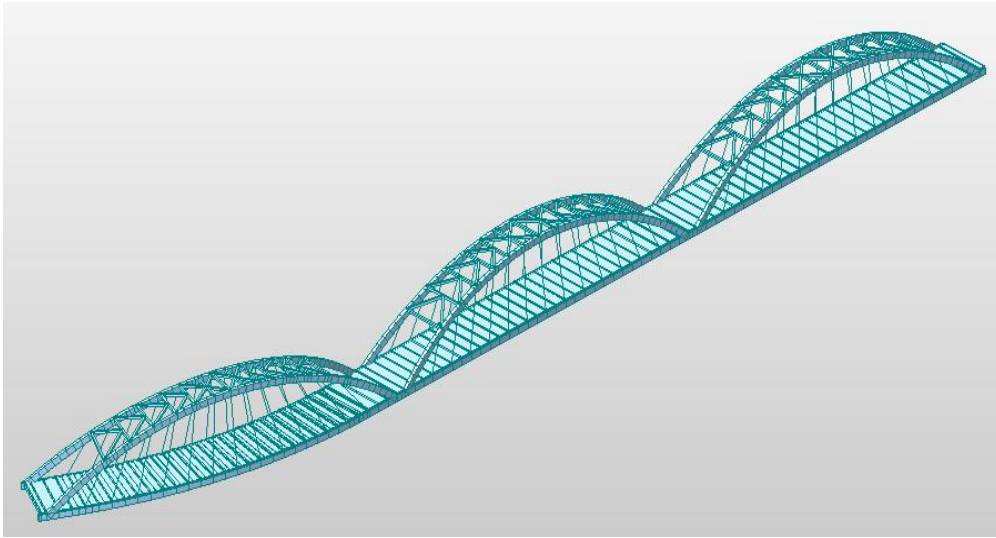


Figure III.12. Calculation model of variant 3.

III.5.5. Verification of Bridge Components

The selected cross-sections must withstand the internal forces induced by the applied loads. Therefore, stress verifications are performed according to the structural behavior of each component to ensure adequate sizing.

At this stage, the following elements are checked:

III.5.5.1. Verification of Hangers under Tension

The hangers are vertical elements working in pure tension. Their role is to connect the arch to the deck and transfer loads between them.

For this project, Macalloy M64 bars were selected, characterized by:

- Diameter: 60 mm
- Minimum elastic resistance: 1204 kN

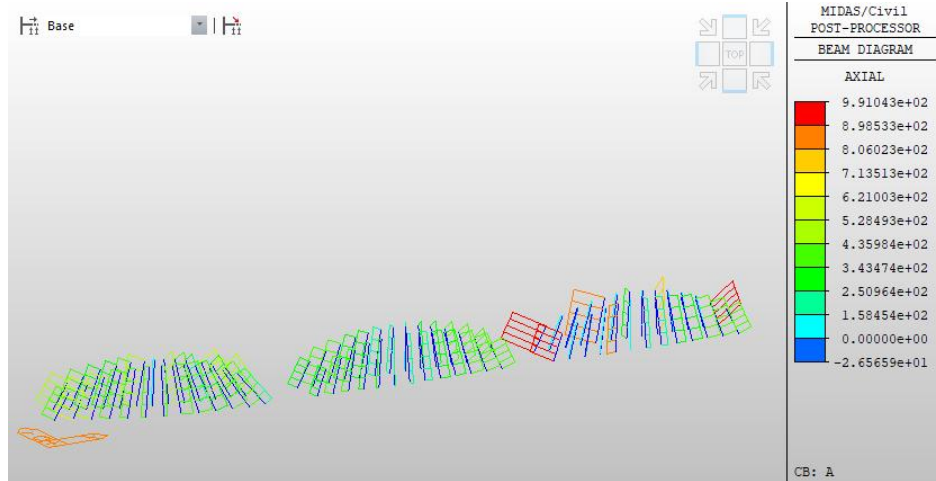


Figure III.13. Maximum tensile force in the hangers at SLS.

From the structural analysis model, the maximum tensile force in the hangers is:

$$T_{max} = 991.04 \text{ kN}$$

Since :

$$991.04 \text{ kN} < 1\,204 \text{ kN}$$

the selected hanger diameter is adequate and verified at the Serviceability Limit State (SLS).

III.5.5.2. Verification of the Arch Section Combined Biaxial Bending

The arch, being the main structural element, is primarily subjected to axial compression combined with biaxial bending. Its verification at the Ultimate Limit State (ULS) consists of checking the resistance of its cross-section.

❖ Cross-section classification

According to Eurocode 3, the resistance of a section depends on its classification.

The arch section is classified as Class 1.

- Web Classification

For a compressed and bent plate element where $\alpha \leq 0.5$ we must verify:

$$\frac{c}{t} \leq \frac{36\epsilon}{\alpha}$$

Where :

α : imperfection factor ; $\alpha = 0.34$

c : height of the web ; $c = h - 3t = 1\,080 \text{ mm}$

t : thickness of the plates ; $t = 40 \text{ mm}$

$\epsilon = \sqrt{235/f_y}$; with $f_y = 355 \text{ MPa}$

Result :

$$\frac{c}{t} = 27 \leq 85.76$$

⇒the web is class 1

- Flange Classification

Similarly, the classification of the flanges is verified, where c represents the flange width, defined as:

$$c = b - 3t$$

$$\frac{c}{t} = \frac{580}{40} = 14.50 \leq 85.76$$

⇒The flanges are class 1

❖ Resistance of the cross-section

For a Class 1 section subjected to axial force and biaxial bending, the following condition must be satisfied:

$$\left(\frac{M_{y,Ed}}{M_{Ny,Rd}} \right)^\alpha + \left(\frac{M_{z,Ed}}{M_{Nz,Rd}} \right)^\beta \leq 1$$

Where :

$$\alpha = \beta = 1.66 / (1 - 1.13n^2) \leq 6 ; n = \frac{N_{Ed}}{N_{pl,Rd}} ; \alpha = \beta \leq 6$$

With :

$N_{pl,Rd}$: design plastic axial resistance, given by: $N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$

$M_{y,Ed}$, $M_{z,Ed}$, and N_{Ed} : design bending moments about the y and z axes, and the design axial force, respectively.

$M_{Ny,Rd}$ and $M_{Nz,Rd}$: reduced design bending resistances about the yy and zz axes, respectively, expressed as:

$$\begin{cases} M_{Ny,Rd} = M_{pl,y,Rd} \cdot \frac{1-n}{1-0.5a_w} & \text{with } M_{Ny,Rd} \leq M_{pl,y,Rd} \\ M_{Nz,Rd} = M_{pl,z,Rd} \cdot \frac{1-n}{1-0.5a_f} & \text{with } M_{Nz,Rd} \leq M_{pl,z,Rd} \end{cases}$$

Where :

$$M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}}$$

$$a_w = \frac{A - 2bt_f}{A} \text{ with } a_w \leq 0.5$$

$$a_f = \frac{A - 2ht_w}{A} \text{ with } a_f \leq 0.5$$

The axial force and bending moment diagrams are presented in the following section.

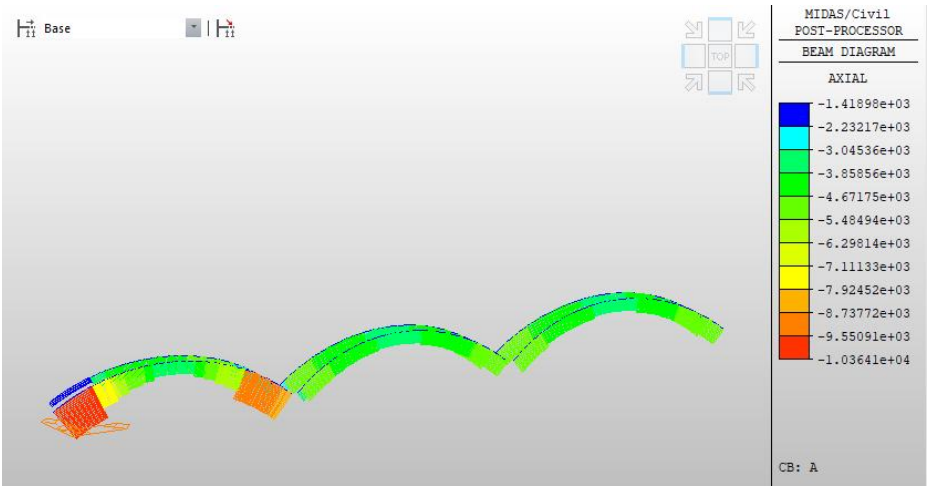


Figure III.14. Diagram of the maximum normal force.

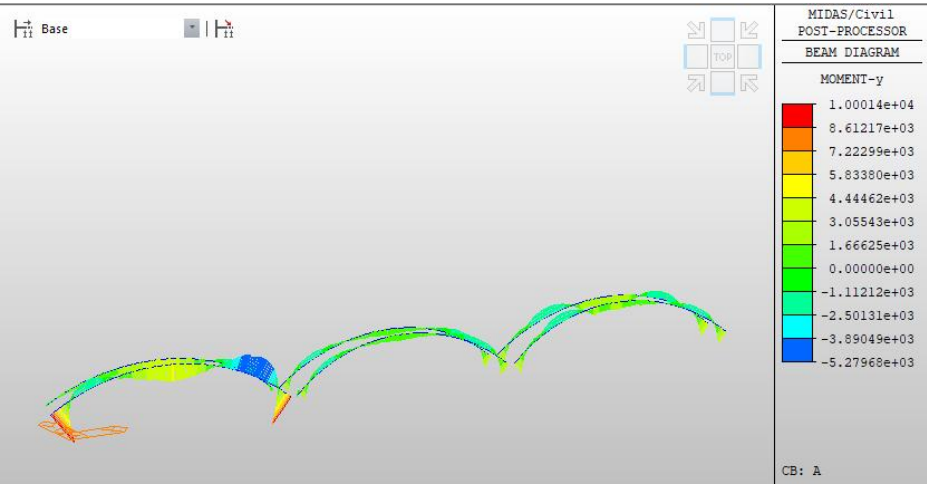


Figure III.15. Maximum bending moment diagram M_y .

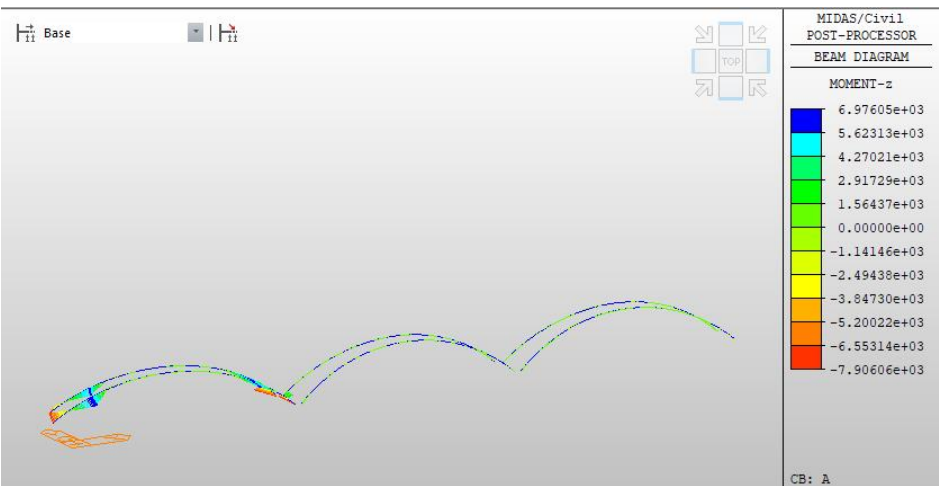


Figure III.16. Maximum bending moment diagram M_z .

The calculated values are summarized in Table III.15.

Table III.15. Table showing the calculated values.

Parameter	Value	Parameter	Value
A	0.146 m ²	N _{pl.Rd}	54.53 MN
W _{pl.y}	0.06 m ³	M _{pl.y.Rd}	20.44 MN.m
W _{pl.z}	0.04 m ³	M _{pl.z.Rd}	13.98 MN.m
t _f = t _w	40 mm	M _{N.y.Rd}	22.08 MN.m
a _w	0.5	M _{N.z.Rd}	13.64 MN.m
a _f	0.34	N _{Ed}	10.36 MN
n	0.19	M _{y.Ed}	10.00 MN.m
α = β	1.59	M _{z.Ed}	7.91 MN.m

III.5.6. Conclusion

Therefore, the selected section is adequate and safe for the applied loads.

III.6. QUANTITATIVE AND COST ESTIMATE

All detailed calculations of the cost estimates for the three proposed variants are presented in [Tables III.16](#), [III.17](#), and [III.18](#).

Based on these calculations, the total cost including all taxes (TTC) for each variant is determined as follows:

Table III.16. Quantitative and Cost Estimate for Variant 1.

Quantitative and Cost Estimate for Variant 1					
N°	Description of Items	U	Quantity	Unit Price (ET)	Amount
Poste 1 : Piles					
2.1	Drilling of Ø1200 mm vertical piles in non-rocky soil	ML	1 080	55 000.00	59 400 000.00
2.2	Supply and installation of Ø50/60 steel tubes for sonic testing	ML	3 240	2 500.00	8 100 000.00
2.3	Sonic integrity testing	U	54	45 000.00	2 430 000.00
2.4	Pile head trimming	U	54	25 000.00	1 350 000.00
2.5	Static load test of isolated piles under axial compression	U	2	400 000.00	800 000.00
TOTAL (Poste 1)					72 080 000.00

Poste 2 : Concrete					
2.1	Lean concrete (10 cm) for foundation beds	M ³	60	10 500.00	630 000.00
2.2	Concrete RN25 for piles	M ³	1 222	13 500.00	16 497 000.00
2.3	Concrete RN30 for footings (piers, abutments)	M ³	1 262	15 000.00	18 930 000.00
2.4	Concrete RN30 for piers	M ³	263	16 000.00	4 208 000.00
2.5	Concrete RN30 for abutments	M ³	341	16 000.00	5 456 000.00
2.6	Concrete RN35 for segments	M ³	1 680	35 000.00	58 800 000.00
TOTAL (Poste 2)					104 521 000.00

Poste 3 : Steel					
2.1	Reinforcing steel Fe E50 for substructure	T	379	132 000.00	50 028 000.00
2.2	Reinforcing steel Fe E50A for segments	T	320	132 000.00	42 240 000.00
2.3	Prestressing steel for segments	T	68	490 000.00	33 320 000.00
2.4	Prestressing steel for segment erection	T	440	14 000.00	6 160 000.00
TOTAL (Poste 3)					131 748 000.00

Poste 4 : Lump Sum – Bridge Works					
4.1	a. Site installation and mobilization				
	b. Equipement removal				
	c. Demobilization, clearing	F	-	40 000 000.00	40 000 000.00
	d. stripping of topsoil,				
	e. Execution studies				
4.2	Segment shoring using temporary towers (including relocation between supports)	F	-	15 000 000.00	15 000 000.00
TOTAL (Poste 4)					55 000 000.00

Total Excluding VAT	363 349 000.00
VAT (19%)	69 036 310.00
Total Including VAT (ATI)	432 385 310.00

Four hundred thirty-two million, three hundred eighty-five thousand, three hundred ten Algerian dinars.

Table III.17. Quantitative and Cost Estimate for Variant 2.

Quantitative and Cost Estimate for Variant 2					
N°	Description of Items	U	Quantity	Unit Price (ET)	Amount
Poste 1 : Piles					

2.1	Drilling of Ø1200 mm vertical piles in non-rocky soil	ML	840	55 000.00	46 200 000.00
2.2	Supply and installation of Ø50/60 steel tubes for sonic testing	ML	2 520	2 500.00	6 300 000.00
2.3	Sonic integrity testing	U	42	45 000.00	1 890 000.00
2.4	Pile head trimming	U	42	25 000.00	1 050 000.00
2.5	Static load test of isolated piles under axial compression	U	2	400 000.00	800 000.00
TOTAL (Poste 1)					56 240 000.00

Poste 2 : Concrete					
2.1	Lean concrete (10 cm) for foundation beds	M ³	46	10 500.00	483 000.00
2.2	Concrete RN25 for piles	M ³	951	13 500.00	12 838 500.00
2.3	Concrete RN30 for footings	M ³	701	15 000.00	10 515 000.00
2.4	Concrete RN30 for piers	M ³	212	16 000.00	3 392 000.00
2.5	Concrete RN30 for abutments	M ³	354	16 000.00	5 664 000.00
2.6	Concrete RN35 for deck slab	M ³	650	18 000.00	11 700 000.00
TOTAL (Poste 2)					44 592 500.00

Poste 3 : Steel					
2.1	Reinforcing steel Fe E50 for substructure	T	273	12 500.00	3 412 500.00
2.2	Reinforcing steel Fe E50A for deck slab	T	163	12 500.00	2 037 500.00
2.3	Structural steel for steel box girder	T	871	340 000.00	296 140 000.00
2.4	Assembly and erection of steel deck	T	871	80 000.00	69 680 000.00
TOTAL (Poste 3)					371 270 000.00

Poste 4 : Lump Sum – Bridge Works					
4.1	a. Site installation and mobilization b. Equipement removal c. Demobilization, clearing d. stripping of topsoil, e. Execution studies	F	-	40 000 000.00	40 000 000.00
4.2	Scaffolding for deck erection	F	-	15 000 000.00	15 000 000.00
TOTAL (Poste 4)					55 000 000.00

Total Excluding VAT	527 102 500.00
VAT (19%)	100 149 475.00
Total Including VAT (ATI)	627 251 975.00

Six hundred twenty-seven million, two hundred fifty-one thousand, nine hundred seventy-five Algerian dinars.

Table III.18. Quantitative and Cost Estimate for Variant 3.

Quantitative and Cost Estimate for Variant 3					
N°	Description of Items	U	Quantity	Unit Price (ET)	Amount
Poste 1 : Piles					
2.1	Drilling of Ø1200 mm vertical piles in non-rocky soil	ML	960	55 000.00	52 800 000.00
2.2	Supply and installation of Ø50/60 steel tubes for sonic testing	ML	2 880	2 500.00	7 200 000.00
2.3	Sonic integrity testing	U	48	45 000.00	2 160 000.00
2.4	Pile head trimming	U	48	25 000.00	1 200 000.00
2.5	Static load test of isolated piles under axial compression	U	2	400 000.00	800 000.00
TOTAL (Poste 1)					64 160 000.00
Poste 2 : Concrete					
2.1	Lean concrete (10 cm) for foundation beds	M³	52	10 500.00	546 000.00
2.2	Concrete RN25 for piles	M³	1 085	13 500.00	14 647 500.00
2.3	Concrete RN30 for footings	M³	1 086	15 000.00	16 290 000.00
2.4	Concrete RN30 for piers	M³	261	16 000.00	4 176 000.00
2.5	Concrete RN30 for abutments	M³	350	16 000.00	5 600 000.00
2.6	Concrete RN35 for deck slab	M³	610	18 000.00	10 980 000.00
TOTAL (Poste 2)					52 239 500.00
Poste 3 : Steel					
2.1	High-bond reinforcing steel Fe E50A for substructure	T	342	132 000.00	45 144 000.00
2.2	High-bond reinforcing steel Fe E50A for deck slab	T	153	132 000.00	20 196 000.00
2.3	Structural steel for arch	T	721	360 000.00	259 560 000.00
2.4	Structural steel for deck (girders + cross beams)	T	487	320 000.00	155 840 000.00
2.5	Assembly and erection of steel deck and arches	T	1 208	80 000.00	96 640 000.00
2.6	Supply and installation of galvanized steel tie rods (Macalloy M64 type) for stay cables	T	7.20	800 000.00	5 760 000.00
TOTAL (Poste 3)					583 140 000.00
Poste 4 : Lump Sum – Bridge Works					

4.1	a. Site installation and mobilization b. Equipement removal c. Demobilization, clearing d. stripping of topsoil, e. Execution studies	F	-	50 000 000.00	50 000 000.00
4.2	Scaffolding for deck erection	F	-	10 000 000.00	10 000 000.00
TOTAL (Poste 4)					60 000 000.00

Total Excluding VAT	759 539 500.00
VAT (19%)	144 312 505.00
Total Including VAT (ATI)	903 852 005.00

Nine hundred three million, eight hundred fifty-two thousand and five Algerian dinars.

Table III.19. Total Cost of Each Variant.

Variant 1	Variant 2	Variant 3
432 385 310.00 DA	627 251 975.00 DA	903 852 005.00 DA

III.7. MULTICRITERIA ANALYSIS

The multicriteria analysis constitutes a decision-support approach used to compare the different design alternatives based on a set of predefined criteria.

In this study, the three proposed bridge variants are evaluated with respect to their aesthetic quality, economic cost, maintenance requirements, construction techniques, and execution time. This assessment is carried out in accordance with standard engineering practices and supported by the techno-economic judgment of field engineers.

Table III.20 Multicriteria Analysis of the Variants.

Variant	Advantages	Disadvantages
Variant 1: Prestressed concrete bridge constructed by balanced cantilever method	- Rapid construction when using precast segments - Flexible execution by multiplying casting/erection fronts - Good cost–aesthetic ratio. - Well-mastered and widely used technique	- Significant structural depth (≈ 2.4 m segments) - Requires qualified labor for prestressing - Large structural elements - Complex execution
Variant 2: Steel box girder bridge	- Slim deck (reduced structural depth) - Lightweight structure - Good architectural appearance - Suitable for medium to long spans	- Requires highly skilled labor (welding) - More complex execution - High cost compared to Variant 1

Variant 3: Arch bridge with lower deck	- Excellent adaptation to functional and site constraints - Slim deck - Lightweight and efficient structural behavior - Outstanding architectural and aesthetic value	- Requires highly skilled labor (welding and assembly) - Complex construction technique - Relatively high cost compared to other variants
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III.8. CONCLUSION

After analyzing the different comparison criteria and based on the quantitative cost estimate, it is observed that, in terms of economy and construction time, Variant 1 is the most advantageous solution. It ensures a significant reduction in overall cost while allowing relatively rapid execution, thanks to the use of well-mastered construction techniques such as the balanced cantilever method.

However, from a techno-economic perspective, Variant 2 provides certain advantages, particularly due to its lightweight structure and its suitability for medium to long spans. It also offers a more refined architectural appearance and better structural efficiency. Nevertheless, its higher cost and the complexity of execution, especially due to welding requirements, reduce its competitiveness compared to the first variant.

As for aesthetics, Variant 3 is clearly the most attractive solution, offering outstanding architectural quality and excellent adaptation to site constraints. It also provides a slender structure and efficient structural behavior. However, this variant involves a high level of technical complexity and significantly higher costs, which make it less suitable in the context of this project.

Therefore, considering that the primary objective of this study is to achieve an optimal balance between cost, feasibility, and construction time, Variant 1 is selected as the final solution. This choice is justified by its economic efficiency, reliability, and the use of well-established construction methods, making it the most appropriate option for the project.

CHAPTER IV

LOADS AND CALCULATION HYPOTHESES

IV.1. INTRODUCTION

The design and analysis of bridge elements are governed by the choice and quality of construction materials, as well as by the loads applied to the structure and the predefined calculation assumptions in accordance with current standards and project requirements. The structure must ensure proper behavior during both the construction and service phases under the effect of various actions, including self-weight, traffic loads, superstructure loads, and sidewalk loads. This chapter defines the construction materials and their characteristics, and evaluates the different loads and actions, whether permanent, functional, or natural, that may affect the structure throughout its service life.

IV.2. DESIGN CODES AND CALCULATION RULES

The design of the structure will be carried out in accordance with the codes and regulations applied in Algeria, as well as relevant Eurocodes:

- **BAEL 91 revised 99**: Design and calculation rules for reinforced concrete structures using the limit state method;
- **BPEL 91**: Design and calculation rules for prestressed concrete structures using the limit state method;
- **RCPR**: Rules defining loads for the calculation and testing of road bridges, including traffic and thermal actions;
- **RPOA 2008**: Seismic design rules for civil engineering structures;
- **Fascicule 62**: Design and calculation rules for foundations of civil engineering structures;
- **SETRA Guide**: Technical guidelines from the Service d'Étude Technique des Routes et Autoroutes;
- **Eurocode 3**: Design of steel structures;
- **Eurocode 4**: Design of composite steel-concrete structures;
- **D.T.R. C 2-4.7 RNV (2013)**: Regulatory document for snow and wind loads.

IV.3. MATERIAL CHARACTERISTICS

The choice of construction materials largely determines the design and calculation of the bridge. The main materials used in the structure are concrete and steel (both active and passive).

IV.3.1. Concrete

IV.3.1.1. Mechanical Properties

Concrete is a generic term for a monolithic composite construction material made from aggregates (sand, gravel) bound together by a binder. The mechanical properties of concrete, such as compressive strength, modulus of elasticity, and durability, are considered in the design to ensure structural performance throughout the service life of the bridge.

Table IV.1. Mechanical Properties of Concrete.

	Element	f_{c28} (MPa)	f_{t28} (MPa)
Superstructure	Deck	35	2.70
Substructure	Blinding Concrete	16	1.56
	Piles	25	2.10
	Foundations	30	2.40

Remark :

- Concrete density: 25 kN/m³.
- Tensile strength at 28 days: $f_{t28} = 0.6 + 0.06 \times f_{c28}$.

IV.3.1.2. Allowable Compressive Stresses

❖ **At Ultimate Limit State (ULS) :**

$$f_{bu} = 0.85 \times f_{cj} / (\theta \times \gamma_b) \text{ (MPa)}$$

Coefficient θ depends on the probable duration of the considered action:

$$\theta = \begin{cases} 1 & \text{if } t > 24 \text{ h} \\ 0.9 & \text{if } 1 \leq t \leq 24 \text{ h} \\ 0.85 & \text{if } t < 1 \text{ h} \end{cases}$$

Coefficient γ_b strength reduction factor, depending on the situation:

$$\gamma_b = \begin{cases} 1.5 & \text{durable situation} \\ 1.5 & \text{transient situation} \\ 1.15 & \text{accidental situation} \end{cases}$$

❖ **At Serviceability Limit State (SLS) :**

$$f_{bc} = 0.6 \times f_{c28} \text{ (MPa)}$$

Table IV.2. Allowable Concrete Stresses.

States	Limit Stresses		Deck	Piles	Foundations
ULS	f_{bu} (MPa)	$\gamma_b = 1.50$	19.83	14.17	17.00
		$\gamma_b = 1.15$	30.43	21.74	26.09
SLS	$f_{bc} (\overline{\sigma}_{bc})$ (MPa)		21.00	15.00	18.00

IV.3.1.3. Concrete Deformation

❖ Longitudinal :

The longitudinal modulus of elasticity of concrete, E_{ij} , as defined by the BAEL rules, is given as follows:

$$E_{ij} = \begin{cases} 11\,000 \times \sqrt[3]{f_{cj}} & \text{instantaneous modulus, } < 24 \text{ h} \\ 3\,700 \times \sqrt[3]{f_{cj}} & \text{long-term loading} \end{cases}$$

❖ Transversal :

According to Hooke's law, the transverse modulus of elasticity is given by:

$$G = \frac{E}{2(1 + \nu)}$$

❖ Poisson's ratio (ν) :

where ν is the Poisson's ratio, defined as the ratio between lateral and longitudinal deformation, as follows:

$$\nu = \begin{cases} 0.2 & \text{at Serviceability Limit State (SLS)} \\ 0 & \text{at Ultimate Limit State (ULS)} \end{cases}$$

IV.3.2. Reinforcements

The steels used in the prestressed concrete structure are of two types:

- ❖ **Passive steel:** to resist shear forces and control cracking.
- ❖ **Active steel:** for prestressing.

IV.3.2.1. Passive Reinforcements

The chosen type of passive reinforcement is high-bond steel of grade Fe E500, with the following characteristics:

- ❖ **Yield strength:** $f_e = 500 \text{ MPa}$
- ❖ **Modulus of elasticity:** $E_s = 200\,000 \text{ MPa}$

The standard commercial diameters of independent HA bars are (in mm):

8 10 12 14 16 20 25 32 40

The usual delivery lengths are: 12 meters for straight bars.

The table below presents the allowable stress values according to BAEL:

Table IV.3. Allowable Stresses in Passive Reinforcement.

cracking	Allowable Stress Formula		Obtained Value (MPa)	
			Super	Infra
Slightly harmful	$\sigma_s = \frac{f_e}{\gamma_s}$	S – Durable or transient: $\gamma_s = 1.15$	434.78	434.78
		S – Accidental: $\gamma_s = 1.00$	500.00	500.00
harmful	$\sigma_s = \max \left\{ \frac{f_e}{2}, 110\sqrt{n \times f_{tj}} \right\}, n = 1.6 \text{ for HA bars}$		250.00	250.00
Very harmful	$\sigma_s = \max \{0.4 \times f_e, 88\sqrt{n \times f_{tj}}\}, n = 1.6 \text{ for HA bars}$		200.00	200.00

IV.3.2.2. Active Reinforcement (Prestressing Steel)

The selected prestressing reinforcement is composed of T15 TBR strands (Very Low Relaxation), characterized by the following mechanical properties:

- Ultimate tensile strength: $f_{prg} = 1\,860 \text{ MPa}$
- Yield strength: $f_{peg} = 1\,674 \text{ MPa}$
- Modulus of elasticity: $E_p = 190\,000 \text{ MPa}$
- Cross-sectional area of one strand: $A_p = 140 \text{ mm}^2$
- Duct diameter: $\varnothing = 80 \text{ to } 100 \text{ mm}$
- Relaxation coefficient at 1000 hours: $\rho = 2.5\%$
- Anchorage slip: $g = 6 \text{ mm}$

❖ Prestressing Justification Class

Class II The structure is designed according to Prestressing Justification Class II. This class allows the development of tensile stresses in concrete under service conditions, but strictly prohibits the formation of cracks.

Class	Execution	Service		
		<u>Rares</u>	<u>Frequent</u>	<u>Q-Perm</u>
Class II				

Figure IV.1. Permissible stress limits of Class II of prestressing.

IV.4. EVALUATION OF ACTIONS

IV.4.1. Evaluation of Non-Traffic Actions

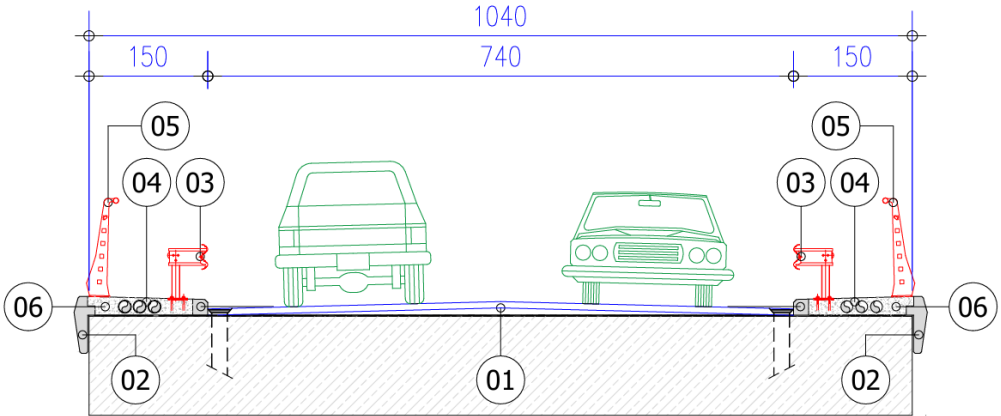
IV.4.1.1. Permanent Loads

Permanent loads arise from the self-weight of structural materials and all integrated permanent elements, such as pavement, equipment, and backfill. These loads are typically calculated automatically using structural analysis software based on the geometry and material properties. The unit weights of materials are taken according to design standards:

- **Reinforced concrete:** 25 kN/m^3
- **Plain concrete:** 24 kN/m^3
- **Steel:** 78.5 kN/m^3
- **Asphalt:** 23.0 kN/m^3
- **Compacted fill (backfill soil):** 20 kN/m^3
- **Natural ground:** 18 kN/m^3

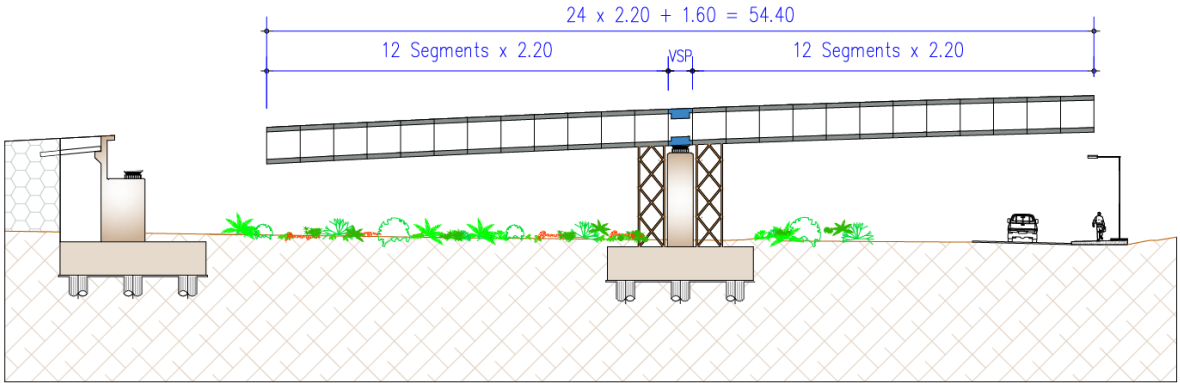
❖ Additional Permanent Loads CCP

Table IV.4. Estimation of Additional Permanent Loads.



Element	Density (kN/m ³)	Linear Load (kN/ml)	Load (kN)
1. Asphalt Pavement BB	24	14.80	3 581.60
2. Precast cornices	25	2.44	590.48
3. Safety Barrier	/	2×0.60	290.40
4. Sidewalk	25	16.53	4 000.26
5. Guardrail	/	2×0.50	242.00
Total Load	/	35.97	8 704.74

Table IV.5. Estimation of Total Weight of Segments.



N° Segment	L Segment (m)	Section (m ²)	Load (KN)
VSC1	1.20	13.40	402.08
	0.40	6.84	68.38
VC	1.60	6.84	273.51
V01	2.20	6.84	376.07
V02	2.20	6.84	376.07
V03	2.20	6.84	376.07
V04	2.20	6.84	376.07
V05	2.20	6.84	376.07
V06	2.20	6.84	376.07
V07	2.20	6.84	376.07
V08	2.20	6.84	376.07
V09	2.20	6.84	376.07
V10	2.20	6.84	376.07
V11	2.20	6.84	376.07
V12	2.20	6.84	376.07
1/2 VSP	0.60	13.40	201.04
	0.20	6.84	34.19
TOTAL			42 549.57

❖ Total Weight of the Deck

The total weight of the deck is calculated by summing:

Table IV.6. Load Summary.

	Linear load	Load
Segments	175.82	42 549.57
CCP	35.97	8 704.74
Total	211.79	51 254.31

IV.4.2. Evaluation of Traffic Loads

IV.4.2.1. Load Systems to Consider

According to **R.C.P.R**, the following live load systems are considered:

- Standard road traffic system – Type A(L)
- Standard road traffic system – Type B (B_c , B_t)
- Military load system – Mc 120
- Exceptional convoy – D240
- Overloads on sidewalks

IV.4.2.2. Bridge Characteristics

- ❖ **Carriageway Width:** The carriageway width is defined as the distance between the containment devices or curbs.
- ❖ **Loadable Width:** The carriageway width minus 0.5 m for each containment device.

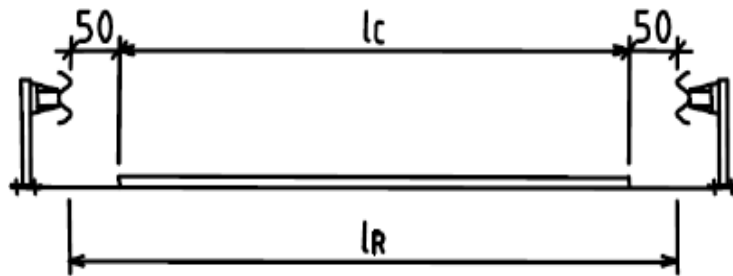


Figure IV.2. Carriageway and loadable width.

The characteristics of the bridge are summarized in the Table IV.7.

Table IV.7. Bridge Characteristics.

Carriageway Width	7.40
Loadable Width	6.40
Number of Lanes	2
Lane Width	3.20

- ❖ **Bridge Classes :** The classification of bridges according to loadable width is summarized in the following table:

Table IV.8. Road Bridge Classes.

Bridge Class	Description
1 st Class	$L_R \geq 7$
2 nd Class	$5.50 < L_R < 7$
3 rd Class	$L_R \leq 5.50$

Based on its Carriageway width, our bridge is classified as 1st Class according to RCPR standards.

IV.4.2.3. Load Cases

The different arrangements of live loads on the deck are illustrated in the figure below:

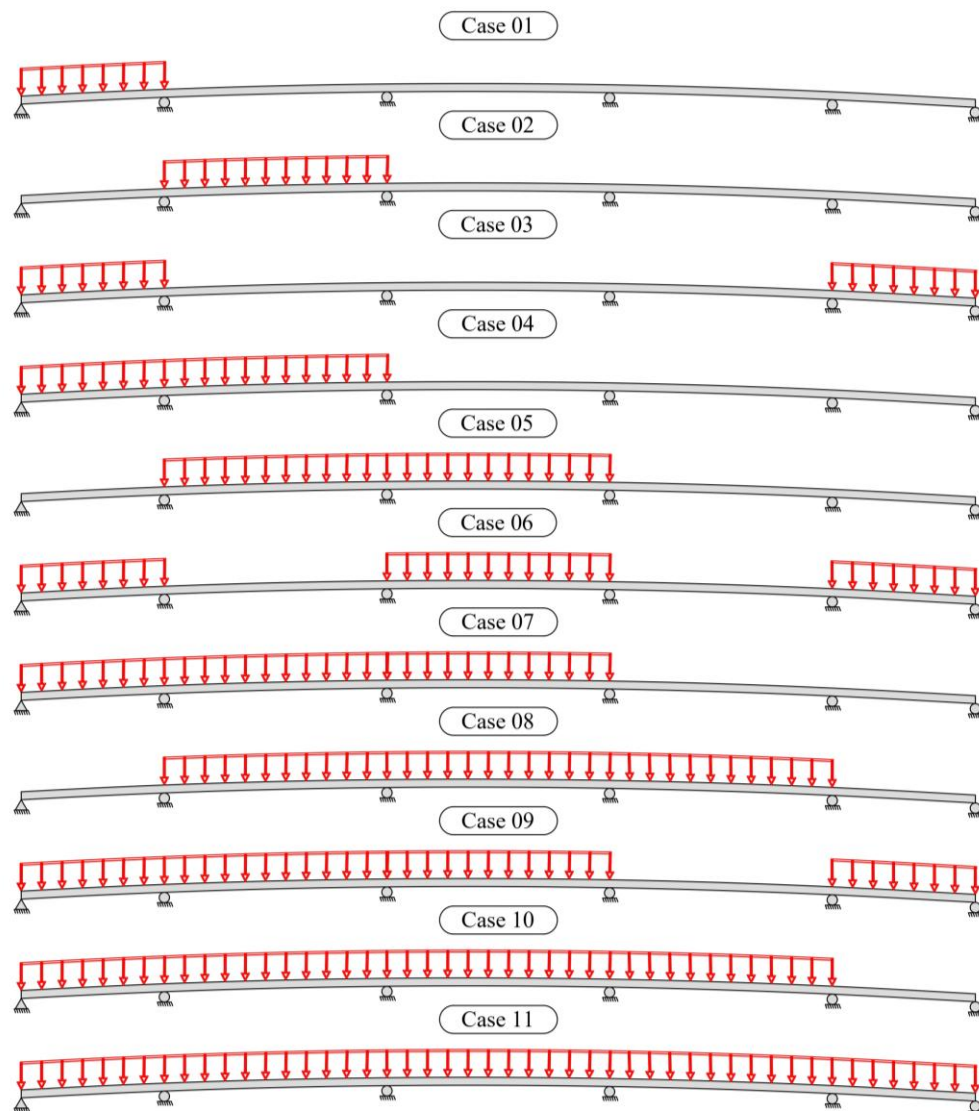


Figure IV.3. The 11 possible load cases.

IV.4.2.4. Load System A(L)

The Type A traffic load is calculated as:

$$A(L) = 2.30 + \frac{360}{12 + L}$$

The coefficient a_1 is defined as the maximum of the following two values:

$$A_1 = \max \left\{ \begin{array}{l} a_1 \cdot A(L) \\ 4 - 0.002 \cdot L \end{array} \right\}$$

L = loaded length of the deck (m)

Depending on the bridge class and the number of loaded lanes, the value of $A(L)$ is multiplied by the coefficient a_1 given in [Table IV.9.](#)

Table IV.9. Coefficient a_1 .

Bridge Class	Number of loaded lanes				
	1	2	3	4	5
1 st Class	1.00	1.00	0.90	0.75	0.75
2 nd Class	1.00	0.90	-	-	-
3 rd Class	0.90	0.80	-	-	-

The load A_1 is multiplied by a coefficient: $a_2 = \frac{v_0}{v}$, where v is the lane width and $v_0 = 3.50$ m. For this bridge $a_2 = 1.09375$.

The results are summarized in [Table IV.10.](#)

Table IV.10. Values of $A(L)$ for the 11 possible load cases.

Load Case	L (m)	$A_1 = A(L)$ (kN/m ²)	$A_2 = a_2 A_1$ (kN/m ²)	A(L) as a function of the number of loaded lanes (kN/m)	
				1 lane	2 lanes
Case 1	37	9.65	10.55	33.76	67.53
Case 2	56	7.59	8.31	26.58	53.16
Case 3	74	6.49	7.09	22.70	45.40
Case 4	93	5.73	6.27	20.05	40.10
Case 5	112	5.20	5.69	18.21	36.42
Case 6	130	4.84	5.29	16.92	33.85
Case 7	149	4.54	4.96	15.88	31.75
Case 8	168	4.30	4.70	15.05	30.10

Case 9	186	4.12	4.50	14.41	28.83
Case 10	205	3.96	4.33	13.86	27.71
Case 11	244	3.71	4.05	12.97	25.94

IV.4.2.5. Load System B

The B load system consists of three independent subsystems, which must be applied separately:

- **System Bc:** composed of standard trucks (300 kN)
- **System Bt:** composed of groups of two axles, known as “tandem axles”
- **System Br:** composed of a single isolated wheel (100 kN)

❖ Bc Load System / Article 4.5.1 (RCPR)

The Bc system consists of standard trucks (300 kN), as shown in Figure IV.3.

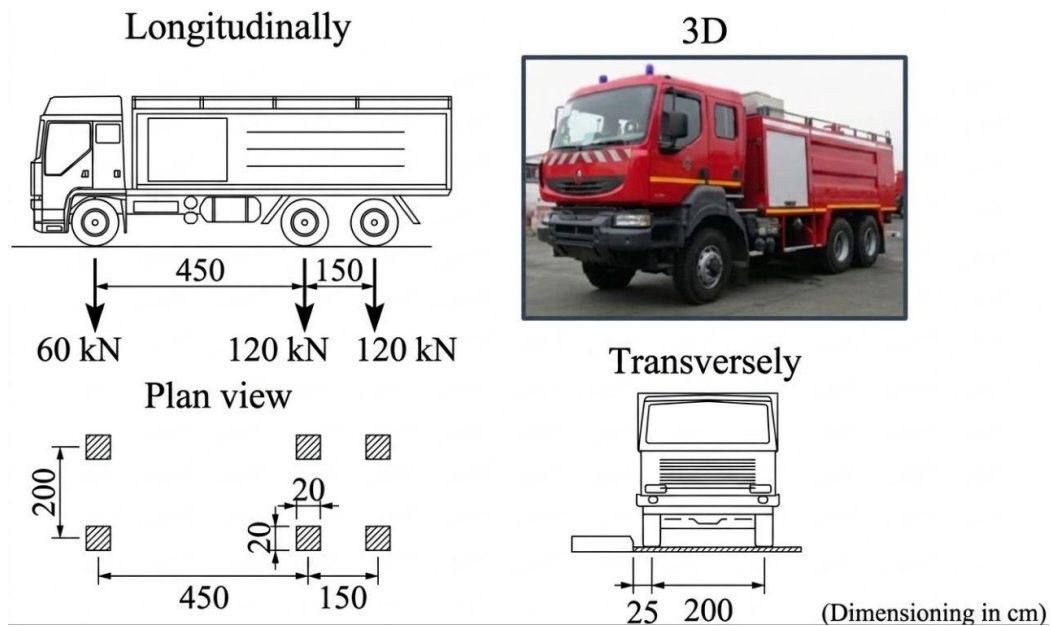


Figure IV.4. Bc loading system.

$$B_c = 2 \times (300 \times N)$$

N = number of lanes considered in the calculation

The load values of the B_c system are multiplied by a coefficient b_c , which depends on the bridge class and the number of traffic lanes considered. The values of b_c are presented in the table below

Table IV.11. Values of b_c .

Bridge Class	Number of loaded lanes				
	1	2	3	4	5
1 st Class	1.20	1.10	0.95	0.80	0.70
2 nd Class	1.00	1.00	-	-	-
3 rd Class	1.00	1.00	-	-	-

Therefore, the values to be considered are:

Table IV.12. Values of B_c .

Number of lanes	B_c (kN)	b_c	$B_c \times b_c$ (kN)
1	600	1.20	720
2	1 200	1.10	1 320

❖ **Bt Load System / Article 4.5.3 (RCPR)**

The Bt system consists of groups of two axles, referred to as tandem axles (160 kN).

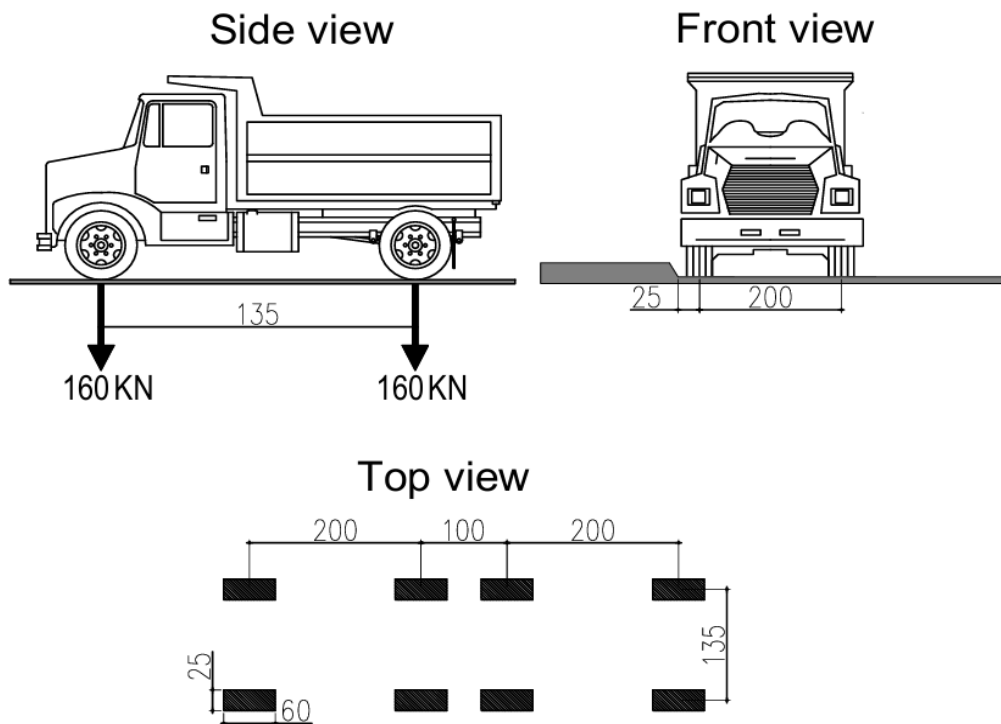


Figure IV.5. Bt loading system.

$$B_t = 2 \times (160 \times N)$$

N = number of lanes considered in the calculation

The B_t load values are adjusted by a coefficient b_t , depending on the bridge class.

Tableau IV.13. Values of b_t .

Bridge Class	Value
1 st Class	1.20
2 nd Class	1.00

The values to be considered are shown in the table below:

Table IV.14. Values of B_t

Number of lanes	B_t (kN)	B_t	$B_t \times b_t$ (kN)
1	320	1.20	384
2	640	1.20	768

❖ Br Load System / Article 4.5.2 (RCPR)

The Br system consists of a single isolated wheel (100 kN).

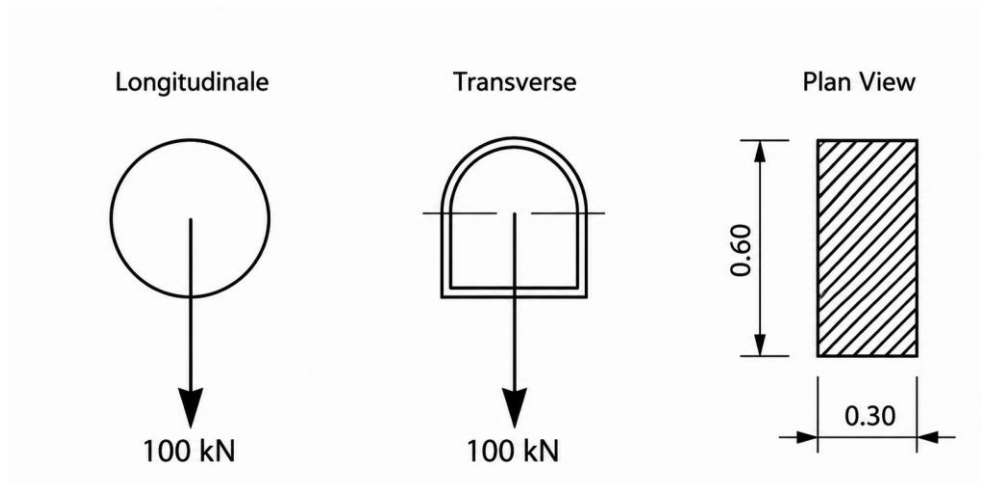


Figure IV.6. Br loading system.

IV.4.2.6. Military Loads – Mc120

- **Total weight:** 1 100 kN
- **Track length:** 6.10 m
- **Track width:** 1.00 m
- **Transverse direction:** only one convoy, regardless of the roadway width
- **Longitudinal direction:** the number of vehicles is limited, with a spacing of 30.5 m

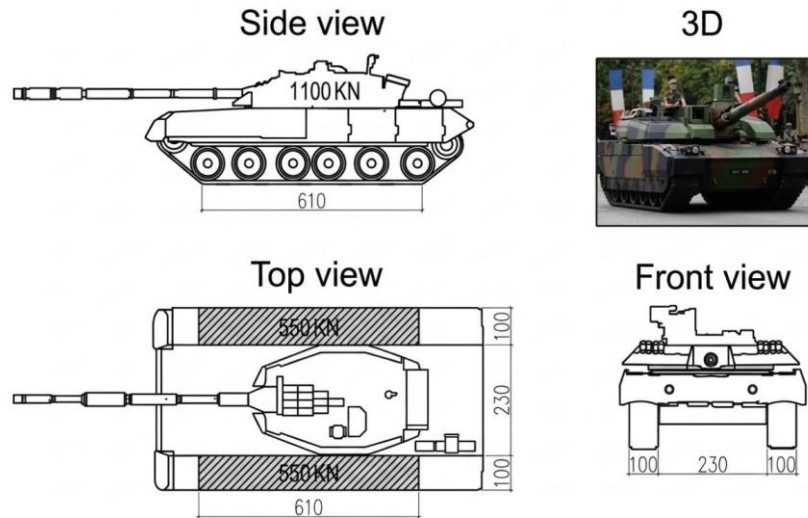


Figure IV.7. Mc120 load convoy.

IV.4.2.7. Exceptional Loads – D240

The D240 convoy consists of a trailer carrying 2 400 kN, with the weight assumed to be uniformly distributed. It travels alone, both longitudinally and transversely, regardless of the width or length of the bridge.

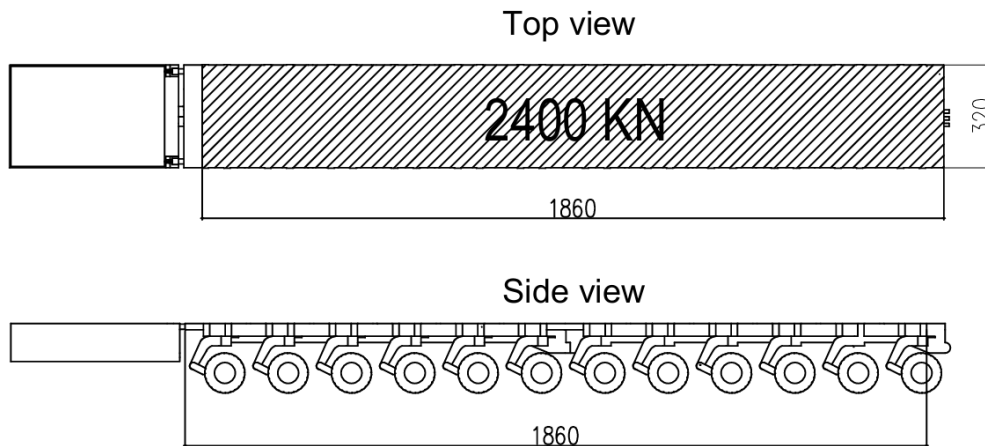


Figure IV.8. D240 exceptional convoy.

IV.4.2.8. Dynamic Amplification Coefficient

The B and Mc120 load systems are increased by a coefficient δ , known as the dynamic amplification coefficient. This coefficient allows converting a dynamic study into a pseudo-static analysis, where dynamic effects are replaced by static loads amplified by δ .

The D240 load system is not amplified because of its slow speed; its load is considered directly as a static load.

$$\delta = 1 + \frac{0.6}{1 + 4 \times \frac{G}{S}} + \frac{0.4}{1 + 0.2 \times L}$$

Where :

L= length of the element (m)

G= permanent load (self-weight + CCP)

S= maximum live load

The table below summarizes the values of the dynamic amplification coefficient δ :

Table IV.15. Dynamic Amplification Coefficients for the Deck.

S (KN)			1 320	768	100	1 100
Load Case	L (m)	G (KN)	B_c (KN)	B_t (KN)	B_r (KN)	M_c 120
Case 1	37	7 951.15	1.072	1.062	1.049	1.068
Case 2	56	11 784.00	1.049	1.042	1.034	1.046
Case 3	74	15 902.31	1.038	1.032	1.026	1.036
Case 4	93	19 735.15	1.030	1.026	1.021	1.029
Case 5	112	23 568.00	1.025	1.022	1.018	1.024
Case 6	130	27 686.31	1.022	1.019	1.015	1.021
Case 7	149	31 519.16	1.019	1.017	1.013	1.018
Case 8	168	35 352.00	1.017	1.015	1.012	1.016
Case 9	186	39 470.31	1.015	1.013	1.011	1.015
Case 10	205	43 303.16	1.014	1.012	1.010	1.013
Case 11	244	51 254.31	1.012	1.010	1.008	1.011

IV.4.3. Other Live Loads

IV.4.3.1. Braking Forces

The roadway loads from the A and Bc load systems can generate braking forces, which act on the surface of the roadway in either traffic direction. In typical cases, the resultant of these forces can be assumed to be centered along the longitudinal axis of the roadway.

❖ Bc Load System

The braking force that can be developed under Load System Bc is limited by the weight of a standard vehicle, taken as 300 kN.

❖ A Load System

The braking force corresponding to the A load system is calculated as:

$$F = \frac{A \times S}{20 + 0.0035 \times S}$$

Where :

S = loaded area in m²

F = braking force in KN

Table IV.16. Braking force F (kN) of the A load system on the deck.

Load Case	L (m)	1 Lane	2 Lanes
Case 1	37	61.20	119.96
Case 2	56	72.16	140.06
Case 3	74	80.65	155.13
Case 4	93	88.62	168.87
Case 5	112	95.96	181.23
Case 6	130	102.54	192.04
Case 7	149	109.17	202.72
Case 8	168	115.55	212.80
Case 9	186	121.40	221.87
Case 10	205	127.40	231.02
Case 11	244	139.23	248.58

IV.4.3.2. Sidewalk Loads

A uniform load of 1.5 kN/m² is applied on the sidewalks, reserved exclusively for pedestrian and bicycle traffic, in order to produce the maximum anticipated effect.

IV.4.4. Climatic Actions

IV.4.4.1. Thermal Action

❖ Uniform Temperature Variation:

According to RCPR, for a northern Algeria region (temperate climate), the uniform temperature variation ranges between +35 °C and –15 °C.

The linear thermal expansion is expressed as:

$$\Delta l = \alpha \times \Delta T \times L$$

Where :

α = thermal expansion coefficient ($1.1 \times 10^{-5} \text{ }^{\circ}\text{C}^{-1}$ for concrete and $1.22 \times 10^{-5} \text{ }^{\circ}\text{C}^{-1}$ for steel)

ΔT = temperature variation, with reference temperature $T_0 = 15 \text{ }^{\circ}\text{C}$

L = length of the structure (m)

❖ Thermal Gradient

According to the RCPR regulations, our structure is Type 3 (concrete deck). Therefore, the thermal gradient resulting from a short-duration, unilateral heating or cooling of the load-bearing structure during the service phase is taken as:

- **During the service phase:** $\Delta_t(\theta) = \pm 7 \text{ }^{\circ}\text{C}$.
- **During the construction phase:** $\Delta_t(\theta) = \pm 12 \text{ }^{\circ}\text{C}$.

Table IV.17. Thermal gradient.

	Construction phase	Service phase
Type 01 (steel deck)	$\pm 18^{\circ}$	$\pm 10^{\circ}$
Type 02 (composite deck)	$\pm 15^{\circ}$	$\pm 8^{\circ}$
Type 03 (concrete deck)	$\pm 12^{\circ}$	$\pm 7^{\circ}$

IV.4.4.2. Wind Action

The wind acts horizontally on the exposed surfaces of the deck and substructures. It is considered as a constant pressure of 2.00 kN/m², with a design wind variation of $\pm 7^{\circ}$.

IV.4.5. Accidental Actions

Forces acting on the structure in the event of vehicle derailment are considered eccentric by ± 1.5 m from the lane axis.

IV.4.6. Seismic Action

Seismic actions are considered in accordance with RPOA. The seismic zone classification for this structure is Zone IIa (medium seismicity).

IV.4.6.1. Zone Acceleration Coefficient

Based on seismic zone IIa and the importance of the bridge (Bridge Group 2): $A = 0.20$

IV.4.6.2. Site Classification:

According to the geotechnical study, the site is Class S3 (soft soil).

IV.4.6.3. Response Spectrum

❖ Horizontal Components

The elastic response spectrum (S_{ae}^H) for the two horizontal components is given as a function of the elastic period T and damping ratio ξ by:

$$S_{ae}^H(T, \xi) = \begin{cases} A \cdot g \cdot S \cdot \left[1 + \frac{T}{T_1} \cdot (2.5\eta - 1) \right] & 0 \leq T \leq T_1 \\ 2.5\eta \cdot A \cdot g \cdot S & T_1 < T \leq T_2 \\ 2.5\eta \cdot A \cdot g \cdot S \cdot \frac{T_2}{T} & T_2 < T \leq 3.0 \text{ s} \\ 2.5\eta \cdot A \cdot g \cdot S \cdot \left(\frac{3T_2}{T^2} \right) & T > 3.0 \text{ s} \end{cases}$$

Where :

A = acceleration coefficient 0.20

g = gravitational acceleration 9.81 m/s²

S = site coefficient 1.20

T_1, T_2 = site characteristic periods $T_1 = 0.20 \text{ s}$, $T_2 = 0.50 \text{ s}$

η = damping correction factor $\eta = \sqrt{7/(2 + \xi)}$

ξ = damping ratio 5%

T = fundamental period of the structure $T = \sqrt{(M/K)}$

M = total mass of the system

K = total stiffness of the system

From these parameters, the response spectrum curve for the structure is obtained, as shown below.

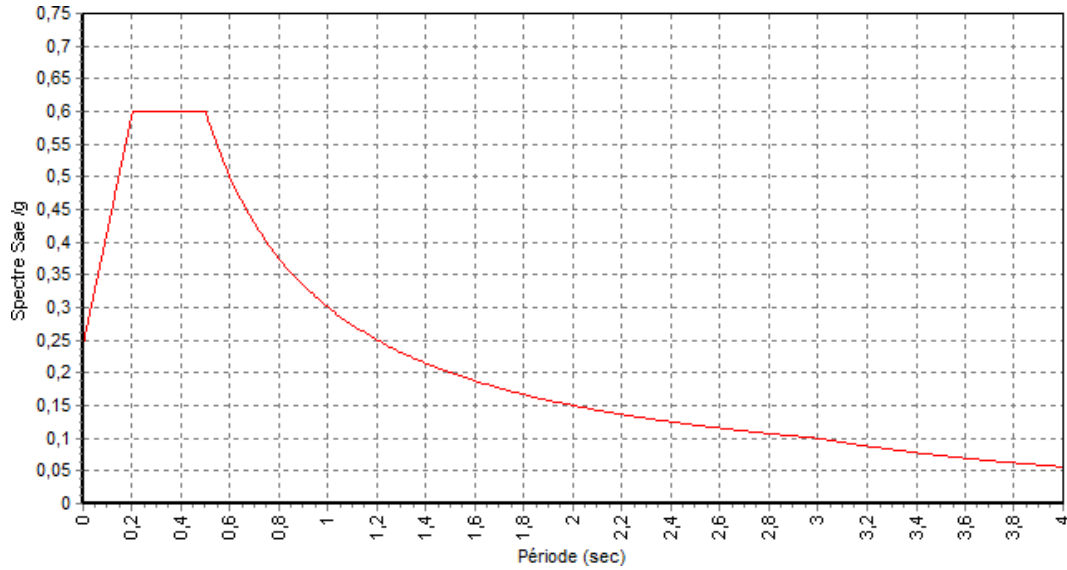


Figure IV.9. Elastic response spectrum : horizontal component.

❖ Vertical Component

The elastic response spectrum for the vertical component is expressed as a function of the elastic period T and the damping ratio ξ by:

$$S_{ae}^V(T, \xi) = \begin{cases} A \cdot g \cdot \alpha \cdot \left[1 + \frac{T}{T_1} \cdot (2.5\eta - 1) \right] & 0 \leq T \leq T_1 \\ 2.5\eta \cdot A \cdot g \cdot \alpha & T_1 < T \leq T_2 \\ 2.5\eta \cdot A \cdot g \cdot \alpha \cdot \frac{T_2}{T} & T_2 < T \leq 3.0 \text{ s} \\ 2.5\eta \cdot A \cdot g \cdot \alpha \cdot \left(\frac{3T_2}{T^2} \right) & T > 3.0 \text{ s} \end{cases}$$

Where :

α = coefficient accounting for the importance of the vertical component in high-seismicity zones $\alpha = 0.7$

T_1, T_2 = characteristic periods associated with the site $T_1 = 0.20 \text{ s}$, $T_2 = 0.40 \text{ s}$

A = zone acceleration coefficient

g = gravitational acceleration 9.81 m/s^2

η = damping correction factor

ξ = damping ratio

The response spectrum curve for the vertical component is obtained as shown below:

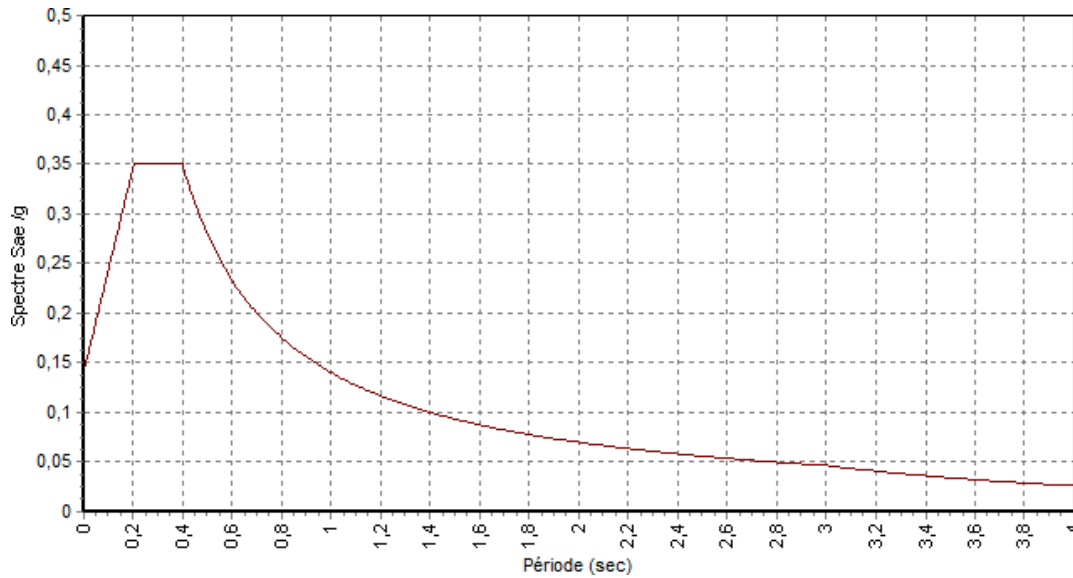


Figure IV.10. Elastic response spectrum : vertical component.

IV.5. LOAD COMBINATIONS

IV.5.1. Combination of Seismic Components

According to RPOA, the effects of the different components of the overall seismic motion are combined as follows:

$$\begin{aligned} E_1 &= E_x \pm 0.3E_y \pm 0.3E_z \\ E_2 &= E_y \pm 0.3E_x \pm 0.3E_z \\ E_3 &= E_z \pm 0.3E_x \pm 0.3E_y \end{aligned}$$

where E_x , E_y , and E_z are the seismic effects in the X, Y, and Z directions, respectively. The effect of the vertical seismic component on piers can generally be neglected in low-seismicity zones.

IV.6. CONCLUSION

The loads evaluated in this chapter will be input into MIDAS Civil 2022 to determine the maximum internal forces, which form the basis for the structural design addressed in the following chapter.

CHAPTER V

LONGITUDINAL STUDY

V.1. INTRODUCTION

The construction of bridges using the balanced cantilever method consists of building the deck from the piers by casting segments, either in prefabrication workshops or directly in formworks supported by movable equipment. These segments are connected by prestressing tendons and installed symmetrically at the ends of the already constructed part of the deck. When the two advancing ends reach the vicinity of the closure joint between adjacent spans on either side of a pier, the structure is referred to as a cantilever. A key characteristic of this type of bridge is that it must be analyzed during both the construction phase and the service phase. In this chapter, the calculations corresponding to these two phases are carried out.

V.2. STABILITY DURING THE CONSTRUCTION PHASE

V.2.1. Different Phases of Cantilever Construction

The different construction phases of a cantilever starting from a pier are illustrated in the figure below:

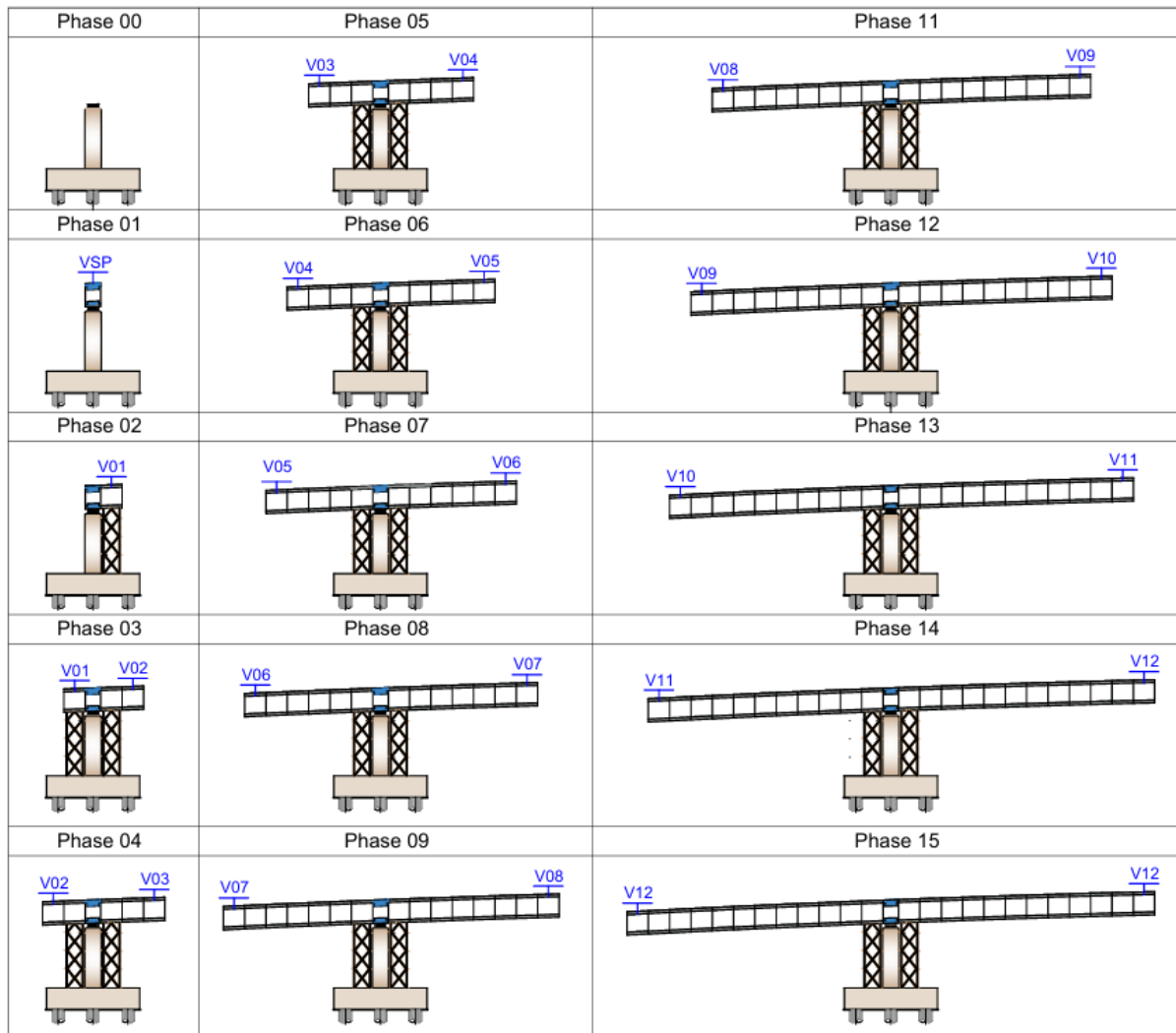


Figure V.1. Sequence for constructing a balanced cantilever.

V.2.2. Loads to Be Considered

The loads considered for the verification of a cantilever are defined according to SETRA guidelines. These include the self-weight, random construction loads, and accidental actions.

V.2.2.1. Self-Weight of the Cantilever

The self-weight corresponds to the weight of the cantilever, determined from the cross-section using a unit weight of 25 kN/m³.

To obtain the most unfavorable loading condition, one half-cantilever is increased by 2% to define the maximum value, while the other is reduced by 2% to define the minimum value:

$$G_{max} = 1.02 G$$

$$G_{min} = 0.98 G$$

Table V.1. Evaluation of Self-Weight of Each Half-Cantilever.

Phase	Half-Cantilever (Reduced)		Half-Cantilever (Increased)	
	G (kN)	G _{min} (kN)	G (kN)	G _{max} (kN)
Phase 1	235.23	230.52	235.23	239.93
Phase 2	235.23	230.52	611.30	623.53
Phase 3	611.30	599.07	987.37	1 007.12
Phase 4	987.37	967.63	1 363.44	1 390.71
Phase 5	1 363.44	1 336.17	1 739.51	1 774.30
Phase 6	1 739.51	1 704.72	2 115.59	2 157.90
Phase 7	2 115.59	2 073.28	2 491.66	2 541.49
Phase 8	2 491.66	2 441.83	2 876.73	2 925.08
Phase 9	2 876.73	2 819.20	3 243.80	3 308.68
Phase 10	3 243.80	3 178.92	3 619.87	3 692.27
Phase 11	3 619.87	3 547.47	3 995.94	4 075.86
Phase 12	3 995.94	3 916.02	4 372.02	4 459.46
Phase 13	4 372.02	4 284.58	4 748.09	4 843.05
Phase 14	4 748.09	4 653.13	4 748.09	4 843.05

V.2.2.2. Random Construction Loads

Random construction loads correspond to personnel and materials placed on the cantilever during construction. These loads are represented by two actions:

❖ Uniformly distributed load (Q_{PRA1}):

Applied on the increased half-cantilever, $Q_{PRA1} = 0.2 \text{ kN/m}^2$. For a deck width of $B = 10.4 \text{ m}$:

$$Q_{PRA1} = 0.2 \times 10.4 = 2.08 \text{ kN/m}$$

❖ Concentrated load (Q_{PRA2}):

Applied at the tip of the cantilever on the last completed segment, representing the weight of rollers, compressors, and small equipment.

$$Q_{PRA2} = 50 + 5B = 50 + 5 \times 10.4 = 102 \text{ kN}$$

❖ Vertical Wind Effect

The additional vertical effect of wind, Q , is represented as a uniformly distributed load with an intensity of 100 to 200 N/m, depending on the site characteristics. This distributed load is applied vertically, from the bottom to the top, on the main horizontal truss of a half-cantilever.

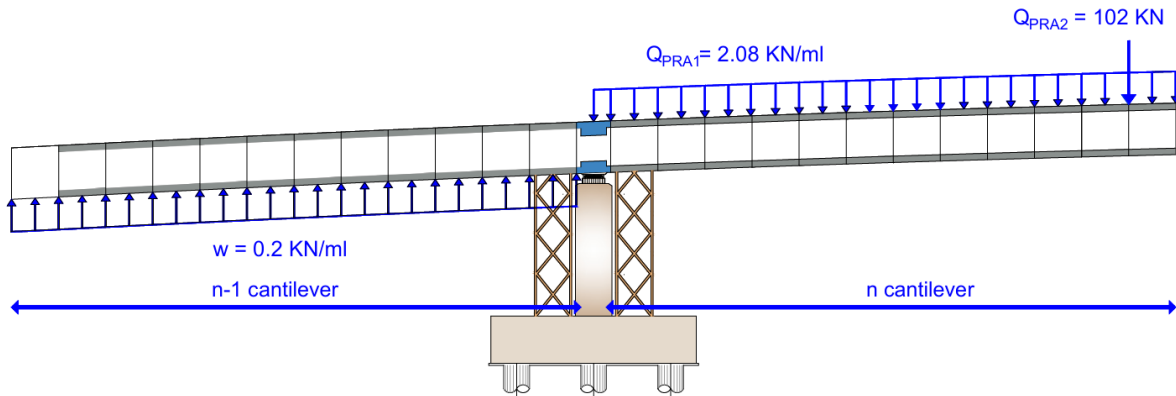


Figure V.2. Temporary construction situation to be considered.

V.2.2.3. Accidental Actions

The accidental action considered is the fall of a precast segment during placement. A dynamic amplification factor of 2 is applied.

To represent the most unfavorable scenario, it is assumed that the fall occurs during the placement of the last pair of segments:

$$F_a = G_{\text{segment}} \times 2 = -752.14 \text{ kN}$$

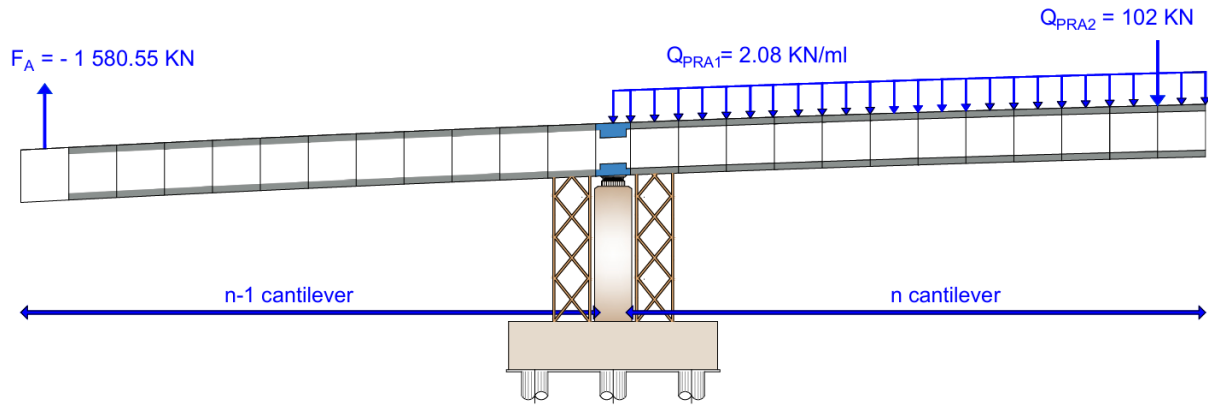


Figure V.3. Accidental situation to be considered.

V.2.3. Load Combinations

The critical phase is considered to be the placement of the last pair of segments. It is assumed that both cantilever arms have been advanced, with one segment already positioned while the other is yet to be placed.

According to the SETRA design guidelines, two types of load combinations must be considered during construction:

- ❖ **Fundamental Combination (Type A):** Verification with respect to the ultimate limit state of static equilibrium. The cantilever must not lift off its bearings.
- ❖ **Accidental Combination (Type B):** Verification with respect to the ultimate limit state under accidental loads. In this case, the cantilever may lift off its bearings, but structural safety must be ensured.

Table V.2. Summary of Load Combinations.

Type of combination		G_{max}	G_{min}	Q_{PRA1}	Q_{PRA2}	Q_W	F_a
Type A	A1	1.10	1.10	1.25	1.25	1.25	/
	A2	0.90	0.90	1.25	1.25	1.25	/
Type B	B1	1.10	1.10	1.00	1.00	/	1.00
	B2	0.90	0.90	1.00	1.00	/	1.00

V.2.4. Evaluation of Internal Forces

For the evaluation of internal forces, a finite element software model is used. In this model, the deck is considered to act as a bi-cantilever.

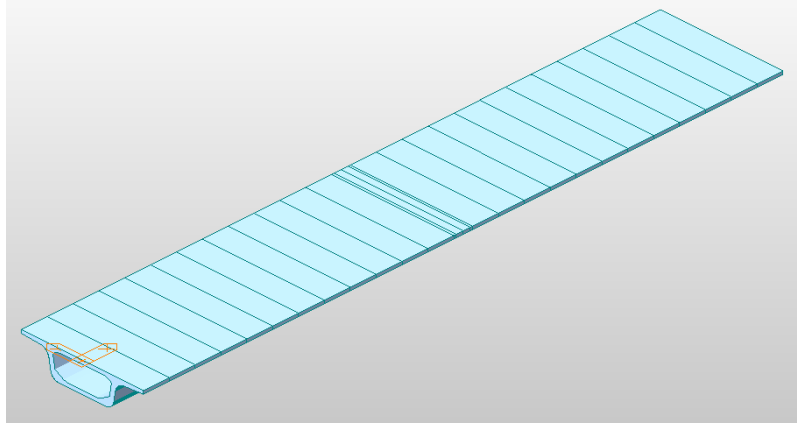


Figure V.4. Calculation model for the balanced cantilever.

The 12th construction phase produces the maximum internal forces. The results are summarized in the tables below:

Table V.3. Evaluation of forces in temporary and accidental situations.

Phase	Combination A1		Combination A2		Combination B1		Combination B2	
	M (MN.m)	T (MN)	M (MN.m)	T (MN)	M (MN.m)	T (MN)	M (MN.m)	T (MN)
Axis VSP	76.60	5.53	63.43	4.56	75.77	5.49	62.60	4.52
End VSP	72.24	5.37	59.83	4.43	71.44	5.33	59.04	4.39
End V01	60.91	4.93	50.49	4.07	60.20	4.89	49.78	4.03
End V02	50.55	4.49	41.93	3.71	49.92	4.46	41.30	3.67
End V03	41.15	4.06	34.16	3.35	40.59	4.02	33.61	3.32
End V04	32.70	3.62	27.18	2.99	32.22	3.58	26.71	2.96
End V05	25.22	3.18	20.99	2.64	24.82	3.15	20.59	2.60
End V06	18.69	2.75	15.59	2.28	18.37	2.71	15.26	2.24
End V07	13.13	2.31	10.97	1.92	12.87	2.28	10.72	1.89
End V08	8.53	1.87	7.15	1.56	8.34	1.84	6.96	1.53
End V09	4.88	1.44	4.11	1.20	4.76	1.41	3.98	1.17
End V10	2.20	1.00	1.86	0.84	2.14	0.97	1.80	0.82
End V11	0.48	0.56	0.39	0.49	0.48	0.54	0.39	0.46
End V12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

V.3. EVALUATION OF INTERNAL FORCES DURING SERVICE

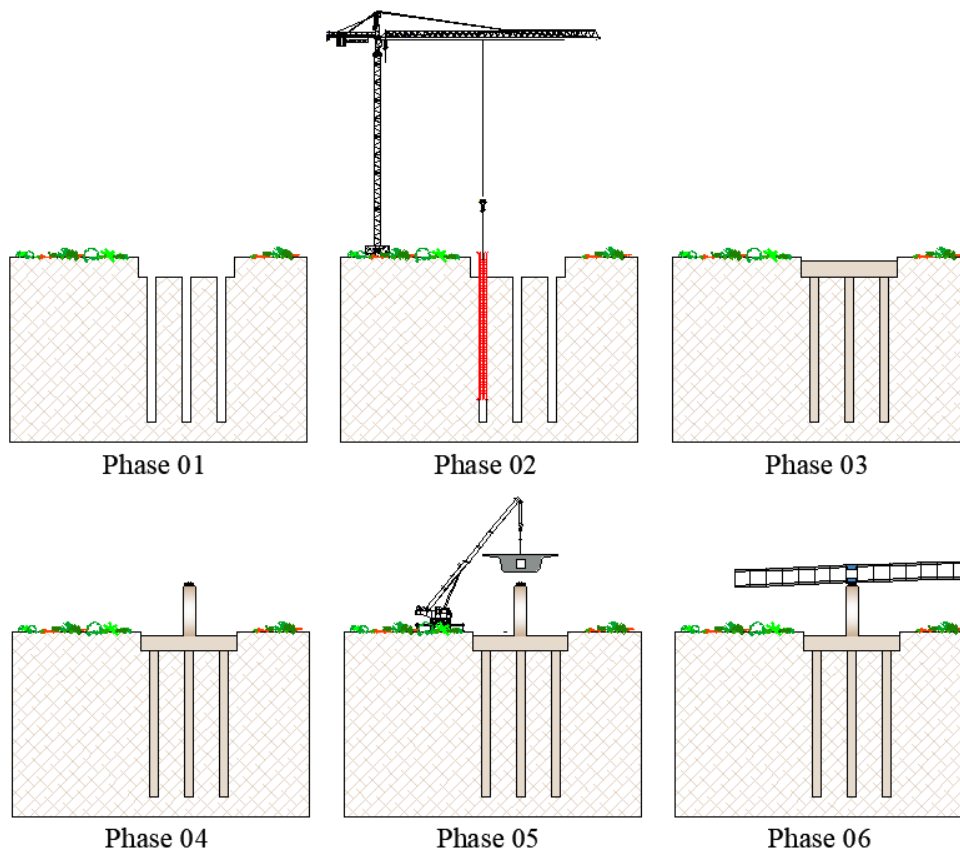
A key feature of balanced cantilever bridges is that they must be analyzed both during construction and in the service phase.

In this chapter, the calculations are carried out for the service phase, when the bridge is in use and all traffic loads are applied. The analysis is performed using MIDAS Civil 2022 structural analysis software.

V.3.1. Phases of Deck Construction

The construction phases of the deck are sequenced as illustrated in the figures below:

- Phase 1:** Excavation and pile drilling
- Phase 2:** Placement of reinforcement and concreting of the piles
- Phase 3:** Construction of the pile footing
- Phase 4:** Construction of the pier
- Phase 5:** Installation of the first segment on the pier
- Phase 6:** Arrangement of the segments
- Phase 7:** Construction of the first cantilever
- Phase 8:** Construction of the second cantilever
- Phase 9:** Construction of the third cantilever and installation of the closing joint between pier 1 and 2
- Phase 10:** Construction of the fourth cantilever and installation of the closing joint between pier 2 and 3
- Phase 11:** Installation of the closing joint between pier 3 and 4, and arrangement of the last edge segments
- Phase 12:** Installation of the edge closing joints



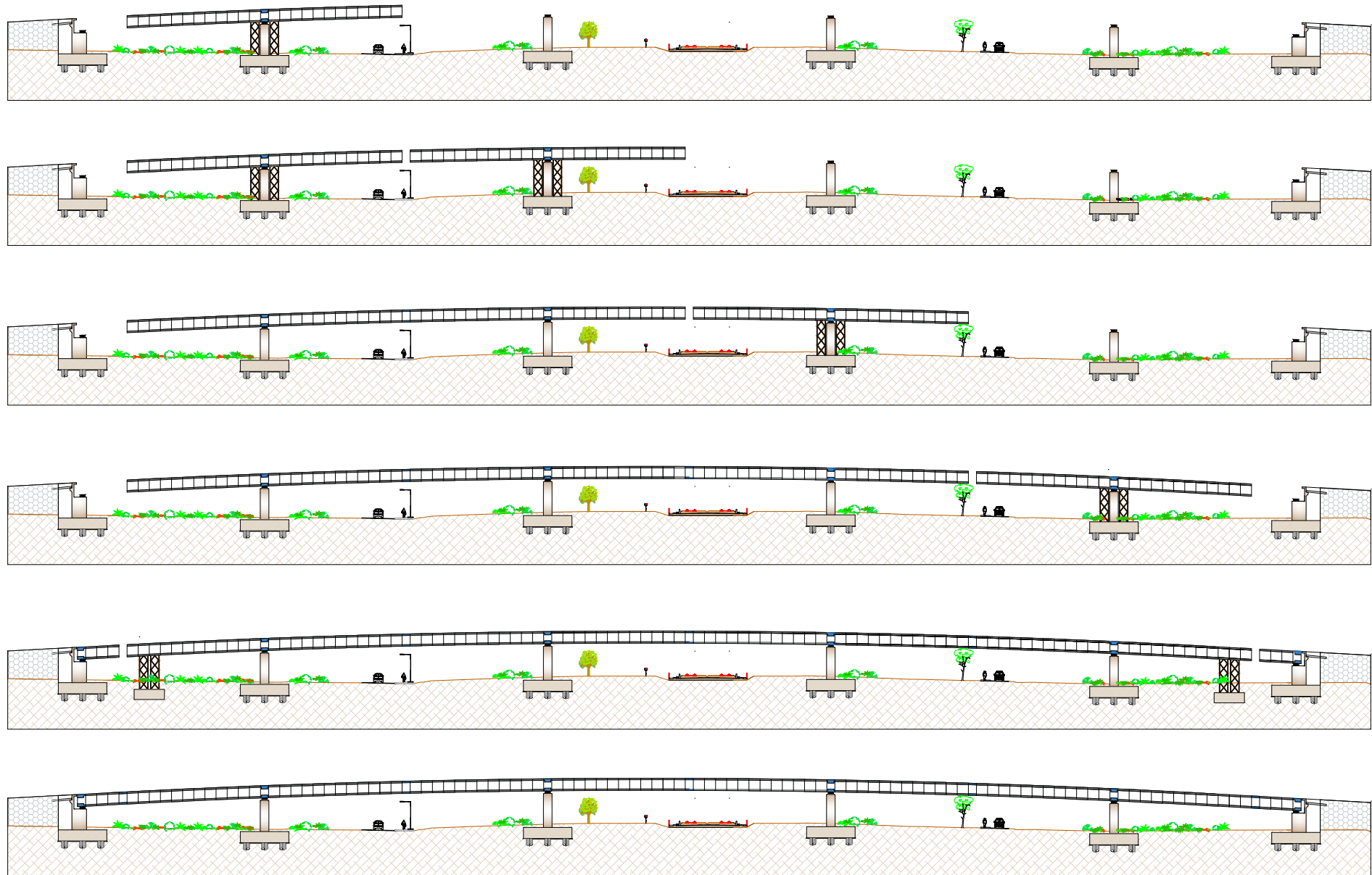


Figure V.5. Bridge construction sequence.

V.3.2. Modeling

Modeling is a critical part of any structural study. Regardless of the complexity of the model, the objective is to develop a model capable of accurately representing the behavior of the structure under different conditions.

For this project, the modeling is performed as follows:

- The structure is defined using beam elements.
- Sections and materials have already been defined in previous chapters.
- Supports are considered as simple supports, except for one at abutment C1, which is modeled as a double support.
- Loads and load combinations are defined according to RCPR 2008, as described in Chapter IV

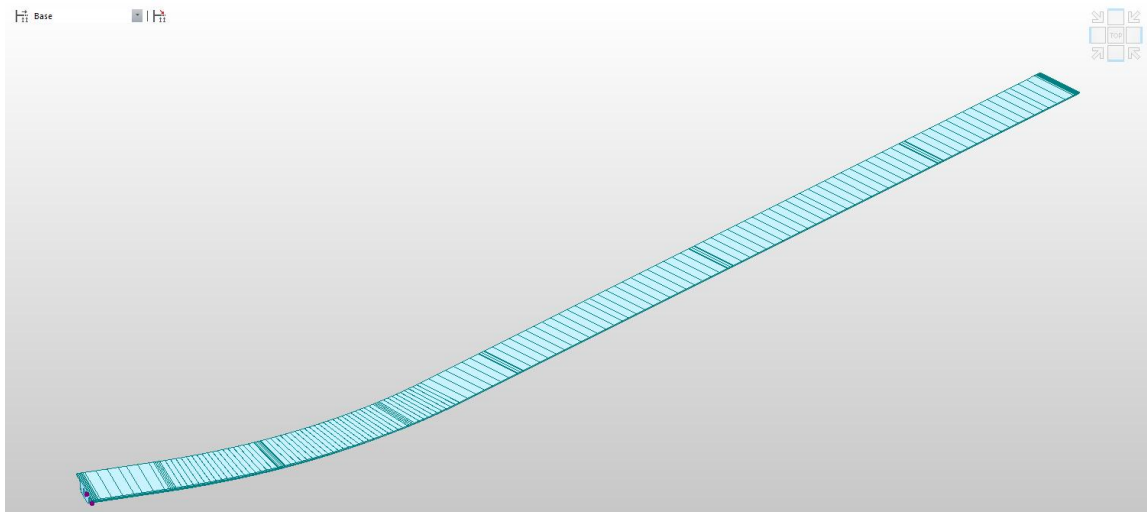


Figure V.6. Analysis model.

V.3.3. Load Combinations

According to RCPR 2008, the following load combinations must be considered for the service phase using MIDAS Civil 2022 VERSION :

Table V.4. Load Combinations During Service Phase.

Combination		G	A(L)	Bc	Bt	Mc120	D240	St
ULS	COMB 1	1.35	1.60	0.00	0.00	0.00	0.00	1.60
	COMB 2	1.35	0.00	1.60	0.00	0.00	0.00	1.60
	COMB 3	1.35	0.00	0.00	1.60	0.00	0.00	1.60
	COMB 4	1.35	0.00	0.00	0.00	1.35	0.00	0.00
	COMB 5	1.35	0.00	0.00	0.00	0.00	1.35	0.00

SLS	COMB 1	1.00	1.20	0.00	0.00	0.00	0.00	1.00
	COMB 2	1.00	0.00	1.20	0.00	0.00	0.00	1.00
	COMB 3	1.00	0.00	0.00	1.20	0.00	0.00	1.00
	COMB 4	1.00	0.00	0.00	0.00	1.00	0.00	0.00
	COMB 5	1.00	0.00	0.00	0.00	0.00	1.00	0.00

V.3.4. Results

The structural analysis software uses the finite element method and elasticity theory to determine the internal forces (M, N, and T) and the deformations of the structural elements.

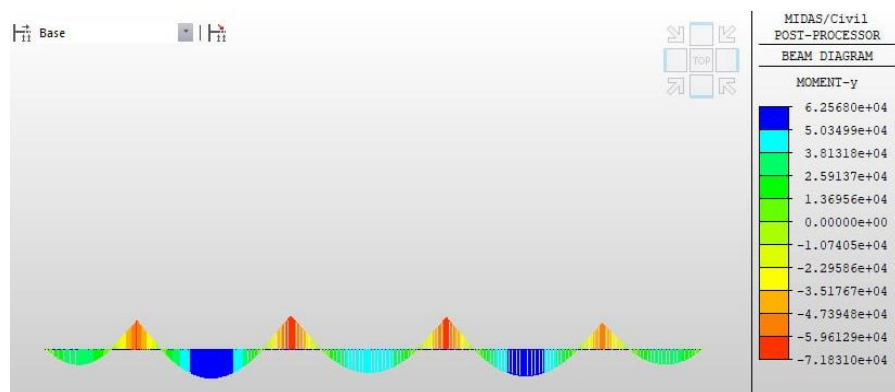


Figure V.7. Bending moment at ULS (envelope).

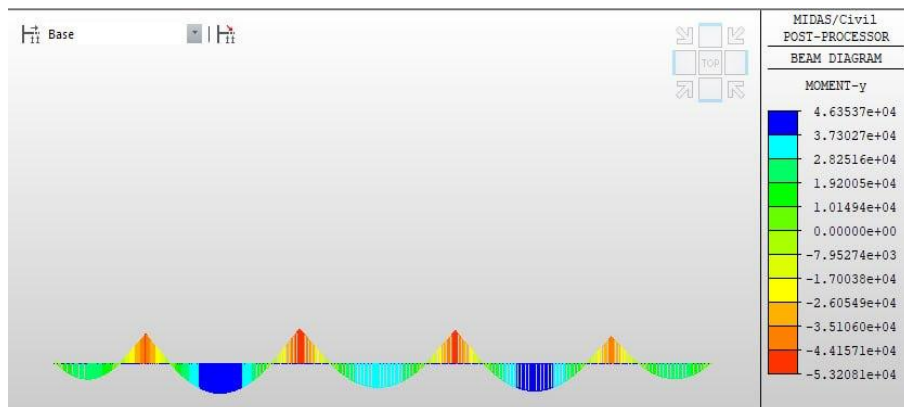


Figure V.8. Bending moment at SLS (envelope).

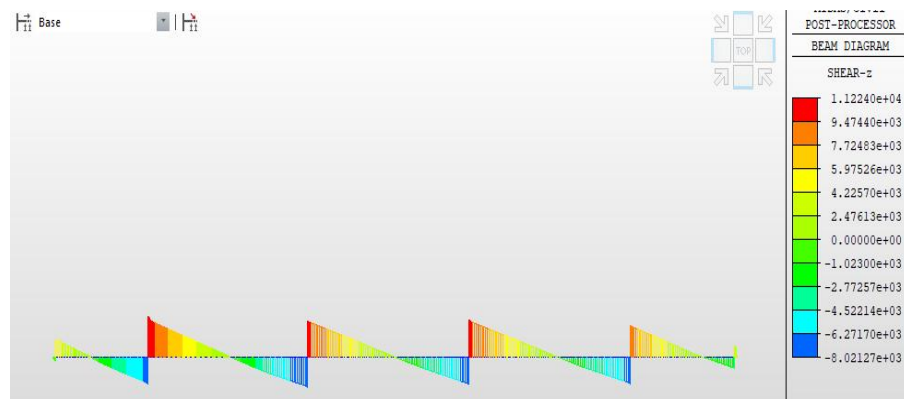


Figure V.9. Shear force at ULS (envelope).

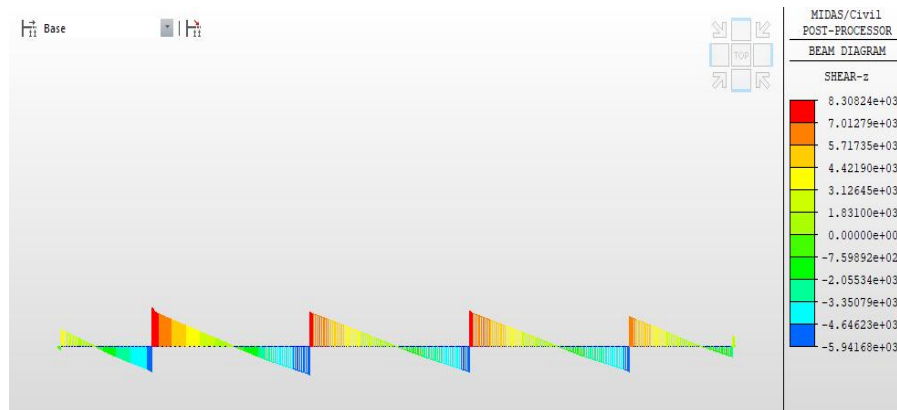


Figure V.10. Shear force at SLS (envelope).

V.4. CONCLUSION

In this chapter, we have seen that the study of a cantilever bridge deck built with precast segments is divided into two phases.

The first phase occurs before the cantilevers are completed, where the stability analysis is mandatory. In this phase, the mechanical system is isostatic, and the maximum moment occurs at the supports on the piers.

The second phase occurs after the cantilever segments are joined and the continuity tendons are tensioned, at which point the structural system becomes hyperstatic. The behavior of the continuity tendons will be analyzed in the following chapter.

CHAPTER VI

PRESTRESSING STUDY

VI.1. INTRODUCTION

The application of prestressing in cantilever bridges is carried out mainly in two stages:

- ❖ **Cantilever Stage:** The cantilevers are maintained in equilibrium on their piers, and the segments are joined together by horizontal prestressing (cantilever tendons).
- ❖ **Continuity Stage:** The cantilevers are connected to each other and to the end spans, which are constructed on falsework, using continuity prestressing.

These stages are explained in more detail below, starting with the determination of the required number of tendons, followed by the calculation of tendon losses, and ending with the verification of concrete stresses.

VI.2. CANTILEVER PRESTRESSING STUDY

Cantilever tendons are placed near the top flange of the box girder and are tensioned symmetrically relative to the pier axis as construction progresses. Their purposes are:

- To assemble the successive segments during construction and resist negative moments due to the self-weight of the cantilevers and construction loads.
- To contribute, together with continuity tendons, to resisting negative moments caused by permanent and live loads during service.

VI.2.1. Determination of the Number of Cantilever tendons

The number of cantilever tendons is determined by the following relation:

$$n \geq \frac{P}{(1 - \Delta P) \cdot P_0}$$

Where :

P = prestressing force

n = number of tendons

P_0 = limit prestressing force provided by a 15T15 tendon, class 1 860

ΔP = estimated losses, 25%

- ❖ Evaluation of P_0 :

Initial stress:

$$\sigma_{p0} = \min(0.8f_{prg}, 0.9f_{peg}) = 1\,488 \text{ MPa}$$

$$P_0 = \sigma_{p0} \times S_{\text{tendon}}, S = 2\,100 \text{ mm}^2 \Rightarrow P_0 = 3.12 \text{ MN}$$

- ❖ Evaluation of P :

$$P = \frac{\frac{M \cdot v}{I}}{\frac{1}{S} + \frac{e \cdot v}{I}}, e = v - d$$

Where :

v= distance from the centroid of the section to the top fiber

I= longitudinal moment of inertia of the section

e= tendon eccentricity relative to the centroid

M= maximum moment due to self-weight and live loads

S= cross-section of the segment at the pier axis

Numerical Application :

$M = 76.60 \text{ MN}\cdot\text{m}$ (Load Combination A1 – Axis VSP, see [Chapter V](#))

$d = 0.15 \text{ m}$, $I = 7.35 \text{ m}^4$, $S = 13.40 \text{ m}^2$, $v = 1.04 \text{ m}$

$P_0 = 3.12 \text{ MN}$, $\Delta P = 25\%$

$P = 54.04 \text{ MN} \Rightarrow n \geq 23.06 \Rightarrow n = 24$ tendons of 15T15

The number of tendons required for each segment is summarized in the table below.

Table VI.1. Estimation of the number of cantilever tendons.

Position	x (m)	M (MN.m)	H (m)	v (m)	I (m ⁴)	S (m ²)	e (m)	P (MN)	N Tendons	Value
Axis VSP	0	76.60	2.40	1.04	7.35	13.40	0.89	54.04	23.06	24
End VSP	0.80	72.24	2.40	0.94	5.46	6.84	0.79	44.07	18.81	24
End V01	3.00	60.91	2.40	0.94	5.46	6.84	0.79	37.16	15.86	24
End V02	5.20	50.55	2.40	0.94	5.46	6.84	0.79	30.84	13.16	22
End V03	7.40	41.15	2.40	0.94	5.46	6.84	0.79	25.10	10.71	20
End V04	9.60	32.70	2.40	0.94	5.46	6.84	0.79	19.95	8.51	18
End V05	11.80	25.22	2.40	0.94	5.46	6.84	0.79	15.38	6.56	16
End V06	14.00	18.69	2.40	0.94	5.46	6.84	0.79	11.40	4.87	14
End V07	16.2	13.13	2.40	0.94	5.46	6.84	0.79	8.01	3.42	12
End V08	18.4	8.53	2.40	0.94	5.46	6.84	0.79	5.20	2.22	10
End V09	20.6	4.88	2.40	0.94	5.46	6.84	0.79	2.98	1.27	8
End V10	22.80	2.20	2.40	0.94	5.46	6.84	0.79	1.34	0.57	6
End V11	25.00	0.48	2.40	0.94	5.46	6.84	0.79	0.29	0.13	4
End V12	27.20	0.00	2.40	0.94	5.46	6.84	0.79	0.00	0.00	2

***NB** The number of prestressing tendons must be even and integer, as the box girder has two webs. tendons must be symmetrically placed to prevent torsional effects.*

VI.2.2. Tendon Layout and Concrete Cover

To ensure proper concrete placement through the construction funnel and avoid interaction between duct bundles during tensioning, minimum spacing and concrete cover must be respected:

- Horizontal spacing between tendon axes: $e_h \geq 2 \times \Phi_g \rightarrow e_h \geq 20 \text{ cm}$
- Vertical spacing between tendon axes: $e_v \geq 2 \times \Phi_g \rightarrow e_v \geq 20 \text{ cm}$
- Concrete cover: $C = \max(3/4a, \Phi, d) \rightarrow C \geq 10 \text{ cm}$

Where :

a = horizontal dimension of the rectangle enclosing the duct or bundle of ducts

Φ = duct diameter (100 mm)

d = 4 cm for typical structures

VI.2.3. Tendon Layout in Plan

In plan view, prestressing tendons run parallel to the longitudinal axis of the box girder. To anchor into the web axis, tendons follow an S-shaped path, though straight paths without plan variation are also possible.

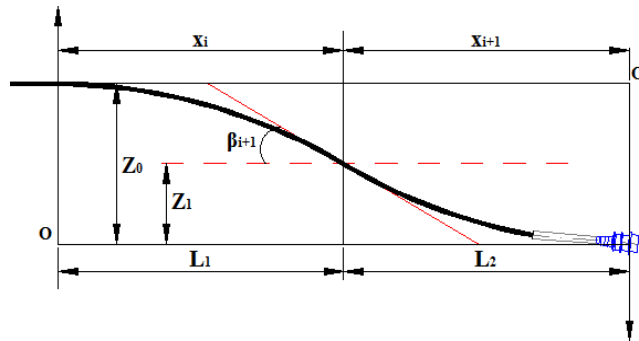


Figure VI.1. Tendons plan layout.

- Parabolic equation in plan: $Z(x) = Z_0 + (Z_1 - Z_0) \left(\frac{x}{l}\right)^2$
- Radius of curvature at a point: $R_i = \frac{(x_i)^2}{2(z_i - z_{i+1})}$, $R_{i+1} = \frac{(x_{i+1})^2}{2(z_{i+1} - z_{i+2})}$
- Slope in intermediate section: $\beta_{i+1} = \arctan\left(\frac{2(z_i - z_{i+1})}{x_i}\right)$
- Slope in anchorage section: $\beta_{i+1} = \arctan\left(\frac{2(z_{i+1})}{x_{i+1}}\right)$

Where :

Z_i, Z_{i+1} = vertical deviation of the tendon

L_i, L_{i+1} = length over which curvature occurs (length of a segment)

β = deviation angle

$R_{\min}$ = minimum radius of curvature, $R_{\min} > 3 \text{ m}$

For this study, both curves have the same shape but opposite directions, implying $x_i = x_{i+1}$.

VI.2.4. Tendon Layout in Elevation

Tendon paths must satisfy the following:

- Avoid tendon crossing, especially at the webs
- tendons follow a straight path up to the last segment, then a parabolic deviation occurs

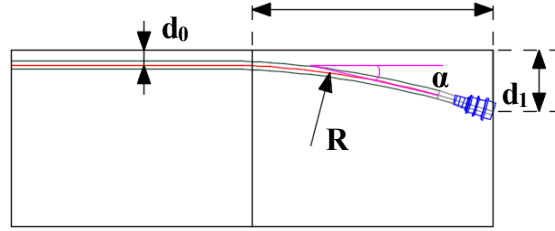


Figure VI.2. Tendons elevation layout.

- Parabolic equation in elevation: $y(x) = d_0 + (d_1 - d_0) \left(\frac{x}{l}\right)^2$
- Radius of curvature at a point: $R(x) = \frac{x^2}{2(d_1 - d_0)} \geq R_{\min}$
- Deviation angle: $\alpha(x) = \arctan \left(\frac{2(d_1 - d_0)x}{l^2} \right)$

Where :

d_0 = distance from tendon centroid to top fiber

d_1 = distance from anchorage to top fiber

l = length over which curvature occurs

$R_{\min} = 3 \text{ m}$

Equations and parameters are detailed in Appendix C.

VI.3. CONTINUITY PRESTRESSING STUDY

Continuity prestressing consists of tendons tensioned after keying to resist the hyperstatic moments generated in the structure, as well as positive bending moments caused by service loads.

VI.3.1. Determination of Continuity tendons in the Main Span

In the intermediate span, continuity tendons are intended to resist maximum positive bending moments. The required number of tendons is calculated based on the maximum positive moment obtained from a finite element analysis using the following relation:

$$n \geq \frac{P}{0.75 \times P_0}, P = \frac{\frac{M \cdot v'}{I}}{\frac{1}{S} + \frac{e \cdot v'}{I}}, e = v' - d$$

Where :

M = bending moment from the most unfavorable load combination

v' = distance from the centroid of the section to the top fiber

d = distance from the section centroid to the tendon anchor

I = moment of inertia of the section

S = section area of the segment

ΔP = estimated prestress losses (25%)

P_0 = ultimate prestressing force provided by a single tendon

The number of continuity tendons for the main span (P1 – P2 – P3– P4) is summarized in Table VI.2. An example calculation for the most stressed segment V12 is given below:

Numerical Example :

$M = 46.36 \text{ MN}\cdot\text{m}$ (COMB-SLS: Envelope, see Chapter V)

$d = 0.15 \text{ m}$; $I = 5.46 \text{ m}^4$; $S = 6.84 \text{ m}^2$; $v' = 1.46 \text{ m}$

$P_0 = 3.12 \text{ MN}$; $\Delta P = 24.12 \%$ $\Rightarrow P = 24.93 \text{ MN}$

$n \geq 10.51 \Rightarrow \text{adopt } n = 12 \text{ tendons of } 15\text{T}15$

Table VI.2. Estimation of the number of continuity tendons for the main span.

Section	M (MN.m)	v' (m)	v (m)	I (m ⁴)	S (m ²)	e (m)	P (MN)	N Tendons	Value
End V03	0.00	1.46	0.94	5.46	6.84	1.31	0.00	0.00	2
End V04	3.06	1.46	0.94	5.46	6.84	1.31	1.65	0.69	2
End V05	12.70	1.46	0.94	5.46	6.84	1.31	6.84	2.88	4
End V06	21.16	1.46	0.94	5.46	6.84	1.31	11.40	4.81	6
End V07	28.41	1.46	0.94	5.46	6.84	1.31	15.30	6.45	8
End V08	34.42	1.46	0.94	5.46	6.84	1.31	18.54	7.82	10
End V09	39.23	1.46	0.94	5.46	6.84	1.31	21.13	8.91	10
End V10	42.93	1.46	0.94	5.46	6.84	1.31	23.12	9.75	12
End V11	45.28	1.46	0.94	5.46	6.84	1.31	24.39	10.28	12
End V12	46.36	1.46	0.94	5.46	6.84	1.31	24.93	10.51	12
End V12	45.95	1.46	0.94	5.46	6.84	1.31	24.75	10.44	12
End V11	44.04	1.46	0.94	5.46	6.84	1.31	23.72	10.00	12
End V10	41.12	1.46	0.94	5.46	6.84	1.31	22.14	9.34	12

End V09	36.98	1.46	0.94	5.46	6.84	1.31	19.92	8.40	10
End V08	31.55	1.46	0.94	5.46	6.84	1.31	16.99	7.16	8
End V07	24.85	1.46	0.94	5.46	6.84	1.31	13.38	5.64	6
End V06	16.97	1.46	0.94	5.46	6.84	1.31	9.14	3.85	4
End V05	7.95	1.46	0.94	5.46	6.84	1.31	4.28	1.80	2
End V04	0.00	1.46	0.94	5.46	6.84	1.31	0.00	0.00	2

VI.3.2. Layout of Continuity tendons in the main span

After determining the required number of tendons for each segment, their layout is illustrated in three views: plan view, elevation, and cross-section.

The equations and parameters governing the tendon paths are provided in [Appendix C](#).

VI.3.3. Determination of Continuity tendons in the End Span

The continuity tendons in the end spans have the same function and are calculated using the same formula as for the main span tendons. The required number of tendons for the end span is summarized in [Table VI.3](#).

Numerical Example :

$M = 24.47 \text{ MN}\cdot\text{m}$ (COMB-SLS: Envelope, see [Chapter V](#))

$d = 0.15 \text{ m}$; $I = 5.46 \text{ m}^4$; $S = 6.84 \text{ m}^2$; $v' = 1.36 \text{ m}$

$P_0 = 2.71 \text{ MN}$; $\Delta P = 25\% \Rightarrow P = 13.18 \text{ MN}$

$n \geq 6.41 \Rightarrow \text{adopt } n = 8 \text{ tendons of } 13\text{T}15$

Table VI.3. Estimation of the number of continuity tendons for the side span.

Section	M (MN.m)	v' (m)	v (m)	I (m ⁴)	S (m ²)	e (m)	P (MN)	N Tendons	Value
End V04	0.00	1.46	0.94	5.46	6.84	1.31	0.00	0.00	2
End V05	6.12	1.46	0.94	5.46	6.84	1.31	3.29	1.60	4
End V06	12.61	1.46	0.94	5.46	6.84	1.31	6.79	3.30	4
End V07	17.72	1.46	0.94	5.46	6.84	1.31	9.54	4.64	6
End V08	21.43	1.46	0.94	5.46	6.84	1.31	11.54	5.62	6
End V09	23.74	1.46	0.94	5.46	6.84	1.31	12.79	6.22	8
End V10	24.47	1.46	0.94	5.46	6.84	1.31	13.18	6.41	8
End V11	23.75	1.46	0.94	5.46	6.84	1.31	12.79	6.22	8

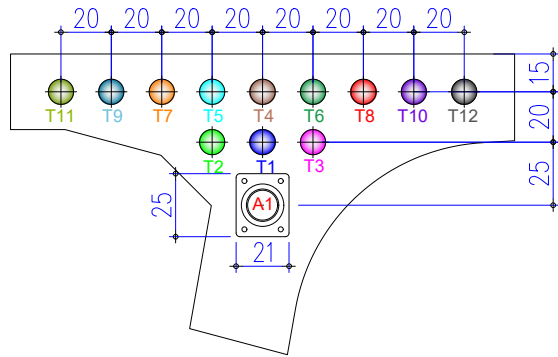
End V12	21.68	1.46	0.94	5.46	6.84	1.31	11.68	5.68	8
End VC3	19.32	1.46	0.94	5.46	6.84	1.31	10.41	5.06	6
End VC2	14.89	1.46	0.94	5.46	6.84	1.31	8.02	3.90	6
End VC1	9.10	1.46	0.94	5.46	6.84	1.31	4.90	2.39	4
End VSC	1.95	1.46	0.94	5.46	6.84	1.31	1.05	0.51	4
VSC	0.00	1.36	1.04	7.35	13.40	1.21	0.00	0.00	2

VI.3.4. Layout of Continuity tendons in the end span

After determining the required number of tendons for each segment, their layout is illustrated in three views: plan view, elevation, and cross-section.

The equations and parameters governing the tendon paths are provided in Appendix C.

Layout of cantilever tendons VSP axis Scale 1:20



Tendon characteristics :

Tendon type : **15T15**

Section $A_p = 2\,100\text{ mm}^2$

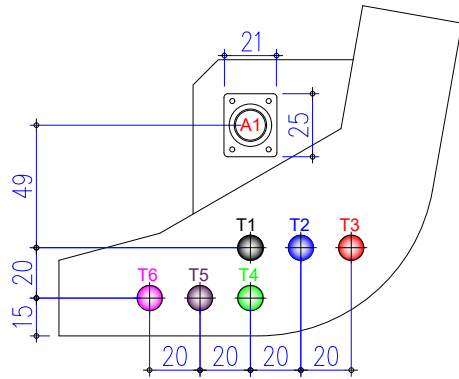
Class : **1 860**

Modulus of elasticity : **190 000 MPa**

Number of tendons : **24**

Anchorage plate dimensions : **25x21 cm²**

Layout of continuity tendons –main span– Scale 1:20



Tendon characteristics :

Tendon type : **15T15**

Section $A_p = 2\,100\text{ mm}^2$

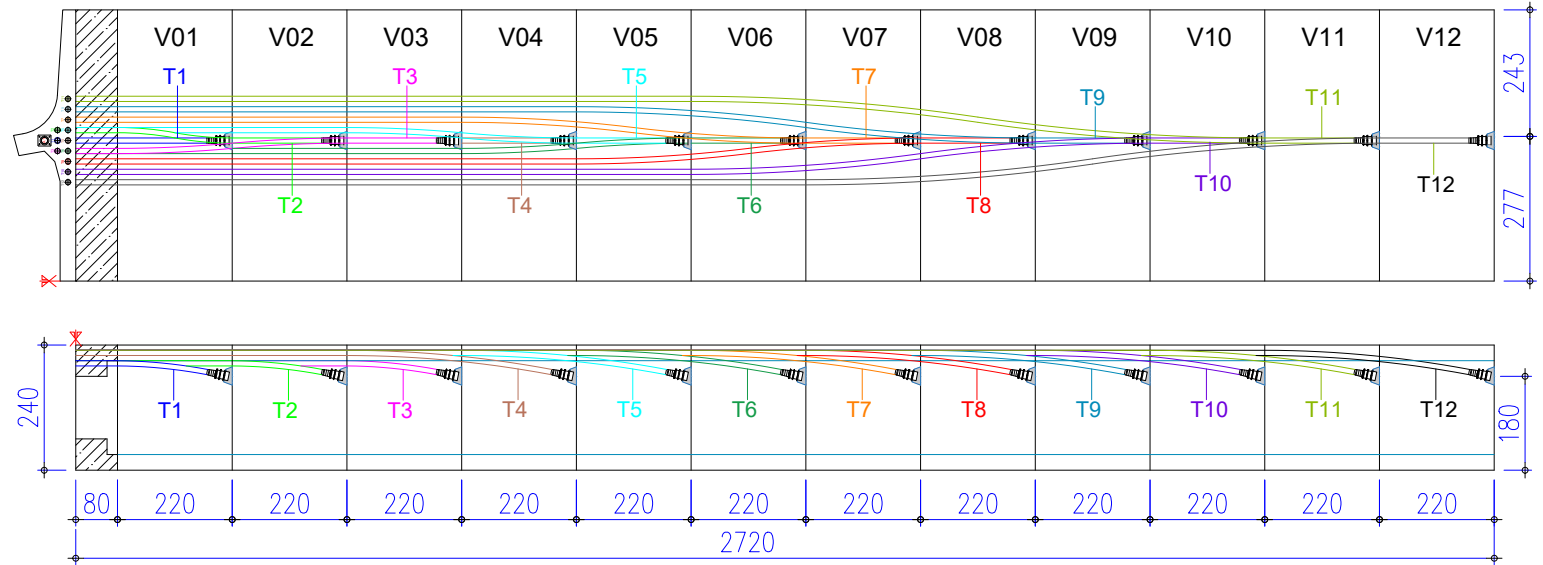
Class : **1 860**

Modulus of elasticity : **190 000 MPa**

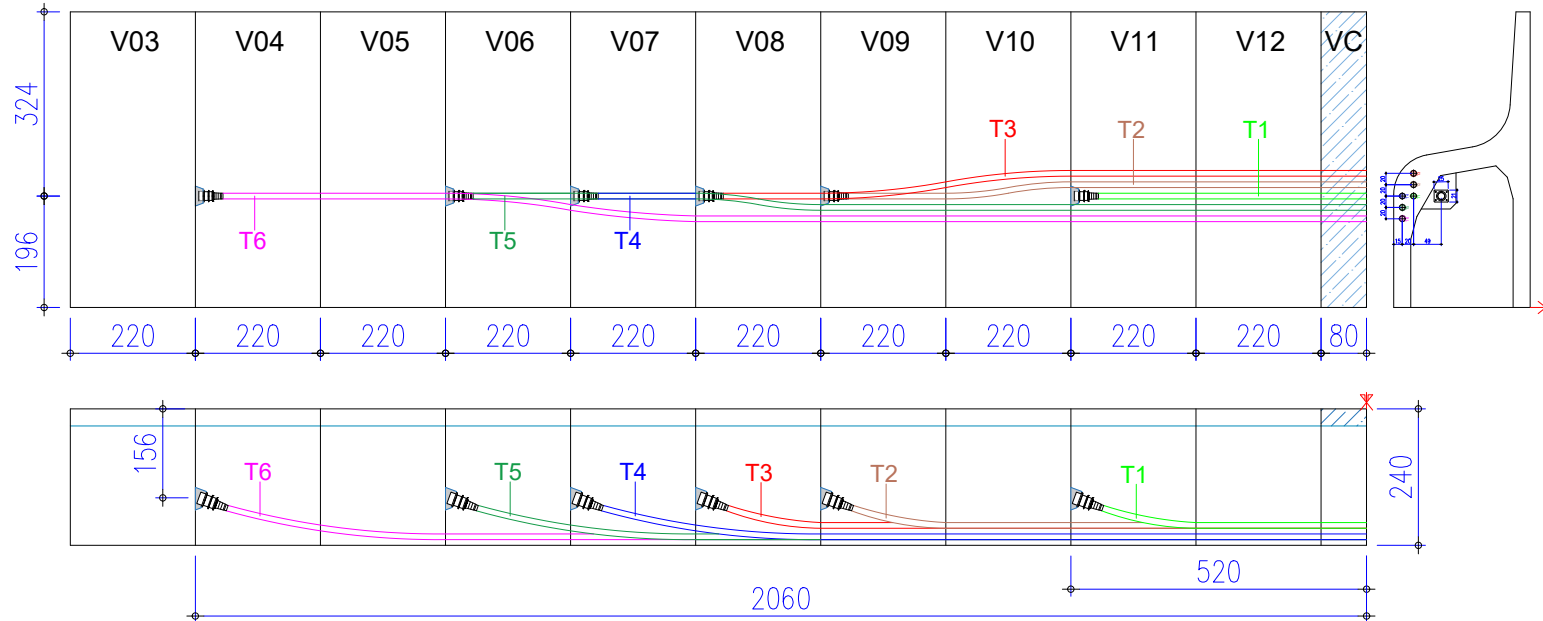
Number of tendons : **12**

Anchorage plate dimensions : **25x21 cm²**

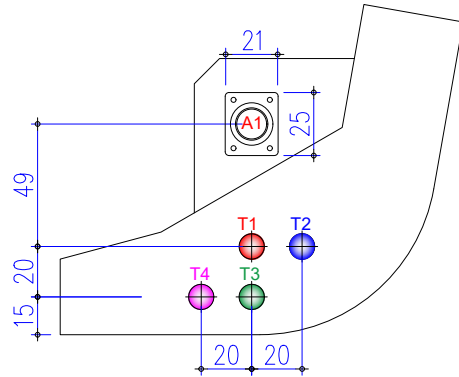
Combined profile of cantilever tendons Scale 1:100



Combined profile of continuity tendons –main span– Scale 1:100



Layout of continuity tendons –edge span– Scale 1:20



Tendon characteristics :

Tendon type : **13T15**

Section $A_p = 1\,820\text{ mm}^2$

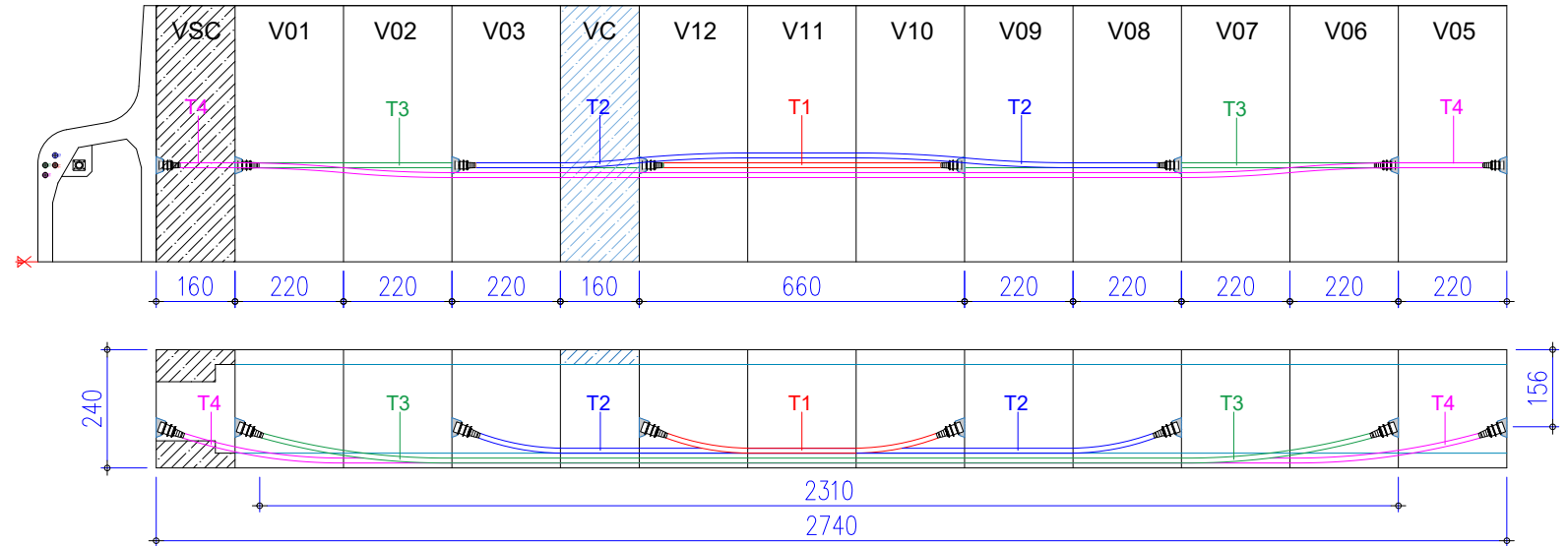
Class : **1 860**

Modulus of elasticity : **190 000 MPa**

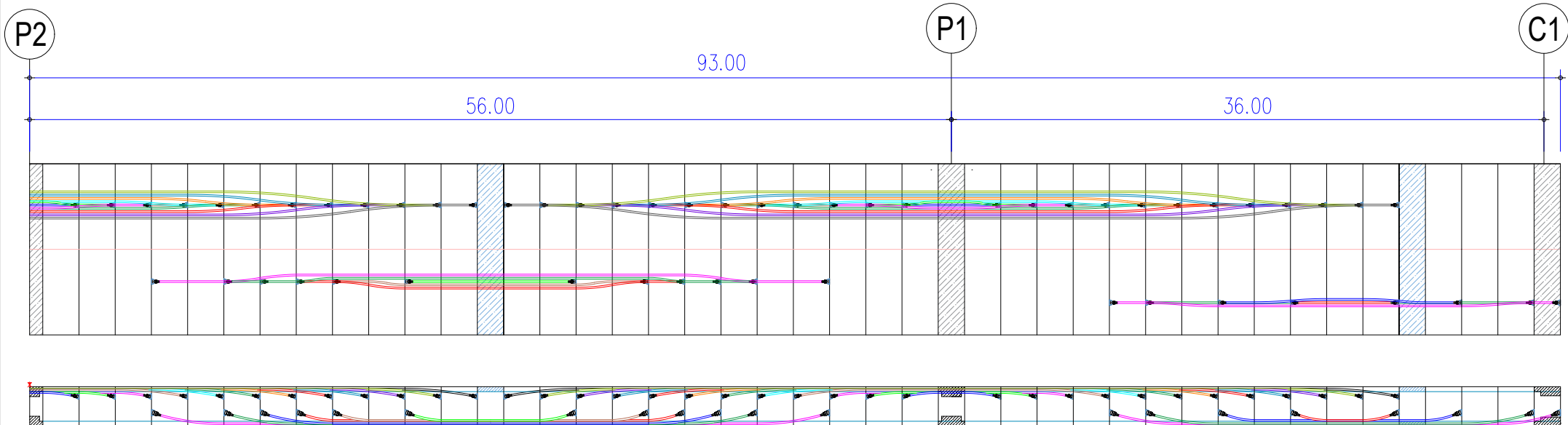
Number of tendons : **8**

Anchorage plate dimensions : **25x21 cm²**

Combined profile of continuity tendons –edge span– Scale 1:100



Combined profile of prestressing tendons Scale 1:325



VI.4. EVALUATION OF PRESTRESS LOSSES

In general, prestress loss refers to the difference between the force applied by the jack during tensioning and the actual force present in a tendon at a specific time.

In prestressed concrete, these losses are far from negligible and can reach 20% to 30% of the initial prestress. They can be categorized into instantaneous (short-term) losses and time-dependent (long-term) losses.

VI.4.1. Instantaneous Prestress Losses

Instantaneous losses occur at the moment the tendons are tensioned. They include:

- Friction losses
- Anchor slip losses
- Elastic shortening of concrete

The total instantaneous loss is expressed as:

$$\Delta\sigma_i = \Delta\sigma_\phi + \Delta\sigma_g + \Delta\sigma_e$$

VI.4.1.1. Friction Losses

Friction losses occur due to the interaction of the tendon with its duct, especially in curved paths or deviations in the tendon trajectory. The actual tendon path rarely follows a perfectly straight line, and even in linear segments, small parasitic deviations exist. These losses can be expressed by the following formula:

$$\Delta\sigma_\phi(x) = \sigma_{p0} \cdot (1 - e^{-(f \cdot \theta + \phi \cdot x)})$$

Where:

σ_{p0} : initial tendon stress

e : base of natural logarithms

f : coefficient of friction along curves (rad^{-1})

ϕ : coefficient of friction along straight segments (m^{-1})

θ : cumulative angular deviation of the tendon over distance x (rad)

x : distance from the section considered (m)

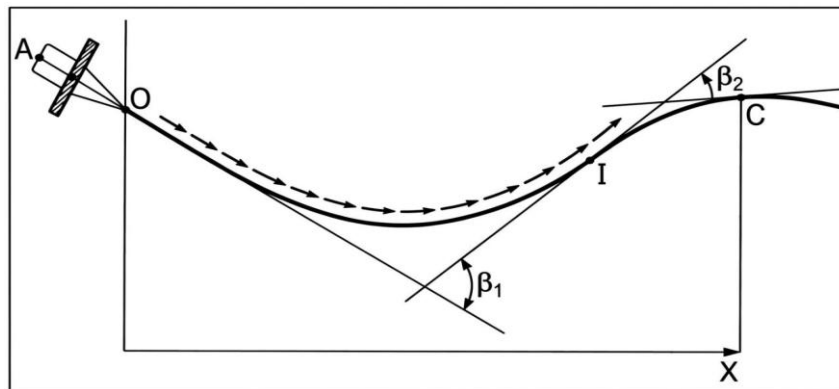


Figure VI.3. Friction along the tendon during tensioning.

During construction, the tendon may experience angular deviations α in elevation and β in plan, yielding:

$$\theta = \alpha + \beta \quad (\text{successive deviations})$$

$$\theta = \sqrt{\tan^2 \alpha + \tan^2 \beta} \quad (\text{simultaneous deviations})$$

VI.4.1.2. Anchor Slip Losses

Anchor slip occurs due to the movement of the tendon relative to its anchor, or deformation/settlement of the anchor itself. The effect diminishes with distance from the anchor and can be defined as:

$$\Delta\sigma_g(x) = 2 \cdot \sigma_{p0} \cdot k(d - x)$$

$$k = \frac{f \cdot \beta}{l} + \phi, d = \sqrt{\frac{E_p \cdot g}{\sigma_{p0} \cdot k}}$$

Where :

d: length over which anchor slip occurs

g = 2 mm: magnitude of anchor slip

E_p = 190 000 MPa: modulus of elasticity of the tendon steel

VI.4.1.3. Elastic Shortening Losses

Since prestressing is applied sequentially to multiple tendons, the initial shortening of the concrete affects subsequently tensioned tendons.

If a member contains n tendons, the first tendon experiences the effect of the n-1 subsequent tendons, whereas the last tendon does not experience this loss. The average loss due to concrete elastic shortening can be calculated as:

$$\sigma_e = \frac{1}{2} \cdot \frac{E_p}{E_{bi}} \sigma_{bi}$$

$$\sigma_{bi} = \frac{P}{B} + \frac{P \cdot e^2}{I_G} - \frac{M_p \cdot e}{I_G}$$

Where :

σ_b: final concrete stress after stabilization of instantaneous losses

E_p: modulus of elasticity of prestressing steel

E_{ci}: instantaneous modulus of concrete

P: prestressing force, **P = P₀(1 - ΔP)**

M_p: bending moment due to self-weight of the member

VI.4.2. Time-Dependent Prestress Losses

Long-term losses occur gradually over time and include:

- Losses due to concrete shrinkage
- Losses due to concrete creep

- Losses due to steel relaxation

The total long-term loss is:

$$\Delta\sigma_d = \Delta\sigma_r + \Delta\sigma_{fl} + \frac{5}{6}\Delta\sigma_\rho$$

VI.4.2.1. Shrinkage Losses

During hardening, concrete undergoes volumetric reduction due to evaporation of excess water and chemical reactions. If t_0 is the age of the concrete at the time of tendon tensioning, the remaining shrinkage is:

$$\Delta\sigma_r = E_p \cdot \epsilon_r \cdot [1 - r(t_0)]$$

In practice, $r(t_0)$ can be neglected compared to 1, giving: $\Delta\sigma_r \approx E_p \cdot \epsilon_r$, with typical $\epsilon_r \approx 2 \times 10^{-4}$ for temperate humid climates.

VI.4.2.2. Creep Losses

Creep is the time-dependent deformation of concrete under sustained stress. It develops gradually after loading and in addition to shrinkage. The creep-related prestress loss is:

$$\Delta\sigma_{fl} = E_p \cdot \epsilon_f, \epsilon_f = \frac{\sigma_M - \sigma_b}{E_{bj}}$$

Where : σ_M is the maximum concrete stress after instantaneous losses; according to BPEL, $\sigma_M \approx 1.5 \sigma_b$.

Where : $\Delta\sigma_{fl} = 2.5 \sigma_b(x) \frac{E_p}{E_{bi}}$, σ_b : Final stress in the concrete at the tendon level.

VI.4.2.3. Steel Relaxation Losses

Steel relaxation leads to a reduction of tendon stress over time, depending on steel type and treatment. It can be estimated as:

$$\Delta\sigma_\rho = \frac{6}{100} \cdot \rho_{1000} \cdot (\mu - \mu_0) \cdot \sigma_i(x), \mu = \frac{f_{prg}}{\sigma_i(x)}$$

Where :

$\rho_{1000} = 2.5\%$: guaranteed relaxation at 1 000 hours

σ_i : initial tendon stress

$\mu_0 = 0.43$: coefficient for very low relaxation (TBR steel)

VI.4.2.4. Total Prestress Losses

Total prestress losses are the sum of instantaneous losses and time-dependent losses:

$$\Delta\sigma_{tot} = \Delta\sigma_i + \Delta\sigma_d$$

VI.4.3. Application to Cantilever Tendons

The results of the prestress loss calculations for each tendon are presented in Table VI.4 and detailed in Appendix C.

Table VI.4. Average tension loss percentages (%) of the cantilever tendons.

Section	$\Delta\sigma_\phi$	$\Delta\sigma_g$	$\Delta\sigma_e$	$\Delta\sigma_i$	$\Delta\sigma_r$	$\Delta\sigma_n$	$\Delta\sigma_p$	$\Delta\sigma_d$	$\Delta\sigma_{tot}$
Axis VSP	7.80	0.28	0.51	8.58	2.55	3.78	4.13	9.78	18.36
End VSP	7.65	0.30	1.25	9.21	2.55	7.53	4.04	13.45	22.66
End V01	6.59	0.92	1.54	9.05	2.55	8.73	4.06	14.67	23.72
End V02	6.41	1.81	1.53	9.75	2.55	8.52	3.95	14.37	24.12
End V03	6.17	1.83	1.53	9.54	2.55	8.37	3.99	14.25	23.79
End V04	5.97	1.76	1.48	9.21	2.55	7.98	4.03	13.90	23.11
End V05	5.71	2.01	1.40	9.12	2.55	7.45	4.05	13.38	22.50
End V06	5.14	2.31	1.30	8.75	2.55	6.84	4.11	12.81	21.57
End V07	4.70	2.72	1.17	8.60	2.55	6.09	4.13	12.09	20.68
End V08	4.10	3.27	1.02	8.39	2.55	5.24	4.16	11.26	19.65
End V09	3.30	4.11	0.84	8.25	2.55	4.28	4.18	10.32	18.57
End V10	2.69	5.49	0.63	8.82	2.55	3.18	4.10	9.15	17.97
End V11	1.69	7.28	0.40	9.38	2.55	2.02	4.03	7.91	17.29
End V12	0.00	9.05	0.17	9.22	2.55	0.93	4.01	6.76	15.97

VI.4.4. Verification of Normal Stresses

The application of prestressing induces normal stresses in both the top and bottom fibers, during the construction phase as well as the service phase. These stresses must be verified against the allowable limits specified by the applicable regulations.

The verification condition is:

$$\overline{\sigma}_{bc} \leq \sigma \leq \overline{\sigma}_{bt}$$

The stress at a given fiber is calculated as:

$$\sigma(y) = \frac{P}{B} + \frac{P \cdot e_0}{I} \cdot y + \frac{M}{I} \cdot y$$

Where :

P = prestressing force including losses

B = cross-sectional area

I = moment of inertia of the considered section

e_0 = eccentricity of the resultant prestressing tendon

M = external bending moment

y = distance from the centroid to the extreme fiber

VI.4.4.1. Verification of Stresses During Construction Phase

The first stage of stress verification is governed by:

$$\overline{\sigma}_{bc} = 0.6 f_{c28} = 21 \text{ MPa}$$

$$\overline{\sigma}_{bt1} = -1.5 f_{t28} = -4.05 \text{ MPa (outside cover zone)}$$

$$\overline{\sigma}_{bt2} = -0.7 f_{t28} = -1.89 \text{ MPa (cover zone)}$$

VI.4.4.2. Verification of Stresses During Service Phase

For the service phase analysis, two verification cases are considered:

- ❖ **Unloaded condition:** only the self-weight of the deck is taken into account
- ❖ **Loaded condition:** both permanent loads and live loads are considered

The verification criteria in this phase are:

$$\overline{\sigma}_{bc} = 0.6 f_{c28} = 21 \text{ MPa}$$

$$\overline{\sigma}_{bt1} = -1.5 f_{t28} = -4.05 \text{ MPa (outside cover zone)}$$

$$\overline{\sigma}_{bt2} = -1.0 f_{t28} = -2.70 \text{ MPa (cover zone)}$$

The results are presented in Appendix C, where all stresses are verified.

VI.4.5. Reinforcement in Tensile Zones

In zones where tensile stresses occur in concrete, it is necessary to provide a minimum area of passive reinforcement A_s in accordance with Article 6.1.32.4.1 of the BPEL rules.

The required reinforcement area is given by:

$$A_s = \frac{B_t}{1000} + \frac{N_{Bt}}{f_e} \cdot \frac{f_{tj}}{\sigma_{Bt}}$$

Where :

B_t = net area of concrete in tension

σ_{Bt} = absolute value of the maximum tensile stress

N_{Bt} = resultant tensile force

f_e = yield strength of the reinforcement steel

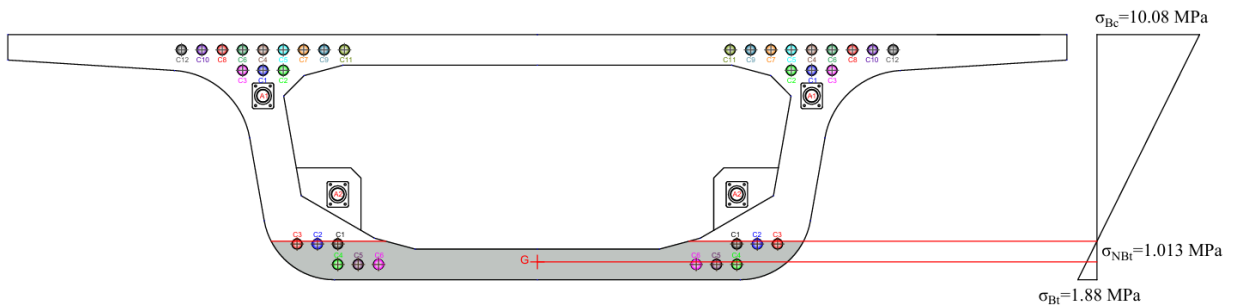


Figure VI.4. Detail of the tension zone under longitudinal actions.

According to the previous figure and the stress verification table (Appendix C), we have:

$$B_t = 1.57 \text{ m}^2$$

$$\sigma_{Bt} = 1.88 \text{ MPa}$$

$$\sigma_{NBt} = 1.013 \text{ MPa}$$

Thus :

$$N_{Bt} = 1.59 \text{ MN}$$

$$A_s = \frac{1.57}{1000} + \frac{1.59}{500} \cdot \frac{2.7}{1.88} = 61.37 \text{ cm}^2 (5.90 \text{ cm}^2/\text{m})$$

Adopted reinforcement: $\Rightarrow$ 2 layers of HA14 spaced at 20 cm

VI.5. VERIFICATION OF SHEAR STRESSES

VI.5.1. Verification with Respect to SLS

The shear-resistant part of a box girder is mainly the web, while the top and bottom flanges, due to their inertia relative to the centroid, primarily resist bending.

VI.5.1.1. Determination of Shear Force

❖ Shear Force Due to External Loads

The shear force due to external actions corresponds to the resultant of all applied loads and is expressed as:

$$V = V_q + V_g = 8.31 \text{ MN}$$

Where :

V_g : shear force due to self-weight

V_q : shear force due to variable loads

❖ Shear Force Due to Prestressing

The actions acting on the considered element are reduced to the vertical components of all applied forces:

- A positive external shear force
- A vertical component of the prestressing force $P \cdot \sin \alpha$, which reduces the shear force

Thus :

$$V_p = P \cdot \sin \alpha$$

Numerical Example :

$$P_0 = 3.12 \text{ MN}$$

$$\Delta P = 24.12\%$$

$$\alpha_{moy} = 0.178 \text{ rad}$$

$$n = 24 \text{ tendons}$$

$$P = 56.91 \text{ MN}$$

$$V_P = 10.08 \text{ MN}$$

❖ **Reduced Shear Force at Serviceability Limit State**

The following condition must be verified:

$$\tau_{max} < \min(\bar{\tau}_1, \bar{\tau}_2)$$

With :

$$\tau = \frac{V_{red}}{b_n \cdot z}$$

Such that :

$$\begin{aligned}\bar{\tau}_2 &= 0.4 \cdot f_{tj} \cdot (f_{tj} + \sigma_x) \\ \bar{\tau}_1 &= 2 \cdot \frac{f_{tj}}{f_{cj}} (0.6 \cdot f_{cj} - \sigma_x) \cdot (f_{tj} + \sigma_x)\end{aligned}$$

Where :

σ_x : longitudinal normal stress at the centroid $\frac{P \times \cos \alpha}{B_n}$

z : lever arm ($z = I/S$), and by simplification: $z \approx \frac{h}{2}(1 + \rho)$

ρ : mechanical reinforcement ratio of the section: $\rho = \frac{I}{v' \cdot v \cdot B}$

b_n : effective web thickness, equal to the difference between the total web thickness and the diameters of the ducts crossing the web: $b_n = 2(b_0 - n \cdot k \cdot \Phi)$

b_0 : total web thickness

n : number of tendons

$k = 0.5$: coefficient for cement grout injection

Φ : diameter of the duct

Numerical Example :

$$\alpha_{moy} = 0.178 \text{ rad} \Rightarrow V_{ser} = 8.31 \text{ MN}$$

$$P = 56.91 \text{ MN}, V_P = 10.08 \text{ MN}, V_{red} = 1.77 \text{ MN}$$

$$B_n = 6.55 \text{ m}^2 \Rightarrow \sigma_x = 8.55 \text{ MPa}$$

$$f_{c28} = 35 \text{ MPa}, f_{t28} = 2.7 \text{ MPa},$$

$$I_n = 5.22 \text{ m}^4, B_n = 6.55 \text{ m}^3, b_n = 0.7 \text{ m}$$

$$\rho = 0.578, z = 1.894 \text{ m}$$

$$\bar{\tau}_1 = 3.49 \text{ MPa}, \bar{\tau}_2 = 4.65 \text{ MPa}$$

$$\tau_{max} = 1.68 \text{ MPa}$$

$$\text{Conclusion: } \tau_{max} < \bar{\tau}$$

$\Rightarrow$ Shear stresses are verified near the studied section

VI.5.2. Verification at Ultimate Limit State (ULS)

The verification of structural elements at the ultimate limit state involves:

- ❖ Checking the concrete compression struts
- ❖ Checking the resistance of transverse reinforcement

VI.5.2.1. Concrete Compression Struts

First, the angle β_u formed by the concrete struts with the mean fiber of the beam is determined using:

$$\tan(2\beta_u) = \frac{2\tau_u}{\sigma_x - \sigma_y}, \text{ with } \beta_u \geq 30^\circ$$

Where :

$$\sigma_x, \sigma_y: \text{normal stresses at the centroid } \sigma_x = \frac{P \cdot \cos \alpha}{B_n}, \sigma_y = 0$$

$$\tau_u: \text{reduced ultimate shear stress } \tau_u = \frac{V_{u,red} \cdot S_n}{b_n \cdot I_n}$$

$V_{u,red}$: maximum reduced shear force at ULS in the considered section

❖ Verification Condition for Concrete Struts

The verification of concrete compression is expressed as:

$$\tau_u \leq \frac{0.85 \cdot f_{c28}}{\gamma_b \cdot \sin(2\beta_u)}$$

For $\beta_u = 30^\circ$, this simplifies to:

$$\tau_u \leq \frac{f_{c28}}{6}$$

Numerical Application :

$$V_{ULS} = 11.23 \text{ MN}, V_P = 10.08 \text{ MN}, V_{u,red} = V_{ULS} - V_P = 1.15 \text{ MN}$$

$$I_n = 5.22 \text{ m}^4, S_n = 6.55 \text{ m}^3, b_n = 0.7 \text{ m}, \tau_u = 2.06 \text{ MPa}$$

$$\sigma_x = 8.55 \text{ MPa}, \sigma_y = 0, \beta_u = 12.86^\circ \Rightarrow \beta_u = 30^\circ \text{ (adopted)}$$

$$\text{Verification: } \tau_u = 2.06 \leq \frac{35}{6} = 5.83 \text{ MPa}$$

$\Rightarrow$ Concrete compression struts are verified

VI.5.2.2. Resistance of Transverse Reinforcement

The required transverse reinforcement area A_t is given by:

$$\frac{A_t \cdot f_e}{b_n \cdot s_t \cdot \gamma_s} \geq \tan \beta_u \cdot \left(\tau_u - \frac{f_{tj}}{3} \right)$$

Where :

A_t : total area of transverse reinforcement per spacing

s_t : spacing of reinforcement

f_e : yield strength of steel

$\gamma_s = 1.15$: safety factor

❖ **Minimum Reinforcement Condition**

$$\frac{A_t}{s_t} \geq 0.6 \cdot \frac{b_n \cdot \gamma_s}{f_e}$$

Numerical Application :

$$\tau_u = 2.06 \text{ MPa } f_{t28} = 2.7 \text{ MPa } f_e = 500 \text{ MPa } \gamma_s = 1.15 \quad b_n = 0.7 \text{ m } \beta_u = 30^\circ$$

$$\frac{A_t}{s_t} \geq \frac{0.6 \cdot b_n}{f_e \cdot \gamma_s} \geq 9.66 \text{ cm}^2/\text{m}$$

⇒ To be verified in the transverse design study

VI.6. CONCLUSION

The study of prestressing is an essential part in the design of a segmental bridge constructed by the balanced cantilever method. The different types of tendons are determined according to the bending moments that must be resisted.

The decrease of moments from the fixed support makes it possible to progressively terminate the cantilever tendons within each segment. Following the same principle, and based on the moments at mid-span, the required continuity tendons are determined.

Finally, a verification of stresses at the joints is carried out in order to identify the sections where additional passive reinforcement is required to compensate for tensile stresses in the concrete.

CHAPTER VII

TRANSVERSE STUDY

VII.1. INTRODUCTION

This chapter focuses on the determination of transverse reinforcement (passive reinforcement) in the webs, as well as in the upper and lower slabs.

Since the structure has a constant depth, the transverse bending analysis is carried out on the typical cross-section, with particular attention given to the standard segment.

At the level of the segment cross-section, the self-weight and the various service loads generate external forces, which are balanced by shear stresses resulting from the shear force.

VII.2. EVALUATION OF FORCES

VII.2.1. Modeling

The modeling is carried out using a finite element-based software. The segment is represented by a closed frame, supported at the lower part of the webs by two simple supports (double-simple) located at the ends of each web.

A unit length of 1 m is adopted for each segment, and the sections of the different elements of the box girder are considered as equivalent sections in terms of both weight and inertia.

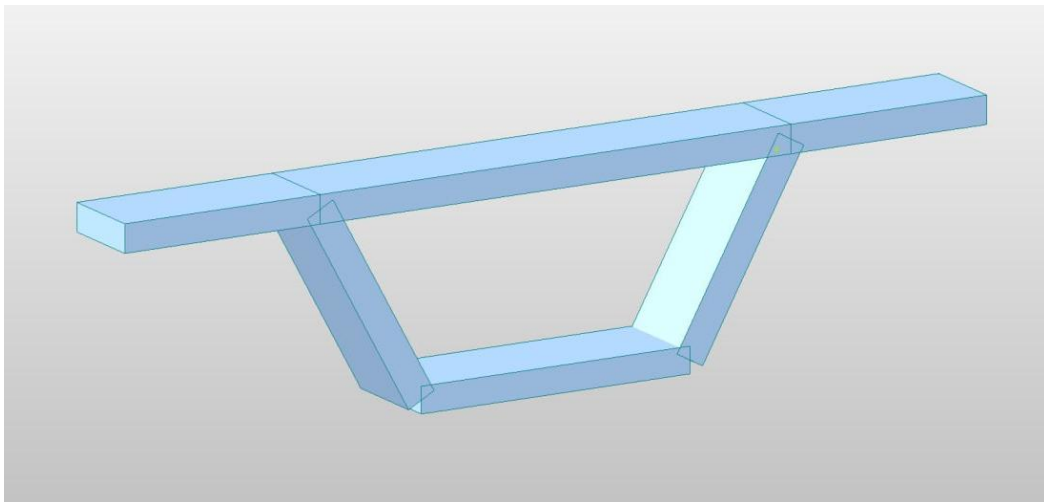


Figure VII.1. 3D view of the typical segment.

VII.2.2. Determination of internal forces/stresses

The loads to be considered are defined beforehand in Chapter IV. The segment must be reinforced under a moment specific to the most unfavorable combination, given by RCPR.

These combinations are those listed in Tables V.4 established in Chapter V.

VII.2.3. Results

The results are summarized in the following table:

Table VII.1. Extreme moments in the segment at the keystone.

		Upper slab	Lower slab	Web
ULS	$M_{\max}$ (kN·m)	249.90	46.70	276.70
	$M_{\min}$ (kN·m)	-336.00	- 61.30	- 60.80
SLS	$M_{\max}$ (kN·m)	185.10	34.60	208.30
	$M_{\min}$ (kN·m)	-251.60	- 45.40	- 45.00

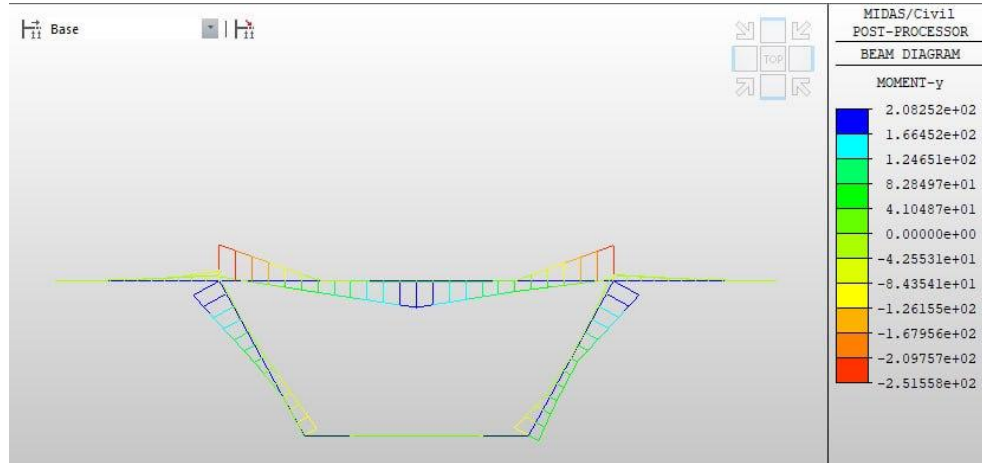


Figure VII.2. Maximum moment for the SLS combination G + D240.

VII.3. REINFORCEMENT

To ensure proper reinforcement design, the segment is subdivided into several elements, defined as follows: a top slab working in simple bending, two webs working in combined bending, and a bottom slab working in simple bending.

VII.3.1. Reinforcement Design of the Upper Slab of VSP

Cracking is considered detrimental; therefore, the design of the rectangular section is carried out at the Serviceability Limit State (SLS).

❖ Bottom reinforcement layer:

Numerical Application :

$b=1.00$ m (calculated per linear meter)

$h = 0.35$ m , $c = 0.05$ m , $M_{ser} = 185.10$ kN·m

$\overline{\sigma}_{bc} = 21$ MPa , $\overline{\sigma}_s = 250$ MPa , $d = 0.9 \times h = 31.5$ cm

$x = \frac{15 \overline{\sigma}_{bc}}{15 \overline{\sigma}_{bc} + \overline{\sigma}_s} \times d = 17.56$ cm , $z = d - \frac{x}{3} = 25.65$ cm

$M_1 = 0.5 \times b \times x \times \overline{\sigma}_{bc} \times z = 471.28$ kN·m

Since $M_1 \geq M_{ser} \Rightarrow$ the section is without compression reinforcement (S.S.A.C).

$$A_{ser} = \frac{M_{ser}}{z \times \bar{\sigma}_{st}} = 28.72 \text{ cm}^2 \Rightarrow \text{Proposed reinforcement: 6HA25}$$

$$\text{Minimum reinforcement (CNF): } A_{min} = 0.23 \times \frac{f_{tj}}{f_e} \times b \times d = 3.91 \text{ cm}^2$$

Since $A_{min} \leq A_{ser} \Rightarrow$ the non-fragility condition is satisfied.

❖ Top Reinforcement Layer

Numerical Application :

$$h = 0.35 \text{ m}, d = 0.32 \text{ m}, x = 0.18 \text{ m}, z = 0.26 \text{ m}, M_{ser} = 251.60 \text{ kN}\cdot\text{m}$$

$$M_1 = 0.5 \times b \times x \times \bar{\sigma}_{bc} \times z = 346.50 \text{ kN}\cdot\text{m}$$

Since $M_1 \geq M_{ser} \Rightarrow$ the section is without compression reinforcement (S.S.A.C).

$$A_{ser} = 38.97 \text{ cm}^2 \Rightarrow \text{Proposed reinforcement: 6HA32}$$

$$A_{min} = 3.91 \text{ cm}^2 \Rightarrow \text{The non-fragility condition is satisfied.}$$

❖ Longitudinal Reinforcement

The longitudinal reinforcement is considered as construction reinforcement only, since the total approach is adopted, meaning that prestressing tendons carry all loads and surcharges of the structure.

Numerical Application :

$$A_{sl} = \frac{A_{st}}{4} = 16.08 \text{ cm}^2 \Rightarrow \text{A reinforcement of 10HA14 is proposed.}$$

$$\text{(C.N.F): } A_{min} = 0.23 \times \frac{f_{tj}}{f_e} \times b \times d = 3.35 \text{ cm}^2 \leq A_{ser} \Rightarrow \text{Condition verified.}$$

VII.3.2. Passive Reinforcement of the Segments

The reinforcement results are summarized in the following table:

Table VII.2. Passive reinforcement of the segments.

Element	Transverse			Longitudinal
	Layer	As	Ø	Ø
Upper slab	Sup	39.70	6HA32	5HA20
	Inf	28.50	6HA25	5HA16
Lower slab	Sup	6.60	5HA14	5HA10
	Inf	5.00	5HA14	
Web	External	24.80	6HA25	5HA16
	Internal	24.80	6HA25	

N.B: The bending moment diagrams as well as the shear force diagrams are presented in the appendix B.

VII.4. TORSION EFFECT

Torsion is a common phenomenon in structures, often neglected due to its complexity and generally limited impact. However, it must be considered when it contributes to structural stability, as is the case for the segment loaded outside its plane of symmetry.

Torsional effects are introduced in the webs by adding longitudinal and transverse reinforcement to the previously determined reinforcement. The torsion effect is checked in the curved zone with a radius of 125 m.

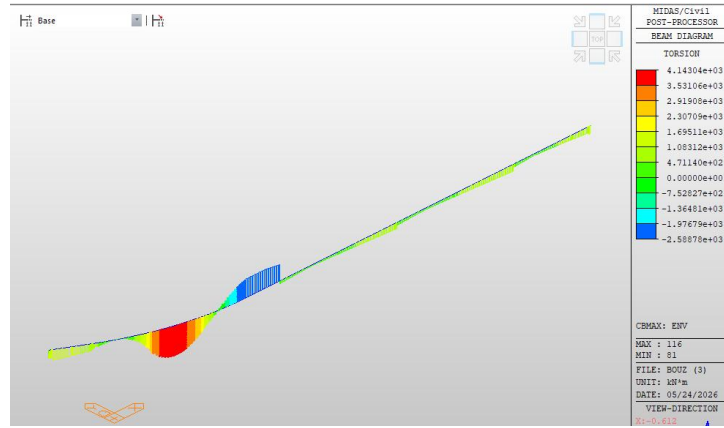


Figure VII.3. Torsional moment diagram.

The torsional moment $M_t = M_{max, SLS} = 4\,143.41\text{ kN}\cdot\text{m}$

The shear stress due to this low torsion is given by: $\tau = \frac{M_t}{b\Omega}$

Where :

M_t : maximum torsional moment, $M_t = 4\,143.41\text{ kN}\cdot\text{m}$

b : thickness of the two webs, $b = 2 \times 0.40 = 0.80\text{ m}$

Ω : torsional area, $\Omega = 10.65\text{ m}^2$

U : perimeter of the segment, $U = 24.04\text{ m}$

$\bar{\tau}$: allowable shear stress, $\bar{\tau} = 0.1 \times f_{c28} = 3.5\text{ MPa}$

S_t : spacing of the bars, $S_t = 20\text{ cm}$

γ_s : partial safety factor, taken as $\gamma_s = 1.15$

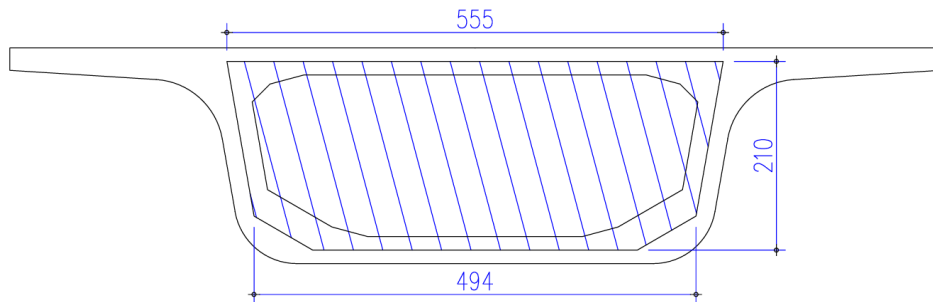


Figure VII.4. Section Ω .

VII.4.1. Reinforcement Calculation

$$\tau = \frac{M_t}{b \Omega} = \frac{4\,143.41 \times 10^{-3}}{0.8 \times 10.65} = 0.52 \text{ MPa} < 3.50 \text{ MPa} \Rightarrow \tau_{\max} < \bar{\tau}$$

$$A_l = \frac{M_t U \gamma_s}{2 \Omega f_e} = \frac{4.14 \times 24.04 \times 1.15}{2 \times 10.65 \times 500} = 107.47 \text{ cm}^2$$

$$A_t = \frac{M_t S_t \gamma_s}{2 \Omega f_e} = \frac{4.14 \times 0.2 \times 1.15}{2 \times 10.65 \times 500} = 0.89 \text{ cm}^2$$

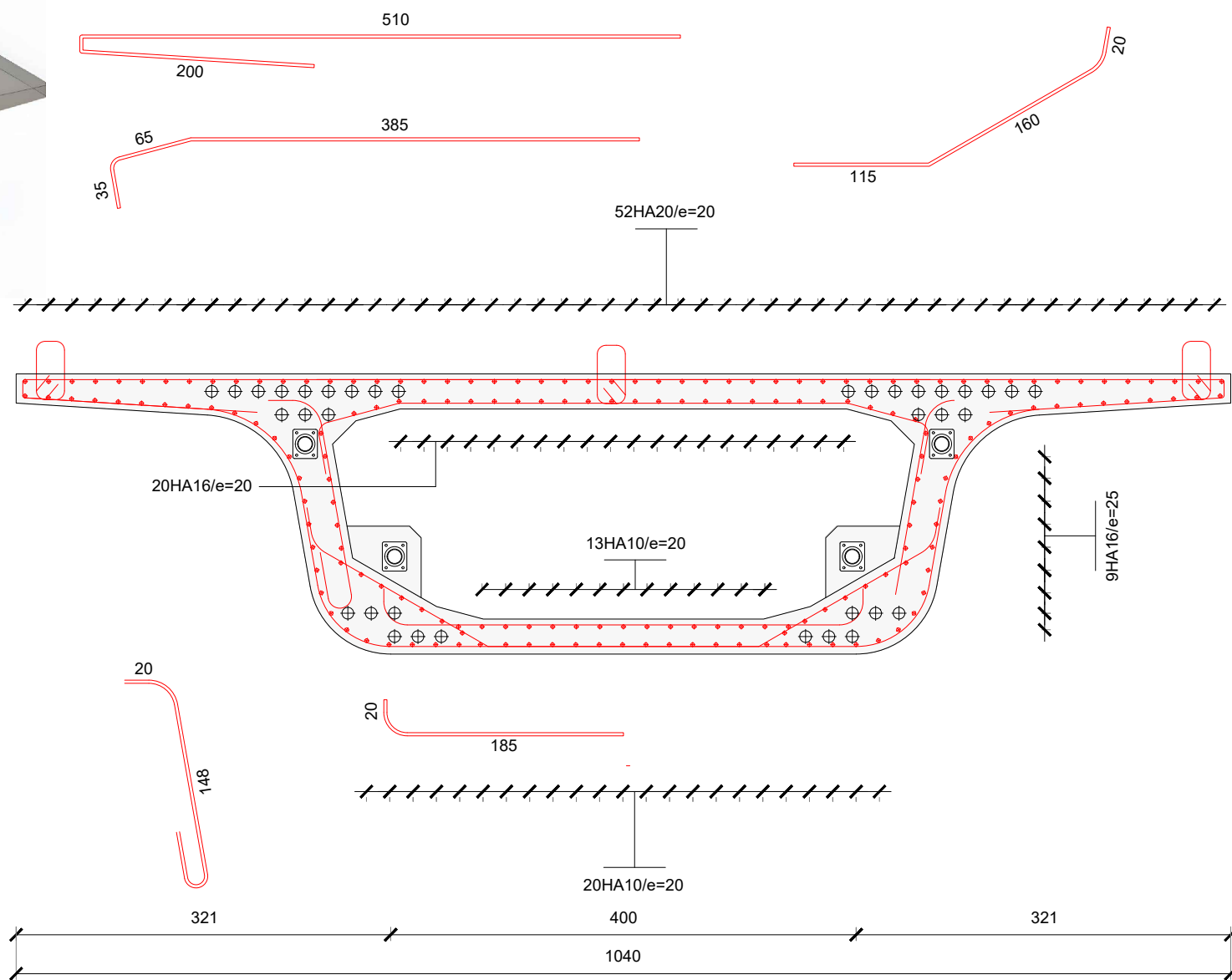
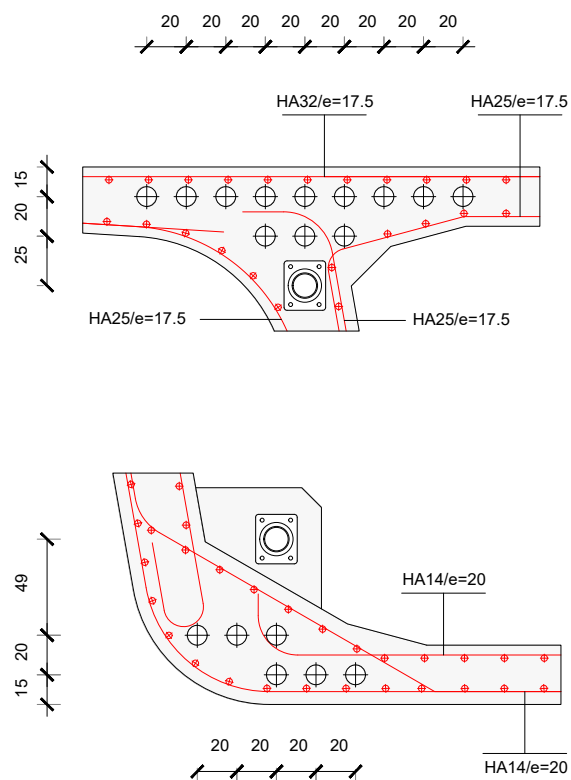
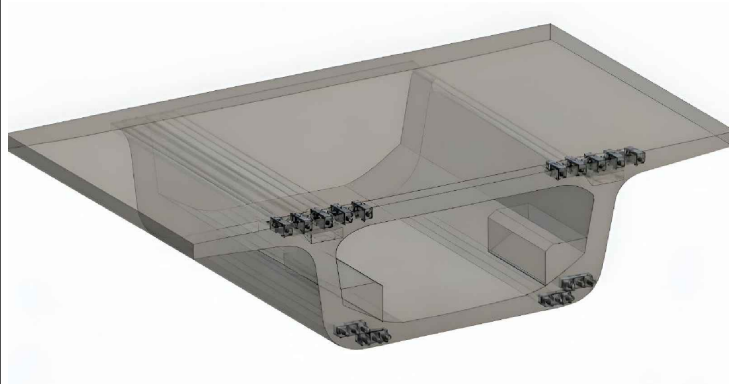
These areas are divided by two since the calculation is performed for both webs.

Therefore, the required reinforcement is:

❖ **Longitudinal (per web):** $53.74 \text{ cm}^2 \Rightarrow 8HA32$ per web

❖ **Transverse (per web):** $0.45 \text{ cm}^2 \Rightarrow$ (negligible)

The figure below illustrates the reinforcement details for the typical segment:



VII.5. STUDY OF PRESTRESSING DIFFUSION AND ANCHORAGE

The anchorage force induces high compressive stresses over a small concrete surface. This force then spreads over a larger area within the segment. After a certain distance, called the regularization length, the Saint-Venant principle is assumed to apply, and the normal stress distribution becomes linear.

Four cases of prestressing force diffusion are distinguished:

- Centered straight tendon
- Eccentric straight tendon
- Multiple straight tendons
- General case: multiple curved and inclined tendons

Let :

S_a : end section where the tendons are anchored

S_r : regularization section located at a distance

d : height of section S_a

The tendon stressing is performed after 4 days of concrete curing:

$$f_{cj} = \frac{j}{4.76+0.83 \times j} = 17.33 \text{ MPa for } f_{cj} \leq 40 \text{ MPa}$$

$$f_{tj} = 1.64 \text{ MPa}$$

VII.5.1. Regulatory Provisions According to BPEL

VII.5.1.1. Surface Effect

In the immediate vicinity of the anchorage, surface confinement reinforcement is required:

$$A_s = \frac{0.04 \times P_{max}}{\bar{\sigma}_s} \quad \text{with} \quad \bar{\sigma}_s = \frac{2}{3} f_e$$

Where :

P_{max} : the maximum prestressing force in the considered section, $P_{max} = 2.84 \text{ MN}$

Result :

$$A_s = 3.41 \text{ cm}^2 \Rightarrow 5HA10$$

VII.5.1.2. Bursting Effect

❖ Concrete tensile stress:

$$\sigma_{tj} = 0.5 \left(1 - \frac{a_j}{d_j} \right) \times \frac{P_{max}}{e \times d_j} \leq 1.25 f_{tj}$$

With :

d_j : height of the symmetric prism at level j,

$$d_j = \min(c, c') = \min(1.20; 0.48) = 0.48 \text{ m}$$

a_j : dimension of the anchorage plate at level j, $a_j = 0.25 \text{ m}$

e : width of the web, $e = 2 \times 0.4 = 0.8 \text{ m}$

Result : $\sigma_{tj} = 1.77 \text{ MPa} \leq 2.05 \text{ MPa} \Rightarrow \text{Condition verified}$

❖ **Concrete Compressive Stress**

$$\sigma_{xj} = \frac{P_{max}}{e \times d_j} \leq \frac{2}{3} f_{cj}$$

Result : $\sigma_{xj} = \frac{2.84}{0.8 \times 0.48} = 7.40 \text{ MPa} \leq 11.55 \text{ MPa} \Rightarrow \text{Condition verified}$

❖ **Bursting Reinforcement**

For each anchorage at level j , the required steel area is:

$$A_{ej} = \frac{0.25 \left(1 - \frac{a_j}{d_j}\right) P_{max}}{K_j \times \frac{2}{3} f_e} = 6.80 \text{ cm}^2$$

Note : $K_j = 1.5$ for an intermediate anchorage.

The final transverse steel area is:

$$A_e = \max \left(A_{ej}; \frac{0.15 P_{max}}{\frac{2}{3} f_e} \right) = 6.80 \text{ cm}^2$$

Note : These reinforcements are distributed over a length d_j .

VII.5.2. Bossage Study

The continuity tendons are housed in the bottom slab and may be deviated within special prisms called bossages.

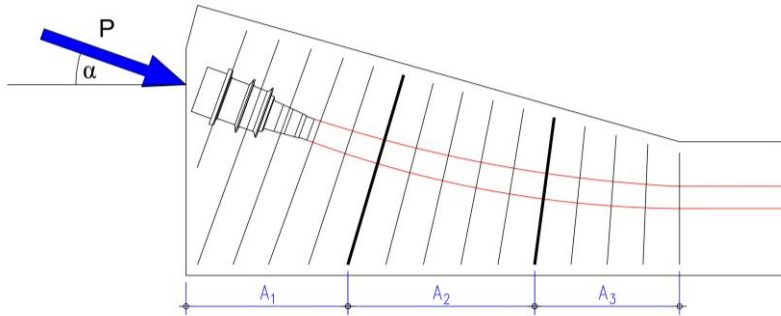


Figure VII.5. Bossage steel.

Under the effect of the prestressing force P , transverse fibers of the slab are subjected to deformation, generating a dragging force behind the bossages.

To limit this effect, specific passive reinforcement is introduced. Three categories of passive reinforcement are distinguished (for tendon CC01).

❖ **Bursting Reinforcement A_1**

These reinforcements are placed over a distance of $1.2 C$ ahead of the anchorage plate, measured from the end of the boss.

$$A_1 = \frac{0.15 P}{f_{yd}} \quad \text{with} \quad f_{yd} = \frac{f_e}{1.15}$$

Where :

P : is the prestressing force at anchorage, $P = 3.12 \text{ MN}$

Result :

$$A_1 = \frac{0.15 \times 3.12}{434.78} = 10.78 \text{ cm}^2$$

❖ Intermediate Stitching Reinforcement A₂

These reinforcements are distributed between the boss head and the beginning of the tendon curvature.

$$A_2 = \frac{0.4 P}{f_{yd}} = 28.75 \text{ cm}^2$$

❖ Reinforcement for Deviation Force A_3

These reinforcements are located in the curved zone of the tendon.

$$A_3 = \frac{2}{3} \times \frac{P}{f_{yd}} \times \sin \alpha = 16.43 \text{ cm}^2$$

Where :

α : is the angle between the prestressing tendon and the upper fiber of the slab,
 $\alpha = 0.35$ rad.

Adopted Reinforcement

- ❖ $A_1 = 10.76 \text{ cm}^2 \rightarrow 2 \text{ double stirrups HA14}$
- ❖ $A_2 = 28.75 \text{ cm}^2 \rightarrow 4 \text{ double stirrups HA16}$
- ❖ $A_3 = 16.43 \text{ cm}^2 \rightarrow 5 \text{ stirrups HA16}$

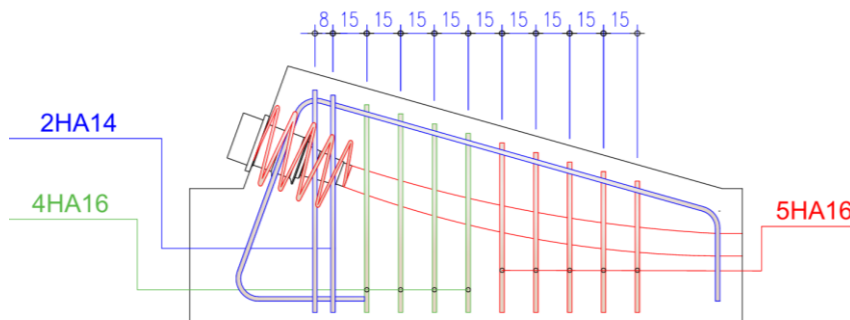


Figure VII.6. Bossage reinforcement diagram.

VII.6. CONCLUSION

The modeling of the segment using dedicated software made it possible to determine the different internal forces under various load combinations. This subsequently enabled the design of the passive reinforcement for the structural elements of the segment.

CHAPTER VIII

SEISMIC STUDY AND BRIDGE COMPONENTS

VIII.1. INTRODUCTION

Bridge equipment plays a fundamental role in ensuring the serviceability and durability of the structure. It consists of a set of devices with various characteristics, designs, and operating principles, whose purpose is to enable the bridge deck to properly fulfill its function, particularly with respect to user safety and comfort.

Among these equipment elements, particular attention is given to bearing devices, pedestals, and expansion joints.

In this study, the focus is placed on the analysis of essential equipment, especially the bearing devices, considering their behavior at the Ultimate Limit State and under exceptional loading conditions when required.

VIII.2. BEARING DEVICES

Bearing devices are structural components that ensure the connection between the bridge deck and its supports. Their primary function is to transfer vertical loads from the superstructure to the substructure, while also accommodating horizontal forces and movements such as displacements and rotations.

VIII.2.1. Selection of Bearing Type

For the present bridge, laminated elastomeric bearings have been adopted. These bearings are composed of layers of elastomer—generally neoprene—stacked together with interleaved steel plates acting as reinforcement.

This type of bearing is widely used in reinforced and prestressed concrete bridges due to the advantages it offers:

- Ease of installation
- Simplicity in adjustment and inspection
- Efficient distribution of horizontal forces among multiple supports
- Relatively low cost

The selected bearings are of Type B, consisting of n elastomer layers and $n + 1$ steel plates. The entire assembly is covered by a peripheral elastomer layer of sufficient thickness to ensure durability and protection.

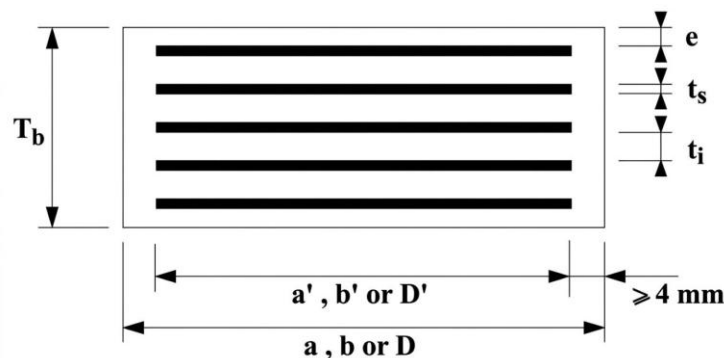


Figure VIII.1. Geometric definition of a bearing device.

The main parameters defining the bearing are:

T : total thickness of elastomer

T_b : total thickness of the bearing

n : number of elastomer layers

t_i : thickness of a single elastomer layer

e : cover thickness (protective elastomer layer)

VIII.2.2. Evaluation of Normal Forces on Bearings

The reactions and the corresponding vertical forces applied to the bearings are determined at the Ultimate Limit State (ULS).

Table VIII.1. Reaction applied to the bearing devices.

Combination	Abutment 01	Pier 01	Pier 02	Pier 03	Pier 04	Abutment 02
G	1.58	6.38	7.07	6.06	5.50	1.49
1.35G+1.6A(l)	2.51	10.19	10.76	8.96	8.17	2.33
1.35G+1.6B _C	3.09	9.90	10.45	9.36	8.60	3.11
1.35G+1.6B _t	2.57	9.33	10.23	8.79	8.04	2.54
1.35(G+M _C 120)	2.91	9.54	10.01	9.09	8.35	2.95
1.35(G+D240)	3.47	10.08	10.88	9.63	8.89	3.49

VIII.2.3. Design of the Bearing Device

According to the SETRA technical guide, the plan dimensions of the bearing device are determined based on the normal stress under the maximum reaction at the Ultimate Limit State.

VIII.2.3.1. Bearing Area

The verification of the non-crushing condition allows the determination of the bearing dimensions.

$$\frac{N_{max}}{a' \times b'} \leq \sigma_{adm} = 20 \text{ MPa}$$

Numerical Application :

For abutments :

$$N_{max} = 3.49 \text{ MN}$$

$$a \times b \geq 1\,745 \text{ cm}^2$$

$$\text{Adopted: } a \times b = 500 \times 500 \text{ mm}^2$$

For piers :

$$N_{max} = 10.88 \text{ MN}$$

$$a \times b \geq 5\,440 \text{ cm}^2$$

$$\text{Adopted: } a \times b = 800 \times 800 \text{ mm}^2$$

VIII.2.3.2. Thickness of the bearing T

Verification of the non-buckling condition to determine the elastomer height:

$$\frac{a}{10} \leq T \leq \frac{a}{5}$$

Where :

T : thickness of the bearing

a : larger side of the elastomer

Numerical Application :

For abutments :

$$a = 500 \Rightarrow 50 \leq T \leq 100 \text{ mm}$$

For piers:

$$a = 800 \Rightarrow 80 \text{ mm} \leq T \leq 160 \text{ mm}$$

VIII.2.3.3. Design of Steel shims

To determine the thickness of the steel shims, the following tensile stress conditions must be verified:

with:

$$\sigma_m = \frac{N_{max}}{a \cdot b}, \beta = \frac{a \cdot b}{2 \cdot G(a + b)}, \sigma_e = 235 \text{ MPa}, G = 0.9 \text{ MPa}, t_s > \frac{a}{\beta} \cdot \frac{\sigma_m}{\sigma_e}$$

Numerical Application :

For abutments :

$$\sigma_m = 13.96 \text{ MPa}$$

$$\beta = 13.86 \Rightarrow t_s > 2.14 \text{ mm}$$

$$\text{Adopted: } t_s = 3 \text{ mm}$$

For piers :

$$\sigma_m = 17.00 \text{ MPa}$$

$$\beta = 22.22 \Rightarrow t_s > 2.60 \text{ mm}$$

$$\text{Adopted: } t_s = 3 \text{ mm}$$

The dimensions are summarized as follows:

For abutments :

6 elastomer layers: $t_1 = 10 \text{ mm}$

7 steel shims: $t_s = 3 \text{ mm}$

Edge cover: 4 mm

External cover: 5 mm

$$500 \times 500 \times 91 \text{ mm}^3$$

For piers :

7 elastomer layers: $t_1 = 16 \text{ mm}$

8 steel shims: $t_s = 3 \text{ mm}$

Edge cover: 4 mm

External cover: 8 mm

$$800 \times 800 \times 152 \text{ mm}^3$$

VIII.2.4. Evaluation of Horizontal Displacements

VIII.2.4.1. Displacements due to static actions

The static force at the top of each support is given by:

$$H_{sta} = \frac{\Delta x \cdot G \cdot a \cdot b}{T_q}$$

Where :

G: shear modulus of the bearing, $G = 0.9 \text{ MPa}$

Δx : deformation due to the applied force

$a \times b$: dimensions of the bearing

T: thickness of the elastomer layers

❖ **Action due to shrinkage**

$$H_r = \frac{\Delta_r \cdot G \cdot a \cdot b}{T}$$

With :

$$\Delta_r = \varepsilon_r \cdot |X_i|$$

Where :

ε_r is the shrinkage coefficient ; $\varepsilon_r = 2 \times 10^{-4}$

❖ **Action due to temperature variation**

$$H_T = \frac{\Delta_T \cdot G \cdot a \cdot b}{T}$$

With :

$$\Delta_T = \varepsilon_T \cdot |X_i|, \varepsilon_T = \Delta T \cdot \alpha$$

Where :

$$\Delta_T = 35^\circ\text{C}, \alpha = 10^{-5}, \varepsilon_T = 3.5 \times 10^{-4}$$

❖ **Action due to creep**

$$H_{fl} = \frac{\Delta_{fl} \cdot G \cdot a \cdot b}{T}$$

With :

$$\Delta_{fl} = \varepsilon_{fl} \cdot |X_i|$$

And :

$$\varepsilon_{fl} = K_{fl} \times 10^{-4}$$

Where :

K_{fl} : creep coefficient (between 2 and 3)

Adopted: $K_{fl} = 2.5 \Rightarrow \varepsilon_{fl} = 2.5 \times 10^{-4}$

VIII.2.4.2. Displacements due to dynamic actions

The only dynamic action considered is the braking force:

- Due to truck Bc: $F_{fr} = 300 \text{ kN}$
- Due to load Al: $F_{fr} = 248.58 \text{ kN}$

The displacement is calculated as:

$$\Delta_{fr} \Rightarrow \frac{F_{fr} \cdot T_q}{2 \cdot G \cdot a \cdot b}$$

The braking forces are distributed according to the stiffness of each support.

VIII.2.4.3. Center of rigidity

The center of rigidity with respect to abutment C_1 is:

$$X_{CR/C1} = \frac{\sum X_i K_i}{\sum K_i} = 120.03 \text{ m}$$

Where :

K_i : stiffness of support

X_i : position of the supports

$|X_i|$: distance between the support and the center of rigidity

The following table summarizes the horizontal forces and displacements in the deck:

Table VIII.2. Forces and horizontal displacement of the deck.

	K_i (MN/m)	 X_i (m)	Δ_r (m)	Δ_T (m)	Δ_n (m)	Δ_{fr} (m)	H_r (MN)	H_T (MN)	H_n (MN)	H_{fr} (MN)
C1	6.43	120.03	0.024	0.042	0.030	0.010	0.077	0.135	0.096	0.023
P1	8.89	84.03	0.017	0.029	0.021	0.013	0.076	0.132	0.094	0.033
P2	8.86	28.03	0.006	0.01	0.007	0.013	0.025	0.044	0.031	0.033
P3	8.89	27.97	0.006	0.01	0.007	0.013	0.025	0.044	0.031	0.033
P4	8.89	83.97	0.017	0.029	0.021	0.013	0.076	0.132	0.094	0.033
C2	6.43	119.97	0.024	0.042	0.030	0.010	0.077	0.135	0.096	0.024

VIII.3. SEISMIC ANALYSIS

The verification of structures with respect to seismic design actions must ensure that the risk of failure under a potential earthquake remains sufficiently low.

VIII.3.1. Structural Characteristics Related to Seismic Analysis

The RPOA 2008 provides a set of classifications required to define the seismic condition of the studied structure. These classifications constitute an essential prerequisite for the selection of the analysis model, the calculation method, and the parameters to be used:

- ❖ The bridge is classified, with respect to seismic risk, as Group 2 (important bridge).
- ❖ The structure is located in seismic zone IIa (moderate seismicity).
- ❖ The zone acceleration coefficient is $A = 0.2$ (RPOA, Table 3.1).
- ❖ The soil belongs to category S3 (soft soil).
- ❖ The damping ratio of the material constituting the supports is $\xi = 5\%$.
- ❖ The shear modulus of elastomeric bearings is $G = 0.9 \text{ MPa}$.

VIII.3.2. Calculation Method

In general, the analysis of a bridge is carried out either using the single-mode spectral method (fundamental mode) or the response spectrum method (linear dynamic analysis).

The single-mode spectral method is applicable to bridges that simultaneously satisfy several criteria, including the curvature criterion: *“The curvature (angle swept in plan by the tangent to the axis) must be less than 30 grads.”* This condition is not satisfied in our case, since the curvature of the structure is 32 grads.

Therefore, the response spectrum method is adopted.

VIII.3.3. Response Spectrum

For the multimodal analysis, the use of calculation software is required. For this purpose, the longitudinal model is reused by combining the beam and segmental parts, while modeling the piers and bearings, and performing a modal spectral analysis.

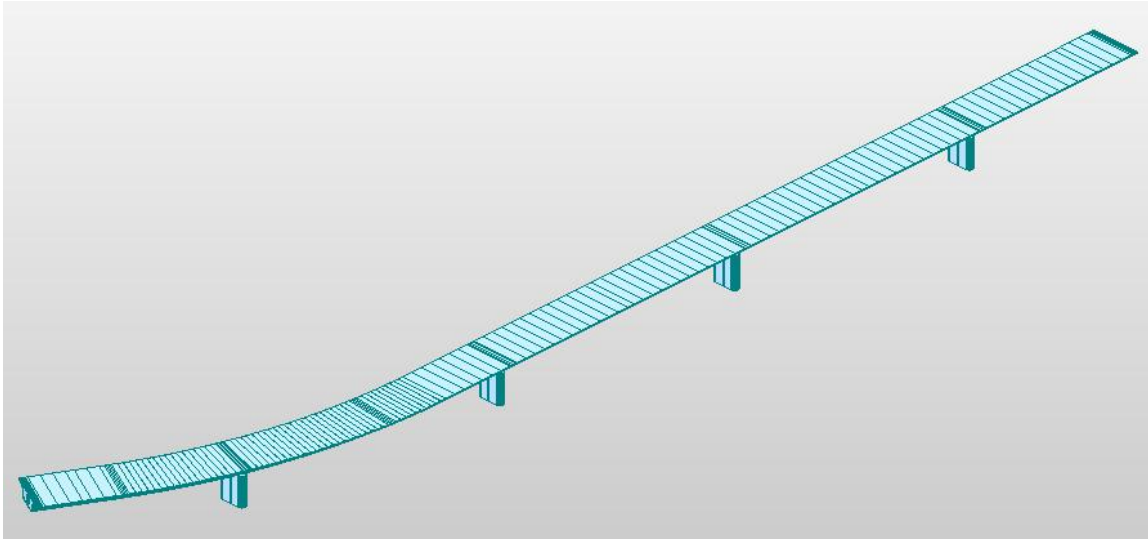


Figure VIII.2. Multimodal calculation model.

VIII.3.4. Significant Modes

All modes that significantly contribute to the total structural response must be taken into account. In other words, the modes considered are those for which the sum of the effective modal masses $\sum(M_i)_c$ reaches at least 90% of the total mass of the bridge.

VIII.3.5. Bearing Verification

The transfer of vertical and horizontal loads, as well as rotations through the bearings, leads to deformations accompanied by stresses that must be limited to avoid any damage.

The verification is carried out according to RPOA 2008, and we check:

VIII.3.5.1. Compressive Strength

The average compressive stress must satisfy:

$$\sigma_{ap} = \frac{N_{max}}{a \times b} \leq 20 \text{ MPa}$$

Numerical Application :

For abutments :

$$\sigma_m = \frac{3.49}{0.5 \times 0.5} = 13.96 \text{ MPa}$$

$$13.96 \leq \sigma_{adm} = 20 \text{ MPa} \Rightarrow \text{OK}$$

For piers :

$$\sigma_m = \frac{10.88}{0.8 \times 0.8} = 17.00 \text{ MPa}$$

$$17.00 \leq \sigma_{adm} = 20 \text{ MPa} \Rightarrow \text{OK}$$

VIII.3.5.2. Buckling Safety

We must verify: $\gamma_s \cdot P_u < P_c$

Where :

γ_s : safety factor ; $\gamma_s = 3$

P_u : maximum vertical load at ULS

P_c : critical buckling load

$$P_c = \frac{4 \cdot G \cdot B \cdot S^2 \cdot d}{h}$$

Where :

G : shear modulus ; $G = 0.9 \text{ MPa}$

B : bearing area ; $B = a \times b$

d : sum of one elastomer layer thickness and one steel shim thickness;

$$d = t_i + t_s$$

h : total height of the bearing

$$S: \text{shape factor ; } S = \frac{a \times b}{2t(a + b)}$$

Numerical Application :

For abutments :

$$S = \frac{500 \times 500}{2 \times 10(500 + 500)} = 12.5$$

$$d = 0.01 + 0.003 = 0.013 \text{ m}$$

$$h = 0.091 \text{ m}, B = 0.5 \times 0.5 = 0.25 \text{ m}^2$$

$$P_c = \frac{4 \times 0.9 \times 0.25 \times 12.5^2 \times 0.013}{0.091}$$

$$P_c = 20.08 \text{ MN}$$

$$P_u = 3.49 \text{ MN}$$

$$\gamma_s \cdot P_u = 3 \times 3.49 = 10.47 \text{ MN}$$

$$\gamma_s \cdot P_u \leq P_c \Rightarrow \text{Condition satisfied}$$

For piers :

$$S = \frac{800 \times 800}{2 \times 16(800 + 800)} = 12.5$$

$$d = 0.016 + 0.003 = 0.019 \text{ m}$$

$$h = 0.152 \text{ m}, B = 0.8 \times 0.8 = 0.64 \text{ m}^2$$

$$P_c = \frac{4 \times 0.9 \times 0.64 \times 12.5^2 \times 0.019}{0.152}$$

$$P_c = 45.00 \text{ MN}$$

$$P_u = 10.88 \text{ MN}$$

$$\gamma_s \cdot P_u = 3 \times 10.88 = 32.64 \text{ MN}$$

$$\gamma_s \cdot P_u \leq P_c \Rightarrow \text{Condition satisfied}$$

VIII.3.5.3. Distortion Limitation

The distortion $\delta = \tan(\gamma)$ represents the ratio of the horizontal displacement d to the total thickness of the elastomer T . It must be limited as follows:

$$\begin{cases} \delta_c = 0.7 + 4(\delta_0 - 0.7)(0.25 - p) & \text{if } p \leq 0.25 \\ \delta_c = 1.4(1 - 2p) & \text{if } p > 0.25 \end{cases}$$

Where :

$$p = \frac{P_u}{P_c} ; \delta = \frac{V_x}{T}$$

$$\delta_0 = \frac{b}{T} \text{ bounded by the intervall } 0.70 < \delta_0 < 2$$

T: total thickness of the elastomer

b: the smaller side of the rectangular bearing

Numerical Application :

For abutments :

$$p = \frac{3.49}{20.08} = 0.17 ; p < 0.25 ; \delta_0 = 7.14$$

we opt $\delta_0 = 2$

$$\delta_c = 0.7 + 4(2 - 0.7)(0.25 - 0.17)$$

$$\delta_c = 1.12$$

$$\delta = \frac{212}{70} = 3.03$$

$$\delta < \delta_c \Rightarrow \text{Condition not satisfied}$$

For piers :

$$p = \frac{10.88}{45.00} = 0.24 ; p < 0.25 ; \delta_0 = 6.25$$

we opt $\delta_0 = 2$

$$\delta_c = 0.7 + 4(2 - 0.7)(0.25 - 0.24)$$

$$\delta_c = 0.75$$

$$\delta = \frac{206}{128} = 1.61$$

$$\delta < \delta_c \Rightarrow \text{Condition not satisfied}$$

Therefore, non-distortion devices must be installed at the abutments and piers.

VIII.3.5.4. Condition for Non-Sliding

To ensure the stability of the bearing pads against sliding under dynamic conditions, the following condition must be verified: $H < f \cdot N$

Where :

H: horizontal force

N: vertical force

f: friction coefficient

$$f = 0.1 + \frac{0.6}{\sigma_m}$$

Numerical Application :

For abutments :

$$N = 2.06 \text{ MN}$$

$$\sigma_m = 8.24 \text{ MPa} \Rightarrow f = 0.17$$

$$H = 1.09 \text{ MN}$$

$$H > 2.06 \times 0.17 = 0.35 \text{ MN}$$

$\Rightarrow$ Condition not satisfied

For piers :

$$N = 7.54 \text{ MN}$$

$$\sigma_m = 11.78 \text{ MPa} \Rightarrow f = 0.15$$

$$H = 4.36 \text{ MN}$$

$$H > 7.54 \times 0.15 = 1.13 \text{ MN}$$

$\Rightarrow$ Condition not satisfied

Therefore, anti-sliding devices must be installed at the abutments and the piers.

VIII.4. CONCRETE BEARING PEDESTALS

The bearing pedestal allows the distribution of concentrated loads coming directly from the deck, which may cause cracking. For this purpose, confinement and bursting reinforcement are provided to transfer these stresses to the supporting element, as well as to correct any slope of the deck.

VIII.4.1. Design of Bearing Pedestals for piers and abutments

The dimensions (A, B, h) are chosen so as to accommodate the bearing devices. The following dimensions are selected:

❖ Piers

$$A = B + 200 \Rightarrow A = 800 + 200 = 1\,000 \text{ mm}$$

$$B = A + 200 \Rightarrow B = 800 + 200 = 1\,000 \text{ mm}$$

$$h \geq \frac{2B_0}{P_0} \Rightarrow h \geq 0.50 \text{ m We opt: } h = 0.55 \text{ m}$$

❖ Abutments

$$A = B + 200 \Rightarrow A = 500 + 200 = 700 \text{ mm}$$

$$B = A + 200 \Rightarrow B = 500 + 200 = 700 \text{ mm}$$

$$h \geq \frac{2B_0}{P_0} \Rightarrow h \geq 0.35 \text{ m We opt: } h = 0.40 \text{ m}$$

With :

$$B_0 = A \times B$$

$$P_0 = 2(A + B)$$

VIII.4.2. Reinforcement of Bearing Pedestals for the Pier

The reinforcement areas are determined according to the SETRA guide (P.P.73).

❖ Tie Beam Reinforcement:

The reinforcement area for confinement can be determined using rules analogous to the strut-and-tie method or the method based on the equilibrium of forces.

$$A_c = \frac{0.25 \times N}{\sigma_s}$$

❖ Horizontal Reinforcement:

To limit crack propagation, the horizontal reinforcement placed below the confinement must resist 12.5% of the applied load:

$$A_h = \frac{0.125 \times N}{\sigma_s}$$

❖ Upper Reinforcement:

Placed as close as possible to the top face of the bearing, consisting of crossed reinforcement bars:

$$A_s = \frac{0.04 \times N}{\sigma_s}$$

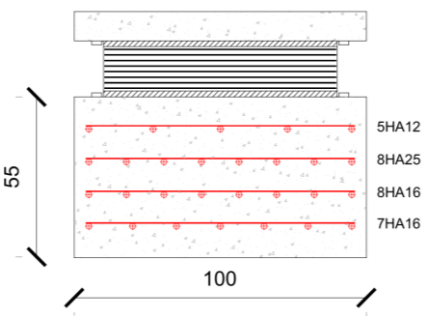
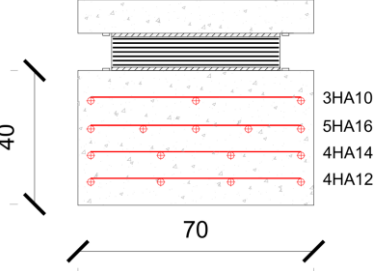
❖ **Lower Reinforcement:**

A lower bursting reinforcement is placed at a depth between $h/3$ and h from the top face of the bearing:

$$A_i = \frac{0.10 \times N}{\sigma_s}$$

The reinforcement of the bearing pedestal is shown in the Table VIII.3.

Table VIII.3. Summary of the reinforcement for the bearing pedestals.

	Piers		Abutments	
Reinforcement Type	Reinforcement	Spacing (cm)	Reinforcement	Spacing (cm)
Tie Beam Reinforcement	16HA25	12.5	10HA16	15
Horizontal Reinforcement	16HA16	12.5	8HA14	17.5
Upper Reinforcement	10HA12	20	6HA10	20
Lower Reinforcement	14HA16	15	8HA12	17.5
Figure				

VIII.5. ROADWAY EXPANSION JOINT

Bridge decks undergo longitudinal dimensional variations due to temperature changes, shrinkage, creep, and service load deformations. It is therefore necessary to allow the deck to expand freely by providing a gap at one or both ends.

VIII.5.1. Design

According to the regulations, the actions to be considered are temperature effects and time-dependent concrete deformations (shrinkage and creep).

$$W = W_D + 0.3W_T + \frac{W_S}{3}$$

Where :

W : total joint movement

W_D : movement due to time-dependent deformations (shrinkage + creep)

$W_D = 5.40 \text{ cm}$

W_T : thermal movement ; $W_T = 4.20 \text{ cm}$

W_S : seismic movement ; $W_S = 12.59 \text{ cm}$

The seismic movement has been determined during the seismic load analysis.

Thus : the total expansion joint movement is equal to: $W = 10.86 \text{ cm}$

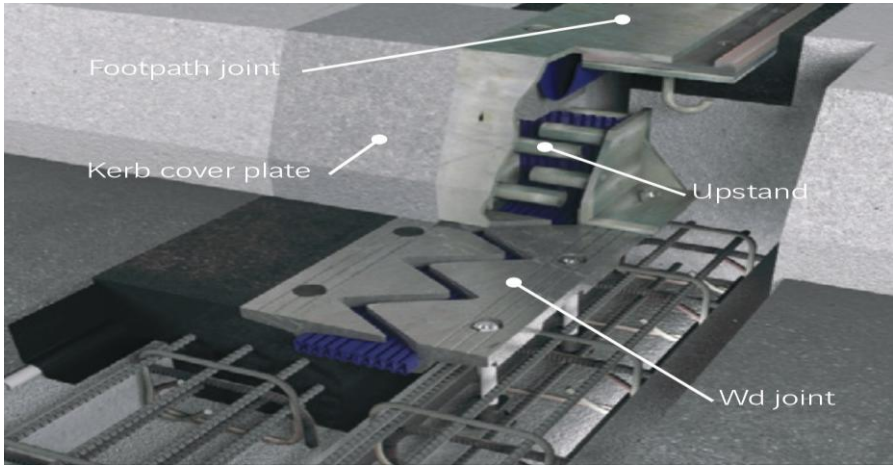
VIII.5.2. Type of Joint

The type of roadway expansion joint is selected based on the calculated movement.

Referring to the CIPEC expansion joint catalog, for a movement of 10.86 cm, the suitable type is the Wd joint family (medium movement joints).

These joints belong to the cantilever tooth-type joints, designed for heavy and intensive traffic. This type of joint offers exceptional durability and robustness.

Table VIII.4. Wd joint models.



Type	100 grades	80 grades	60 grades	40 grades	30 grades
Wd60	60	61	71	66	67
Wd80	80	84	92	85	86
Wd110	110	116	104	92	90
Wd160	160	169	158	141	139
Wd230	230	185	127	102	97

The selected model, based on the joint movement and the skew of the structure, is the Wd110 expansion joint.

The allowable movements for this type of joint permit transverse displacements under service conditions, as well as longitudinal displacements up to 110 mm.

VIII.6. GUARDRAILS AND RESTRAINT SYSTEMS

The sidewalks provided on the structure are intended for maintenance access; therefore, guardrails are installed to ensure user safety. Since guardrails also play an important role in the architectural appearance of the structure, an appropriate type is selected so as not to obstruct the view.

Vehicle restraint systems are designed to contain vehicles, control their trajectory after impact, and prevent accidents.

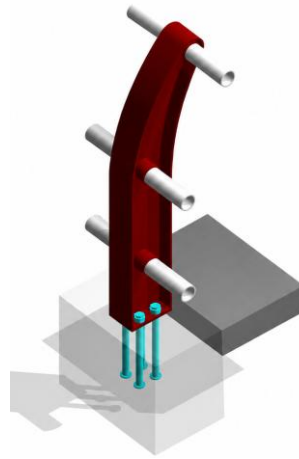


Figure VIII.3. Guardrail model.

VIII.7. DECK DRAINAGE

The deck must be equipped with appropriate drainage systems such as inlets, gutters, and drainage pipes to ensure proper surface water evacuation and to prevent rainwater accumulation on the roadway during precipitation.

VIII.8. DECK WATERPROOFING

The purpose of deck waterproofing is to enhance the durability of the slab by preventing reinforcement corrosion and concrete deterioration caused by water infiltration.

Several techniques can be used for deck waterproofing, such as mastic asphalt systems, bonded thin-film membranes, and prefabricated sheet membranes.

For the present project, a bonded thin-film waterproofing system is selected, as it is more efficient and economical for a prestressed concrete box girder bridge.

VIII.9. CONCLUSION

In this chapter, the role of each deck equipment element and their design methods have been presented in order to ensure user comfort and safety.

The arrangement of the deck with these components represents one of the final stages in the design of the bridge superstructure, before proceeding to the study of the substructure in the next chapter.

CHAPTER IX

SUBSTRUCTURE STUDY

IX.1. INTRODUCTION

The transmission of loads applied to the superstructure to the ground, more specifically to the underlying substratum, is ensured by the substructure. The latter must be properly designed to meet functional requirements of the crossing while also resisting both static and dynamic actions.

In this chapter, the main components of the substructure are examined, including piers, abutments, their foundations, and associated structural elements. The chapter concludes with an overview of the construction phases of the bridge.

IX.2. ABUTMENT STUDY

Abutments are the end supports of a bridge, serving as fundamental components of the substructure that ensure the connection and continuity between the bridge structure and the underlying soil.

The abutment plays a dual role: on one hand, it acts as a support, transferring the loads from the deck to the foundation; on the other, it functions as a retaining wall, bearing the earth pressure from the embankment behind the abutment.

Considering the natural and functional characteristics of the site (such as the presence of retaining walls behind the abutment to support the access embankment), a backfilled abutment is chosen for this design.

IX.2.1. Elements of a Backfilled Abutment

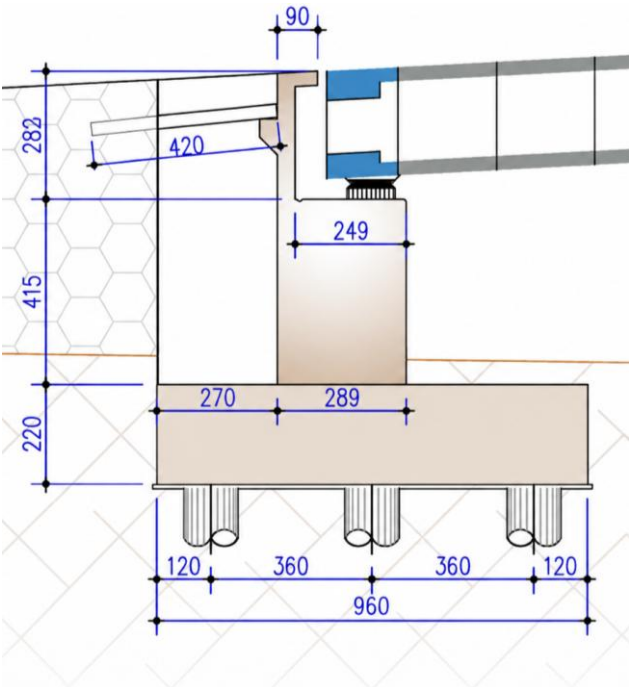
A backfilled abutment consists of the following key elements:

- ❖ **Parapet Wall:** Designed to isolate the deck from contact with the backfill.
- ❖ **Bearing Slab:** Supports the end of the deck and transfers the loads to the foundation.
- ❖ **Protection Wall:** Shields the bearing devices from direct contact with the embankment or external forces.
- ❖ **Front Wall:** Transmits vertical loads from the deck and supports the embankment behind the abutment.
- ❖ **Return Wall:** Wall parallel to the bridge axis, serving to support the backfill.
- ❖ **Rear Corbel:** A trapezoidal-shaped structure at the back, supporting the transition slab.
- ❖ **Front Corbel:** Supports the roadway joint and provides clearance beneath the abutment.
- ❖ **Transition Slab:** A slab implemented to prevent settlements occurring between the roadway and the access ramps.
- ❖ **Footing:** The part that supports loads from the deck as well as its own self-weight.

IX.2.2. Dimensioning of the Abutment

The chosen dimensions are presented in the table on the following page.

Table IX.1. Dimensions of the abutment.



Element	Dimensions (m)	Element	Dimensions (m)
Front wall	$h = 4.15$	Transition slab	$h = 0.30$
	$l = 10.40$		$l = 9.40$
	$e = 2.89$		$b = 4.20$
Parapet wall	$h = 2.82$	Corbel	$b = 0.4$
	$l = 10.40$		$l = 10.40$
	$e = 0.40$		$S = 0.24 \text{ m}^2$
Return wall	$e = 0.50$	Footing	$h = 2.20$
			$l = 10.80$
			$b = 9.60$

IX.2.3. Evaluation of Actions

Two critical sections must be considered for this evaluation:

1. The section at the base of the front wall concerning the reinforcement design ($Z = 2.2 \text{ m}$).
2. The lower face of the footing concerning the foundation's stability ($Z = 0 \text{ m}$).

Since the checks in both directions can be performed independently, only the longitudinal direction check is presented in this chapter. The transverse direction check follows a similar process.

IX.2.3.1. Determination of Static Actions

The calculation of the static earth pressure and the inertia forces, performed with standard assumptions, results in the following parameters:

$$K_a = 0.33 \quad \gamma_{\text{concrete}} = 25 \text{ kN/m}^3 \quad \gamma_{\text{soil}} = 18 \text{ kN/m}^3$$

Table IX.2. Determination of static stresses.

Designation	N (KN)	H (KN)	Ifv(x/o)	Ifh(y/o)	Mfv/o	Mfh/o
Footing	5702.40	/	0.00	1.10	0.00	/
Front wall	3118.31	/	0.66	4.28	2 058.08	/
Transition Slab	148.05	/	3.55	5.92	525.58	/
Return wall	463.50	/	2.79	3.43	1 293.17	/
Corbel	62.40	/	2.28	5.63	142.27	/
Parapet wall	293.28	/	1.90	5.54	557.23	/
Upstream earth weight	3 425.76	/	2.80	3.41	9 592.13	/
Earth weight on transition slab	297.65	/	3.64	6.44	1 083.44	/
Earth pressure	/	1 390.70	/	-2.24	/	-3 110.54
Live load surcharge (10 kN/m ²)	390.60	/	1.45	/	566.37	/
Surcharge pressure (10 kN/m ²)	/	238.18	/	-3.47	/	-826.49

The support reaction of the deck leads to:

Table IX.3. Actions transmitted by the deck.

Designation	N (KN)	H (KN)	Ifv(x/o)	Ifh(y/o)	Mfv/o	Mfh/o
Al	373.00	/	0.00	/	0.00	/
Bc	1 197.50	/	0.00	/	0.00	/
Mc120	1 325.90	/	0.00	/	0.00	/
D240	1 997.80	/	0.00	/	0.00	/
St	80.40	/	0.00	/	0.00	/

Hf+r	/	173.30	/	-6.35	/	-1 098.55
HT°	/	135.40	/	-6.35	/	-857.25
Braking (Al)	/	18.60	/	-6.35	/	-118.11
Braking (Bc)	/	24.50	/	-6.35	/	-155.58

Note: The results of the horizontal forces induced by temperature variations (T), wind (W), and live loads are negligible.

The table below summarizes the load combinations for the Serviceability Limit State (SLS) and Ultimate Limit State (ULS).

Table IX.4. Combinations of actions applied to the abutment.

	Designation	N (KN)	H (KN)	M _H (kN.m)	M _V (kN.m)	ΔM (kN.m)
SLS	G+0.6T	13 511.35	1 644.70	-5 549.93	1 5251.90	9 701.97
	G+0.6T+1.2Q	13 980.07	1 930.52	-6 541.71	1 5931.54	9 389.83
	G+0.6T+1.2A(l)+ST	14 039.35	1 667.02	-5 691.66	1 5251.90	9 560.24
	G+0.6T+1.2Bc+ST	15 028.75	1 674.10	-5 736.62	1 5251.90	9 515.28
	G+0.6T+Mc120	14 837.25	1 644.70	-5 549.93	1 5251.90	9 701.97
	G+0.6T+D240	15 509.15	1 644.70	-5 549.93	1 5251.90	9 701.97
ULS	1.35G+0.9T	18 240.32	2 232.50	-6 453.80	20 590.06	14 136.26
	1.35G+0.9T+1.6Q	18 865.28	2 613.59	-7 776.18	21 496.25	13 720.07
	1.35G+0.9T+1.6(A(l)+ST)	18 965.76	2 262.26	-6 642.78	20 590.06	13 947.29
	1.35G+0.9T+1.6(Bc+ST)	20 284.96	2 271.70	-6 702.72	20 590.06	13 887.34
	1.35G+0.9T+1.35Mc120	20 030.28	2 232.50	-6 453.80	20 590.06	14 136.26
	1.35G+0.9T+1.35D240	20 937.35	2 232.50	-6 453.80	20 590.06	14 136.26

IX.2.3.2. Determination of Seismic Forces

The dynamic calculation of the abutment takes into account: Inertia forces, dynamic earth pressure and loads transmitted from the deck

❖ Calculation of Seismic Inertia Forces on the Abutment:

According to the RPOA, the abutment is divided into elementary volumes on which the horizontal acceleration is applied. The forces are calculated at the base of the front wall and at the lower face of the footing.

The horizontal and vertical seismic forces are determined using the following formulas:

$$F_h = P_p \times k_h \quad F_v = P_p \times k_v$$

Where :

P_p : is the self-weight of the element

$k_h = A$, where A is the dynamic acceleration coefficient, which is equal to 0.2 in our case.

$$k_v = 0.3 \times k_h$$

Thus, the static loads previously calculated are adapted for dynamic conditions as follows:

Table IX.5. Calculation of forces due to the abutment under dynamic conditions.

Designation	N (KN)	H (KN)	I _{FV} (x/o)	I _{FH} (y/o)	M _{FV} /o	M _{FH} /o
Footing	342.14	1 140.48	0.00	1.10	0.00	1 254.53
Front wall	187.10	623.66	0.66	4.28	123.49	2 666.16
Transition Slab	8.88	29.61	3.55	5.92	31.53	175.29
Return wall	27.81	92.70	2.79	3.43	77.59	317.96
Corbel	3.74	12.48	2.28	5.63	8.54	70.26
Parapet wall	17.60	58.66	1.90	5.54	33.43	324.95
Upstream earth weight	205.55	685.15	2.80	3.41	575.53	2 336.37
Earth weight on transition slab	17.86	59.53	3.64	6.44	65.00	383.37
Sum	810.68	2 702.27	/	/	915.11	7 528.89

❖ Calculation of Dynamic Thrust:

According to the RPOA (Regulation for Seismic Design of Structures), during an earthquake, the total dynamic thrust incorporates both the static and dynamic effects of the active earth pressure. The dynamic thrust is given by the following formula:

$$F_{ad} = \frac{1}{2} \gamma (1 \pm K_v) H^2 K_{ad}$$

$$K_{ad} = \frac{\cos^2(\varphi - \theta)}{\cos^2 \theta \left[1 + \sqrt{\frac{\sin \varphi \sin(\varphi - \theta - \beta)}{\cos \theta \cos \beta}} \right]^2}$$

Where :

θ = Angle determined by seismic forces, calculated as: $\theta = \arctan \left(\frac{k_h}{1 \pm k_v} \right)$

ϕ = Angle of internal friction of the embankment ($^\circ$)

k_h = Horizontal seismic coefficient

k_v = Vertical seismic coefficient

β = Angle between the embankment surface and the horizontal (usually $\beta = 0$)

Table IX.6. Coefficient for calculating dynamic earth pressure.

$\gamma(\text{kN/m}^3)$	18.00	$\theta_1^\circ (1+k_v)$	10.68
$\varphi^\circ =$	30.00	$\theta_2^\circ (1-k_v)$	12.01
$k_h =$	0.20	$K_{ad} (1+k_v)$	0.46
$k_v =$	0.06	$K_{ad} (1-k_v)$	0.48

The dynamic thrusts acting on the abutment for both cases are equal to:

Table IX.7. Calculation of dynamic earth pressures.

Designation	N (kN)	H (kN)	$M_{FV/o}$ (kN.m)	$M_{FH/o}$ (kN.m)
$P_{ad} (1+k_v)$	/	2 216.08	/	-5 126.53
$P_{ad} (1-k_v)$	/	2 052.22	/	-4 747.46

❖ Calculation of Seismic Loads Due to the Deck

The forces induced by the deck in the case of an earthquake are determined using the Response Spectrum Method (as defined in [Chapter VIII](#)), utilizing the calculation software MIDAS 2022.

The calculation results for a horizontal earthquake (where the vertical seismic forces are neglected, as discussed in [Chapter VIII](#)) are provided in the table below:

Table IX.8. Seismic forces due to the deck.

	N (kN)	H (kN)	M_V (kN.m)	M_H (kN.m)
Total	Neglected	1 037.10	0.00	-6 585.59

❖ Combination of Seismic Actions on the Abutment

The evaluation of the total seismic action is carried out for three cases:

- **Case 1:** Horizontal earthquake ($1+k_v$)
- **Case 2:** Horizontal earthquake ($1-k_v$)
- **Case 3:** Vertical earthquake

Since all inertial forces act in the same direction, it is appropriate to consider them simultaneously. These forces include the inertia effects of the abutment mass, the soil above the footing, as well as the dynamic earth pressures.

The combination of actions is performed as follows:

❖ **Inertial forces are directly summed:** $F_{\text{inertial}} = \sum F$

where F represents the inertia forces of the abutment, the soil, the footing, and the dynamic thrust.

❖ **Forces in each direction are combined quadratically:** $F = \sqrt{F_{\text{deck}}^2 + F_{\text{inertial}}^2}$

Table IX.9. Evaluation of seismic actions on the abutment.

	N (kN)	H (kN)	M (kN.m)
Case 1	0.00	5 026.50	7 010.08
Case 2	0.00	4 866.28	7 148.87
Case 3	810.68	0.00	915.11

IX.2.3.3. Total Seismic Load Combination

According to RPOA 2008, the load combination at the Seismic Limit State (ALS) is defined as:

$$G + 0.2 \cdot Q + 0.4 \cdot T + E$$

Let :

$$R = G + 0.2 \cdot Q + 0.4 \cdot T$$

Table IX.10. Determination of Rmax and Rmin.

	N (kN)	H (kN)	M (kN.m)
R _{max}	13 910.91	231.90	13 786.83
R _{min}	13 602.03	230.72	13 779.33

It is essential to identify the critical load combinations in order to reduce the extent of the calculations. Therefore, the focus is placed on the following governing combinations:

Table IX.11. Combinations of total seismic actions on the abutment.

	N (kN)	H (kN)	M (kN.m)
R _{max} - E _H + 0.3E _V	14 154.11	5 258.40	7 051.28
R _{min} - E _H - 0.3 E _V	13 358.82	5 097.00	6 355.93
R _{max} + E _V - 0.3 E _H	14 721.59	1 739.85	12 598.91
R _{min} - E _V - 0.3 E _H	12 791.35	1 690.61	10 719.56

IX.2.4. Reinforcement Design of Abutment Elements

IX.2.4.1. Front Wall

The front wall, considered as fixed at the base into the footing, is subjected to combined bending and axial force. It is mainly loaded by:

- Inertia forces
- Earth pressures
- Reactions transmitted by the bridge deck

The vertical reinforcement of the wall is determined using the software ROBOT-Expert, by considering a rectangular section with dimensions: $1.0 \times 2.88 \text{ m}^2$

The unfavorable design forces used for the reinforcement calculation are presented in the following table:

Table IX.12. Unfavorable stresses at ULS and SLS for the front wall.

ULS		SLS	
$M_{\max}$	151.09	$M_{\max}$	117.41
N_{cor}	1 161.91	N_{cor}	860.67
$T_{\max}$	185.72	$T_{\max}$	136.94
N_{cor}	932.95	N_{cor}	691.35

Reinforcement Results

- Vertical reinforcement: $A_s = 28.9 \text{ cm}^2$ Provided as: 6HA25, with spacing $s = 17.5 \text{ cm}$
- Horizontal reinforcement: $A_{st} = \frac{A_s}{4}$ Provided as: 6HA14, with spacing $s = 17.5 \text{ cm}$
- Secondary (construction) reinforcement: $A'_s = \frac{A_s}{3}$ Provided as: 6HA16, with spacing $s = 17.5 \text{ cm}$

IX.2.4.2. Parapet Wall

The parapet wall, assumed to be fixed at its base, is designed under simple bending due to the following horizontal actions:

- Earth pressure
- Pressure from a localized surcharge behind the wall
- Braking force

❖ **Moment Due to Earth Pressure**

The bending moment at the fixed base due to earth pressure is expressed as: $M_p = \frac{P \cdot H}{3}$

With :
$$P = \frac{1}{2} K_a \gamma H^2 L$$

Where :

P: Earth pressure force

K_a: Active earth pressure coefficient **K_a = 0.33**

γ: Unit weight of the backfill **γ = 18 kN/m³**

H: Height of the parapet wall **H = 2.82 m**

L: Unit length (per linear meter) **L = 1 m**

Final Results :

M_p = 22.20 kN.m/m at SLS

M_p = 29.97 kN.m/m at ULS

❖ Moment Due to Local Surcharge

The bending moment at the fixed base due to a localized load is given by:

$$M_{Pc} = \frac{12K}{0.75 + 2h} \int_0^h \frac{h-x}{0.25+x} dx$$

Where : **K = K_a × δ × γ**

With :

γ: Load factor (**γ = 1.2** at SLS and **γ = 1.6** at ULS)

δ: Dynamic amplification factor **δ = 1.06**

h: Height of the parapet wall

Values of K :

K = 0.51 at SLS

K = 0.68 at ULS

The values of M_{Pc}/K for different values of **h** are presented in the following table:

Table IX.13. Moment M_{Pc}/K due to a local load as a function of h.

h (m)	0.50	0.75	1.00	1.50	2.00	2.50	3.00
M_{Pc} /K (t.m/ml)	2.23	3.40	4.41	6.11	7.45	8.56	9.49

By performing a linear interpolation for **h = 2.82 m**, we obtain: $\frac{M_{Pc}}{K} = 9.16 \text{ t.m}$

Thus :

M_{Pc} = 46.64 kN.m/m at SLS

M_{Pc} = 62.18 kN.m/m at ULS

❖ Moment Due to Braking Force

The braking effect of a single wheel from an exceptional convoy load is considered. The fixed-end moment is given by:

$$M_f = \frac{2\mu \cdot P \cdot h}{0.25 + 2h}$$

Where :

μ : Load factor ($\mu = 1.2$ at SLS and $\mu = 1.6$ at ULS)

P : Load per wheel $P = 60$ kN

h : Height of the wall $h = 2.82$ m

Results :

$M_f = 68.94$ kN.m/m at SLS

$M_f = 91.93$ kN.m/m at ULS

❖ Total Bending Moment

The total moment at the fixed base is obtained by summing all previously calculated contributions:

$M = 137.78$ kN.m/m at SLS

$M = 184.08$ kN.m/m at ULS

❖ Reinforcement Design

The main reinforcement is determined using ROBOT-Expert, assuming simple bending of a rectangular section with dimensions: 1.0×0.40 m²

Results :

- Main reinforcement (backfill side): $A_s = 17.6$ cm² Provided as: 6HA20, with spacing $s = 17.5$ cm
- Secondary reinforcement (deck side): $A'_s = \frac{A_s}{3}$ Provided as: 6HA12, with spacing $s = 17.5$ cm
- Transverse reinforcement: $A_{st} = \frac{A_s}{4}$ Provided as: 6HA10, with spacing $s = 17.5$ cm

IX.2.4.3. Return Wall

The return wall ensures the retention of the embankment soil at the bridge approach. The structural analysis of the wall is carried out using ROBOT STRUCTURAL ANALYSIS SOFTWARE 2026 VERSION. The wall is assumed to be fixed at both ends and is subjected to the following loads:

- Earth pressure: $P_{soil} = 0.5 \times (0.333 \times 18 \times 6.94^2 \times 1) = 144.35$ kN/m
- Surcharge pressure: $P_{sr} = 3.33$ kN/m²
- CCP weight: $P_{eq} = 13.15$ kN/m.
- sidewalk overload $S_t = 0.75$ kN/m.
- Self-weight of the wall

The numerical model of the return wall developed using the software is illustrated in the following figure.

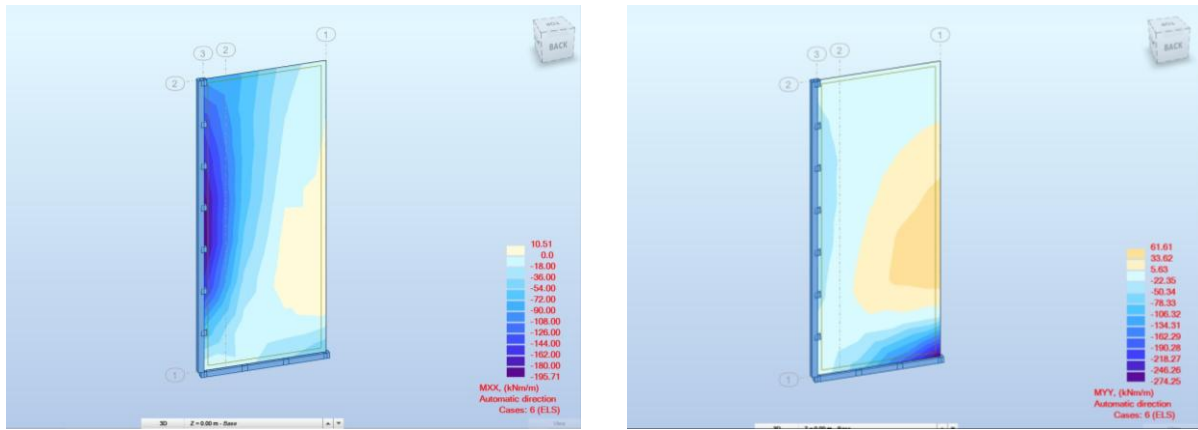


Figure IX.1. Modeling of the return wall / Mxx and Myy moments.

The reinforcement design is carried out using ROBOT-Expert software, and the results are as follows:

Table IX.14. Calculation of reinforcement for the return wall.

Designation	M _{max.SLS} (kN.m/ml)	M _{max.ULS} (kN.m/ml)	As (cm ²)	Reinforcement
Horizontal	113.76	161.39	11.00	6HA16
Vertical	90.00	113.42	8.60	5HA16

IX.2.4.4. Transition Slab

The transition slab is a reinforced concrete slab placed beneath the road surface at the ends of the bridge. Its primary function is to prevent any settlement or differential settlement that may occur due to the potential compression of the backfill behind the abutment.

The slab is treated as a simply supported rectangular slab, although in reality, it is supported by elastic foundations across its entire surface (in direct contact with the ground). The slab is subjected to its own weight, superimposed loads, and the weight of the embankment above it.

Table IX.15. Evaluation of forces on the transition slab.

Load type	Q (kN/ml)	M (kN.m/ml)	V (kN)
Self-Weight	7.50	16.54	15.75
Pavement	1.92	4.23	4.03
Backfill	14.80	32.63	31.08
Superimposed loads	10.00	22.05	21.00

Results :

$$\text{Service Bending Moment: } M_{\text{ser}} = M_G + 1.2 \times M_Q = 79.87 \text{ kN}\backslash\text{m/m}$$

$$\text{Service Shear Force: } V_{\text{ser}} = V_G + 1.2 \times V_Q = 76.06 \text{ kN/m}$$

$$\text{Ultimate Bending Moment: } M_{\text{ult}} = 1.35 \times M_G + 1.6 \times M_Q = 107.38 \text{ kN}\backslash\text{m/m}$$

Ultimate Shear Force: $V_{ult} = 1.35 \times V_G + 1.6 \times V_Q = 102.26 \text{ kN/m}$

The main reinforcement is calculated using ROBOT-Expert software, assuming simple bending for a rectangular section with dimensions $1.0 \times 0.3 \text{ m}^2$.

Results :

- Main Reinforcement: $A_s = 14.4 \text{ cm}^2$ Provided as: 5HA20, with spacing $s = 20 \text{ cm}$
- Construction Reinforcement: $A'_s = \frac{A_s}{3}$ Provided as: 5HA12, with spacing $s = 20 \text{ cm}$
- Transverse Reinforcement: $A_{st} = \frac{A_s}{4}$ Provided as: 5HA12, with spacing $s = 20 \text{ cm}$

IX.2.4.5. Corbel

The corbel is a support for the transition slab. It is subjected to the reactions from the permanent loads and the surcharge from the embankment. The reaction at the supports of a simply supported beam is given by the formula: $R = \frac{q \cdot L}{2}$

Where :

L : Length of the transition slab $L = 4.20 \text{ m}$

$q = 24.22 \text{ kN/m}$ for permanent loads

$q = 10 \text{ kN/m}$ for surcharge on embankment

For the reinforcement design, the crow is assumed to work in axial compression.

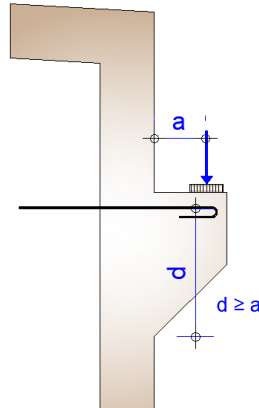


Figure IX.2. Upper reinforcement of the corbel.

The reactions at the limit states are calculated as follows:

At SLS: $R = 76.06 \text{ kN/m}$

At ULS: $R = 102.26 \text{ kN/m}$

The main reinforcement is calculated using ROBOT-Expert software for a rectangular section with dimensions $1.0 \times 0.60 \text{ m}^2$.

Results :

- Main Reinforcement: $A_s = 11.2 \text{ cm}^2$ Provided as: 5HA20, with spacing $s = 20 \text{ cm}$
- Construction Reinforcement: $A'_s = \frac{A_s}{3}$ Provided as: 5HA12, with spacing $s = 20 \text{ cm}$
- Transverse Reinforcement: $A_{st} = \frac{A_s}{4}$ Provided as: 5HA12, with spacing $s = 20 \text{ cm}$

IX.2.5. Foundation Calculation

The number of piles required to transfer the loads to the ground and their depth depends on the bearing capacity of a pile, which is determined by the tip resistance q_u and the unit friction resistance q_s , as given in the geotechnical report, along with the vertical load due to the deck and the abutment.

IX.2.5.1. Estimating the Pile Bearing Capacity

According to the Fascicule No. 62, the bearing capacity of a pile is determined by the tip resistance Q_{pu} and the unit friction resistance Q_{su} . It is calculated as follows:

$$\begin{cases} Q_{pu} &= \frac{\pi D^2}{4} \cdot q_{pu} \\ Q_{su} &= \pi D \int_{z_t}^{z_p} q_{su} dz \end{cases}$$

Where :

- q_{pu} : Point rupture stress
- q_{su} : Unit friction resistance
- z_t : Depth of the top of the pile
- z_p : Depth of the base of the pile

Note :

- The values of q_{pu} and q_{su} are determined from pressuremeter tests, and these values will be provided in the annex of the report.
- After reviewing the pressuremeter test results from the geotechnical report, a pile depth of 20 meters has been chosen for the abutment piles.
- At this depth, the piles will be anchored in a very hard marl layer, which showed limit pressures 1.4 MPa which will be more than sufficient to carry the loads.

In accordance with the guidance provided in Chapter C.4 of Fascicule No. 62, the axial load at the top of the pile must remain between two limits, Q_{min} and Q_{max} , defined below based on the combination of actions considered:

Table IX.16. Q_{max} and Q_{min} for each limit state.

Limit State	Q_{min}	Q_{max}
ULS (Fundamental)	- $Q_{tu} / 1.4$	$Q_u / 1.4$
SLS (Rare)	- $Q_{tc} / 1.4$	$Q_c / 1.1$
ALS (Seismic)	- $Q_{tu} / 1.3$	$Q_u / 1.2$

Where :

- Q_u : Ultimate compressive load capacity of the pile $Q_u = Q_{pu} + Q_{su}$
- Q_{tu} : Ultimate tensile load capacity of the pile $Q_{tu} = Q_{su}$
- Q_c : Creep (service) load in compression $Q_c = 0.5 Q_{pu} + 0.7 Q_{su}$

Q_{tc} : Creep (service) load in tension $Q_{tc} = 0.7 Q_{su}$

The calculated bearing capacities at the ULS and the SLS are presented in the following table:

Table IX.17. Estimation of the bearing capacity of the abutment piles.

ULS		Designation	Value
$Q_{\max}$	3 756.43 kN	q_{pu}	1800 kPa
$Q_{\min}$	- 2 302.14 kN	Σq_{su}	855 kPa
SLS		Q_{su}	3 223 kN
$Q_{\max}$	2 976.45 kN	Q_{pu}	2 036 kN
$Q_{\min}$	- 1 611.50 kN	Q_u	5 259 kN
ALS		Q_c	3 274.10 kN
$Q_{\max}$	4 382.50 kN	Q_{tu}	3 223 kN
$Q_{\min}$	- 2 479.23 kN	Q_{tc}	2 256.10 kN

IX.2.5.2. Calculation of Actions on Piles

The load applied to each pile is determined based on the total number of piles and their spacing, as follows:

$$R_i = \frac{N}{n} \pm \frac{M_x \cdot y_i}{\Sigma(y_i^2)} \pm \frac{M_y \cdot x_i}{\Sigma(x_i^2)}$$

Where :

R_i : Load carried by each pile

N : Total axial load

n : Total number of piles, with: $n \geq \frac{N}{Q_{\max}}$

M_x, M_y : Bending moments about the x and y axes

x_i, y_i : Coordinates of each pile relative to the centroid of the footing

The adopted pile configuration is as follows:

- Number of piles: Three rows of 3 piles (total = 9 piles)
- Spacing: 3.60 m
- Diameter: 1.2 m
- Depth: 20 m

The actions on the piles at the following limit states are presented in the tables below:

Table IX.18. Estimation of actions on abutment piles at SLS.

SLS	1 st row	2 nd row	3 rd row
	R ₁ (kN)	R ₂ (kN)	R ₃ (kN)
G+0.6T	1 147.38	1 501.26	1 855.14
G+0.6T+1.2Q	1 213.91	1 553.34	1 892.77
G+0.6T+1.2A(l)+ST	1 212.61	1 559.93	1 907.25
G+0.6T+1.2Bc+ST	1 324.62	1 669.86	2 015.10
G+0.6T+Mc120	1 294.70	1 648.58	2 002.47
G+0.6T+D240	1 369.36	1 723.24	2 077.12

Table IX.19. Estimation of actions on abutment piles at ULS.

ULS	1 st row	2 nd row	3 rd row
	R ₁ (kN)	R ₂ (kN)	R ₃ (kN)
1.35G+0.9T	1 500.88	2 026.70	2 552.53
1.35G+0.9T+1.6Q	1 589.58	2 096.14	2 602.70
1.35G+0.9T+1.6(A(l)+ST)	1 590.23	2 107.31	2 624.38
1.35G+0.9T+1.6(Bc+ST)	1 739.58	2 253.88	2 768.19
1.35G+0.9T+1.35Mc120	1 699.76	2 225.59	2 751.41
1.35G+0.9T+1.35D240	1 800.55	2 326.37	2 852.20

Table IX.20. Estimation of actions on abutment piles at ALS.

ALS	1 st row	2 nd row	3 rd row
	R ₁ (kN)	R ₂ (kN)	R ₃ (kN)
R _{max} - E _H + 0.3 E _V	1 343.23	1 572.68	1 802.13
R _{min} + E _H - 0.3 E _V	1 283.62	1 484.31	1 685.00
R _{max} - E _V - 0.3 E _H	1 153.45	1 635.73	2 118.02
R _{min} - E _V - 0.3 E _H	1 014.55	1 421.26	1 827.97

Observations : It can be clearly observed that:

- Maximum pile loads:

$$R_{\max, \text{SLS}} = 2\,077.12 \text{ kN} < Q_{\max, \text{SLS}} = 2\,976.45 \text{ kN}$$

$$R_{\max, \text{ULS}} = 2\,852.20 \text{ kN} < Q_{\max, \text{ULS}} = 3\,756.43 \text{ kN}$$

$$R_{\max, \text{ALS}} = 2\,118.02 \text{ kN} < Q_{\max, \text{ALS}} = 4\,382.50 \text{ kN}$$

⇒ No risk of punching failure

- Minimum pile loads:

$$R_{\min, \text{SLS}} = 1\,147.38 \text{ kN} > 0$$

$$R_{\min, \text{ULS}} = 1\,590.23 \text{ kN} > 0$$

$$R_{\min, \text{ALS}} = 1\,014.55 \text{ kN} > 0$$

⇒ No risk of uplift

IX.2.5.3. Reinforcement of the Abutment Footing

The footing is a key element of the substructure, performing a dual function:

- ensuring the fixity of the support,
- and distributing the loads to the piles.

For the reinforcement of the footing, the strut-and-tie method is used, governed by the following conditions:

$$\left\{ \begin{array}{l} \alpha \geq 45^\circ \\ h \geq \frac{L}{2} - \frac{b}{4} \\ h = H - d' \end{array} \right.$$

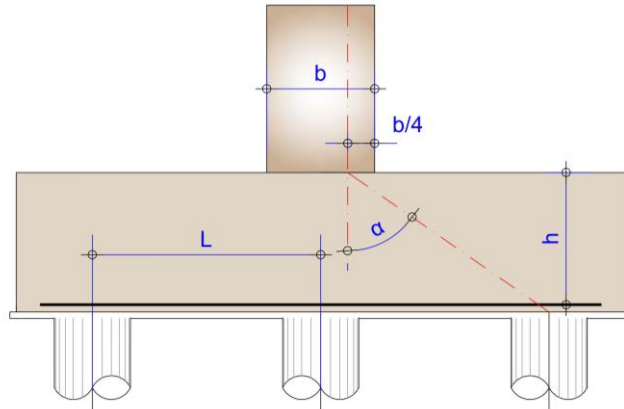


Figure IX.3. Notations of the strut-and-tie method.

Where :

L: spacing between piles

b: thickness of the front wall

Numerical Example :

$$L = 3.60 \text{ m} ; b = 2.89 \text{ m} ; H = 2.20 \text{ m}$$

$$d' = 0.1 \text{ m} ; h = 2.10 \text{ m}$$

$$\frac{L}{2} - \frac{b}{4} = 1.08 \text{ m}$$

$$\alpha_1 = \alpha_2 = \arctan\left(\frac{h}{\frac{L}{2} - \frac{b}{4}}\right) = 62.78^\circ \Rightarrow \text{Condition satisfied}$$

According to the SETRA document, the required area of lower transverse reinforcement determined by the strut-and-tie method is:

$$A_s = \frac{R_{\max}}{\bar{\sigma}_a} \times \frac{\frac{L}{2} - \frac{b}{4}}{h}$$

Numerical Example :

$$\begin{aligned} R_{\max, SLS} &= 2\,077.12 \text{ kN} ; R_{\max, ALS} = 2118.02 \text{ kN} \\ \bar{\sigma}_{a, SLS} &= \frac{2}{3} f_e = 333.33 \text{ MPa} ; \bar{\sigma}_{a, ALS} = f_e = 500 \text{ MPa} \\ A_{s, SLS} &= 32.09 \text{ cm}^2 ; A_{s, ALS} = 21.81 \text{ cm}^2 \\ \Rightarrow A_s &= 32.09 \text{ cm}^2 \end{aligned}$$

Transverse reinforcement placed in the strips centered on the piles, with a width (L) defined as $L = H + \Phi_{\text{pile}}$.

The adopted reinforcement layout is as follows:

- Lower transverse reinforcement: $A_s = \frac{A_{s1}}{3.4} = 9.44 \text{ cm}^2/\text{m}$ Adopted: 4HA25 bars with a spacing of 25 cm
- Upper transverse reinforcement: $A_{s1} = \frac{A_s}{3} = 3.15 \text{ cm}^2/\text{m}$ Adopted: 4HA14 bars with a spacing of 25 cm
- Lower longitudinal reinforcement: $A_{s2} = \frac{A_s}{2} = 4.72 \text{ cm}^2/\text{m}$ Adopted: 4HA16 bars with a spacing of 25 cm
- Upper longitudinal reinforcement: $A_{s3} = \frac{A_{s1}}{4} = 2.36 \text{ cm}^2/\text{m}$ Adopted: 4HA12 bars with a spacing of 25 cm
- Side reinforcement: $A_{s4} = \frac{A_s}{10} = 0.974 \text{ cm}^2/\text{m}$ Adopted: 4HA10 bars with a spacing of 25 cm

IX.2.5.4. Reinforcement of Abutment Piles

The pile is a reinforced concrete element subjected to combined bending and axial loading. Its behavior in the soil is governed by the following differential equation:

$$EI \frac{d^4 y}{dx^4} + C_u b y = 0$$

Where :

- b**: Diameter of the pile
- C_u**: Subgrade reaction modulus
- y**: Lateral displacement of the pile

The solution of the equation introduces the parameter: $\alpha = \sqrt[4]{\frac{EI}{C_u b}}$

Where :

α: represents the elastic length of the pile

The pile is subjected to bending moments along its length. Assuming zero displacement and rotation at the pile head (fixed-head piles), the bending moment is determined using the Winkler–Werner method:

$$\bar{M} = -\frac{\chi_{\theta P}}{\chi_{\theta M}} \cdot \frac{\bar{P}_{\theta P}}{\lambda}$$

With :

$$\lambda = \sqrt[4]{\frac{C_u b}{4EI}}$$

Where :

λ : Damping coefficient (Werner parameter)

$\chi_{\theta P}, \chi_{\theta M}$: Coefficients obtained from Werner charts, depending on $\lambda \cdot L$

$\bar{P}$: Shear force at the pile head: $\bar{P} = \frac{H}{n}$

H : Total horizontal force

n : Number of piles

Results :

The calculated values of internal forces and moments in the piles are summarized in the following table:

Table IX.21. Determination of stresses at the head of abutment piles.

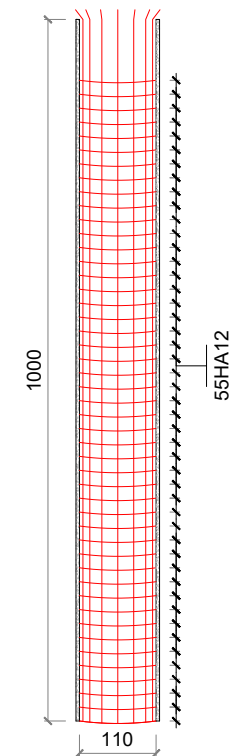
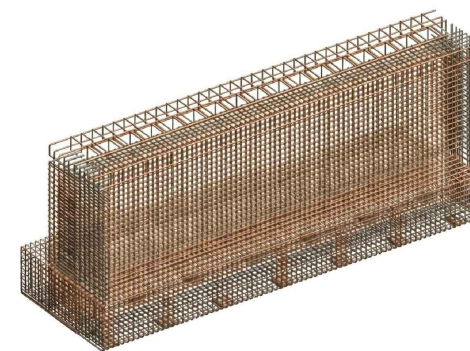
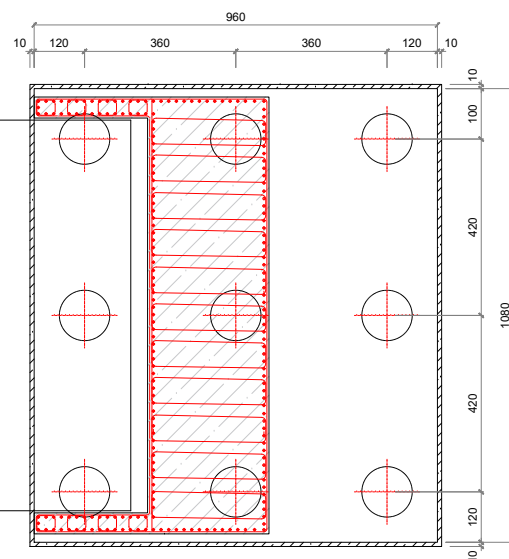
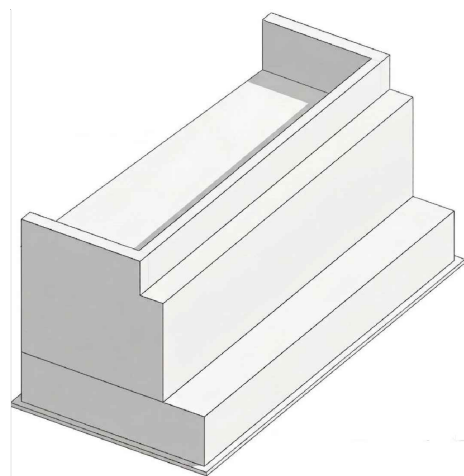
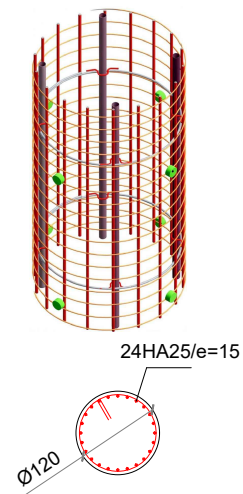
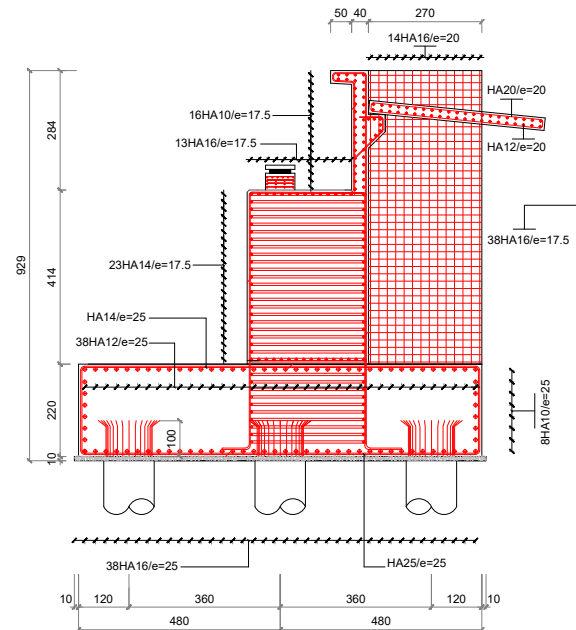
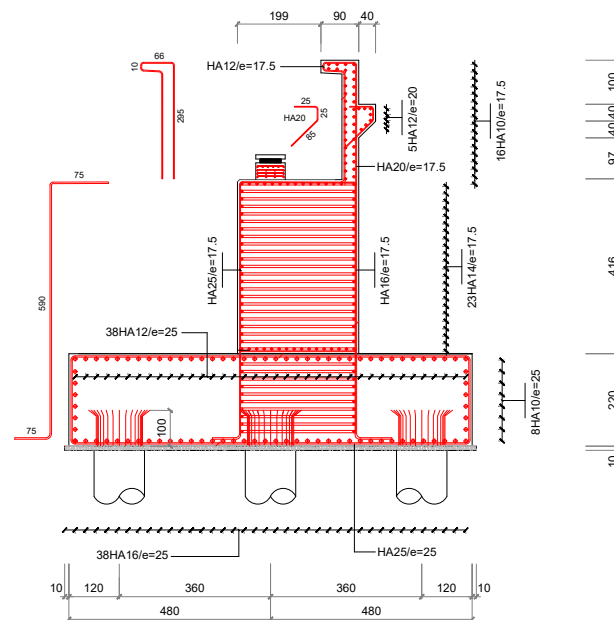
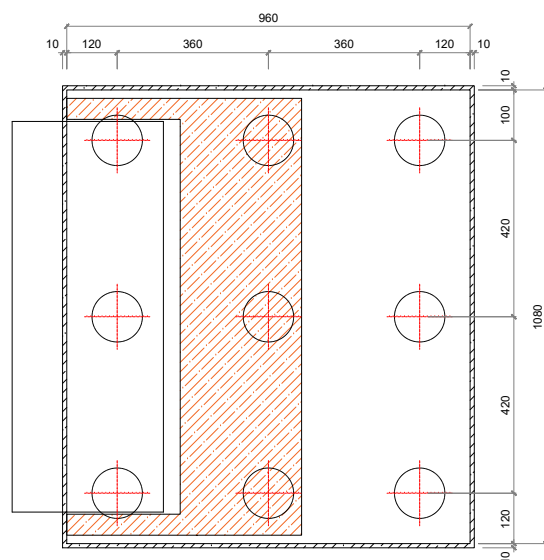
L (m)	20	λ	0.229
E (MPa)	32 164	$\lambda \times L$	4.58
C_u (MN/m³)	30	$\chi_{\theta P}$	1.32
b (m)	1.20	$\chi_{\theta M}$	1.57
I (m⁴)	0.1018	$\bar{P}$ (kN)	584.26
n	9	$\bar{M}$ (kN.m)	2 134.62

The reinforcement design for the piles is performed using ROBOT-Expert software for a circular section with a diameter of 1.2 meters, subjected to simple bending.

Results :

- Main Reinforcement: $A_s = 113.10 \text{ cm}^2$ we opt 24HA25, with a spacing of $S_t = 15 \text{ cm}$.
- Horizontal Reinforcement: $A_{st} = \frac{A_s}{4}$ For the stirrups, we use 14HA/ml, with 12.5 cm spacing in the critical zone and 20 cm in the normal zone.

The figure below illustrates the reinforcement details for the abutment, pile and its foundation:



IX.3. PIER STUDY

Piers are intermediate supports that transfer loads from the superstructure to the foundations and, ultimately, to the underlying soil. They play a crucial role in the structural behavior of the bridge deck, which may be simply supported or partially restrained depending on the design.

For the present structure, wall-type piers are adopted. These piers have a rectangular cross-section with rounded connections at the edges, which improves both their structural performance and aerodynamic behavior while reducing stress concentrations.

Table IX.22. Dimensions of the piers.

Element		Value / Description	Figure
Section	Shape:	Rectangular with circular connections	
	Dimensions	$3.82 \times 1.60 \text{ m}^2$ $\Phi = 1.60 \text{ m}$	
Height	P1	6.18 m	
	P2	6.75 m	
	P3	6.27 m	
	P4	6.16 m	

IX.3.1. Determination of Actions

Two critical sections need to be considered:

1. The section at the base of the anchor block, concerning the reinforcement of the block itself ($Z = 2.2 \text{ m}$).
2. The lower face of the foundation slab, concerning the equilibrium of the foundation ($Z = 0 \text{ m}$).

IX.3.1.1. Determination of Actions under Static Loading

The calculation of inertia loads, carried out with the classical assumptions, leads to:

Table IX.23. Forces due to inertial loads.

Designation	N (KN)	H (KN)	IFV(x/o)	IFH(y/o)	MFV/o	MFH/o
Footing	5 068.80	/	0.00	1.10	0.00	/
Pier	1 207.43	/	0.00	5.58	0.00	/
Sum	6 276.23	/	/	/	0.00	/

Table IX.24. Forces due to the deck.

Designation	N (KN)	H (KN)	I _{FV} (x/o)	I _{FH} (y/o)	M _{FV/o}	M _{FH/o}
G	13 026.90	/	0.00	/	0.00	/
Al	1 559.70	/	0.00	/	0.00	/
Bc	944.60	/	0.00	/	0.00	/
Bt	516.80	/	0.00	/	0.00	/
Mc120	886.40	/	0.00	/	0.00	/
D240	1 894.80	/	0.00	/	0.00	/
St	316.80	/	0.00	/	0.00	/
Hf+r	/	170.00	/	8.85	/	1 521.50
HT°	/	132.00	/	8.85	/	1 181.40
Braking (Al)	/	27.80	/	8.85	/	248.81
Braking (Bc)	/	33.60	/	8.85	/	300.72

Note: The values of the load combinations for the pier at SLS and the ULS are provided in Appendix C.

IX.3.1.2. Evaluation of Seismic Forces Acting on the Pier

Table IX.25 summarizes the seismic actions applied to the piers, including inertia forces generated by the structure and seismic reactions transmitted from the bridge deck.

Table IX.25. Evaluation of seismic actions on the pier.

Designation	N (KN)	H _L (KN)	H _T (KN)	M _N (kN.m)	M _L (kN.m)	M _T (kN.m)
Pier	72.45	241.49	241.49	0.00	1 345.10	1 345.10
Footing	304.13	1 013.76	1 013.76	0.00	1 115.14	1 115.14
Seismic deck	0.00	1 603.00	1 858.50	0.00	10 138.10	15 153.00

In this case, the actions are combined quadratically: $F = \sqrt{F_{deck}^2 + F_{inertielles}^2}$

IX.3.1.3. Combinations of Seismic Actions on the Pile

According to the RPOA 2008, the seismic action combination is defined as follows:

$$G + 0.2 \cdot Q + 0.4 \cdot T + E$$

It is recommended to combine these actions in order to determine the most unfavorable case.

IX.3.1.4. Combinations of Seismic Directions

According to the RPOA, the effects of different components of the overall motion are combined as follows:

$$E_1 = E_L + 0.3 \cdot (E_T + E_V)$$

$$E_2 = E_T + 0.3 \cdot (E_L + E_V)$$

$$E_3 = E_V + 0.3 \cdot (E_L + E_T)$$

Table IX.26. Combinations of seismic directions.

	E_L	E_T	E_V	E_1	E_2	E_3
H_L (kN)	2 035.99	0.00	0.00	2 035.99	610.80	610.80
H_T (kN)	0.00	2 242.69	0.00	672.81	2 242.69	672.81
M_L (kN.m)	10 242.72	0.00	0.00	10 242.72	3 072.82	3 072.82
M_T (kN.m)	0.00	15 223.20	0.00	4 566.96	15 223.20	4 566.96
N (kN)	0.00	0.00	376.58	112.97	112.97	376.58

IX.3.1.5. Calculation of forces from the deck

The calculation for the efforts coming from the deck in the dynamic case is described as follows, according to the RPOA:

$$R = G + 0.2 \cdot Q + 0.4 \cdot T$$

Table IX.27. Forces from the deck.

	R_{max}	R_{min}
H_L (kN)	229.52	228.36
H_T (kN)	0.00	0.00
M_L (kN.m)	1 549.26	1 541.43
M_T (kN.m)	0.00	0.00
N (kN)	13 405.86	13 204.18

For the pier and foundation analysis, we have 6 combinations that account for the maximum and minimum possible effects of seismic forces:

$$E_1 = E_1 + R_{max} + PP_{PS}$$

$$E_2 = E_2 + R_{max} + PP_{PS}$$

$$E_3 = E_3 + R_{\max} + PP_{PS}$$

$$E_4 = -E_1 + R_{\min} + PP_{PS}$$

$$E_5 = -E_2 + R_{\min} + PP_{PS}$$

$$E_6 = -E_3 + R_{\min} + PP_{PS}$$

Where :

PP_{PS}: The normal load on the pier and the foundation, which is 6 276.23 kN.

Table IX.28. Seismic stresses on the pier.

	E ₁	E ₂	E ₃	E ₄	E ₅	E ₆
H _L (kN)	2 265.51	840.32	840.32	-1 807.63	-382.44	-382.44
H _T (kN)	672.81	2 242.69	672.81	-6 72.81	-2 242.69	-672.81
M _L (kN.m)	11 791.98	4 622.08	4 622.08	-8 701.29	-1 531.39	-1 531.39
M _T (kN.m)	4 566.96	15 223.2	4 566.96	-4 566.96	-15 223.2	-4 566.96
N (kN)	19 795.06	19 795.06	20 058.67	19 367.44	19 367.44	19 103.83

IX.3.2. Pier Reinforcement:

The pier is subjected to combined biaxial bending, with a normal force from the deck and bending moments.

Table IX.29. Evaluation of maximum pier forces at limit states.

SLS		ULS		ALS	
M _{L,max}	2 591.20	M _{L,max}	3 598.44	M _{L,max}	11 791.98
M _{T,max}	0.00	M _{T,max}	0.00	M _{T,max}	15 223.20
T _{max}	289.52	T _{max}	402.06	T _{max}	2 265.51
N _{cor}	20 753.45	N _{cor}	28 077.47	N _{cor}	19 795.06

The main reinforcement is calculated using the ROBOT-Expert software for a rectangular section with two circular connections, subjected to bending with combined deflection. The results are as follows:

- Main Reinforcement: $A_s = 17 \text{ cm}^2/\text{ml}$ we opt 4HA25/ml, with a spacing of $S_t = 25 \text{ cm}$.
- Horizontal Reinforcement: $A_{st} = \frac{A_s}{4}$ For the stirrups, we use HA12/ml, with 20 cm spacing .

IX.3.3. Foundation Calculation:

The determination of the number of piles required to transfer loads to the soil and the necessary depth depends on the bearing capacity of the pile. This is calculated based on point resistance (q_{pu}) and unit side friction resistance (q_{su}), which are determined from the geotechnical report.

IX.3.3.1. Estimating the Pile Bearing Capacity

We will follow the same approach as in section [IX.2.5.1](#) to calculate the bearing capacities of the pile foundations under the limit states.

The results of these calculations are shown in the table below:

Table IX.30. Calculation of the bearing capacity of pier piles.

ULS		Designation	Value
Q_{max}	4 437.14 kN	q_{pu}	1 893 kPa
Q_{min}	- 2 907.86 kN	Σq_{su}	1 080 kPa
SLS		Q_{su}	4 071 kN
Q_{max}	3 563.82 kN	Q_{pu}	2 141 kN
Q_{min}	- 2 035.50 kN	Q_u	6 212 kN
ALS		Q_c	3 920.20 kN
Q_{max}	5 176.67 kN	Q_{tu}	4 071 kN
Q_{min}	- 3 131.54 kN	Q_{tc}	2 849.70 kN

IX.3.3.2. Calculation of Actions on Piles

For the pile foundation, due to the significant forces transmitted, we opt for a configuration with three rows of 9 piles. One row will be positioned along the centerline of the foundation slab, while the other two rows will be spaced 3.6 meters apart.

The piles have the following specifications:

- Diameter: 1.2 meters.
- Depth: 20 meters.
- Total number of piles: 9 piles.

The calculations for actions on the piles are based on the approach outlined in [IX.2.5.2](#) for limit states. The results of these calculations are detailed in the corresponding tables below.

Table IX.31. Calculation of actions on pier piles at SLS.

SLS	1 st row	2 nd row	3 rd row
	R ₁ (kN)	R ₂ (kN)	R ₃ (kN)
G+0.6T	2 041.54	2 144.79	2 248.05
G+0.6T+1.2A(l)+ST	2 270.87	2 387.95	2 505.03
G+0.6T+1.2Bc+ST	2 185.98	2 305.94	2 425.90
G+0.6T+Mc120	2 140.02	2 243.28	2 346.54
G+0.6T+D240	2 252.07	2 355.33	2 458.58

Table IX.32. Calculation of actions on pier piles at ULS.

ULS	1 st row	2 nd row	3 rd row
	R ₁ (kN)	R ₂ (kN)	R ₃ (kN)
1.35G+0.9T	2 751.15	2 895.47	3 039.79
1.35G+0.9T+1.6(A(l)+ST)	3 066.32	3 229.07	3 391.82
1.35G+0.9T+1.6(Bc+ST)	2 953.12	3 119.72	3 286.31
1.35G+0.9T+1.35Mc120	3 035.37	3 179.69	3 324.01
1.35G+0.9T+1.35D240	2 798.67	2 942.99	3 087.31

Table IX.33. Calculation of actions on pier piles at ALS.

ALS	1 st row	2 nd row	3 rd row
	R ₁ (kN)	R ₂ (kN)	R ₃ (kN)
E ₁	1 653.53	2 199.45	2 745.38
E ₂	1 985.47	2 199.45	2 413.44
E ₃	2 014.76	2 228.74	2 442.73
E ₄	2 554.78	2 151.94	1 749.10
E ₅	2 222.84	2 151.94	2 081.04
E ₆	2 193.55	2 122.65	2 051.75

Observations: It can be clearly observed that:

- Maximum pile loads:

$$R_{\max, SLS} = 2\,505.03 \text{ kN} < Q_{\max, SLS} = 3\,563.82 \text{ kN}$$

$$R_{\max, ULS} = 3\,391.82 \text{ kN} < Q_{\max, ULS} = 4\,437.14 \text{ kN}$$

$$R_{\max, ALS} = 2\,745.38 \text{ kN} < Q_{\max, ALS} = 5\,176.67 \text{ kN}$$

⇒ No risk of punching failure

- Minimum pile loads:

$$R_{\min, SLS} = 2\,041.54 \text{ kN} > 0$$

$$R_{\min, ULS} = 2\,751.15 \text{ kN} > 0$$

$$R_{\min, ALS} = 1\,653.53 \text{ kN} > 0$$

⇒ No risk of uplift

IX.3.3.3. Pile Footing Reinforcement

The reinforcement of the pile footing is calculated using the same method applied for the abutment footing.

The adopted reinforcement layout is as follows:

- Lower transverse reinforcement: $A_s = \frac{A_s}{3.4} = 14.47 \text{ cm}^2/\text{m}$ Adopted: 4HA25 bars with a spacing of 25 cm
- Upper transverse reinforcement: $A_{s1} = \frac{A_s}{3} = 4.82 \text{ cm}^2/\text{m}$ Adopted: 4HA14 bars with a spacing of 25 cm
- Lower longitudinal reinforcement: $A_{s2} = \frac{A_s}{2} = 7.24 \text{ cm}^2/\text{m}$ Adopted: 4HA16 bars with a spacing of 25 cm
- Upper longitudinal reinforcement: $A_{s3} = \frac{A_{s1}}{4} = 3.62 \text{ cm}^2/\text{m}$ Adopted: 4HA12 bars with a spacing of 25 cm
- Side reinforcement: $A_{s4} = \frac{A_s}{10} = 1.45 \text{ cm}^2/\text{m}$ Adopted: 4HA10 bars with a spacing of 25 cm

IX.3.3.4. Pile Footing Reinforcement

The reinforcement of the pile footing is calculated using the same method applied for the abutment footing.

Table IX.34. Determination of stresses at the head of pier piles.

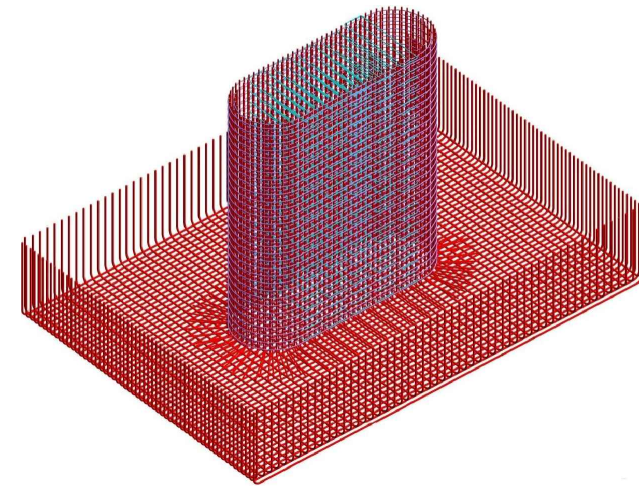
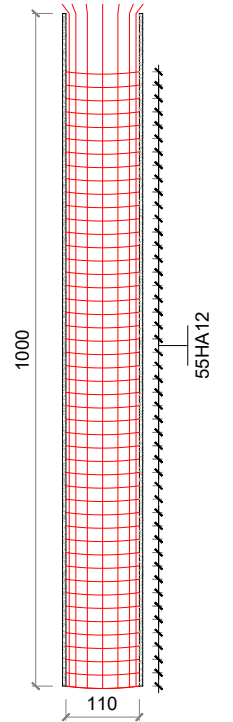
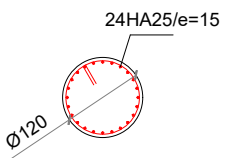
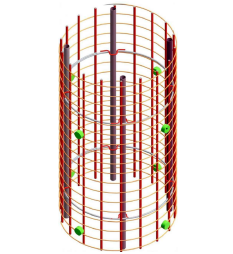
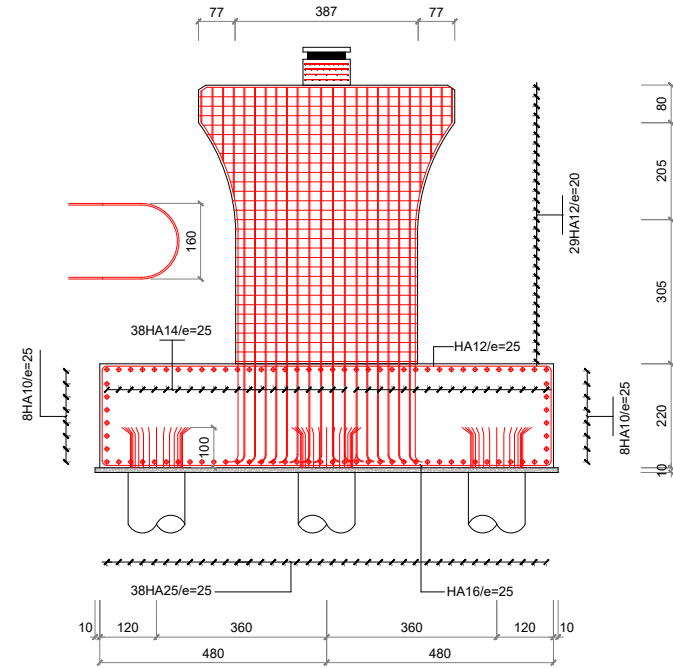
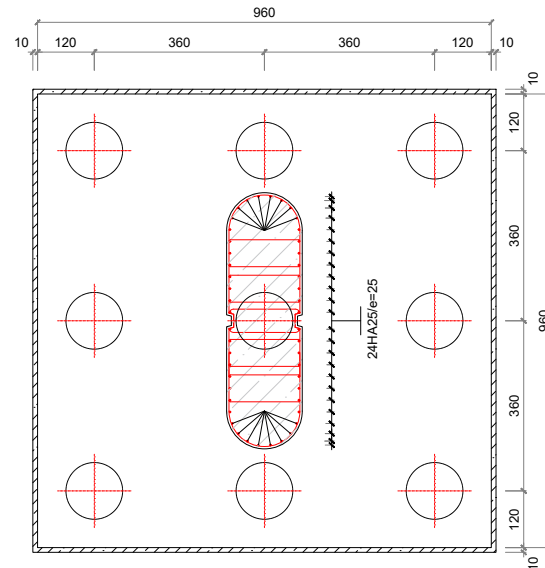
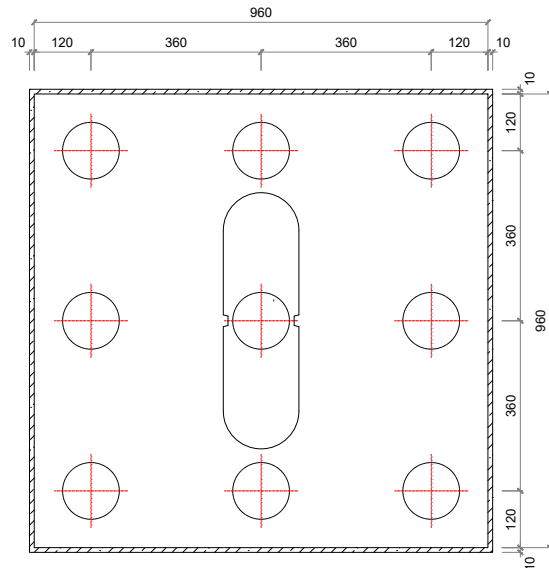
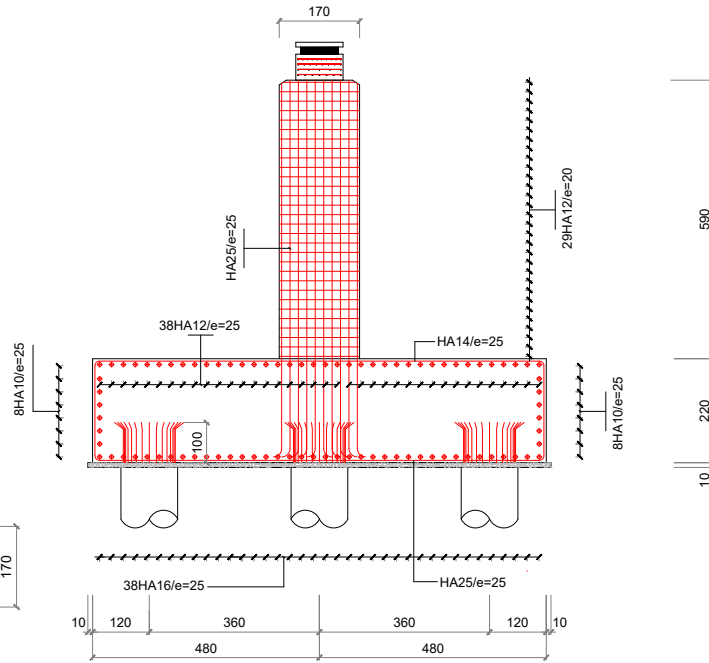
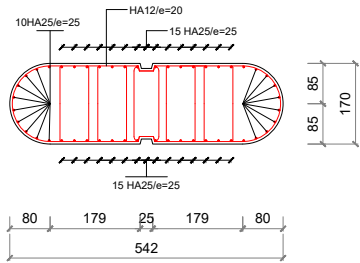
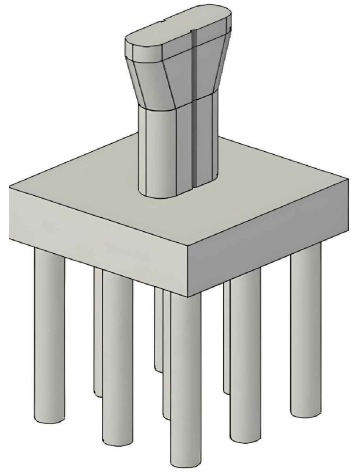
L (m)	20	λ	0.229
E (MPa)	32 164	$\lambda \times L$	4.58
C_u (MN/m³)	30	$\chi_{\theta P}$	1.32
b (m)	1.20	$\chi_{\theta M}$	1.57
I (m⁴)	0.1018	$\bar{P}$ (kN)	251.72
n	9	$\bar{M}$ (kN.m)	919.73

The reinforcement design for the piles is performed using ROBOT-Expert software for a circular section with a diameter of 1.2 meters, subjected to simple bending.

Results :

- Main Reinforcement: $A_s = 113.10 \text{ cm}^2$ we opt 24HA25, with a spacing of $S_t = 15 \text{ cm}$.
- Horizontal Reinforcement: $A_{st} = \frac{A_s}{4}$ For the stirrups, we use 14HA/ml, with 12.5 cm spacing in the critical zone and 20 cm in the normal zone.

The figure below illustrates the reinforcement details for the pile and its foundation:



GENERAL CONCLUSION

The work presented in this report is part of our final year project, entitled: “Design and Analysis of a Road Bridge For Railway Line Reinstatement Including The Removal of The Level Crossing at KERMA ORAN PK 00+262.”

This study allowed us to acquire the necessary skills to design multiple alternatives for a structure, perform a multi-criteria analysis to select the most suitable solution, and size, calculate, and verify the chosen option. Throughout this process, we relied on an extensive bibliography, applicable standards and references, and the use of several design and calculation software, including AUTOCAD, ROBOT STRUCTURAL ANALYSIS, ROBOT EXPERT, REVIT, and MICROSOFT OFFICE EXCEL.

Studying a bridge highlights the application of all theoretical knowledge acquired during our engineering curriculum, such as strength of materials, soil mechanics, structural mechanics, and steel framework, among others.

Through this project, we were able to visualize the different phases of execution of a civil engineering structure and apply various regulations (BAEL, RPOA, Eurocode), while integrating technical, economic, safety, and aesthetic constraints into the design.

This final year project represents a synthesis of the knowledge gained over three years at the National Higher School of Public Works and constitutes an important step toward a professional career as well as potential advanced studies.

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APPENDICES

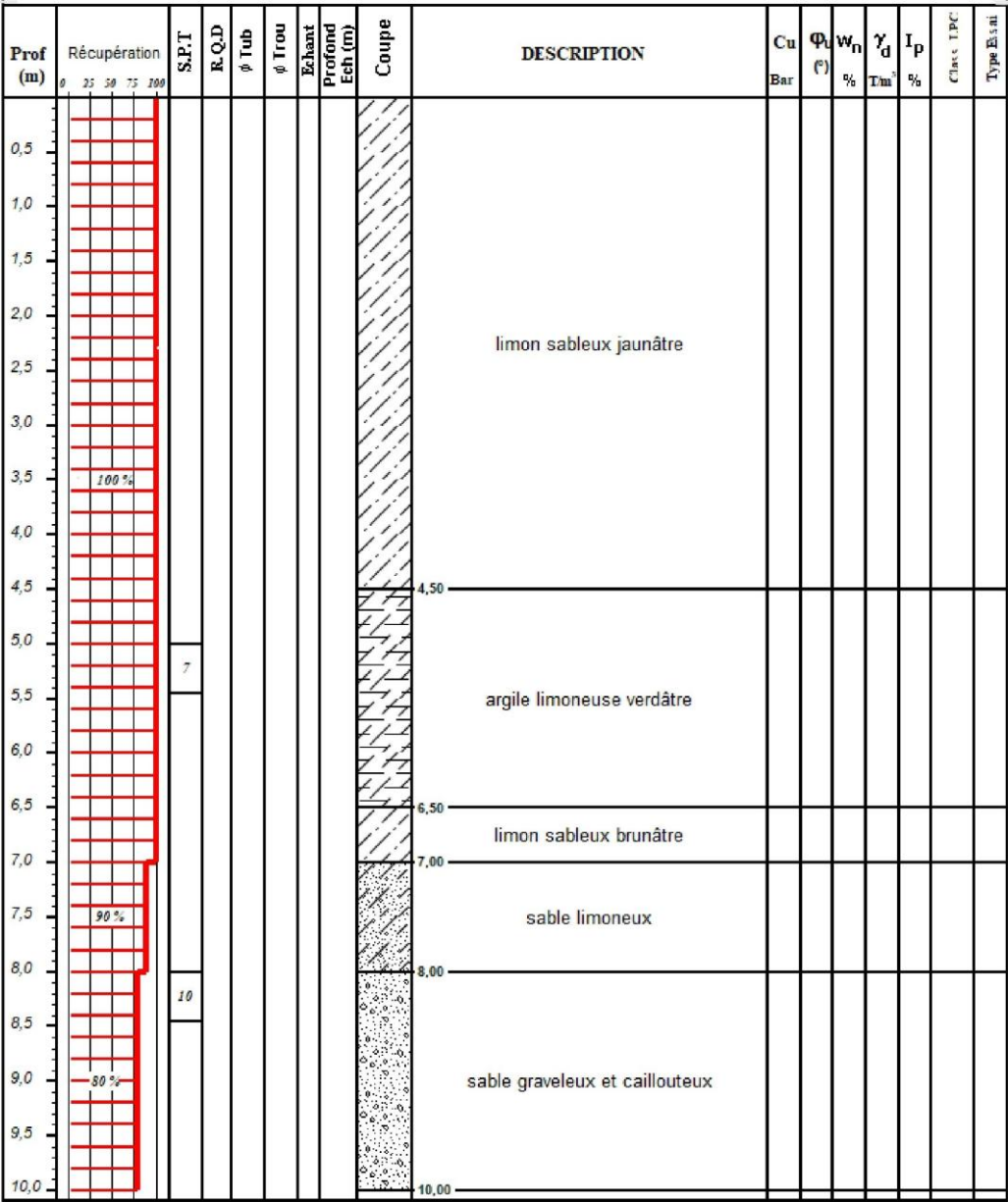


Figure A.1. Cored borehole log SC1.



Figure A.2. SC1 cored borehole to a depth of 20 meters.

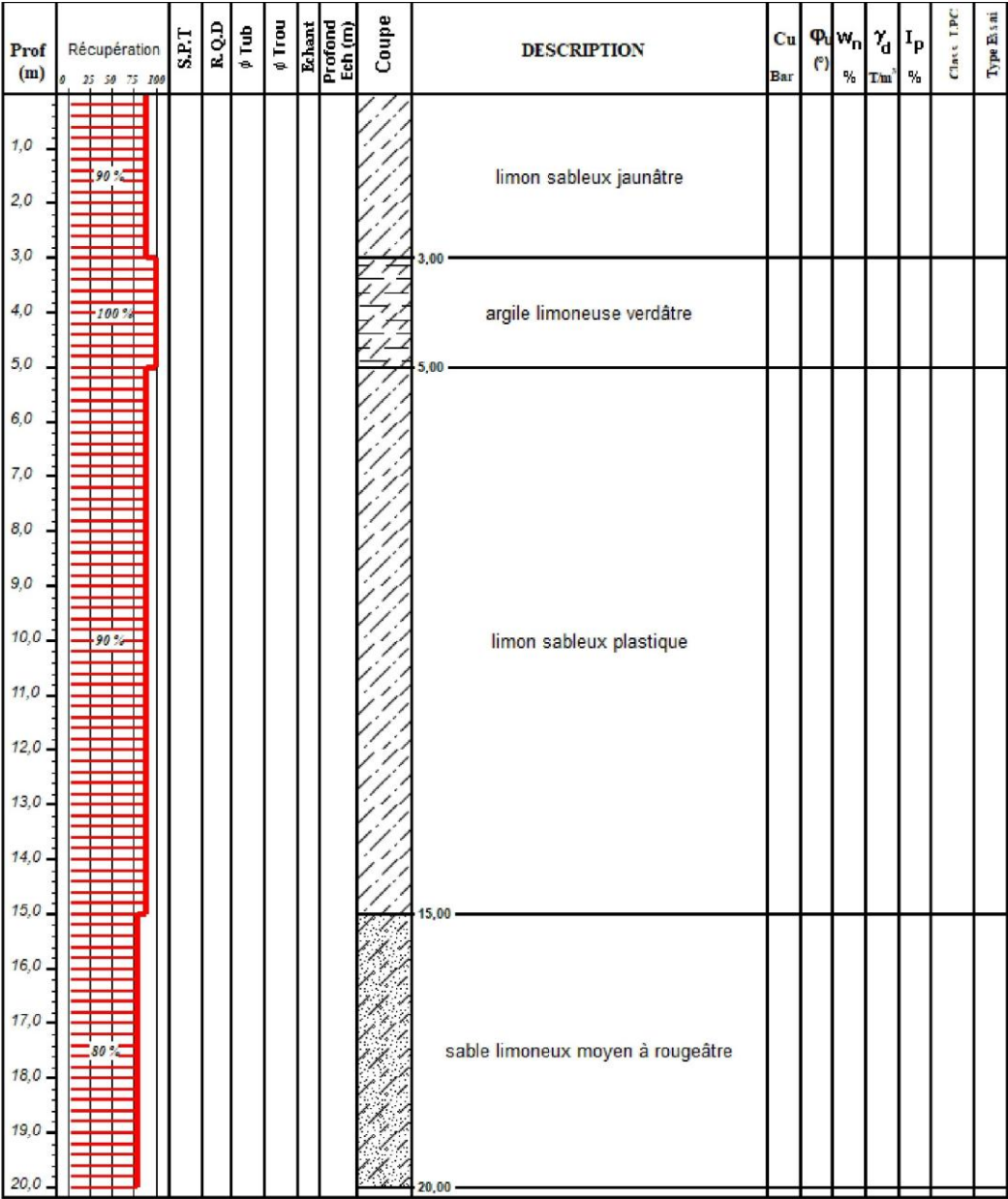


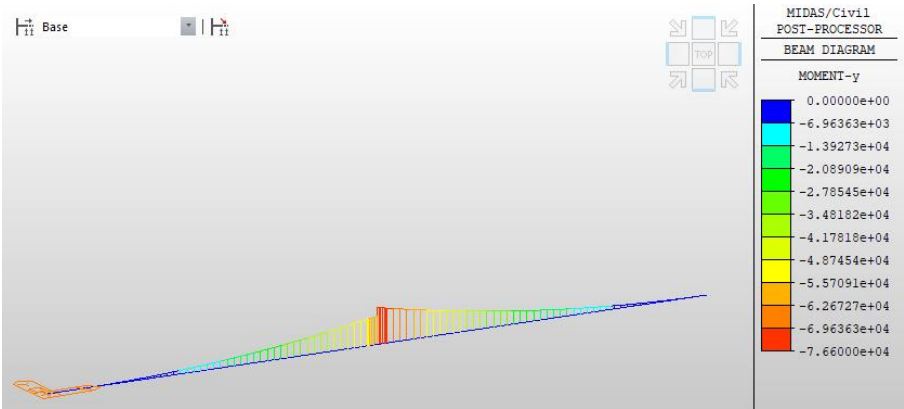
Figure A.3. Cored borehole log SC2.



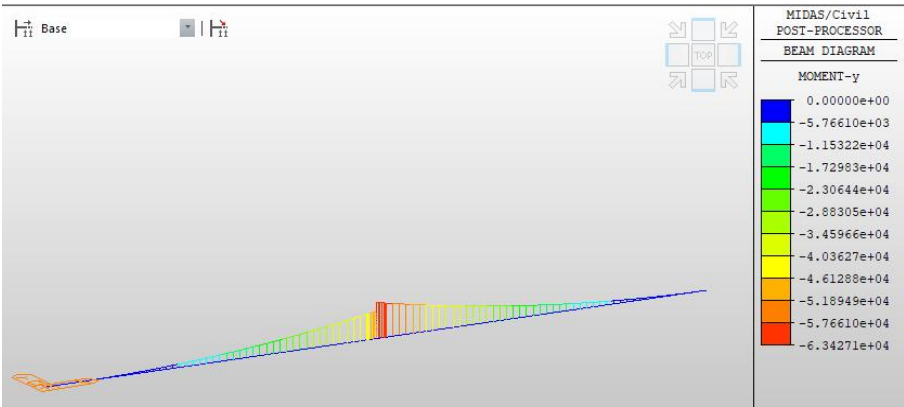
Figure A.4. cored borehole to a depth of 10 meters.

Table A.1. Menard pressuremeter test SCP-1 to a depth of 30 meters.

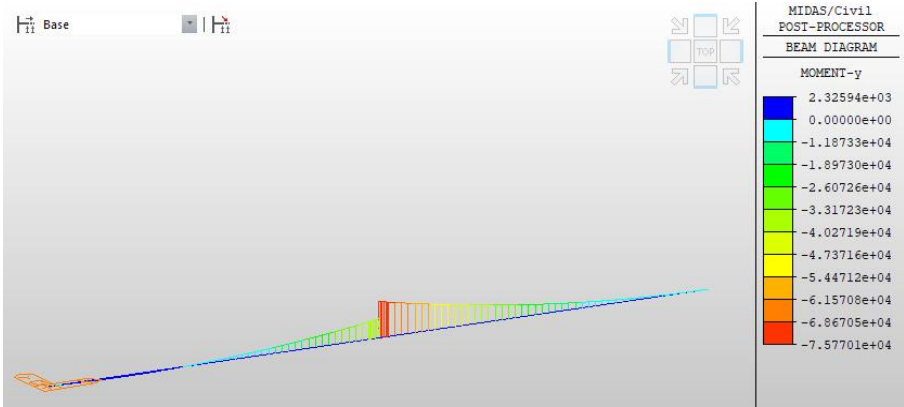
Depth	E (bars)	PL (bars)	Pf (bars)
02.00	040.82	04.66	03.98
04.00	134.48	10.64	08.75
06.00	086.24	10.41	07.02
08.00	136.33	11.80	07.55
10.00	110.32	11.74	09?61
12.00	077.62	7.30	05.64
14.00	094.06	7.53	05.50
16.00	190.81	13.81	10.12
18.00	96.25	14.83	12.21
20.00	158.96	14.11	11.46
22.00	093.82	14.18	09.16
24.00	128.40	12.87	10.53
26.00	155.84	14.78	10.69
28.00	244.02	17.98	13.79
30.00	222.81	17.64	13.96



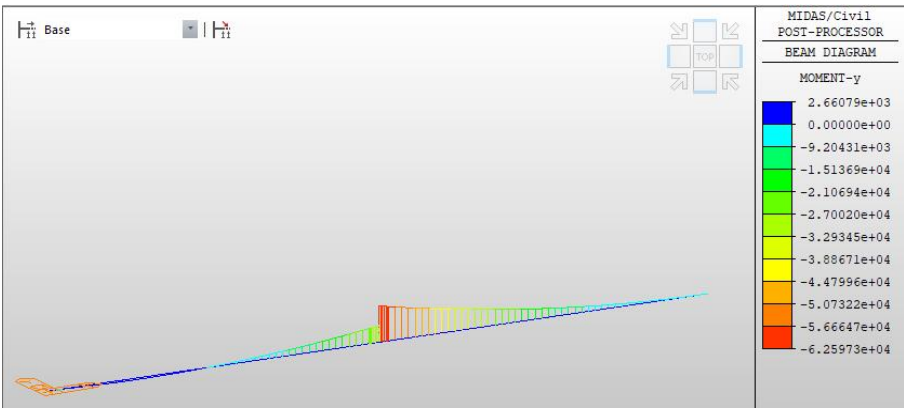
Appendix B.1. Bending moment of a cantilever under combination A1.



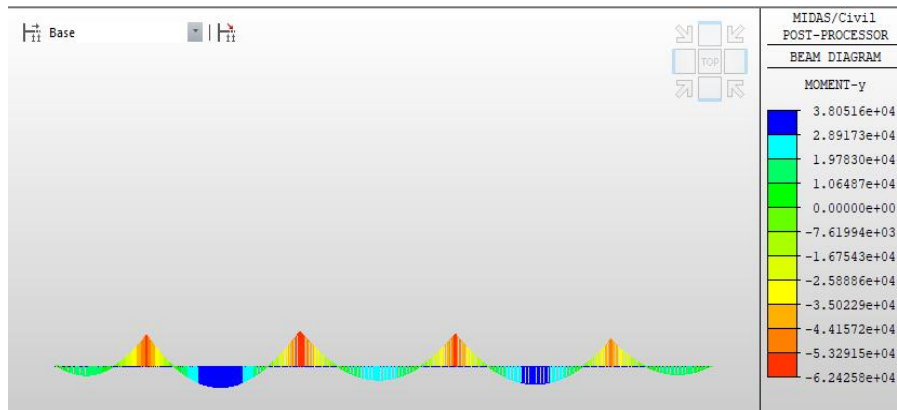
Appendix B.2. Bending moment of a cantilever under combination A2.



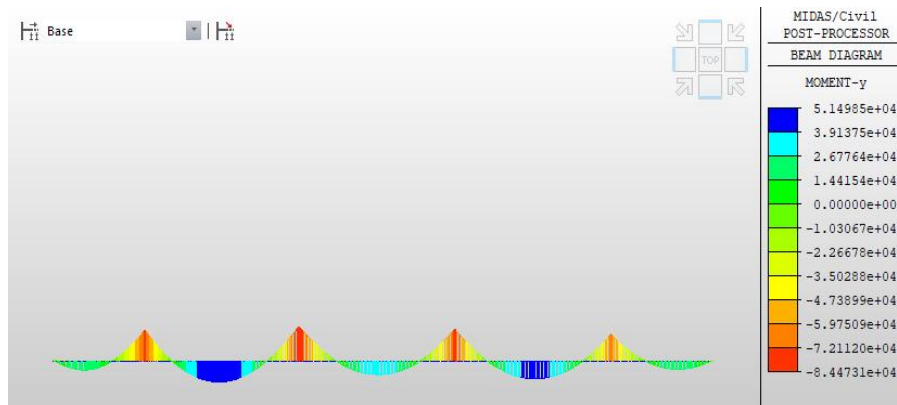
Appendix B.3. Bending moment of a cantilever under combination B1.



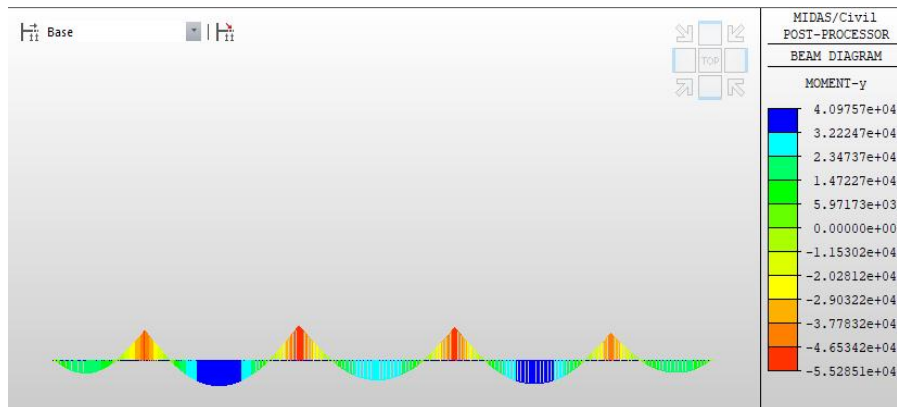
Appendix B.4. Bending moment of a cantilever under combination B2.



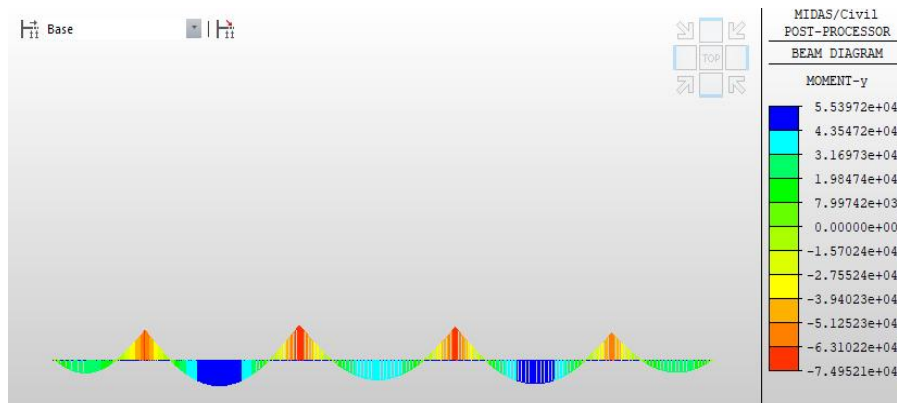
Appendix B.5. Longitudinal bending moment under combination G+1.2Al+St.



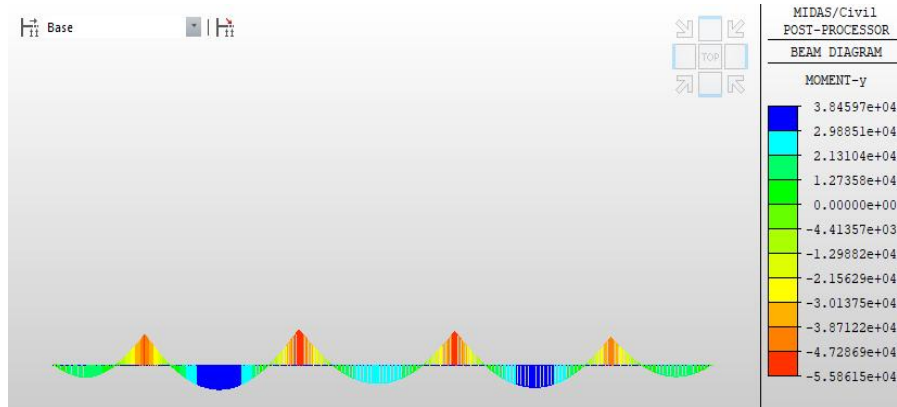
Appendix B.6. Longitudinal bending moment under combination
1.35G+1.6Al+1.6St.



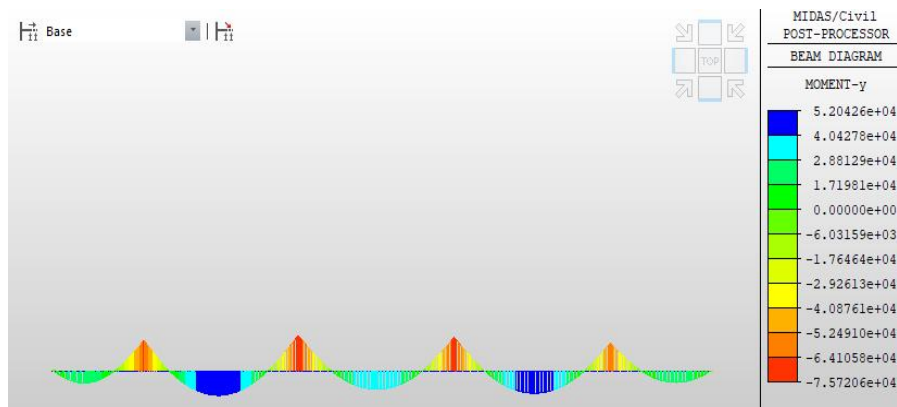
Appendix B.7. Longitudinal bending moment under combination G+1.2Al+St.



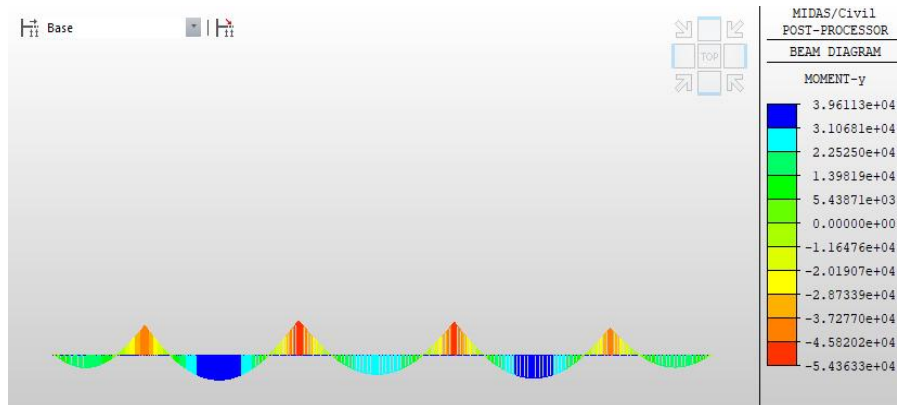
Appendix B.8. Longitudinal bending moment under combination
1.35G+1.6Al+1.6St.



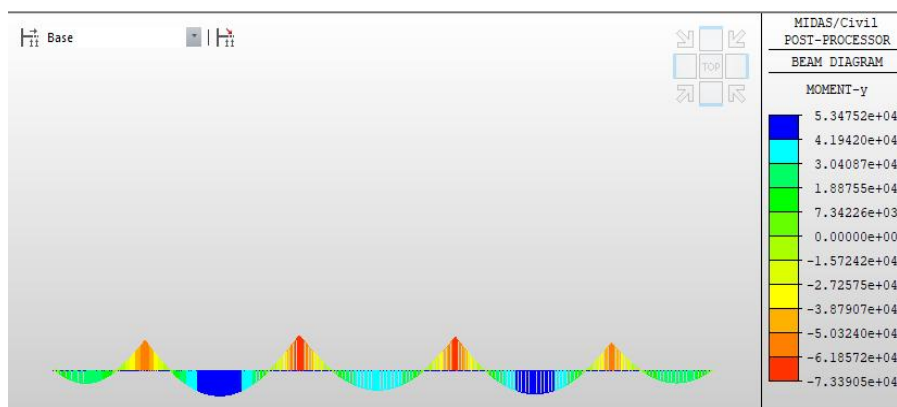
Appendix B.9. Longitudinal bending moment under combination $G+1.2Bt+St$.



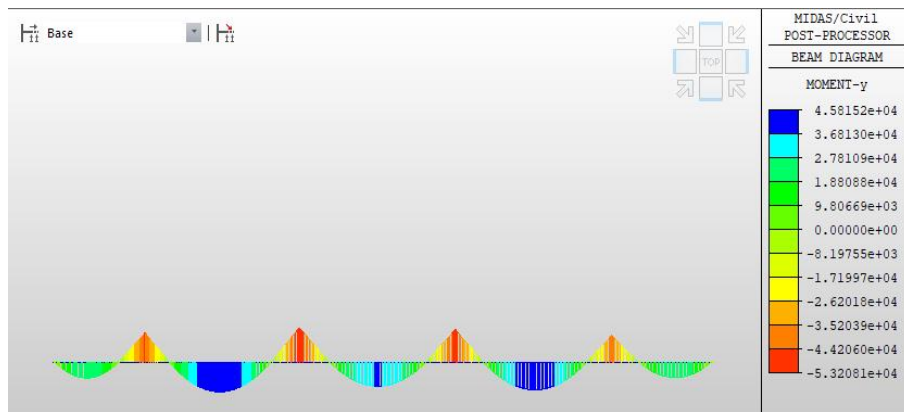
Appendix B.10. Longitudinal bending moment under combination $1.35G+1.6Bt+1.6St$.



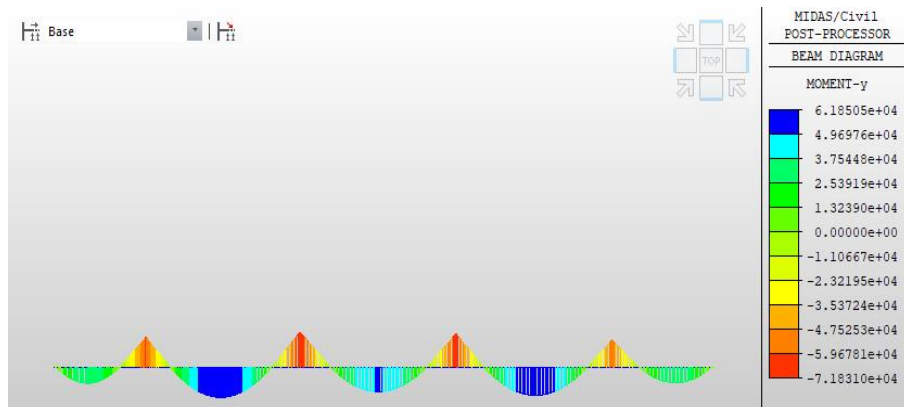
Appendix B.11. Longitudinal bending moment under combination $G+Mc120$.



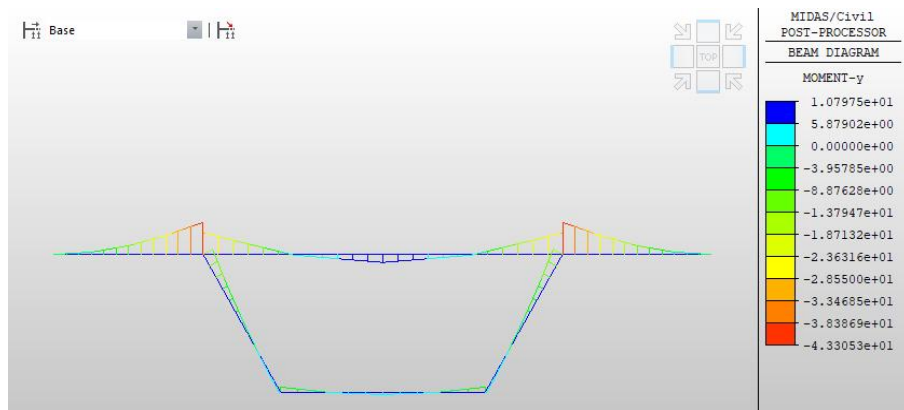
Appendix B.12. Longitudinal bending moment under combination $1.35G+1.35Mc120$.



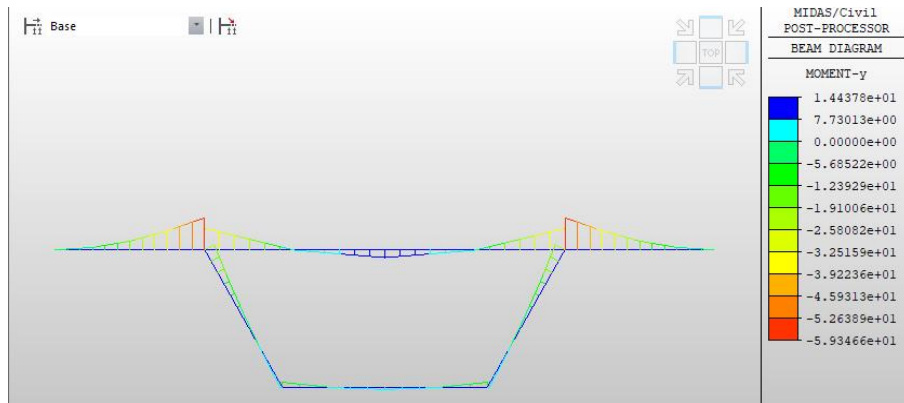
Appendix B.13. Longitudinal bending moment under combination G+D240.



Appendix B.14. Longitudinal bending moment under combination 1.35G+1.35D240.

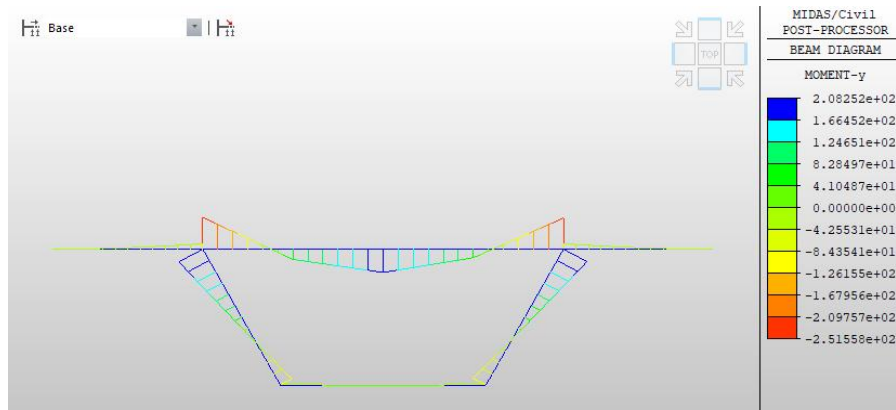


Appendix B.15. Transverse bending moment under combination G+1.2AI+St.

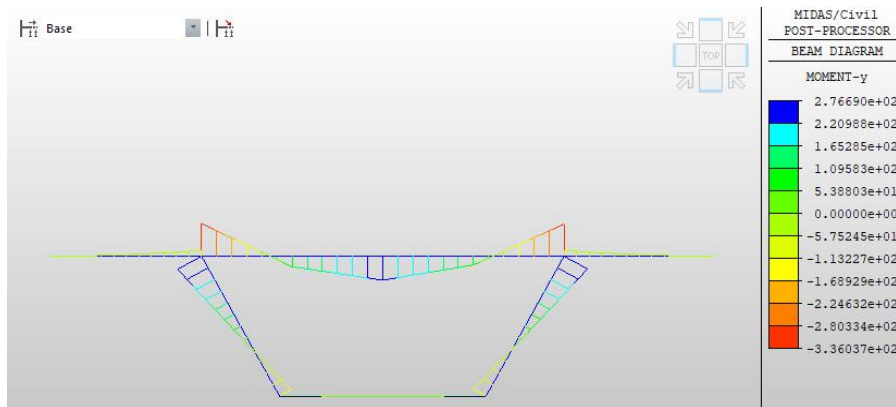


Appendix B.16. Transverse bending moment under combination 1.35G+1.6AI+1.6St.

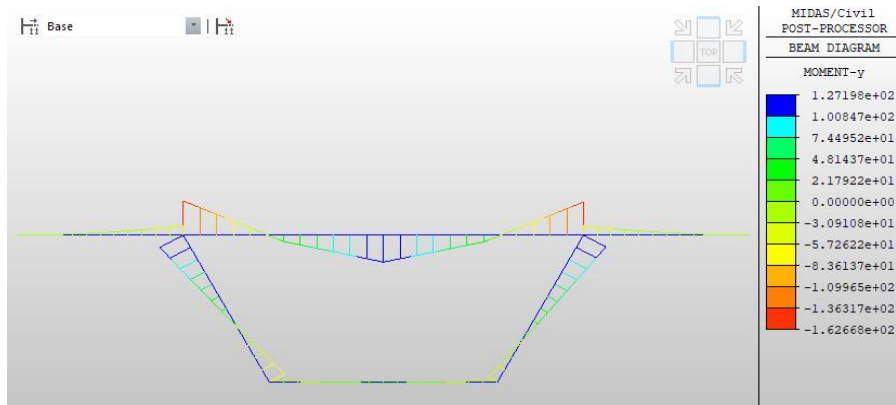
APPENDIX B. RESULTS OF NUMERICAL CALCULATION



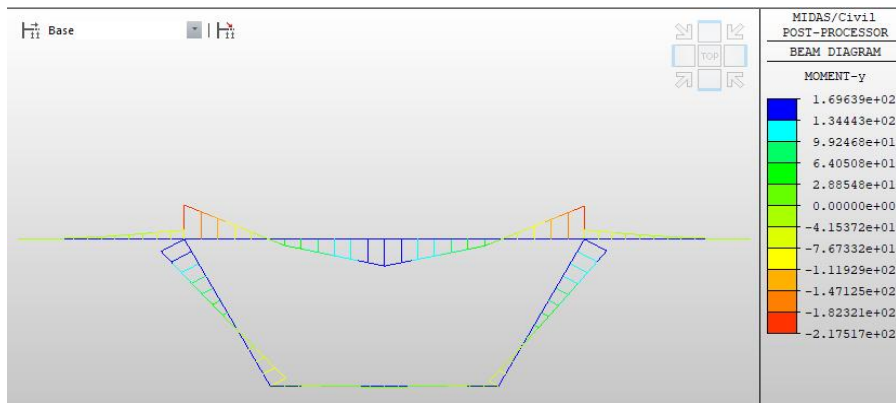
Appendix B.17. Transverse bending moment under combination $G+1.2Bc+St$.



Appendix B.18. Transverse bending moment under combination $1.35G+1.6Bc+1.6St$.

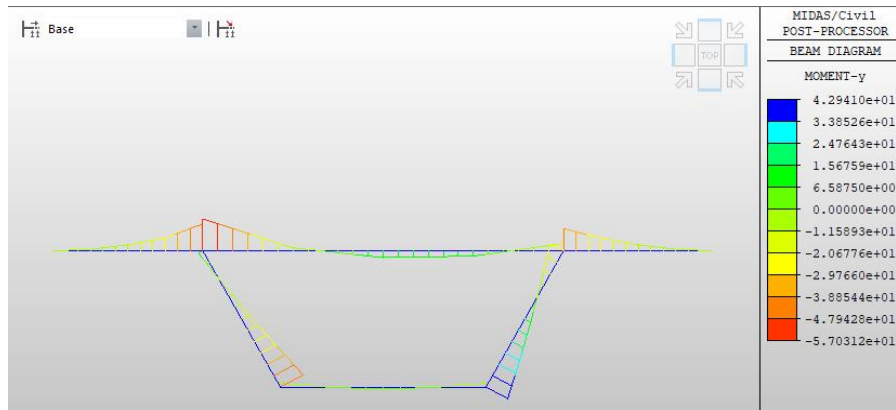


Appendix B.19. Transverse bending moment under combination $G+1.2Bt+St$.

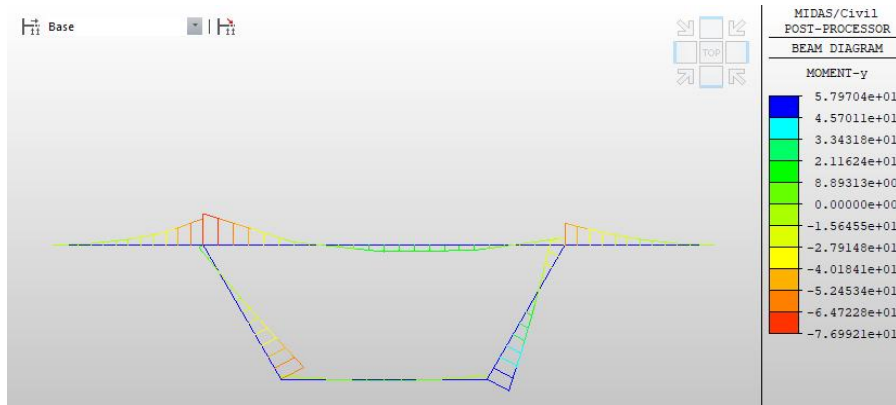


Appendix B.20. Transverse bending moment under combination $1.35G+1.6Bt+1.6St$.

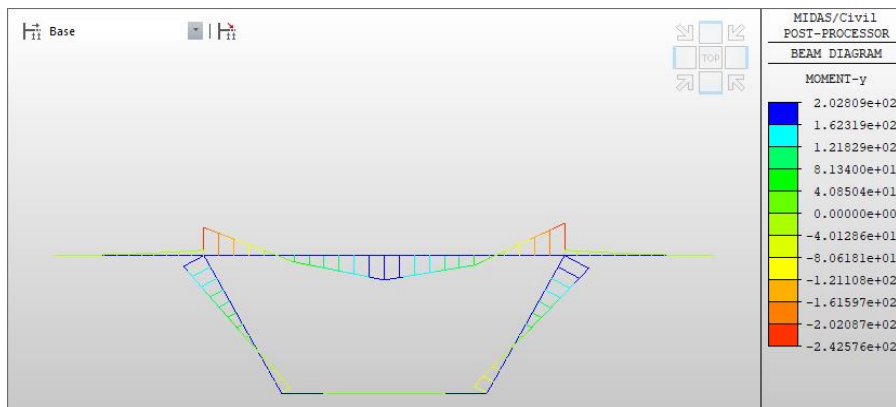
APPENDIX B. RESULTS OF NUMERICAL CALCULATION



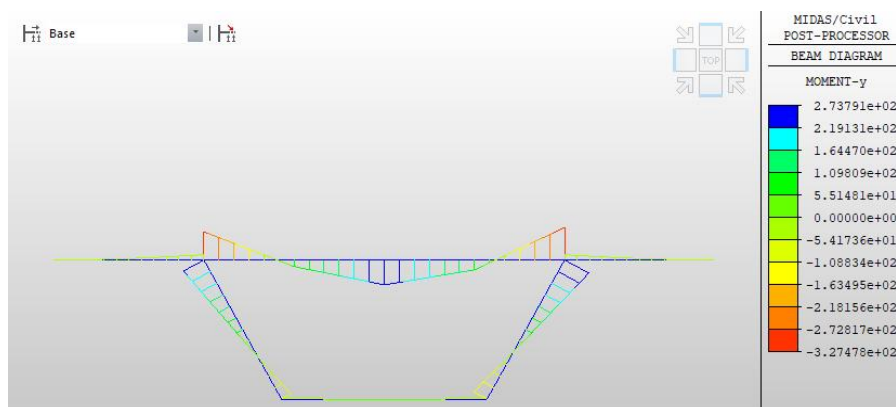
Appendix B.21. Transverse bending moment under combination G+Mc120.



Appendix B.22. Transverse bending moment under combination 1.35G+1.35Mc120.



Appendix B.23. Transverse bending moment under combination G+D240.



Appendix B.24. Transverse bending moment under combination 1.35G+1.35D240.

Study of prestressing and justification of segments

Student Engineers : GAHAM & ARAR

Project : Final Year Project

STUDY OF THE CANTILEVER PRESTRESSING

Data

Prestressing reinforcements

Tendon type	15T15
Class	1 860
Ultimate strength f_{prg}	1 860 MPa
Elastic limit f_{peg}	1 674 MPa
Modulus of elasticity E_p	190 000 MPa
Strand section A_{1t}	140 mm ²
Tendon section A_p	2 100 mm ²
Maximum force F_{p0}	206.1 kN

D duct	0.10 m
f	0.18 (rd ⁻¹)
φ	0.002 (m ⁻¹)
g	4 mm
μ ₀	0.43
ρ _{1000h}	2.5

Calculation of P_0

$$P_0 = \sigma_{p0} \times S_{tendon15T15}$$

$$\sigma_{p0} = \min (0.8 f_{prg} ; 0.9 f_{peg})$$

$$n \geq \frac{P}{0.75 \times P_0}$$

$$P = \frac{\frac{M \cdot v'}{I}}{\frac{1}{S} + \frac{e \cdot v''}{I}}$$

$$e = v' - d$$

σ _{p0}	1 488 MPa
P ₀	3.12 MN
ΔP	25 %
d	15 cm

Segment data

L _{vsp}	1.60 m
L _{vs}	2.20 m
n _{vs}	12

Determination of tendons number

	x (m)	M (kN.m)	H (m)	v'(m)	v (m)	I (m ⁴)	S (m ²)	e (m)	P (MN)	Nbre tendons	Nbre retenus	σ_{sup} (MPa)	σ_{inf} (MPa)	OBS
Axe VSP	0	76 599.97	2.40	1.36	1.04	7.35	13.40	0.89	54.04	23.06	24	0.44	9.11	OK
End VSP	0.80	72 243.10	2.40	1.46	0.94	5.46	6.84	0.79	44.07	18.81	24	3.44	15.61	OK
End V01	3.00	70 619.59	2.40	1.46	0.94	5.46	6.84	0.79	37.16	15.86	24	5.39	12.60	OK
End V02	5.20	50 550.61	2.40	1.46	0.94	5.46	6.84	0.79	30.84	13.16	22	5.85	10.15	OK
End V03	7.40	41 145.15	2.40	1.46	0.94	5.46	6.84	0.79	25.10	10.71	20	6.14	7.95	OK
End V04	9.60	32 700.21	2.40	1.46	0.94	5.46	6.84	0.79	19.95	8.51	18	6.28	6.00	OK
End V05	11.80	25 215.79	2.40	1.46	0.94	5.46	6.84	0.79	15.38	6.56	16	6.24	4.31	OK
End V06	14.00	18 691.90	2.40	1.46	0.94	5.46	6.84	0.79	11.40	4.87	14	6.04	2.88	OK
End V07	16.20	13 128.53	2.40	1.46	0.94	5.46	6.84	0.79	8.01	3.42	12	5.68	1.70	OK
End V08	18.40	8 525.68	2.40	1.46	0.94	5.46	6.84	0.79	5.20	2.22	10	5.15	0.77	OK
End V09	20.60	4 883.35	2.40	1.46	0.94	5.46	6.84	0.79	2.98	1.27	8	4.45	0.10	OK
End V10	22.80	2 201.54	2.40	1.46	0.94	5.46	6.84	0.79	1.34	0.57	6	3.59	-0.31	OK
End V11	25.00	480.26	2.40	1.46	0.94	5.46	6.84	0.79	0.29	0.13	4	2.56	-0.47	OK
End V12	27.20	0.00	2.40	1.46	0.94	5.46	6.84	0.79	0.00	0.00	2	1.32	-0.30	OK

Allowable concrete stress characteristics

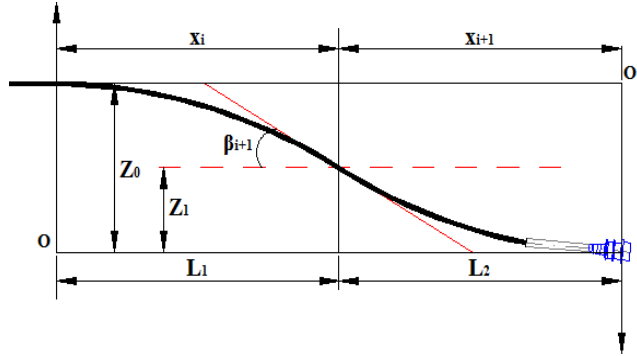
f_{c28}	35 MPa	$\overline{\sigma}_{bt1}$	-4.05 MPa	$\overline{\sigma}_{bc}$	21 MPa	f_{t7}	1.99 MPa	$\overline{\sigma}_{bt2}$	-2.99 MPa
f_{t28}	2.70 MPa	$\overline{\sigma}_{bt2}$	-1.89 MPa	f_{c7}	23.18 MPa	$\overline{\sigma}_{bt1}$	-1.39 MPa	$\overline{\sigma}_{bc}$	13.91 MPa

Tendon plan layout

$$Z(x) = Z_0 + (Z_1 - Z_0) \left(\frac{x}{l}\right)^2 \quad R_i(x) = \frac{(x_i)^2}{2 \times (z_i - z_{i+1})}; \quad R_{i+1}(x) = \frac{(x_{i+1})^2}{2 \times (z_{i+1} - z_{i+2})}$$

$$\beta_i(x) = \text{Arctg} \frac{2 \times (z_i - z_{i+1})}{x_i}$$

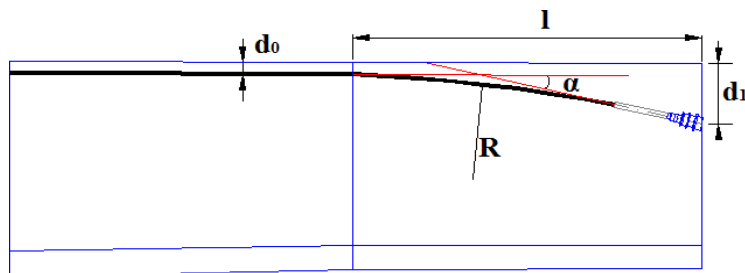
$$\beta_{i+1}(x) = \text{Arctg} \frac{2 \times (z_{i+1} - z_{i+2})}{x_{i+1}}$$



Vsr	Tendons	L _{1,2} (m)	Z _i (m)	Z _{i+1} (m)	X _i (m)	X _{i+1} (m)	R _i (m)	R _{i+1} (m)	B _i (rad)	β _{i+1} (rad)	Equation		
End V1	T1	Straight line											
End V2	T2	1.10	0.20	0.10	1.10	1.10	6.05	6.05	0.18	0.18	0.20	-0.0826	X ²
End V3	T3	2,20	0.20	0.10	2.20	2.20	24.20	24.20	0.09	0.09	0.20	-0.0207	X ²
End V4	T4	Straight line											
End V5	T5	2.20	0.20	0.10	2.20	2.20	24.20	24.20	0.09	0.09	0.20	-0.0207	X ²
End V6	T6	2.20	0.20	0.10	2.20	2.20	24.20	24.20	0.09	0.09	0.20	-0.0207	X ²
End V7	T7	3.30	0.40	0.20	3.30	3.30	27.23	27.23	0.12	0.12	0.40	-0.0184	X ²
End V8	T8	3.30	0.40	0.20	3.30	3.30	27.23	27.23	0.12	0.12	0.40	-0.0184	X ²
End V9	T9	4.40	0.60	0.30	4.40	4.40	32.27	32.27	0.14	0.14	0.60	-0.0155	X ²
End V10	T10	4.40	0.60	0.30	4.40	4.40	32.27	32.27	0.14	0.14	0.60	-0.0155	X ²
End V11	T11	5.50	0.80	0.40	5.50	5.50	37.81	37.81	0.14	0.14	0.80	-0.0132	X ²
End V12	T12	5.50	0.80	0.40	5.50	5.50	37.81	37.81	0.14	0.14	0.80	-0.0132	X

Tendon elevation layout

$$y = d_0 + (d_1 - d_0) \times \left(\frac{x}{l}\right)^2 \quad R_c(x) = \frac{x^2}{2 \times (d_1 - d_0)} \geq R_{\min} \quad \alpha_n = \text{artg}(2 \times (d_1 - d_0)) \times \frac{x}{l^2}$$



APPENDIX C : CALCULATIONS OF THE RETAINED

Vsr	Tendons	L (m)	d ₀ (m)	d ₁ (m)	R _i (m)	α (rad)	Equation			
End V1	T1	2.20	0.35	0.60	0.60	0.21	0.35	+	0.052	X ²
End V2	T2	2.20	0.35	0.60	9.68	0.21	0.35	+	0.052	X ²
End V3	T3	2.20	0.35	0.60	9.68	0.21	0.35	+	0.052	X ²
End V4	T4	4.40	0.15	0.60	21.51	0.17	0.35	+	0.023	X ²
End V5	T5	4.40	0.15	0.60	21.51	0.17	0.15	+	0.023	X ²
End V6	T6	4.40	0.15	0.60	21.51	0.17	0.15	+	0.023	X ²
End V7	T7	4.40	0.15	0.60	21.51	0.17	0.15	+	0.023	X ²
End V8	T8	4.40	0.15	0.60	21.51	0.17	0.15	+	0.023	X ²
End V9	T9	4.40	0.15	0.60	21.51	0.17	0.15	+	0.023	X ²
End V10	T10	4.40	0.15	0.60	21.51	0.17	0.15	+	0.023	X ²
End V11	T11	4.40	0.15	0.60	21.51	0.17	0.15	+	0.023	X ²
End V12	T12	4.40	0.15	0.60	21.51	0.17	0.15	+	0.023	X ²

Loss calculation

1. Friction tension losses :

φ	0.002 m ⁻¹
f	0.18
σ_{p0}	1488 MPa

$$\Delta\sigma_{\varphi}(x) = \sigma_{p0} \times (1 - e^{-(f \times \beta + \varphi \times x)})$$

X	Tendons	L (m)	θ_1 (rad)	θ_2 (rad)	θ_3 (rad)	Axe VSP	End VSP	End V1	End V2	End V3	End V4	End V5	End V6	End V7	End V8	End V9	End V10	End V11	End V12
3.00	T1	2.20	0.21	0.21	0.21	63.96	61.68	0											
5.20	T2	2.20	0.41	0.27	0.21	119.75	117.57	61.68	0										
7.40	T3	2.20	0.27	0.23	0.21	92.57	90.34	72.44	61.68	0									
9.60	T4	2.20	0.17	0.17	0.17	71.41	69.14	62.88	56.60	50.29	0								
11.80	T5	2.20	0.24	0.19	0.17	97.10	94.87	88.73	82.56	62.33	50.29	0							
14.00	T6	2.20	0.24	0.19	0.17	103.21	100.99	94.87	88.73	82.56	62.33	50.29	0						
16.20	T7	2.20	0.29	0.20	0.17	120.61	118.42	112.38	106.32	100.22	94.10	66.31	50.29	0					
18.40	T8	2.20	0.29	0.20	0.17	126.61	124.43	118.42	112.38	106.32	100.22	94.10	66.31	50.29	0				
20.60	T9	2.20	0.31	0.21	0.17	138.45	136.38	130.42	124.43	118.42	112.38	106.31	74.79	68.56	50.29	0			
22.80	T10	2.20	0.31	0.21	0.17	144.46	142.31	136.38	130.42	124.43	118.42	112.38	106.31	74.79	68.56	50.29	0		
25.00	T11	2.20	0.33	0.22	0.17	153.97	151.83	145.94	140.02	134.08	128.11	122.11	116.09	110.04	76.21	69.98	50.29	0	
27.20	T12	2.20	0.33	0.22	0.17	159.82	157.70	151.83	145.94	140.02	134.08	128.11	122.11	116.09	110.04	76.21	69.98	50.29	0.00
Average						116.00	113.80	98.00	95.37	91.87	88.88	84.95	76.56	69.96	61.02	49.12	40.09	25.14	0.00
As a (%)						7.80	7.65	6.59	6.41	6.17	5.97	5.71	5.14	4.70	4.10	3.30	2.69	1.69	0.00

2. Anchor set losses :

g	0.002
E_p	190 000 MPa

$$\Delta\sigma_g(x) = 2 \times \sigma_{p0} \times k (d - x) , \text{ avec } k = f \times \frac{\beta}{\ell} + \varphi \quad \text{et} \quad d = \sqrt{\frac{E_p g}{\sigma_{p0} k}}$$

X	Tendons	L	k	d	A.VSP	E VSP	E V1	E V2	E V3	E V4	E V5	E V6	E V7	E V8	E V9	E V10	E V11	E V12
3.00	T1	3.00	0.002	11.30	49.40	54.16	67.26											
5.20	T2	4.40	0.013	4.38	-	-	86.36	173.66										
7.40	T3	4.40	0.011	4.74	-	-	11.39	85.93	160.47									
9.60	T4	6.60	0.007	6.25	-	-	-	35.98	78.81	121.64								
11.80	T5	6.60	0.007	5.97	-	-	-	-	33.45	80.41	127.36							
14.00	T6	6.60	0.007	5.97	-	-	-	-	-	33.45	80.41	127.36						
16.20	T7	6.60	0.008	5.79	-	-	-	-	-	-	31.55	81.37	131.18					
18.40	T8	6.60	0.008	5.79	-	-	-	-	-	-	-	31.55	81.37	131.18				
20.60	T9	6.60	0.008	5.70	-	-	-	-	-	-	-	-	30.43	81.86	133.30			
22.80	T10	6.60	0.008	5.70	-	-	-	-	-	-	-	-	-	30.43	81.86	133.30		
25.00	T11	6.60	0.008	5.65	-	-	-	-	-	-	-	-	-	-	29.70	82.16	134.62	
27.20	T12	6.60	0.008	5.65	-	-	-	-	-	-	-	-	-	-	-	29,70	82.16	134.62
Average					4.12	4.51	13.75	26.87	27.27	26.17	29.91	34.33	40.50	48.69	61.22	81.72	108.39	134.62
As a (%)					0.28	0.30	0.92	1.81	1.83	1.76	2.01	2.31	2.72	3.27	4.11	5.49	7.28	9.05

3. Tension losses due to non-simultaneous tensionin :

$$\Delta\sigma_e(x) = 2,64 \times \sigma_b(x) , \text{ avec : } \sigma_{bi} = \frac{P}{B} + \frac{P \times e^2}{I_G} - \frac{M_p \times e}{I_G}$$

E_{bi}	35 981.73 MPa
E_p	190 000 MPa
P₀	3.12 MN

Abcs	n Tendons	v (m)	d _{moy} (m)	e _{moy} (m)	I (m ⁴)	S (m ²)	M (MN.m)	P (MPa)	Δσ _e (MPa)	Δσ _e (%)
Axe VSP	24	1.04	0.23	0.81	7.35	13.40	76.60	2.87	7.35	0.51
End VSP	24	0.94	0.23	0.71	5.46	6.84	72.24	2.88	18.66	1.25
End V1	24	0.94	0.22	0.72	5.46	6.84	60.92	2.89	22.94	1.54
End V2	22	0.94	0.21	0.73	5.46	6.84	50.55	2.87	22.79	1.53
End V3	20	0.94	0.20	0.75	5.46	6.84	41.15	2.87	22.80	1.53
End V4	18	0.94	0.20	0.74	5.46	6.84	32.70	2.88	22.07	1.48
End V5	16	0.94	0.21	0.73	5.46	6.84	25.22	2.88	20.87	1.40
End V6	14	0.94	0.21	0.73	5.46	6.84	18.69	2.89	19.38	1.30
End V7	12	0.94	0.23	0.72	5.46	6.84	13.13	2.89	17.44	1.17
End V8	10	0.94	0.24	0.70	5.46	6.84	8.53	2.89	15.14	1.02
End V9	8	0.94	0.26	0.68	5.46	6.84	4.88	2.89	12.47	0.84
End V10	6	0.94	0.30	0.64	5.46	6.84	2.20	2.87	9.37	0.63
End V11	4	0.94	0.38	0.57	5.46	6.84	0.48	2.84	6.02	0.40
End V12	2	0.94	0.60	0.34	5.46	6.84	0.00	2.94	2.51	0.17

4.Total instantaneous losses :

	A VSP	E VSP	E V1	E V2	E V3	E V4	E V5	E V6	E V7	E V8	E V9	E V10	E V11	E V12
Instantaneous losses as a (%)	8.58	9.21	9.05	9.75	9.54	9.22	9.12	8.75	8.60	8.39	8.25	8.82	9.38	9.22
Instantaneous losses as a (Mpa)	127.65	136.98	134.68	145.03	141.94	137.12	135.74	130.26	127.90	124.86	122.81	131.18	139.55	137.13

1. Shrinkage losses :

$$\Delta\sigma_r \approx E_p \varepsilon_r$$

E_p	190 000 MPa
ε	0.0002

Shrinkage losses	
Losses (Mpa)	As a (%)
38	2.55

2. Creep losses :

$$\Delta\sigma_n = 2,5 \sigma_b(x) \times \frac{E_p}{E_{ij}}$$

$$\sigma_{bi} = \frac{P}{B} + \frac{P \times e^2}{I_G} - \frac{M_p \times e}{I_G}$$

E_{bi}	35 981.73 MPa
E_p	190 000 MPa

Abcs	N Tendons	v (m)	d _{moy} (m)	e _{moy} (m)	I (m ⁴)	S (m ²)	M _g (MN.m)	P (MPa)	$\Delta\sigma_f$ (MPa)	$\Delta\sigma_f$ (%)
Axe VSP	24	1.04	0.23	0.81	7.35	13.40	63.26	2.86	56.24	3.78
End VSP	24	0.94	0.23	0.71	5.46	6.84	59.57	2.84	112.09	7.53
End V1	24	0.94	0.22	0.72	5.46	6.84	50.06	2.84	129.89	8.73
End V2	22	0.94	0.21	0.73	5.46	6.84	41.37	2.82	126.78	8.52
End V3	20	0.94	0.20	0.75	5.46	6.84	33.51	2.83	124.62	8.37
End V4	18	0.94	0.20	0.74	5.46	6.84	26.48	2.84	118.78	7.98
End V5	16	0.94	0.21	0.73	5.46	6.84	20.27	2.84	110.88	7.45
End V6	14	0.94	0.21	0.73	5.46	6.84	14.89	2.85	101.74	6.84
End V7	12	0.94	0.23	0.72	5.46	6.84	10.34	2.86	90.64	6.09
End V8	10	0.94	0.24	0.70	5.46	6.84	6.62	2.86	77.96	5.24
End V9	8	0.94	0.26	0.68	5.46	6.84	3.72	2.87	63.62	4.28
End V10	6	0.94	0.30	0.64	5.46	6.84	1.65	2.85	47.37	3.18
End V11	4	0.94	0.38	0.57	5.46	6.84	0.41	2.83	30.04	2.02
End V12	2	0.94	0.60	0.34	5.46	6.84	0.00	2.84	12.54	0.84

3. Relaxation losses :

$$\Delta\sigma_p = \frac{6}{100} \times \rho_{1000} (\mu - \mu_0) \times \sigma_i(x), \text{ avec : } \mu = \frac{\sigma_i(x)}{f_{prg}}$$

Abcs	n Tendons	σ_i (Mpa)	μ	μ_0	$\Delta\sigma_p$ (MPa)	$\Delta\sigma_p$ (%)
Axe VSP	24	1360.35	0.731	0.43	61.50	4.13
End VSP	24	1351.02	0.726	0.43	60.06	4.04
End V1	24	1353.32	0.728	0.43	60.41	4.06
End V2	22	1342.97	0.722	0.43	58.83	3.95
End V3	20	1346.06	0.724	0.43	59.30	3.99
End V4	18	1350.88	0.726	0.43	60.04	4.03
End V5	16	1352.26	0.727	0.43	60.25	4.05
End V6	14	1357.74	0.730	0.43	61.09	4.11
End V7	12	1360.10	0.731	0.43	61.46	4.13
End V8	10	1363.14	0.733	0.43	61.93	4.16
End V9	8	1365.19	0.734	0.43	62.25	4.18
End V10	6	1356.82	0.729	0.43	60.95	4.10
End V11	4	1348.45	0.725	0.43	59.66	4.01
End V12	2	1350.87	0.726	0.43	60.03	4.03

7.Total deferred losses :

$$\Delta\sigma_d = \Delta\sigma_r + \Delta\sigma_n + \frac{5}{6}\Delta\sigma_p$$

8. Total losses :

	Axe VSP	End VSP	End V1	End V2	End V3	End V4	End V5	End V6	End V7	End V8	End V9	End V10	End V11	End V12
Deferred losses as a (%)	9.78	13.45	14.67	14.37	14.25	13.90	13.38	12.81	12.09	11.26	10.32	9.15	7.91	6.76
Instantaneous losses as a (%)	8.58	9.21	9.05	9.75	9.54	9.22	9.12	8.75	8.60	8.39	8.25	8.82	9.38	9.22
Total (%)	18.36	22.66	23.72	24.12	23.79	23.11	22.50	22.57	20.68	19.65	18.57	17.97	17.29	15.97
Total (MPa)	273.14	337.10	352.91	358.83	353.97	343.93	334.82	320.91	307.75	292.42	276.30	267.34	257.31	237.70

Stress verification for the construction phase

σ_{p0}	1 488 MPa
P_0	3.12 MPa
ΔP	24.12 MPa

$$\sigma(y) = \frac{P}{B} + \frac{Pe_0 + M}{I} y$$

	x (m)	n tendons	M (MN.m)	v' (m)	v (m)	I (m ⁴)	S (m ²)	e _{moy} (m)	P (MN)	σ_{sup} (MPa)	σ_{inf} (MPa)	OBS
Axe VSP	0	24	76.60	1.36	1.04	7.35	13.40	0.81	56.91	-0.08	9.90	OK
End VSP	0.80	24	72.24	1.46	0.94	5.46	6.84	0.71	56.91	2.83	16.85	OK
End V01	3.00	24	60.92	1.46	0.94	5.46	6.84	0.72	56.91	4.88	13.67	OK
End V02	5.20	22	50.55	1.46	0.94	5.46	6.84	0.73	52.17	5.49	10.95	OK
End V03	7.40	20	41.15	1.46	0.94	5.46	6.84	0.75	47.43	5.93	8.49	OK
End V04	9.60	18	32.70	1.46	0.94	5.46	6.84	0.74	42.68	6.05	6.54	OK
End V05	11.80	16	25.22	1.46	0.94	5.46	6.84	0.73	37.94	6.00	4.85	OK
End V06	14.00	14	18.69	1.46	0.94	5.46	6.84	0.73	33.20	5.78	3.41	OK
End V07	16.20	12	13.13	1.46	0.94	5.46	6.84	0.72	28.46	5.40	2.23	OK
End V08	18.40	10	8.53	1.46	0.94	5.46	6.84	0.70	23.71	4.86	1.31	OK
End V09	20.60	8	4.88	1.46	0.94	5.46	6.84	0.68	18.97	4.15	0.64	OK
End V10	22.80	6	2.20	1.46	0.94	5.46	6.84	0.64	14.23	3.27	0.23	OK
End V11	25.00	4	0.48	1.46	0.94	5.46	6.84	0.57	9.49	2.23	0.08	OK
End V12	27.20	2	0.00	1.46	0.94	5.46	6.84	0.34	9.49	1.94	0.52	OK

$f_{c28} =$	35 MPa
$\sigma_{bc} =$	21 MPa
$f_{t28} =$	-2.7 MPa

$\overline{\sigma}_{bt1}$	-4.05 MPa	Outside the cover section
$\overline{\sigma}_{bt2}$	-1.89 MPa	Within the cover section

STUDY OF CONTINUITY PRESTRESSING

Data

Prestressing reinforcements

Type	15T15	For the main span
Type	13T15	For the side span
Class		1860

Tendon type :	15/13T15
Class :	1 860
Ultimate strength f_{prg}	1 860 MPa
Elastic limit f_{peg}	1 674 MPa
Modulus of elasticity E_p	190 000 MPa
Strand section A_{lt}	140 mm ²
Tendon section $A_{p.C}$	2 100 mm ²
Tendon section $A_{p.R}$	1 820 mm ²
Maximum force F_{p0}	206.1 KN

D duct	0.10 m
f	0.18 (rd ⁻¹)
ϕ	0.002 (m ⁻¹)
g	4 mm
μ_0	0.43
ρ_{1000h}	2.5

Calculation of P_0

$$P_0 = \sigma_{p0} \times S_{tendon15/13T15}$$

$$\sigma_{p0} = \min (0.8 f_{prg} ; 0.9 f_{peg})$$

$$n \geq \frac{P}{0.75 \times P_0}$$

$$P = \frac{\frac{M \cdot v'}{I}}{\frac{1}{S} + \frac{e \cdot v'}{I}}$$

$$e = v' - d$$

σ_{p0}	1 488 MPa
P_0	3.12 MN
P_0	2.71 MN

For the main span

For the side span

ΔP	24.12 %
d	15 cm

Segment data

L_{vsp}	1.60 m
L_{vs}	2.20 m
n_{vs}	12

Determination of the tendons number (Main Span)

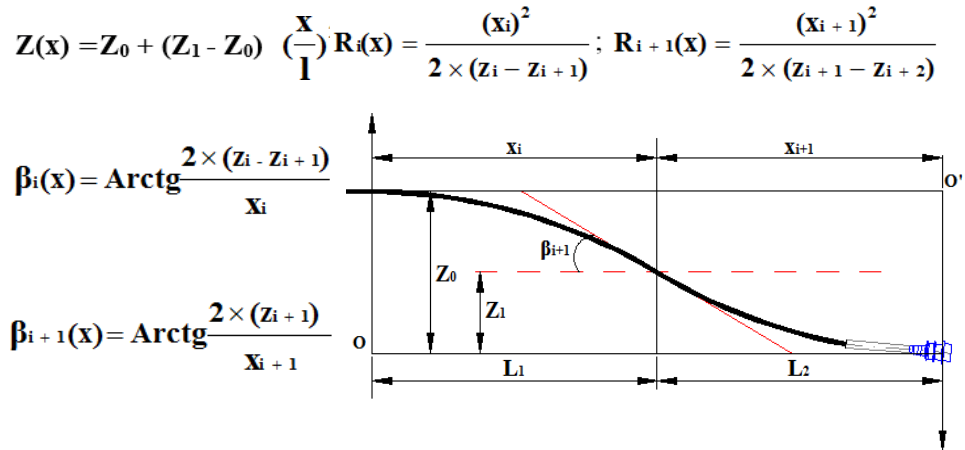
	M (kN.m)	v' (m)	v (m)	I (m⁴)	S (m²)	e (m)	P (MN)	Nbre tendons	N Tendons
End V4	0.00	1.46	0.94	5.46	6.84	1.31	0.00	0.00	2
End V5	7 946.24	1.46	0.94	5.46	6.84	1.31	4.28	1.80	2
End V6	16 968.89	1.46	0.94	5.46	6.84	1.31	9.14	3.85	4
End V7	24 849.89	1.46	0.94	5.46	6.84	1.31	13.38	5.64	6
End V8	31 545.62	1.46	0.94	5.46	6.84	1.31	16.99	7.16	8
End V9	36 980.81	1.46	0.94	5.46	6.84	1.31	19.92	8.40	10
End V10	41 115.98	1.46	0.94	5.46	6.84	1.31	22.14	9.34	10
End V11	44 038.16	1.46	0.94	5.46	6.84	1.31	23.72	10.00	12
End V12	45 951.73	1.46	0.94	5.46	6.84	1.31	24.75	10.44	12
End V12	46 287.06	1.46	0.94	5.46	6.84	1.31	24.93	10.51	12
End V11	45 282.36	1.46	0.94	5.46	6.84	1.31	24.39	10.28	12
End V10	42 930.33	1.46	0.94	5.46	6.84	1.31	23.12	9.75	12
End V9	39 231.79	1.46	0.94	5.46	6.84	1.31	21.13	8.91	10
End V8	34 417.33	1.46	0.94	5.46	6.84	1.31	18.54	7.82	10
End V7	28 410.10	1.46	0.94	5.46	6.84	1.31	15.30	6.45	8
End V6	21 159.62	1.46	0.94	5.46	6.84	1.31	11.40	4.81	6
End V5	12 696.24	1.46	0.94	5.46	6.84	1.31	6.84	2.88	4
End V4	3 055.83	1.46	0.94	5.46	6.84	1.31	1.65	0.69	2
End V3	0.00	1.46	0.94	5.46	6.84	1.31	0.00	0.00	2

Determination of the tendons number (Side Span)

	M (kN.m)	v' (m)	v (m)	I (m⁴)	S (m²)	e (m)	P (MN)	Nbre tendons	N Tendons
End V4	0.00	1.46	0.94	5.46	6.84	1.31	0.00	0.00	2
End V5	6 115.02	1.46	0.94	5.46	6.84	1.31	3.29	1.60	4
End V6	12 606.35	1.46	0.94	5.46	6.84	1.31	6.79	3.30	4
End V7	17 716.72	1.46	0.94	5.46	6.84	1.31	9.54	4.64	6
End V8	21 432.04	1.46	0.94	5.46	6.84	1.31	11.54	5.62	6
End V9	23 741.79	1.46	0.94	5.46	6.84	1.31	12.79	6.22	8
End V10	24 468.99	1.46	0.94	5.46	6.84	1.31	13.18	6.41	8
End V11	23 752.40	1.46	0.94	5.46	6.84	1.31	12.79	6.22	8
End V12	21 684.23	1.46	0.94	5.46	6.84	1.31	11.68	5.68	8
End VC3	19 322.65	1.46	0.94	5.46	6.84	1.31	10.41	5.06	6
End VC2	14 888.54	1.46	0.94	5.46	6.84	1.31	8.02	3.90	6
End VC1	9 101.88	1.46	0.94	5.46	6.84	1.31	4.90	2.39	4

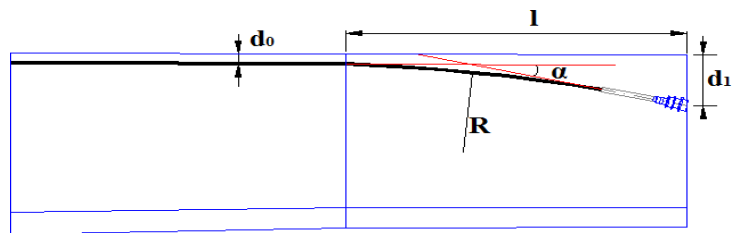
End VSC2	1 952.70	1.46	0.94	5.46	6.84	1.31	1.05	0.51	2
End VSC1	0.00	1.36	1.04	8.04	13.47	1.21	0.00	0.00	2

Plan tendon layout



Vrs	Tendons	Zi (m)	Zi+1 (m)	Xi (m)	Xi+1 (m)	Ri (m)	Ri+1 (m)	βi (rad)	βi+1 (rad)	Equation		
End V3	T1	Straight line										
End V5	T2	0.2	0.1	1.1	1.1	6.05	6.05	0.18	0.18	0.2	-0.083	X ²
End V6	T3	0.4	0.2	2.2	2.2	12.10	12.10	0.18	0.18	0.4	-0.041	X ²
End V7	T4	Straight line										
End V8	T5	0.2	0.1	1.1	1.1	6.05	6.05	0.18	0.18	0.2	-0.083	X ²
End V10	T6	0.4	0.2	2.2	2.2	12.10	12.10	0.18	0.18	0.4	-0.041	X ²

Elevation tendon layout



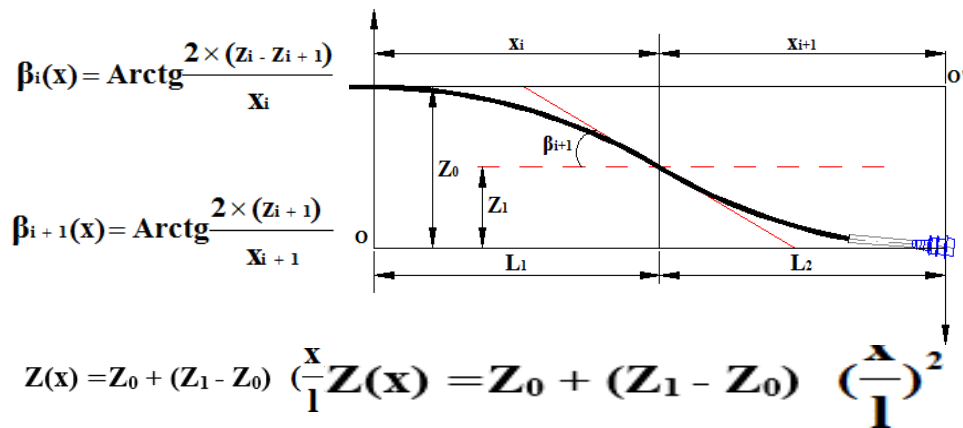
$$y = d_0 + (d_1 - d_0) \times \left(\frac{x}{l} \right)^2$$

$$R_c(x) = \frac{x^2}{2 \times (d_1 - d_0)} \geq R_{\min}$$

$$\alpha_n = \text{artg} \left(2 \times (d_1 - d_0) \right) \times \frac{x}{l^2}$$

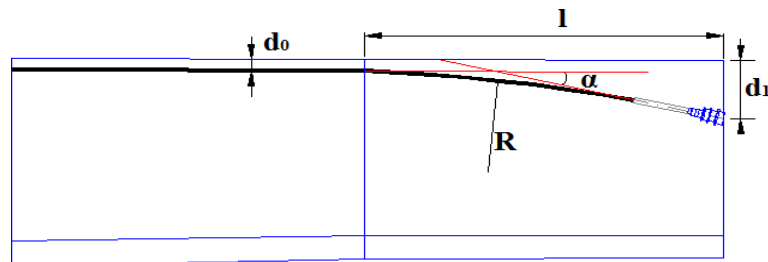
Vrs	Tendons	L (m)	d ₀ (m)	d ₁ (m)	R _i (m)	α (rad)	Equation		
End V3	T1	2.20	0.35	0.84	14.03	0.21	0.15	0.69	x ²
End V5	T2	2.20	0.35	0.84	14.03	0.21	0.15	0.69	x ²
End V6	T3	2.20	0.35	0.84	14.03	0.21	0.15	0.69	x ²
End V7	T4	4.40	0.15	0.84	4.94	0.35	0.35	0.49	x ²
End V8	T5	4.40	0.15	0.84	4.94	0.35	0.35	0.49	x ²
End V10	T6	4.40	0.15	0.84	4.94	0.35	0.35	0.49	x ²

Plan tendon layout



Vrs	Tendons	Z _i (m)	Z _{i+1} (m)	X _i (m)	X _{i+1} (m)	R _i (m)	R _{i+1} (m)	β _i (rad)	β _{i+1} (rad)	Equation		
End VSC	T4	0.20	0.10	2.20	2.20	24.20	24.20	0.09	0.09	0.2	-0.021	X ²
End V1	T3	Straight line										
End V3	T2	0.20	0.10	2.20	1.60	24.20	12.80	0.09	0.12	0.2	-0.028	X ²
End V12	T1	Straight line										
End V10	T1	Straight line										
End V8	T2	0.20	0.10	2.20	2.20	24.20	24.20	0.09	0.09	0.2	-0.021	X ²
End V6	T3	Straight line										
End V5	T4	0.20	0.10	2.20	2.20	24.20	24.20	0.09	0.09	0.2	-0.021	X ²

Elevation tendon layout



APPENDIX C : CALCULATIONS OF THE RETAINED

Vrs	Tendons	L (m)	d ₀ (m)	d ₁ (m)	R _i (m)	α (rad)	Equation		
End VSC	T4	4.40	0.15	0.84	14.03	0.21	0.15	0.69	x^2
End V1	T3	4.40	0.15	0.84	14.03	0.21	0.15	0.69	x^2
End V3	T2	2.20	0.35	0.84	4.94	0.35	0.35	0.49	x^2
End V12	T1	2.20	0.35	0.84	4.94	0.35	0.35	0.49	x^2
End V10	T1	2.20	0.35	0.84	4.94	0.35	0.35	0.49	x^2
End V8	T3	2.20	0.35	0.84	4.94	0.35	0.35	0.49	x^2
End V6	T2	4.40	0.15	0.84	14.03	0.21	0.15	0.69	x^2
End V5	T1	3.80	0.15	0.84	10.46	0.25	0.15	0.69	x^2

E

Stress verification for the service phase

σ_{p0}	1 488 MPa
P_0	3.12 MN
P_0	2.71 MN
ΔP	25 %

For the main span

For the side span

f_{c28}	35 MPa
σ_{bc}	21 MPa
f_{t28}	-2.7 MPa
σ_{bt1}	-4.05 MPa
σ_{bt2}	-2.7 MPa

1. Unloaded bridge :

$$\sigma(y) = \frac{P}{B} + \frac{Pe_0 + M}{I}y$$

	M (kN.m)	Cantilever tendons	Continuity tendons	v' (m)	v (m)	I (m ⁴)	S (m ²)	ef (m)	ec (m)	Pf (MN)	Pc (MN)	σ_{Sup} (MPa)	σ_{Inf} (MPa)	OBS
Beginning VSC1	0	0	2	1.36	1.04	7.35	13.40	0.00	0.52	0.00	4.10	0.00	0.70	OK
Beginnig V01	-1 115.66	0	2	1.46	0.94	5.46	6.84	0.00	0.97	0.00	4.10	0.11	1.37	OK
Beginnig V02	-5 528.56	0	4	1.46	0.94	5.46	6.84	0.00	1.08	0.00	8.21	0.63	2.09	OK
Beginnig V03	-8 992.7	0	6	1.46	0.94	5.46	6.84	0.00	1.09	0.00	12.31	1.04	2.98	OK
Beginnig VC C1-P1	-11 508.06	0	6	1.46	0.94	5.46	6.84	0.00	1.09	0.00	12.31	1.47	2.31	OK
Beginnig V04	-12 734.38	2	8	1.46	0.94	5.46	6.84	0.34	1.09	4.10	16.41	2.35	4.00	OK
Beginnig V05	-13 591.4	4	8	1.46	0.94	5.46	6.84	0.57	1.09	8.21	16.41	3.66	3.50	OK
Beginnig V06	-13 499.64	6	8	1.46	0.94	5.46	6.84	0.64	1.09	12.31	16.41	4.80	3.27	OK
Beginnig V07	-12 459.11	8	8	1.46	0.94	5.46	6.84	0.68	1.09	16.41	16.41	5.79	3.27	OK
Beginnig V08	-10 469.82	10	6	1.46	0.94	5.46	6.84	0.70	1.09	20.51	12.31	6.76	1.75	OK
Beginnig V09	-7 531.75	12	6	1.46	0.94	5.46	6.84	0.72	1.09	24.62	12.31	7.44	2.23	OK
Beginnig V10	-3 644.92	14	4	1.46	0.94	5.46	6.84	0.73	1.08	28.72	8.21	8.11	1.19	OK
Beginnig V11	1 190.69	16	2	1.46	0.94	5.46	6.84	0.73	0.97	32.82	4.10	8.63	0.37	OK
Beginnig V12	6 975.06	18	2	1.46	0.94	5.46	6.84	0.74	0.52	36.93	4.10	9.13	1.13	OK
Beginnig V13	13 708.2	20	0	1.46	0.94	5.46	6.84	0.75	0.00	41.03	0.00	8.94	1.44	OK

Beginnig V14	21 390.12	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	8.59	3.51	OK
Beginnig V15	30 020.8	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	8.13	5.75	OK
Beginnig VSP	39 600.25	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	6.40	8.44	OK
Axis VSP P1	43 381.04	24	0	1.36	1.04	7.35	13.40	0.81	0.00	49.23	0.00	3.18	4.32	OK
Beginnig V16	39 120.57	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	6.48	8.31	OK
Beginnig V17	28 221.99	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	8.44	5.27	OK
Beginnig V18	18 272.18	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	9.12	2.67	OK
Beginnig V19	9 271.14	20	2	1.46	0.94	5.46	6.84	0.75	0.00	41.03	4.10	10.30	0.85	OK
Beginnig V20	1 218.87	18	2	1.46	0.94	5.46	6.84	0.74	0.97	36.93	4.10	9.81	0.08	OK
Beginnig V21	-5 884.63	16	4	1.46	0.94	5.46	6.84	0.73	1.08	32.82	8.21	9.61	0.39	OK
Beginnig V22	-12 039.36	14	6	1.46	0.94	5.46	6.84	0.73	1.09	28.72	12.31	9.37	0.76	OK
Beginnig V23	-17 245.32	12	8	1.46	0.94	5.46	6.84	0.72	1.09	24.62	16.41	8.94	1.43	OK
Beginnig V24	-21 502.51	10	10	1.46	0.94	5.46	6.84	0.70	1.13	20.51	20.51	8.18	2.61	OK
Beginnig V25	-24 810.93	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	7.60	1.98	OK
Beginnig V26	-27 170.58	6	12	1.46	0.94	5.46	6.84	0.64	1.13	12.31	24.62	6.64	3.47	OK
Beginnig V27	-28 581.46	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	5.74	3.34	OK
Beginnig VC P1-P2	-29 043.58	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	4.65	3.50	OK
Beginnig V29	-28 776.62	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	4.60	3.57	OK
Beginnig V30	-27 580.38	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	5.56	3.61	OK
Beginnig V31	-25 435.37	6	10	1.46	0.94	5.46	6.84	0.64	1.13	12.31	20.51	6.54	2.09	OK
Beginnig V32	-22 341.6	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	7.18	2.64	OK
Beginnig V33	-18 299.05	10	8	1.46	0.94	5.46	6.84	0.70	1.09	20.51	16.41	7.94	1.45	OK
Beginnig V34	-13 307.73	12	6	1.46	0.94	5.46	6.84	0.72	1.09	24.62	12.31	8.43	0.69	OK
Beginnig V35	-7 367.64	14	4	1.46	0.94	5.46	6.84	0.73	1.08	28.72	8.21	8.75	0.19	OK
Beginnig V36	-478.79	16	2	1.46	0.94	5.46	6.84	0.73	0.97	32.82	4.10	8.92	-0.07	OK
Beginnig V37	7 358.84	18	2	1.46	0.94	5.46	6.84	0.74	0.52	36.93	4.10	9.07	1.23	OK
Beginnig V38	16 145.23	20	0	1.46	0.94	5.46	6.84	0.75	0.00	41.03	0.00	8.52	2.09	OK
Beginnig V39	25 880.4	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	7.81	4.71	OK
Beginnig V40	36 564.33	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	7.01	7.50	OK
Beginnig VSP	48 197.03	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	4.92	10.74	OK

Axis VSP P2	52 724.46	24	0	1.36	1.04	7.35	13.40	0.81	0.00	49.23	0.00	1.86	6.05	OK
Beginnig V41	48 368.7	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	4.89	10.78	OK
Beginnig V42	37 208.08	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	6.90	7.67	OK
Beginnig V43	26 996.23	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	7.62	5.01	OK
Beginnig V44	17 733.15	20	2	1.46	0.94	5.46	6.84	0.75	0.00	41.03	4.10	8.84	3.11	OK
Beginnig V45	9 418.84	18	2	1.46	0.94	5.46	6.84	0.74	0.97	36.93	4.10	8.40	2.27	OK
Beginnig V46	2 053.3	16	4	1.46	0.94	5.46	6.84	0.73	1.08	32.82	8.21	8.24	2.51	OK
Beginnig V47	-4 363.47	14	6	1.46	0.94	5.46	6.84	0.73	1.09	28.72	12.31	8.05	2.81	OK
Beginnig V48	-9 831.47	12	8	1.46	0.94	5.46	6.84	0.72	1.09	24.62	16.41	7.66	3.41	OK
Beginnig V49	-14 350.71	10	10	1.46	0.94	5.46	6.84	0.70	1.13	20.51	20.51	6.95	4.52	OK
Beginnig V50	-17 921.17	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	6.41	3.82	OK
Beginnig V51	-20 542.86	6	12	1.46	0.94	5.46	6.84	0.64	1.13	12.31	24.62	5.50	5.24	OK
Beginnig V52	-22 215.79	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	4.64	5.05	OK
Beginnig VC P2-P3	-22 939.94	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	3.60	5.13	OK
Beginnig V53	-22 863.57	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	3.59	5.15	OK
Beginnig V54	-21 929.37	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	4.59	5.12	OK
Beginnig V55	-20 046.4	6	10	1.46	0.94	5.46	6.84	0.64	1.13	12.31	20.51	5.62	3.53	OK
Beginnig V56	-17 214.66	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	6.29	4.01	OK
Beginnig V57	-13 434.16	10	8	1.46	0.94	5.46	6.84	0.70	1.09	20.51	16.41	7.10	2.75	OK
Beginnig V58	-8 704.88	12	6	1.46	0.94	5.46	6.84	0.72	1.09	24.62	12.31	7.64	1.92	OK
Beginnig V59	-3 026.84	14	4	1.46	0.94	5.46	6.84	0.73	1.08	28.72	8.21	8.00	1.35	OK
Beginnig V60	3 599.98	16	2	1.46	0.94	5.46	6.84	0.73	0.97	32.82	4.10	8.22	1.02	OK
Beginnig V61	11 175.56	18	2	1.46	0.94	5.46	6.84	0.74	0.52	36.93	4.10	8.41	2.25	OK
Beginnig V61	19 699.92	20	0	1.46	0.94	5.46	6.84	0.75	0.00	41.03	0.00	7.90	3.04	OK
Beginnig V62	29 173.04	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	7.25	5.59	OK
Beginnig V63	39 594.93	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	6.48	8.31	OK
Beginning VSP	50 965.59	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	4.44	11.48	OK
Axis VSP P3	55 397.73	24	0	1.36	1.04	7.35	13.40	0.81	0.00	49.23	0.00	1.48	6.55	OK
Beginnig V64	50 681.65	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	4.49	11.40	OK
Beginnig V65	38 530.15	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	6.67	8.02	OK

Beginnig V66	27 327.41	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	7.57	5.10	OK
Beginnig V67	17 073.45	20	2	1.46	0.94	5.46	6.84	0.75	0.00	41.03	4.10	8.96	2.94	OK
Beginnig V68	7 768.25	18	2	1.46	0.94	5.46	6.84	0.74	0.97	36.93	4.10	8.68	1.83	OK
Beginnig V69	588.17	16	4	1.46	0.94	5.46	6.84	0.73	1.08	32.82	8.21	8.50	2.12	OK
Beginnig V70	-7 995.83	14	6	1.46	0.94	5.46	6.84	0.73	1.09	28.72	12.31	8.67	1.84	OK
Beginnig V71	-14 455.25	12	8	1.46	0.94	5.46	6.84	0.72	1.09	24.62	16.41	8.46	2.18	OK
Beginnig V72	-20 000.38	10	10	1.46	0.94	5.46	6.84	0.70	1.13	20.51	20.51	7.92	3.01	OK
Beginnig V73	-24 493.67	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	7.55	2.06	OK
Beginnig V74	-27 955.09	6	12	1.46	0.94	5.46	6.84	0.64	1.13	12.31	24.62	6.78	3.26	OK
Beginnig V75	-30 383.65	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	6.05	2.86	OK
Beginnig VC P3-P4	-31 770.61	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	5.12	2.77	OK
Beginnig V76	-32 062.82	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	5.17	2.69	OK
Beginnig V77	-31 459.58	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	6.23	2.57	OK
Beginnig V78	-29 819.93	6	12	1.46	0.94	5.46	6.84	0.64	1.13	12.31	24.62	7.10	2.76	OK
Beginnig V79	-27 147.23	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	8.00	1.35	OK
Beginnig V80	-23 442.39	10	10	1.46	0.94	5.46	6.84	0.70	1.09	20.51	20.51	8.66	1.87	OK
Beginnig V81	-18 706.64	12	8	1.46	0.94	5.46	6.84	0.72	1.09	24.62	16.41	9.19	1.04	OK
Beginnig V82	-12 941.53	14	6	1.46	0.94	5.46	6.84	0.73	1.08	28.72	12.31	9.55	0.49	OK
Beginnig V83	-6 148.92	16	4	1.46	0.94	5.46	6.84	0.73	0.97	32.82	8.21	9.81	0.08	OK
Beginnig V84	1 669.01	18	2	1.46	0.94	5.46	6.84	0.74	0.52	36.93	4.10	10.05	-0.29	OK
Beginnig V85	10 509.75	20	0	1.46	0.94	5.46	6.84	0.75	0.00	41.03	0.00	9.49	0.58	OK
Beginnig V86	20 370.49	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	8.76	3.24	OK
Beginnig V87	31 248.09	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	7.92	6.07	OK
Beginnig VSP	43 135.66	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	5.79	9.39	OK
Axis VSP P4	47 866.23	24	0	1.36	1.04	7.35	13.40	0.81	0.00	49.23	0.00	2.54	5.15	OK
Beginnig V88	43 677.61	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	5.70	9.53	OK
Beginnig V89	33 271.92	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	7.57	6.62	OK
Beginnig V90	23 885.49	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	8.16	4.18	OK
Beginnig V91	15 514.95	20	0	1.46	0.94	5.46	6.84	0.75	0.00	41.03	0.00	8.62	1.92	OK
Beginnig V92	8 160.18	18	2	1.46	0.94	5.46	6.84	0.74	0.52	36.93	4.10	8.93	1.44	OK

Beginnig V93	1 828.35	16	2	1.46	0.94	5.46	6.84	0.73	0.97	32.82	4.10	8.52	0.54	OK
Beginnig V94	-3 479.62	14	4	1.46	0.94	5.46	6.84	0.73	1.08	28.72	8.21	8.08	1.23	OK
Beginnig V95	-7 762.17	12	6	1.46	0.94	5.46	6.84	0.72	1.09	24.62	12.31	7.48	2.17	OK
Beginnig V96	-11 018.05	10	6	1.46	0.94	5.46	6.84	0.70	1.09	20.51	12.31	6.86	1.60	OK
Beginnig V97	-13 246.75	8	8	1.46	0.94	5.46	6.84	0.68	1.09	16.41	16.41	5.92	3.06	OK
Beginnig V98	-14 275.93	6	8	1.46	0.94	5.46	6.84	0.64	1.09	12.31	16.41	4.93	3.06	OK
Beginnig V99	-14 239.49	4	8	1.46	0.94	5.46	6.84	0.57	1.09	8.21	16.41	3.78	3.32	OK
Beginnig VC P4-C2	-13 254.28	2	8	1.46	0.94	5.46	6.84	0.34	1.09	4.10	16.41	2.44	3.87	OK
Beginnig V100	-11 934.73	2	6	1.46	0.94	5.46	6.84	0.00	1.09	4.10	12.31	2.14	2.80	OK
Beginnig V101	-9 291.17	0	6	1.46	0.94	5.46	6.84	0.00	1.09	0.00	12.31	1.09	2.90	OK
Beginnig V102	-5 698.84	0	4	1.46	0.94	5.46	6.84	0.00	1.08	0.00	8.21	0.66	2.05	OK
Beginnig VSC	-1 157.74	0	2	1.46	0.94	5.46	6.84	0.00	0.97	0.00	4.10	0.11	1.35	OK
End VSC	0.00	0	2	1.36	1.04	7.35	13.40	0.00	0.52	0.00	0.00	0.00	0.00	OK

2. Unloaded bridge :

	M (kN.m)	Cantilever tendons	Continuity tendons	v' (m)	v (m)	I (m⁴)	S (m²)	ef (m)	ec (m)	Pf (MN)	Pc (MN)	σ^{Sup} (MPa)	σ^{Inf} (MPa)	OBS
Beginning VSC1	0	0	2	1.36	1.04	7.35	13.40	0.00	0.52	0.00	4.10	0.00	0.70	OK
Beginnig V01	-1 119.61	0	2	1.46	0.94	5.46	6.84	0.00	0.97	0.00	4.10	0.11	1.36	OK
Beginnig V02	-5 571.33	0	4	1.46	0.94	5.46	6.84	0.00	1.08	0.00	8.21	0.63	2.08	OK
Beginnig V03	-9 056.71	0	6	1.46	0.94	5.46	6.84	0.00	1.09	0.00	12.31	1.05	2.97	OK
Beginnig VC C1-P1	-11 575.74	0	6	1.46	0.94	5.46	6.84	0.00	1.09	0.00	12.31	1.48	2.29	OK
Beginnig V04	-12 793.07	2	8	1.46	0.94	5.46	6.84	0.34	1.09	4.10	16.41	2.36	3.99	OK
Beginnig V05	-13 621.7	4	8	1.46	0.94	5.46	6.84	0.57	1.09	8.21	16.41	3.67	3.49	OK
Beginnig V06	-13 483.99	6	8	1.46	0.94	5.46	6.84	0.64	1.09	12.31	16.41	4.80	3.27	OK
Beginnig V07	-12 379.94	8	8	1.46	0.94	5.46	6.84	0.68	1.09	16.41	16.41	5.77	3.29	OK
Beginnig V08	-10 309.54	10	6	1.46	0.94	5.46	6.84	0.70	1.09	20.51	12.31	6.74	1.79	OK
Beginnig V09	-7 272.8	12	6	1.46	0.94	5.46	6.84	0.72	1.09	24.62	12.31	7.39	2.30	OK
Beginnig V10	-3 269.71	14	4	1.46	0.94	5.46	6.84	0.73	1.08	28.72	8.21	8.05	1.29	OK

Beginnig V11	1 699.71	16	2	1.46	0.94	5.46	6.84	0.73	0.97	32.82	4.10	8.55	0.51	OK
Beginnig V12	7 635.48	18	2	1.46	0.94	5.46	6.84	0.74	0.52	36.93	4.10	9.02	1.30	OK
Beginnig V13	14 537.6	20	0	1.46	0.94	5.46	6.84	0.75	0.00	41.03	0.00	8.79	1.66	OK
Beginnig V14	22 406.06	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	8.41	3.78	OK
Beginnig V15	31 240.86	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	7.92	6.07	OK
Beginning VSP	41 042	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	6.15	8.83	OK
Axis VSP P1	44 911.23	24	0	1.36	1.04	7.35	13.40	0.81	0.00	49.23	0.00	2.96	4.61	OK
Beginnig V16	40 524.42	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	6.24	8.69	OK
Beginnig V17	29 299.92	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	8.26	5.55	OK
Beginnig V18	19 041.76	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	8.99	2.88	OK
Beginnig V19	9 749.94	20	2	1.46	0.94	5.46	6.84	0.75	0.00	41.03	4.10	10.22	0.98	OK
Beginnig V20	1 424.47	18	2	1.46	0.94	5.46	6.84	0.74	0.97	36.93	4.10	9.77	0.14	OK
Beginnig V21	-5 934.66	16	4	1.46	0.94	5.46	6.84	0.73	1.08	32.82	8.21	9.62	0.37	OK
Beginnig V22	-12 327.44	14	6	1.46	0.94	5.46	6.84	0.73	1.09	28.72	12.31	9.42	0.68	OK
Beginnig V23	-17 753.88	12	8	1.46	0.94	5.46	6.84	0.72	1.09	24.62	16.41	9.03	1.29	OK
Beginnig V24	-22 213.98	10	10	1.46	0.94	5.46	6.84	0.70	1.13	20.51	20.51	8.30	2.42	OK
Beginnig V25	-25 707.74	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	7.75	1.74	OK
Beginnig V26	-28 235.15	6	12	1.46	0.94	5.46	6.84	0.64	1.13	12.31	24.62	6.83	3.18	OK
Beginnig V27	-29 796.22	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	5.94	3.02	OK
Beginnig VC P1-P2	-30 390.94	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	4.88	3.14	OK
Beginnig V29	-30 208.78	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	4.85	3.19	OK
Beginnig V30	-29 113.1	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	5.83	3.20	OK
Beginnig V31	-27 051.08	6	10	1.46	0.94	5.46	6.84	0.64	1.13	12.31	20.51	6.82	1.66	OK
Beginnig V32	-24 022.72	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	7.46	2.19	OK
Beginnig V33	-20 028.01	10	8	1.46	0.94	5.46	6.84	0.70	1.09	20.51	16.41	8.24	0.99	OK
Beginnig V34	-15 066.96	12	6	1.46	0.94	5.46	6.84	0.72	1.09	24.62	12.31	8.73	0.22	OK
Beginnig V35	-9 139.57	14	4	1.46	0.94	5.46	6.84	0.73	1.08	28.72	8.21	9.06	-0.28	OK
Beginnig V36	-2 245.84	16	2	1.46	0.94	5.46	6.84	0.73	0.97	32.82	4.10	9.23	-0.54	OK
Beginnig V37	5 614.24	18	2	1.46	0.94	5.46	6.84	0.74	0.52	36.93	4.10	9.37	0.76	OK
Beginnig V38	14 440.67	20	0	1.46	0.94	5.46	6.84	0.75	0.00	41.03	0.00	8.81	1.63	OK

Beginnig V39	24 233.43	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	8.10	4.27	OK
Beginnig V40	34 992.54	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	7.28	7.08	OK
Beginning VSP	46 717.99	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	5.17	10.34	OK
Axis VSP P2	51 286.97	24	0	1.36	1.04	7.35	13.40	0.81	0.00	49.23	0.00	2.06	5.78	OK
Beginnig V41	47 013.05	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	5.12	10.42	OK
Beginnig V42	36 099.01	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	7.09	7.37	OK
Beginnig V43	26 151.31	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	7.77	4.78	OK
Beginnig V44	17 169.95	20	2	1.46	0.94	5.46	6.84	0.75	0.00	41.03	4.10	8.94	2.96	OK
Beginnig V45	9 154.94	18	2	1.46	0.94	5.46	6.84	0.74	0.97	36.93	4.10	8.44	2.20	OK
Beginnig V46	2 106.27	16	4	1.46	0.94	5.46	6.84	0.73	1.08	32.82	8.21	8.24	2.52	OK
Beginnig V47	-3 976.06	14	6	1.46	0.94	5.46	6.84	0.73	1.09	28.72	12.31	7.98	2.92	OK
Beginnig V48	-9 092.04	12	8	1.46	0.94	5.46	6.84	0.72	1.09	24.62	16.41	7.54	3.61	OK
Beginnig V49	-13 241.68	10	10	1.46	0.94	5.46	6.84	0.70	1.13	20.51	20.51	6.76	4.82	OK
Beginnig V50	-16 424.98	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	6.16	4.22	OK
Beginnig V51	-18 641.93	6	12	1.46	0.94	5.46	6.84	0.64	1.13	12.31	24.62	5.18	5.75	OK
Beginnig V52	-19 892.54	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	4.24	5.67	OK
Beginnig VC P2-P3	-20 176.81	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	3.12	5.87	OK
Beginnig V53	-19 768.86	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	3.05	5.98	OK
Beginnig V54	-18 362.72	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	3.98	6.08	OK
Beginnig V55	-15 990.24	6	10	1.46	0.94	5.46	6.84	0.64	1.13	12.31	20.51	4.92	4.62	OK
Beginnig V56	-12 651.42	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	5.51	5.23	OK
Beginnig V57	-8 346.26	10	8	1.46	0.94	5.46	6.84	0.70	1.09	20.51	16.41	6.23	4.11	OK
Beginnig V58	-3 074.75	12	6	1.46	0.94	5.46	6.84	0.72	1.09	24.62	12.31	6.67	3.42	OK
Beginnig V59	3 163.1	14	4	1.46	0.94	5.46	6.84	0.73	1.08	28.72	8.21	6.94	3.01	OK
Beginnig V60	10 367.3	16	2	1.46	0.94	5.46	6.84	0.73	0.97	32.82	4.10	7.05	2.83	OK
Beginnig V61	18 537.84	18	2	1.46	0.94	5.46	6.84	0.74	0.52	36.93	4.10	7.14	4.22	OK
Beginnig V61	27 674.72	20	0	1.46	0.94	5.46	6.84	0.75	0.00	41.03	0.00	6.53	5.17	OK
Beginnig V62	37 777.94	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	5.77	7.89	OK
Beginnig V63	48 847.51	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	4.89	10.78	OK
Beginning VSP	60 883.42	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	2.73	14.13	OK

Axis VSP P3	65 565.29	24	0	1.36	1.04	7.35	13.40	0.81	0.00	49.23	0.00	0.04	8.43	OK
Beginnig V64	59 665.2	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	2.94	13.81	OK
Beginnig V65	44 529.23	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	5.63	9.63	OK
Beginnig V66	30 508.06	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	7.02	5.95	OK
Beginnig V67	17 601.71	20	2	1.46	0.94	5.46	6.84	0.75	0.00	41.03	4.10	8.87	3.08	OK
Beginnig V68	5 810.17	18	2	1.46	0.94	5.46	6.84	0.74	0.97	36.93	4.10	9.02	1.31	OK
Beginnig V69	-4 866.55	16	4	1.46	0.94	5.46	6.84	0.73	1.08	32.82	8.21	9.44	0.66	OK
Beginnig V70	-14 429.38	14	6	1.46	0.94	5.46	6.84	0.73	1.09	28.72	12.31	9.78	0.12	OK
Beginnig V71	-22 908.05	12	8	1.46	0.94	5.46	6.84	0.72	1.09	24.62	16.41	9.91	-0.08	OK
Beginnig V72	-30 207.18	10	10	1.46	0.94	5.46	6.84	0.70	1.13	20.51	20.51	9.68	0.28	OK
Beginnig V73	-36 162.99	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	9.55	-1.06	OK
Beginnig V74	-40 773.76	6	12	1.46	0.94	5.46	6.84	0.64	1.13	12.31	24.62	8.99	-0.17	OK
Beginnig V75	-44 038.16	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	8.40	-0.79	OK
Beginnig VC P3-P4	-45 951.73	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	7.56	-1.02	OK
Beginnig V76	-46 287.06	2	12	1.46	0.94	5.46	6.84	0.34	1.13	4.10	24.62	7.62	-1.11	OK
Beginnig V77	-45 282.36	4	12	1.46	0.94	5.46	6.84	0.57	1.13	8.21	24.62	8.61	-1.12	OK
Beginnig V78	-42 930.33	6	12	1.46	0.94	5.46	6.84	0.64	1.13	12.31	24.62	9.36	-0.75	OK
Beginnig V79	-39 231.79	8	10	1.46	0.94	5.46	6.84	0.68	1.13	16.41	20.51	10.08	-1.88	OK
Beginnig V80	-34 188	10	10	1.46	0.94	5.46	6.84	0.70	1.09	20.51	20.51	10.51	-1.00	OK
Beginnig V81	-27 800.61	12	8	1.46	0.94	5.46	6.84	0.72	1.09	24.62	16.41	10.76	-1.39	OK
Beginnig V82	-20 071.7	14	6	1.46	0.94	5.46	6.84	0.73	1.08	28.72	12.31	10.77	-1.42	OK
Beginnig V83	-11 003.79	16	4	1.46	0.94	5.46	6.84	0.73	0.97	32.82	8.21	10.65	-1.22	OK
Beginnig V84	-599.77	18	2	1.46	0.94	5.46	6.84	0.74	0.52	36.93	4.10	10.44	-0.90	OK
Beginnig V85	11 137.03	20	0	1.46	0.94	5.46	6.84	0.75	0.00	41.03	0.00	9.38	0.75	OK
Beginnig V86	24 202.86	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	8.10	4.26	OK
Beginnig V87	38 593.58	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	6.66	8.04	OK
Beginnig VSP	54 297.88	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	3.87	12.37	OK
Axis VSP P4	60 706.62	24	0	1.36	1.04	7.35	13.40	0.81	0.00	49.23	0.00	0.73	7.53	OK
Beginnig V88	56 160.51	24	0	1.46	0.94	5.46	6.84	0.71	0.00	49.23	0.00	3.55	12.87	OK
Beginnig V89	44 799.66	24	0	1.46	0.94	5.46	6.84	0.72	0.00	49.23	0.00	5.59	9.70	OK

Beginnig V90	34 474.29	22	0	1.46	0.94	5.46	6.84	0.73	0.00	45.13	0.00	6.34	7.01	OK
Beginnig V91	25 187.53	20	0	1.46	0.94	5.46	6.84	0.75	0.00	41.03	0.00	6.96	4.51	OK
Beginnig V92	16 942.18	18	2	1.46	0.94	5.46	6.84	0.74	0.52	36.93	4.10	7.42	3.79	OK
Beginnig V93	9 740.7	16	2	1.46	0.94	5.46	6.84	0.73	0.97	32.82	4.10	7.16	2.66	OK
Beginnig V94	3 585.24	14	4	1.46	0.94	5.46	6.84	0.73	1.08	28.72	8.21	6.87	3.12	OK
Beginnig V95	-1 522.36	12	6	1.46	0.94	5.46	6.84	0.72	1.09	24.62	12.31	6.40	3.84	OK
Beginnig V96	-5 575.94	10	6	1.46	0.94	5.46	6.84	0.70	1.09	20.51	12.31	5.92	3.06	OK
Beginnig V97	-8 584.77	8	8	1.46	0.94	5.46	6.84	0.68	1.09	16.41	16.41	5.12	4.30	OK
Beginnig V98	-10 348.66	6	8	1.46	0.94	5.46	6.84	0.64	1.09	12.31	16.41	4.26	4.11	OK
Beginnig V99	-11 010.1	4	8	1.46	0.94	5.46	6.84	0.57	1.09	8.21	16.41	3.22	4.19	OK
Beginnig VC P4-C2	-10 705.2	2	8	1.46	0.94	5.46	6.84	0.34	1.09	4.10	16.41	2.00	4.55	OK
Beginnig V100	-9 868.76	2	6	1.46	0.94	5.46	6.84	0.00	1.09	4.10	12.31	1.79	3.35	OK
Beginnig V101	-7 873.45	0	6	1.46	0.94	5.46	6.84	0.00	1.09	0.00	12.31	0.85	3.28	OK
Beginnig V102	-4 911.8	0	4	1.46	0.94	5.46	6.84	0.00	1.08	0.00	8.21	0.52	2.26	OK
Beginnig VSC	-983.81	0	2	1.46	0.94	5.46	6.84	0.00	0.97	0.00	4.10	0.08	1.40	OK
End VSC	0	0	2	1.36	1.04	7.35	13.40	0.00	0.52	0.00	4.10	0.00	0.70	OK

Seismic study
Student Engineers : GAHAM & ARAR
Project : Final Year Ptoject

Bearing Device : Sizing / According to SETRA Technical Guide
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Bearing area :

$$\sigma_{\max} = R_{\max} / a \times b \leq \sigma_{\text{adm}} = 20 \text{ MPa}$$

$R_{\max}$: Bearing reaction at SLS.

$R_{\max}\text{-Piles}$	10.88 MN
$a \times b \geq$	5 440 cm x cm

$R_{\max}\text{-Cul}$	3.49 MN
$a \times b \geq$	1 745 cm x cm

We opt :

Piers	$a \times b \approx$	800 x 800 mm ²
Abutments	$a \times b \approx$	500 x 500 mm ²

2. Calculation of bearing thickness T :

$a/10 \leq T \leq a/5$ (non-buckling condition ; where **a** is the longest side)

Piers	$80.00 \leq T \leq 160.00 \text{ mm}$
Abutments	$50.00 \leq T \leq 100.00 \text{ mm}$

3. Sizing of shims :

The thickness of the shim must satisfy the following two conditions :

$$\begin{cases} t_s \geq \frac{a}{\beta} \frac{\sigma_m}{\sigma_e} \\ t_s \geq 2 \text{ mm} \end{cases} \text{ où } : \sigma_e = 235 \text{ kg/mm}^2 \text{ (for steel 24-1)} \quad \sigma_m = \frac{N_{\max}}{a \times b}; \beta = \frac{a \times b}{2 G (a+b)}$$

	Piers	Abutments
σ_m	17.00	13.96
β	22.22	13.86
t_s	2.60	2.14

Static modulus of elasticity	
G	0.90MPa

For the pier, We opt :

Elastomer layers : t_i	16 mm
Steel shims : t_s	3 mm
Edge covering :	4 mm
External covering top and bottom	8 mm

$$T_d = 152 \text{ mm}$$

$$T = 128 \text{ mm}$$

For the abutment, We opt :

Elastomer layers : t_i	10 mm
Steel shims : t_s	3 mm
Edge covering :	4 mm
External covering top and bottom	5 mm

$$T_d = 91 \text{ mm}$$

$$T = 70 \text{ mm}$$

Study of the abutment elements
Student Engineers : GAHAM & ARAR
Project : Final Year Project

Study of the front wall (Calculation relative to the base of the wall).

Evaluation of self-weight forces

Désignation	N (kN)	H (kN)	IFV(x/o)	IFH(y/o)	MFV/o	MFH/o
Front wall	3 118.31	/	0.66	2.08	2 058.09	/
Transition slab	148.05	/	2.30	5.94	525.58	/
Corbel	62.4	/	2.28	3.43	142.27	/
Prapet wall	293.28	/	1.9	3.34	557.23	/
Earth weight on TS	297.65	/	3.64	4.24	1 083.44	/
Earth pressure	/	976.19	/	-1.57	/	-1 532.61
Surcharge load 10kN/m ²	197.4	0	2.30	0.00	454.02	0
Surcharge pressure 10kN/m ²	/	161.65	/	-3.36	/	-542.33

Forces due to the deck

Rg	3 033.50	/	0.00	/	0.00	/
R(Al)	373.00	/	0.00	/	0.00	/
Rbc	1 197.50	/	0.00	/	0.00	/
RMc120	1 325.90	/	0.00	/	0.00	/
RD240	1 997.80	/	0.00	/	0.00	/
RST	80.40	/	0.00	/	0.00	/
Hf+r	/	173.00	/	-4.15	/	-1 098.55
HT°	/	135.00	/	-4.15	/	-857.25
Braking (Al)	/	18.60	/	-4.15	/	-118.11
Braking (Bc)	/	24.50	/	-4.15	/	-155.58

Load combination

At ULS

	N (kN)	H (kN)	M _H (KN.m)	M _V (KN.m)	ΔM (KN.m)
1.35G+0.9T	9 386.80	1 672.90	-4 323.60	5 894.92	1 571.32
1.35G++0.9T+1.6Q	9 702.64	1 931.54	-5 191.32	6 621.35	1 430.03
1.35G+0.9T+1.6(A(I)+ST)	10 112.24	1 702.66	-4 512.57	5 894.92	1 382.34
1.35G+0.9T+1.6(Bc+ST)	11 431.44	1 712.10	-4 572.52	5 894.92	1 322.40
1.35G+0.9T+1.35Mc120	11 176.77	1 672.90	-4 323.60	5 894.92	1 571.32
1.35G+0.9T+1.35D240	12 083.83	1 672.90	-4 323.60	5 894.92	1 571.32

Most unfavorable stresses

$$M_{\max} = -868.00 \text{ kN.m/ml}$$

$$M_{\max} = -868.00 \text{ kN.m/ml}$$

$$T_{\max} = 308.18 \text{ kN./ml}$$

$$T_{\max} = 308.18 \text{ kN./ml}$$

At SLS

	N (kN)	H (kN)	M _H (KN.m)	M _V (KN.m)	ΔM (KN.m)
G+0.6T	6 953.19	12 30.19	-3 145.51	4 366.60	1 221.09
G+0.6T+1.2Q	7 190.07	14 24.16	-3 796.31	4 911.43	1 115.12
G+0.6T+1.2A(I)+ST	7 481.19	12 52.51	-3 287.25	4 366.60	1 079.36
G+0.6T+1.2Bc+ST	8 470.59	12 59.587	-3 332.20	4 366.60	1 034.40
G+0.6T+Mc120	8 279.09	1 230.19	-3 145.51	4 366.60	1 221.09
G+0.6T+D240	8 950.99	1 230.19	-3 145.51	4 366.60	1 221.09

Most unfavorable stresses

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$$T_{\max} = 308.18 \text{ kN./ml}$$

$$T_{\max} = 308.18 \text{ kN./ml}$$

Determination of WERNER coefficients for pile calculation.

$\lambda \times L$	1	1.5	2	3	4	6
χ_{0P}	4.03	1.96	1.35	1.16	1.26	1.45
χ_{0M}	6.76	2.55	1.62	1.48	1.54	1.65

Study of the pier
Student Engineers : GAHAM & ARAR
Project : Final Year Project

At U L S

	N (kN)	H (kN)	M _H (kN.m)	M _V (kN.m)	ΔM (kN.m)
1.35G	26 059.23	348.30	3 117.29	0.00	3 117.29
1.35G+1.6Al+1.6ST+0.9T	29 061.63	392.78	3 515.38	0.00	3 515.38
1.35G+1.6Bc+1.6ST+0.9T	28 077.47	402.06	3 598.44	0.00	3 598.44
1.35G+1.35Mc120+0.9T	28 617.21	348.30	3 117.29	0.00	3 117.29
1.35G+1.35D240+0.9T	26 486.91	348.30	3 117.29	0.00	3 117.29

Most unfavorable stresses

$$M_{\max} = -868.00 \text{ kN.m/ml}$$

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$$T_{\max} = 308.18 \text{ kN./ml}$$

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At S L S

	N (kN)	H (kN)	M _H (kN.m)	M _V (kN.m)	ΔM (kN.m)
G+0.6T	19 303.13	249.20	2 230.34	0.00	2 230.34
G+1.2Al+ST+0.6T	21 491.57	282.56	2 528.91	0.00	2 528.91
G+1.2Bc+ST+0.6T	20 753.45	289.52	2 591.20	0.00	2 591.20
G+Mc120+0.6T	20 189.53	249.20	2 230.34	0.00	2 230.34
G+D240+0.6T	21 197.93	249.20	2 230.34	0.00	2 230.34

Most unfavorable stresses

$$M_{\max} = -868.00 \text{ kN.m/ml}$$

$$M_{\max} = -868.00 \text{ kN.m/ml}$$

$$T_{\max} = 308.18 \text{ kN./ml}$$

$$T_{\max} = 308.18 \text{ kN./ml}$$

Segment reinforcement

Upper slab reinforcement (section design for simple bending)

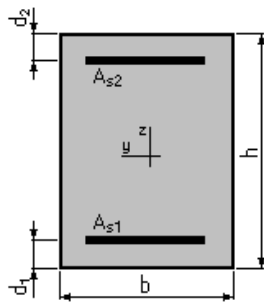
1. Assumptions:

Concrete: $f_{ck} = 35.0$ (MPa)

Reinforcing steel: $f_{yk} = 500.0$ (MPa)

- Cracking category: limited
- Account for compressed reinforcement
- No seismic design provisions
- Calculations according to BAEL 91 mod. 99

2. Section:



$$b = 100.0 \text{ (cm)}$$

$$h = 35.0 \text{ (cm)}$$

$$d_1 = 5.0 \text{ (cm)}$$

$$d_2 = 5.0 \text{ (cm)}$$

3. Applied moments:

	M_{max} (kN*m)	M_{min} (kN*m)
Ultimate Limit State (fundamental)	249.90	-336.00
Serviceability Limit State	185.10	-251.60
Accidental Limit State	0.00	0.00

4. Results:

Steel:

Theoretical area	$A_{s1} = 28.5 \text{ (cm}^2\text{)}$	$A_{s2} = 39.7 \text{ (cm}^2\text{)}$
Minimum area	$A_{s \min} = 4.1 \text{ (cm}^2\text{)}$	
Theoretical	$\rho = 2.27 \text{ (\%)} $	
Minimum	$\rho_{\min} = 0.14 \text{ (\%)} $	

Case analysis:

Case ULS	$M_{max} = 249.90 \text{ (kN*m)}$	$M_{min} = -336.00 \text{ (kN*m)}$
Safety factor:	1.34	Pivot: A
Neutral axis location:	$y = 7.3 \text{ (cm)}$	Internal forces lever arm: $Z = 27.1 \text{ (cm)}$
Concrete strain:	$\epsilon_b = 3.20 \text{ (\%)} $	Stress in steel:
Steel strain:	$\epsilon_s = 10.00 \text{ (\%)} $	in tension $\sigma_s = 434.8 \text{ (MPa)}$
		in compression $\sigma_s' = 200.1 \text{ (MPa)}$

Case SLS	$M_{max} = 185.10 \text{ (kN*m)}$	$M_{min} = -251.60 \text{ (kN*m)}$
Safety factor:	1.00	
Neutral axis location:	$y = 12.2 \text{ (cm)}$	
Internal forces lever arm:	$Z = 25.9 \text{ (cm)}$	
Maximum stress in concrete:	$\sigma_b = 11.3 \text{ (MPa)}$	
Concrete stress limit	$0,6 f_{cj} = 21.0 \text{ (MPa)}$	
Stress in steel:		
tension	$\sigma_s = 247.3 \text{ (MPa)}$	
compression	$\sigma'_s = 100.6 \text{ (MPa)}$	
Steel stress limit:	$\sigma_{s \text{ lim}} = 250.0 \text{ (MPa)}$	

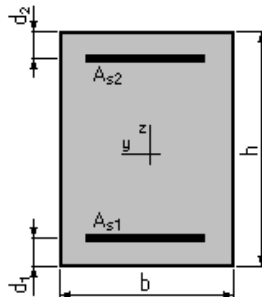
Lower slab reinforcement (section design for simple bending)

1. Assumptions:

Concrete: $f_{ck} = 35.0 \text{ (MPa)}$ **Reinforcing steel: $f_{yk} = 500.0 \text{ (MPa)}$**

- Cracking category: limited
- Account for compressed reinforcement
- No seismic design provisions
- Calculations according to BAEL 91 mod. 99

2. Section:



$b = 100.0 \text{ (cm)}$
 $h = 35.0 \text{ (cm)}$
 $d_1 = 5.0 \text{ (cm)}$
 $d_2 = 5.0 \text{ (cm)}$

3. Applied moments:

	$M_{max} \text{ (kN*m)}$	$M_{min} \text{ (kN*m)}$
Ultimate Limit State (fundamental)	46.70	-61.30
Serviceability Limit State	34.60	-45.40
Accidental Limit State	0.00	0.00

4. Results:

Steel:

Theoretical area	$A_{s1} = 5.0 \text{ (cm}^2\text{)}$	$A_{s2} = 6.6 \text{ (cm}^2\text{)}$
Minimum area	$A_{s \text{ min}} = 4.1 \text{ (cm}^2\text{)}$	
Theoretical	$\rho = 0.38 \text{ (\%)}$	
Minimum	$\rho_{min} = 0.14 \text{ (\%)}$	

Case analysis:

Case ULS	$M_{max} = 46.70$ (kN*m)	$M_{min} = -61.30$ (kN*m)
Safety factor:	1.42	Pivot: A
Neutral axis location:	$y = 2.4$ (cm)	Internal forces lever arm: $Z = 29.0$ (cm)
Concrete strain:	$\varepsilon_b = 0.87$ (‰)	Stress in steel:
Steel strain:	$\varepsilon_s = 10.00$ (‰)	in tension $\sigma_s = 434.8$ (MPa)

Case SLS	$M_{max} = 34.60$ (kN*m)	$M_{min} = -45.40$ (kN*m)
Safety factor:	1.00	
Neutral axis location:	$y = 6.6$ (cm)	
Internal forces lever arm:	$Z = 27.8$ (cm)	
Maximum stress in concrete:	$\sigma_b = 2.1$ (MPa)	
Concrete stress limit	$0,6 f_{cj} = 21.0$ (MPa)	
Stress in steel:		
tension	$\sigma_s = 22.3$ (MPa)	
compression	$\sigma_s' = 22.0$ (MPa)	
Steel stress limit:	$\sigma_{s\ lim} = 250.0$ (MPa)	

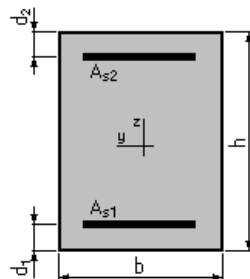
Web reinforcement (section design for combined bending)

1. Assumptions:

Concrete: $f_{ck} = 35.0$ (MPa) **Reinforcing steel: $f_{yk} = 500.0$ (MPa)**

- Cracking category: limited
- No seismic design provisions
- Calculated as a beam
- Calculations according to BAEL 91 mod. 99

2. Section:



$b = 100.0$ (cm)
 $h = 40.0$ (cm)
 $d_1 = 5.0$ (cm)
 $d_2 = 5.0$ (cm)

3. Applied forces:

Case N°	Type	N (kN)	M (kN*m)
1.	ULS	173.90	276.70
2.	SLS	126.10	208.30

4. Results:

Steel section area:

Theoretical area	$A_{s1} = 24.8 \text{ (cm}^2\text{)}$	$A_{s2} = 0.0 \text{ (cm}^2\text{)}$
Minimum area	$A_{s \text{ min}} = 4.6 \text{ (cm}^2\text{)}$	
Theoretical	$\rho = 0.71 \text{ (\%)}\text{}$	
Minimum	$\rho_{\text{min}} = 0.13 \text{ (\%)}\text{}$	

Case analysis:

Case N° 1: Type ULS	N = 173.90 (kN)	M = 276.70 (kN*m)
Safety factor:	1.38	Pivot: A
Neutral axis location:	$y = 8.3 \text{ (cm)}$	Internal forces lever arm: $Z = 31.7 \text{ (cm)}$
Steel strain:	$\varepsilon_s = 10.00 \text{ (\%)}\text{}$	Stress in steel:
Concrete strain:	$\varepsilon_b = 3.12 \text{ (\%)}\text{}$	in tension $\sigma_s = 434.8 \text{ (MPa)}$
Case N° 2: Type SLS	N = 126.10 (kN)	M = 208.30 (kN*m)
Safety factor:	1.00	
Neutral axis location:	$y = 13.8 \text{ (cm)}$	Internal forces lever arm: $Z = 30.4 \text{ (cm)}$
Maximum stress in concrete:	$\sigma_b = 10.8 \text{ (MPa)}$	
Stress limit: $0,6 f_{cj} = 21.0 \text{ (MPa)}$		
Stress in steel:		
tension	$\sigma_s = 250.0 \text{ (MPa)}$	
Stress limit in steel:	$\sigma_{s \text{ lim}} = 250.0 \text{ (MPa)}$	

Front wall reinforcement

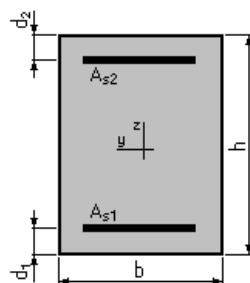
Section design for combined axial load and bending

1. Assumptions:

Concrete: $f_{ck} = 30.0 \text{ (MPa)}$ Reinforcing steel: $f_{yk} = 500.0 \text{ (MPa)}$

- Cracking category: limited
- No seismic design provisions
- Calculated as a column
- Calculations according to BAEL 91 mod. 99

2. Section:



$b = 100.0$	(cm)
$h = 289.0$	(cm)
$d_1 = 5.0$	(cm)
$d_2 = 5.0$	(cm)

3. Applied forces:

Case N°	Type	N (kN)	M (kN*m)
1.	ULS	1161.91	151.09
2.	SLS	860.67	117.41

4. Results:

Steel section area:

Theoretical area	$A_{s1} = 28.9 \text{ (cm}^2\text{)}$	$A_{s2} = 28.9 \text{ (cm}^2\text{)}$
Minimum area	$A_{s \text{ min}} = 57.8 \text{ (cm}^2\text{)}$	Maximum area $A_{s \text{ max}} = 1445.0 \text{ (cm}^2\text{)}$
Theoretical	$\rho = 0.20 \text{ (\%)} $	
Minimum	$\rho_{\text{min}} = 0.20 \text{ (\%)} $	Maximum $\rho_{\text{max}} = 5.00 \text{ (\%)} $

Case analysis:

Case N° 1: Type ULS	N = 1161.91 (kN)	M = 151.09 (kN*m)
Safety factor:	40.55	Pivot: C
Neutral axis location:	$y = 335.1 \text{ (cm)}$	Internal forces lever arm: $Z = 150.0 \text{ (cm)}$
Steel strain:	$\varepsilon_s = 0.00 \text{ (\%)} $	Stress in steel:
Concrete strain:	$\varepsilon_b = 3.17 \text{ (\%)} $	in compression $\sigma_s' = 434.8 \text{ (MPa)}$

Case N° 2: Type SLS	N = 860.67 (kN)	M = 117.41 (kN*m)
Safety factor:	49.06	
Neutral axis location:	$y = 681.7 \text{ (cm)}$	Internal forces lever arm: $Z = 152.5 \text{ (cm)}$
Maximum stress in concrete:	$\sigma_b = 0.4 \text{ (MPa)}$	
Stress limit: $0,6 f_{cj} = 18.0 \text{ (MPa)}$		
Stress in steel:		
Compression $\sigma_s' = 5.5 \text{ (MPa)}$		
Stress limit in steel: $\sigma_{s \text{ lim}} = 250.0 \text{ (MPa)}$		

Parapet wall reinforcement

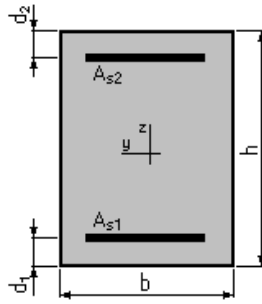
Section design for simple bending

1. Assumptions:

Concrete: $f_{ck} = 25.0 \text{ (MPa)}$ Reinforcing steel: $f_{yk} = 500.0 \text{ (MPa)}$

- Cracking category: limited
- Account for compressed reinforcement
- No seismic design provisions
- Calculations according to BAEL 91 mod. 99

2. Section:



$b = 100.0 \text{ (cm)}$
 $h = 40.0 \text{ (cm)}$
 $d_1 = 5.0 \text{ (cm)}$
 $d_2 = 5.0 \text{ (cm)}$

3. Applied moments:

	$M_{max} \text{ (kN*m)}$	$M_{min} \text{ (kN*m)}$
Ultimate Limit State (fundamental)	184.08	0.00
Serviceability Limit State	137.78	0.00
Accidental Limit State	0.00	0.00

4. Results:

Steel:

Theoretical area	$A_{s1} = 17.6 \text{ (cm}^2\text{)}$	$A_{s2} = 0.0 \text{ (cm}^2\text{)}$
Minimum area	$A_{s \text{ min}} = 3.6 \text{ (cm}^2\text{)}$	
Theoretical	$\rho = 0.50 \text{ (\%)}$	
Minimum	$\rho_{min} = 0.10 \text{ (\%)}$	

Case analysis:

Case ULS	$M_{max} = 184.08 \text{ (kN*m)}$	$M_{min} = 0.00 \text{ (kN*m)}$
Safety factor:	1.34	Pivot: A
Neutral axis location:	$y = 6.8 \text{ (cm)}$	Internal forces lever arm: $Z = 32.3 \text{ (cm)}$
Concrete strain:	$\epsilon_b = 2.40 \text{ (\%)}_0$	Stress in steel:
Steel strain:	$\epsilon_s = 10.00 \text{ (\%)}_0$	in tension $\sigma_s = 434.8 \text{ (MPa)}$

Case SLS	$M_{max} = 137.78 \text{ (kN*m)}$	$M_{min} = 0.00 \text{ (kN*m)}$
Safety factor:	1.00	
Neutral axis location:	$y = 11.2 \text{ (cm)}$	
Internal forces lever arm:	$Z = 31.3 \text{ (cm)}$	
Maximum stress in concrete:	$\sigma_b = 7.9 \text{ (MPa)}$	
Concrete stress limit	$0.6 f_{cj} = 15.0 \text{ (MPa)}$	
Stress in steel:		
tension	$\sigma_s = 250.0 \text{ (MPa)}$	
Steel stress limit:	$\sigma_{s \text{ lim}} = 250.0 \text{ (MPa)}$	

Return wall reinforcement M_{xx}

Section design for simple bending

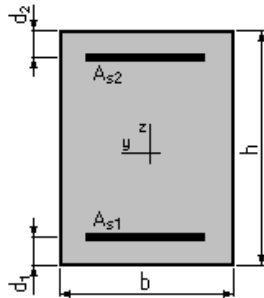
1. Assumptions:

Concrete: $f_{ck} = 30.0 \text{ (MPa)}$

Reinforcing steel: $f_{yk} = 500.0 \text{ (MPa)}$

- Cracking category: limited
- Account for compressed reinforcement
- No seismic design provisions
- Calculations according to BAEL 91 mod. 99

2. Section:



$b = 100.0 \text{ (cm)}$
 $h = 50.0 \text{ (cm)}$
 $d_1 = 5.0 \text{ (cm)}$
 $d_2 = 5.0 \text{ (cm)}$

3. Applied moments:

	$M_{max} \text{ (kN*m)}$	$M_{min} \text{ (kN*m)}$
Ultimate Limit State (fundamental)	161.39	0.00
Serviceability Limit State	113.76	0.00
Accidental Limit State	0.00	0.00

4. Results:

Steel:

Theoretical area	$A_{s1} = 11.0 \text{ (cm}^2\text{)}$	$A_{s2} = 0.0 \text{ (cm}^2\text{)}$
Minimum area	$A_{s \text{ min}} = 4.9 \text{ (cm}^2\text{)}$	
Theoretical	$\rho = 0.24 \text{ (\%)} $	
Minimum	$\rho_{min} = 0.11 \text{ (\%)} $	

Case analysis:

Case ULS	$M_{max} = 161.39 \text{ (kN*m)}$	$M_{min} = 0.00 \text{ (kN*m)}$
Safety factor:	1.29	Pivot: A
Neutral axis location:	$y = 3.5 \text{ (cm)}$	Internal forces lever arm: $Z = 43.6 \text{ (cm)}$
Concrete strain:	$\epsilon_b = 0.85 \text{ (\%)} $	Stress in steel:
Steel strain:	$\epsilon_s = 10.00 \text{ (\%)} $	in tension $\sigma_s = 434.8 \text{ (MPa)}$

Case SLS	$M_{max} = 113.76 \text{ (kN*m)}$	$M_{min} = 0.00 \text{ (kN*m)}$
Safety factor:	1.00	
Neutral axis location:	$y = 10.6 \text{ (cm)}$	
Internal forces lever arm:	$Z = 41.5 \text{ (cm)}$	
Maximum stress in concrete:	$\sigma_b = 5.2 \text{ (MPa)}$	
Concrete stress limit	$0,6 f_{cj} = 18.0 \text{ (MPa)}$	
Stress in steel:		
tension	$\sigma_s = 250.0 \text{ (MPa)}$	
Steel stress limit:	$\sigma_{s \text{ lim}} = 250.0 \text{ (MPa)}$	

Return wall reinforcement M_{yy}

Section design for simple bending

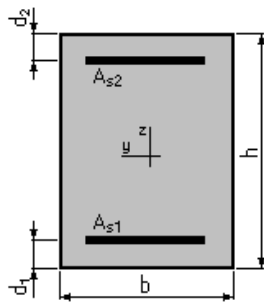
1. Assumptions:

Concrete: $f_{ck} = 30.0$ (MPa)

Reinforcing steel: $f_{yk} = 500.0$ (MPa)

- Cracking category: limited
- Account for compressed reinforcement
- No seismic design provisions
- Calculations according to BAEL 91 mod. 99

2. Section:



$$b = 100.0 \text{ (cm)}$$

$$h = 50.0 \text{ (cm)}$$

$$d_1 = 5.0 \text{ (cm)}$$

$$d_2 = 5.0 \text{ (cm)}$$

3. Applied moments:

	M_{max} (kN*m)	M_{min} (kN*m)
Ultimate Limit State (fundamental)	113.42	0.00
Serviceability Limit State	90.00	0.00
Accidental Limit State	0.00	0.00

4. Results:

Steel:

Theoretical area	$A_{s1} = 8.6 \text{ (cm}^2\text{)}$	$A_{s2} = 0.0 \text{ (cm}^2\text{)}$
Minimum area	$A_{s \text{ min}} = 4.9 \text{ (cm}^2\text{)}$	
Theoretical	$\rho = 0.19 \text{ (\%)} $	
Minimum	$\rho_{min} = 0.11 \text{ (\%)} $	

Case analysis:

Case ULS	$M_{max} = 113.42 \text{ (kN*m)}$	$M_{min} = 0.00 \text{ (kN*m)}$
Safety factor:	1.45	Pivot: A
Neutral axis location:	$y = 2.8 \text{ (cm)}$	Internal forces lever arm: $Z = 43.9 \text{ (cm)}$
Concrete strain:	$\epsilon_b = 0.65 \text{ (\%)} $	Stress in steel:
Steel strain:	$\epsilon_s = 10.00 \text{ (\%)} $	in tension $\sigma_s = 434.8 \text{ (MPa)}$
Case SLS	$M_{max} = 90.00 \text{ (kN*m)}$	$M_{min} = 0.00 \text{ (kN*m)}$
Safety factor:	1.00	

Neutral axis location:	$y = 9.6$	(cm)
Internal forces lever arm:	$Z = 41.8$	(cm)
Maximum stress in concrete:	$\sigma_b = 2.1$	(MPa)
Concrete stress limit	$0,6 f_{cj} = 18.0$	(MPa)
Stress in steel:		
tension	$\sigma_s = 24.1$	(MPa)
Steel stress limit:	$\sigma_{s \text{ lim}} = 250.0$	(MPa)

Transition slab reinforcement

Section design for simple bending

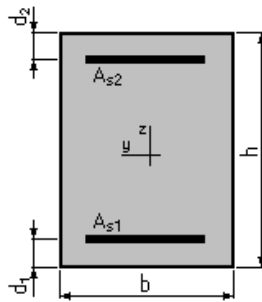
1. Assumptions:

Concrete: $f_{ck} = 30.0$ (MPa)

Reinforcing steel: $f_{yk} = 500.0$ (MPa)

- Cracking category: limited
- Account for compressed reinforcement
- No seismic design provisions
- Calculations according to BAEL 91 mod. 99

2. Section:



$b = 100.0$ (cm)
 $h = 30.0$ (cm)
 $d_1 = 5.0$ (cm)
 $d_2 = 5.0$ (cm)

3. Applied moments:

	$M_{\max}$ (kN*m)	$M_{\min}$ (kN*m)
Ultimate Limit State (fundamental)	107.38	0.00
Serviceability Limit State	79.87	0.00
Accidental Limit State	0.00	0.00

4. Results:

Steel:

Theoretical area	$A_{s1} = 14.4$ (cm ²)	$A_{s2} = 0.0$ (cm ²)
Minimum area	$A_{s \text{ min}} = 3.2$ (cm ²)	
Theoretical	$\rho = 0.58$ (%)	
Minimum	$\rho_{\min} = 0.13$ (%)	

Case analysis:

Case ULS	$M_{max} = 107.38 \text{ (kN*m)}$	$M_{min} = 0.00$	(kN*m)
Safety factor:	1.35	Pivot: A	
Neutral axis location:	$y = 4.6$	Internal forces lever arm: $Z = 23.2$	(cm)
Concrete strain:	$\varepsilon_b = 2.26$	Stress in steel:	
Steel strain:	$\varepsilon_s = 10.00$	in tension	$\sigma_s = 434.8$ (MPa)

Case SLS	$M_{max} = 79.87 \text{ (kN*m)}$	$M_{min} = 0.00$	(kN*m)
Safety factor:	1.00		
Neutral axis location:	$y = 8.5$		(cm)
Internal forces lever arm:	$Z = 22.2$		(cm)
Maximum stress in concrete:	$\sigma_b = 8.5$		(MPa)
Concrete stress limit	$0,6 f_{cj} = 18.0$		(MPa)
Stress in steel:			
tension	$\sigma_s = 250.0$		(MPa)
Steel stress limit:	$\sigma_{s \text{ lim}} = 250.0$		(MPa)

Corbel reinforcement

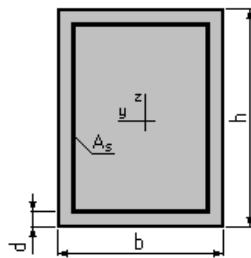
Section design for axial compression

1. Assumptions:

Concrete: $f_{ck} = 30.0$ (MPa) Reinforcing steel: $f_{yk} = 500.0$ (MPa)

- Cracking category: limited
- Account for slenderness
- No seismic design provisions
- More than half of loads applied after 90 days
- Calculations according to BAEL 91 mod. 99

2. Section:



$$b = 100.0 \text{ (cm)}$$

$$h = 40.0 \text{ (cm)}$$

$$d = 5.0 \text{ (cm)}$$

3. Forces:

	N (kN)
Ultimate Limit State (fundamental)	102.26
Serviceability Limit State	76.06
Accidental Limit State	0.00

4. Results:

Material resistance:

Total cross section area	$B = 4000.0$	(cm ²)			
Reduced section area	$B_r = 3724.0$	(cm ²)			
Buckling length	$l_{fy} = 0.6$	(m)	Buckling length	$l_{fz} = 0.6$	(m)
Moment of inertia	$I_y = 533333.3$	(cm ⁴)	Moment of inertia	$I_z = 3333333.3$	(cm ⁴)
Radius of gyration	$i_y = 11.5$	(cm)	Radius of gyration	$i_z = 28.9$	(cm)
Slenderness	$\lambda_y = 5.2$		Slenderness	$\lambda_z = 2.1$	
	$\alpha = 0.85$		N_u (only concrete)	$= 7003.35$	(kN)

Concrete:

σ (SLS) = 0.2 (MPa)

0,6 f_{cj} = 18.0 (MPa)

Steel:

Theoretical area	$A_s = 11.2$ (cm2)	
Minimum area	$A_{s\ min} = 11.2$ (cm2)	Maximum area $A_{s\ max} = 200.0$ (cm2)
Theoretical	$\rho = 0.42$ (%)	
Minimum	$\rho_{min} = 0.28$ (%)	Maximum $\rho_{max} = 5.00$ (%)

Abutment pile reinforcement

Section design for axial load and biaxial bending

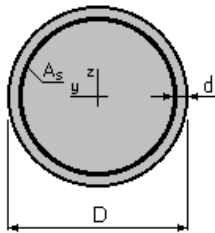
1. Assumptions:

Concrete: $f_{ck} = 25.0$ (MPa)

Reinforcing steel: $f_{yk} = 500.0$ (MPa)

- Cracking category: limited
- Account for seismic design provisions
- Calculation according to BAEL 91 mod. 99

2. Section:



$D = 120.0$ (cm)
 $d = 5.0$ (cm)

3. Applied forces:

Case N°	Type	N (kN)	M_y (kN*m)	M_z (kN*m)
1.	ULS	2096.14	1061.06	0.00
2.	SLS	1553.34	783.74	0.00
3.	ALS	1572.68	2134.62	0.00

4. Results:

Steel section area:

Theoretical area	$A_s = 113.1 \text{ (cm}^2\text{)}$	Maximum area	$A_{s \text{ max}} = 565.5 \text{ (cm}^2\text{)}$
Minimum area	$A_{s \text{ min}} = 113.1 \text{ (cm}^2\text{)}$		
Theoretical ρ	$= 1.00 \text{ (\%)}$		
Minimum ρ_{min}	$= 0.50 \text{ (\%)}$	Maximum ρ_{max}	$= 5.00 \text{ (\%)}$

Case analysis:

Case N° 1: Type ULS **N = 2096.14 (kN)** **M_y = 1061.06 (kN*m)** **M_z = 0.00 (kN*m)**

Safety factor:	3.20	Pivot: B	
Neutral axis location:	$y = 65.8 \text{ (cm)}$	Eccentric:	$Z = 84.1 \text{ (cm)}$
Concrete deformation:	$\varepsilon_b = 3.50 \text{ (‰)}$	Stress in steel:	
Steel deformation:	$\varepsilon_s = 2.60 \text{ (‰)}$	in tension	$\sigma_s = 434.8 \text{ (MPa)}$
		in compression	$\sigma_s' = 434.8 \text{ (MPa)}$

Case N° 2: Type SLS **N = 1553.34 (kN)** **M_y = 783.74 (kN*m)** **M_z = 0.00 (kN*m)**

Safety factor:	2.43		
Neutral axis location:	$y = 61.7 \text{ (cm)}$	Eccentric:	$Z = 89.3 \text{ (cm)}$
Maximum stress in concrete:	$\sigma_b = 6.2 \text{ (MPa)}$		
Stress limit:	$0,6 f_{cj} = 15.0 \text{ (MPa)}$		
Stress in steel:			
in tension	$\sigma_s = 79.6 \text{ (MPa)}$		
in compression	$\sigma_s' = 84.7 \text{ (MPa)}$		
Maximum stress in steel:	$\sigma_{s \text{ lim}} = 250.0 \text{ (MPa)}$		

Case N° 3: Type ALS **N = 1572.68 (kN)** **M_y = 2134.62 (kN*m)** **M_z = 0.00 (kN*m)**

Safety factor:	1.58	Pivot: B	
Neutral axis location:	$y = 38.9 \text{ (cm)}$	Eccentric:	$Z = 96.3 \text{ (cm)}$
Concrete deformation:	$\varepsilon_b = 3.50 \text{ (‰)}$	Stress in steel:	
Steel deformation:	$\varepsilon_s = 6.83 \text{ (‰)}$	in tension	$\sigma_s = 500.0 \text{ (MPa)}$
		in compression	$\sigma_s' = 500.0 \text{ (MPa)}$

Pier reinforcement

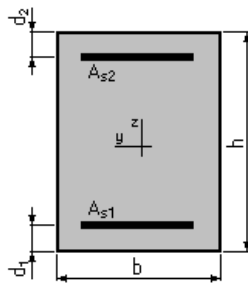
Section design for combined axial load and bending

1. Assumptions:

Concrete: $f_{ck} = 30.0 \text{ (MPa)}$ **Reinforcing steel: $f_{yk} = 500.0 \text{ (MPa)}$**

- Cracking category: limited
- No seismic design provisions
- Calculated as a column
- Calculations according to BAEL 91 mod. 99

2. Section:



$b = 100.0$ (cm)
 $h = 170.0$ (cm)
 $d_1 = 5.0$ (cm)
 $d_2 = 5.0$ (cm)

3. Applied forces:

Case N°	Type	N (kN)	M (kN*m)
1.	ULS	5180.35	663.91
2.	SLS	3829.05	2175.64
3.	ALS	3652.23	2808.67

4. Results:

Steel section area:

Theoretical area	$A_{s1} = 17.0$ (cm ²)	$A_{s2} = 17.0$ (cm ²)
Minimum area	$A_{s \min} = 34.0$ (cm ²)	Maximum area $A_{s \max} = 850.0$ (cm ²)
Theoretical	$\rho = 0.20$ (%)	
Minimum	$\rho_{\min} = 0.20$ (%)	Maximum $\rho_{\max} = 5.00$ (%)

Case analysis:

Case N° 1: Type ULS	N = 5180.35 (kN)	M = 663.91 (kN*m)
Safety factor:	5.00	Pivot: C
Neutral axis location:	$y = 184.3$ (cm)	Internal forces lever arm: $Z = 91.3$ (cm)
Steel strain:	$\epsilon_s = 0.00$ (‰)	Stress in steel:
Concrete strain:	$\epsilon_b = 3.31$ (‰)	in compression $\sigma_s' = 434.8$ (MPa)

Case N° 2: Type SLS	N = 3829.05 (kN)	M = 2175.64 (kN*m)
Safety factor:	2.41	
Neutral axis location:	$y = 101.0$ (cm)	Internal forces lever arm: $Z = 131.3$ (cm)
Maximum stress in concrete:	$\sigma_b = 6.4$ (MPa)	
Stress limit: $0,6 f_{cj} = 18.0$ (MPa)		
Stress in steel:		
tension	$\sigma_s = 26.3$ (MPa)	
compression	$\sigma_s' = 91.9$ (MPa)	
Stress limit in steel:	$\sigma_{s \lim} = 250.0$ (MPa)	

Case N° 3: Type ALS	N = 3652.23 (kN)	M = 2808.67 (kN*m)
Safety factor:	2.67	Pivot: B
Neutral axis location:	$y = 55.1$ (cm)	Internal forces lever arm: $Z = 143.0$ (cm)
Steel strain:	$\epsilon_s = 6.99$ (‰)	Stress in steel:
Concrete strain: $\epsilon_b = 3.50$ (‰)		in tension $\sigma_s = 500.0$ (MPa)

in compression $\sigma_s' = 500.0$ (MPa)

Pier pile reinforcement

Section design for axial load and biaxial bending

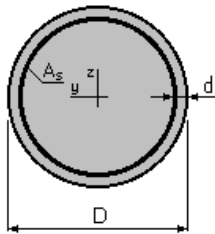
1. Assumptions:

Concrete: $f_{ck} = 25.0$ (MPa)

Reinforcing steel: $f_{yk} = 500.0$ (MPa)

- Cracking category: limited
- Account for seismic design provisions
- Calculation according to BAEL 91 mod. 99

2. Section:



$D = 120.0$ (cm)

$d = 5.0$ (cm)

3. Applied forces:

Case N°	Type	N (kN)	M_y (kN*m)	M_z (kN*m)
1.	ULS	3119.72	163.21	0.00
2.	SLS	2305.94	117.51	0.00
3.	ALS	2199.45	919.73	0.00

4. Results:

Steel section area:

Theoretical area $A_s = 113.1$ (cm²)

Minimum area $A_{s\min} = 113.1$ (cm²)

Theoretical $\rho = 1.00$ (%)

Minimum $\rho_{\min} = 0.50$ (%)

Maximum area $A_{s\max} = 565.5$ (cm²)

Maximum $\rho_{\max} = 5.00$ (%)

Case analysis:

Case N° 1: Type ULS **N = 3119.72 (kN)** **$M_y = 163.21$ (kN*m)** **$M_z = 0.00$ (kN*m)**

Safety factor: 5.70

Neutral axis location: $y = 135.8$ (cm)

Concrete deformation: $\epsilon_b = 3.22$ (‰)

Steel deformation: $\epsilon_s = 0.00$ (‰)

Pivot: C

Eccentric: $Z = 57.4$ (cm)

Stress in steel:
in compression $\sigma_s' = 434.8$ (MPa)

Case N° 2: Type SLS **N = 2305.94 (kN)** **$M_y = 117.50$ (kN*m)** **$M_z = 0.00$ (kN*m)**

Safety factor: 6.41

APPENDIX D : REINFORCEMENT

Neutral axis location: $y = 251.5$ (cm) Eccentric: $Z = 59.4$ (cm)
Maximum stress in concrete: $\sigma_b = 2.3$ (MPa)
Stress limit: $0,6 f_{cj} = 15.0$ (MPa)
Stress in steel:
Maximum stress in steel: $\sigma_{s \text{ lim}} = 250.0$ (MPa)

Case N° 3: Type ALS $N = 2199.45$ (kN) $M_y = 919.73$ (kN*m) $M_z = 0.00$ (kN*m)

Safety factor: 4.40 Pivot: B
Neutral axis location: $y = 69.5$ (cm) Eccentric: $Z = 82.5$ (cm)
Concrete deformation: $\varepsilon_b = 3.50$ (‰) Stress in steel:
Steel deformation: $\varepsilon_s = 2.28$ (‰) in tension $\sigma_s = 455.5$ (MPa)
in compression $\sigma_s' = 500.0$ (MPa)