الجمهورية الجزائرية الديمقراطية الشعبية
People's Democratic Republic of Algeria
وزارة التعليم العالي والبحث العلمي
Ministry of Higher Education and Scientific Research
جامعة عين تموشنت بلحاج بوشعيب
University of Ain Temouchent - Belhadj Bouchaib
Faculty of Science and Technology
Department of Civil Engineering & Public Works

Graduation Thesis for the Degree of Master
Field: Civil Engineering

Specialization: Infrastructure and Structural Engineering

Theme:

ANALYSIS AND OPTIMIZATION OF EXTERNAL CONTINUITY TENDONS IN LONG-SPAN BALANCED CANTILEVER BRIDGES

PRESENTED BY:

BESSAYAH MOHAMED
BRIKI ABDELHAKIM YASSINE

DEFENDED ON / /2026

BEFORE THE EXAMINATION COMMITTEE:

CHAIR: MRS. SALIMA ABDELBARI UNIVERSITY OF AIN TEMOUCHENT
EXAMINER: MRS. AMINA TAHAR BERRABAH UNIVERSITY OF AIN TEMOUCHENT
SUPERVISOR: MR. ABDERRAHMANE SAID UNIVERSITY OF AIN TEMOUCHENT

ACADEMIC YEAR 2025/2026

Acknowledgments

Above all, we would like to thank God Almighty for giving us the power, will, and endurance required to achieve this goal. To our parents, we offer sincere thanks for their unflinching support, sacrifice, and patience during our academic pursuits.

*To our mentor, **Mr. Said Abderrahmane**, we would like to thank him for being available, guiding us, and encouraging us during the entire course of completing this graduation project.*

*Our thanks go out to the respected members of the jury for their readiness to review this thesis. Lastly, we appreciate the contribution made by professors of the University of **Ain Temouchent-Belhadj Bouchaib** in providing an excellent education and enriching us with vast amounts of knowledge.*

Moreover, we owe thanks to all those who helped us directly and indirectly in completing this project.

DEDICATION

We dedicate this work to our parents, whose unwavering support and encouragement have guided us throughout our academic journey.

To our beloved family and friends, for being our pillars of strength and motivation.

Finally, to the entire Civil Engineering cohort, for the camaraderie, shared challenges, and the growth we experienced together along this path

ملخص:

هذه الدراسة بحثٌ حول تحسين كوابل سبق الإجهاد الخارجية في الجسور الخرسانية ذات البحور الكبيرة وفقاً للكود الأوروبي 2(Eurocode 2) . يهدف العمل إلى تجاوز عيوب التصميم التقليدي من خلال إيجاد مسارات مثالية للكوابل تضمن توزيعاً فعالاً للإجهادات وتقليلاً في استهلاك المواد. تم تطبيق النماذج النظرية على حالة عملية لجسر ذي بحر كبير باستخدام برنامج Midas Civil للتحليل الإنشائي. أظهرت النتائج عدم فاعلية الكوابل الخارجية في هذا النوع من الجسور في تقليص إجهادات الشد، على الرغم من تعديل مسار الكوابل ورفع عددها وفق التوصيات التقنية.

الكلمات المفتاحية :الكوابل الخارجية لسبق الإجهاد؛ الجسور ذات البحور الكبيرة؛ التحسين الإنشائي؛ إجهادات الشد؛ العزوم الثانوية؛ النمذجة العددية(Midas Civil) ؛ الكود الأوروبي 2.(Eurocode 2) .

Abstract:

This research investigates the optimization of external prestressing tendons in large-span concrete bridges, in accordance with the Eurocode 2 (EN 1992-2) standards. The study aims to surpass the limitations of conventional design by determining optimal tendon profiles to ensure an efficient stress distribution and material reduction. The theoretical framework was applied to a high-scale bridge case study using Midas Civil structural analysis software. The findings demonstrate the ineffectiveness of external prestressing tendons in mitigating tensile stresses for this specific bridge type, despite systematic modifications to tendon layouts and the application of increased prestressing forces in line with current technical recommendations.

Keywords: *External Prestressing Tendons; Large-Span Bridges; Structural Optimization; Tensile Stresses; Secondary Moments; Numerical Modeling (Midas Civil); Eurocode 2.*

Résume:

Cette étude porte sur l'optimisation des câbles de précontrainte extérieure dans les ponts en béton à grande portée, conformément aux normes de l'Eurocode 2 (EN 1992-2). Le travail vise à dépasser les limites de la conception conventionnelle en déterminant des profils de câblage optimaux permettant une distribution efficace des contraintes et une réduction de la consommation de matériaux. Le cadre théorique a été appliqué à une étude de cas d'un pont à grande portée en utilisant le logiciel d'analyse structurelle Midas Civil. Les résultats démontrent l'inefficacité des câbles de précontrainte extérieure pour atténuer les contraintes de traction dans ce type de pont, malgré des modifications systématiques du tracé des câbles et l'application de forces de précontrainte accrues, conformément aux recommandations techniques en vigueur.

Mots clés: *Câbles de précontrainte extérieure ; Ponts à grande portée ; Optimisation structurelle ; Contraintes de traction ; Moments secondaires ; Modélisation numérique (Midas Civil) ; Eurocode 2.*

TABLE OF CONTENTS

CHAPTER A: GENERALITIES

CHAPTER B: GENERAL INTRODUCTION

CHAPTER C: STRUCTURAL DESIGN

CHAPTER E: DESIGN OF EXTERNAL TENDONS WITHIN SERVICE STAGE

CHAPTER F: TECHNICAL AND ECONOMIC OPTIMIZATION STUDY

GENERAL CONCLUSION

TABLE OF FIGURE:

LIST OF TABLES:

LIST OF NOTATIONS:

Parameter	Symbol	Unit
Abbreviations & Modeling		
Construction Stage	CS	-
Serviceability Limit State	SLS	-
Ultimate Limit State	ULS	-
Finite Element Method	FEM	-
Dead Load / Superimposed Dead Load	DL / SDL	-
Geometric & Cross-Sectional Parameters		
Side span length	L_r	m
Interior span length	L_i	m
Total deck width	B	m
Deck depth at the pier	H_p	m
Deck depth at mid-span (Key segment)	H_c	m
Web thickness at pier	E_{ap}	m
Web thickness at mid-span	E_{ac}	m
Minimum top slab thickness at mid-span	e_1	m (or cm)
Top slab thickness at web root	e_2	m (or cm)
Top slab thickness at the cantilever root	e_3	m (or cm)
Minimum top slab thickness at crown	e_4	m (or cm)
Cantilever length (projecting part beyond support)	C	m
Bottom slab thickness at mid-span	E_{ic}	m (or cm)
Bottom slab thickness at pier	E_p	m (or cm)
Maximum reinforcement bar diameter	$\emptyset$	mm
Prestressing duct spacing	e_a	mm (or cm)
Haunch height	H	m (or cm)
Haunch base length	V	m (or cm)
Haunch deviation/inclination angle	α	° (degrees)
Second Moment of Area (Cross-Section)	I	m^4
Distance from Neutral Axis to Extreme Bottom Fiber	V_{bot}	m
Section Modulus (Bottom Fiber)	Z_b	m^3
Concrete Cross-Sectional Area	A_c	m^2
Concrete & Material Properties		
Unit weight of concrete	γ_c	KN/ m^3
Thermal expansion coefficient	α_T	1/°C

Parameter	Symbol	Unit
Characteristic compressive cylinder strength of concrete	f_{ck}	MPa
Characteristic compressive cube strength of concrete	$f_{ck,cube}$	MPa
Mean compressive strength of concrete	f_{cm}	MPa
Design compressive strength of concrete	f_{cd}	MPa
Mean tensile strength of concrete	f_{ctm}	MPa
Characteristic tensile strength (5% fractile)	$f_{ctk,0.05}$	MPa
Design tensile strength of concrete	f_{ctd}	MPa
Modulus of Elasticity of concrete	E_{cm}	MPa
Concrete compressive strains	$\epsilon_{c1}, \epsilon_{c2}, \epsilon_{cu2}$	%
Poisson's ratio	ν	-
Coefficient for long-term effects (compression)	α_{cc}	-
Coefficient for long-term effects (tension)	α_{ct}	-
Partial safety factor for concrete	γ_c	-
Passive Reinforcement & Prestressing Steel		
Modulus of elasticity of passive reinforcement	E_E (or E_s)	MPa
Characteristic yield strength of reinforcement	f_{yk}	MPa
Characteristic tensile strength of reinforcement	f_t	MPa
Design yield strength of reinforcing steel	f_{yd}	MPa
Ultimate strain under maximum load	ϵ_{uk}	%
Design Ultimate strain	ϵ_{ud}	%
Partial safety factor for steel	γ_s	-
Total Tendon Area	A_p	mm^2
Number of Strands per tendon	N	-
Characteristic tensile strength of prestressing steel	f_{pk} (or f_{pu})	MPa
Characteristic 0.1% proof stress of prestressing steel	$f_{p0.1k}$	MPa
Modulus of elasticity of prestressing steel	E_p	MPa
Long-term effective steel stress post-losses	σ_{pe}	MPa
Maximum permissible stress at jacking	$\sigma_{p,max}$	MPa
Prestressing force (effective after losses)	P (or F')	KN
Tendon eccentricity relative to neutral axis	e (or e_c)	m
Actions, Loadings, and Internal Forces		
Contribution of prestressing force to bending resistance	ΔM_p	KN.m
Dead Load Moment	M_{DL}	KN.m
Thermal Gradient Moment	M_{TG}	KN.m
Tendon Primary (Isostatic) Moment	M_{iso} (or M_1)	KN.m

Parameter	Symbol	Unit
Tendon Secondary (Hyperstatic) Moment	M_{hyp} (or M_2)	KN.m
Total Design/Net Moment	M_{Total} (or M_{net})	KN.m
Sizing scaling factor for baseline force	β	-
Tandem axle load (Load Model 1)	Q_{ik}	KN
Uniformly Distributed Load (UDL) (Load Model 1)	q_{ik}	KN/ m^2
Traffic load adjustment factors	α_{Qi}, α_{qi}	-
Positive Thermal Gradient (Max)	$\Delta T_{M,pos}$	°C
Negative Thermal Gradient (Min)	$\Delta T_{M,neg}$	°C

CHAPTER A:

GENERALITIES

I. Introduction:

Bridges are among the basic elements of infrastructure required for the uninterrupted functioning of transport networks. They are an important element in economic growth and territorial cohesion, which makes bridge construction an integral part of modern civil engineering.

Long-span bridges involve a number of complex demands, such as ensuring structural integrity, good behavior under load, high durability under adverse environmental effects, effective construction and architectural integration. When the span of the bridge goes beyond the usual range, the application of reinforced concrete structures becomes increasingly less efficient, owing to their heavy weight, high moments, and bad serviceability characteristics such as deformations and cracks.

***Figure 1:* Construction of a long-span prestressed concrete bridge using the balanced cantilever method.**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

In meeting these problems, the prestressed concrete system has proved to be a very efficient structure for construction of long span bridges. The application of the compressed force within the concrete greatly increases its resistance against bending, cracking, stiffness, among other properties. Within the modern-day methods of prestressing concrete, the most important method is the use of external prestressing, especially the external continuity tendons in the case of cantilever or segmental bridges. [1] [2] [3].

In this context, prestressed concrete technology has become a fundamental engineering solution for long-span bridge construction. By introducing a state of

pre-compression into the structural system, prestressing counteracts tensile stresses induced by external loading, thereby improving stiffness, crack control, fatigue resistance, and overall structural efficiency.

Among modern prestressing configurations, external prestressing systems particularly external continuity tendons have gained significant importance in the design of segmental and balanced cantilever bridges due to their constructability, durability, and maintenance advantages.

II. Historical Development of External Prestressing Systems:

The evolution of external prestressing reflects the progressive advancement of materials, construction technology, and structural design philosophy in bridge engineering [4].

2.1. Early Development Phase (1920s–1950s):

The conceptual foundation of prestressed concrete was introduced by Eugène Freyssinet, who demonstrated that the application of high-strength steel tendons could effectively overcome the inherent weakness of concrete in tension.

During this early phase, external prestressing was experimentally investigated; however, its practical application remained limited due to insufficient anchorage technology, lack of reliable corrosion protection systems, and inadequate understanding of long-term tendon behavior. Consequently, internal bonded prestressing became the dominant solution in early bridge construction [5].

2.2. Revival and Structural Rehabilitation Phase (1970s–1980s):

The resurgence of external prestressing occurred primarily in the context of bridge strengthening and rehabilitation. Engineers recognized that externally placed tendons offered significant advantages, including ease of inspection, replacement capability, and reduced intervention on existing structural elements [6].

This period also marked the development of segmental construction techniques, where external tendons were increasingly integrated into new bridge systems [6].

A representative example is the Long Key Bridge (Florida; USA: A 3.7 km precast segmental box girder bridge, notable for its span-by-span construction and use of external post-tensioning), which demonstrated the feasibility and efficiency of external prestressing in long-span applications.

2.3. Modern Development Phase (1990s–Present):

Advances in materials science, particularly the introduction of high-density polyethylene (HDPE) sheathing, improved corrosion protection systems, and refined anchorage devices, significantly enhanced the durability and reliability of external tendons.

As a result, external prestressing has become a standard solution in modern bridge engineering, especially for continuous box girder bridges and balanced cantilever systems, where long-term maintenance and structural adaptability are critical design considerations [7].

III. Structural Role of Prestressed Concrete in Bridge Engineering:

Prestressed concrete is defined as a structural system in which internal stresses are intentionally introduced to improve resistance against external loading. This pre-compression modifies the stress distribution within the section, reducing or eliminating tensile stresses under service conditions.

In bridge engineering, post-tensioned systems are predominantly used due to their adaptability to large spans and complex geometries. The post-tensioning process involves tensioning high-strength steel tendons after concrete has reached sufficient compressive strength.

The main structural benefits include:

- Increased flexural capacity without significant increase in cross-sectional area
- Improved control of cracking under service loads
- Enhanced fatigue resistance under cyclic traffic loading
- Reduction of structural self-weight, improving global efficiency

IV. Internal vs External Prestressing Systems:

Prestressing systems in bridge engineering are generally classified into internal bonded systems and external unbonded systems.

4.1. Internal Tendons:

Internal tendons are embedded within ducts inside the concrete section and are bonded through cementitious grout. This bonding ensures composite action between steel and concrete, allowing efficient stress transfer.

However, internal tendons present several limitations:

- Higher friction and wobble losses during stressing
- Inability to inspect or replace tendons after construction
- Susceptibility to long-term durability issues if grout quality is insufficient

4.2. External Tendons:

External tendons are positioned outside the concrete cross-section, typically within the void of box girders, and are deviated using saddles or deviator blocks.

They remain unbonded along their length and interact with the structure only at anchorages and deviation points. Their mechanical behavior is therefore governed by global structural deformation rather than local strain compatibility.

Figure 2: ***Internal layout of external tendons and deviation points within a hollow box girder.***

Source: *Adapted from Zilch & Weiher (2005) box girder.*

Key advantages include:

- Full accessibility for inspection and maintenance
- Possibility of tendon replacement or re-stressing
- Reduced friction losses and improved prestress efficiency
- Enhanced adaptability for structural strengthening and upgrading

V. Structural Function of External Continuity Tendons:

External continuity tendons are longitudinal prestressing elements designed to ensure structural continuity across multiple spans in bridge systems.

Their primary mechanical function is to resist positive bending moments in mid-span regions, where tensile stresses are maximized under service loading conditions.

From a structural perspective, these tendons:

- Enhance global stiffness of the bridge system
- Improve redistribution of internal forces under live loads
- Reduce long-term deflections induced by creep and shrinkage
- Contribute to redundancy and structural robustness

VI. Structural Behavior of External Prestressing Systems:

The behavior of externally prestressed structures differs fundamentally from internally bonded systems due to the absence of strain compatibility along the tendon length.

External tendons contribute to global equilibrium through changes in tendon geometry rather than local strain transfer. As a result, their structural contribution is highly dependent on overall bridge deformation.

Key behavioral characteristics include:

- Nonlinear geometric effects due to large displacements
- Variation in tendon eccentricity under load
- Second-order (P–Δ) effects influencing global stiffness
- Interaction between tendon profile and structural deflection

Accurate analysis therefore requires iterative nonlinear structural modeling that accounts for tendon geometry variation and load-dependent eccentricity changes.

VII. Design Considerations According to Eurocode 2:

The design of prestressed concrete bridges is governed by EN 1992-2 Eurocode 2, which provides comprehensive guidelines for serviceability and ultimate limit state verification.

7.1. Serviceability Limit State (SLS):

At service conditions, stress limitations are imposed to ensure durability and long-term performance. In general, compressive stresses in concrete are limited to approximately:

- 0.6 f_{ck} under frequent load combinations

This limitation ensures control of creep, cracking, and long-term deformation.

7.2. Ultimate Limit State (ULS):

At ultimate conditions, structural safety is verified by ensuring sufficient resistance against bending, shear, and torsion.

The contribution of prestressing force to bending resistance is expressed as:

$$\boldsymbol{\Delta M_p = P \cdot e}$$

Where:

- $\boldsymbol{P}$: is the effective prestressing force after all losses
- $\boldsymbol{e}$: is the eccentricity relative to the neutral axis

VIII. Structural Optimization in Long-Span Bridges:

Optimization in long-span bridge engineering is a multi-variable and highly coupled problem involving geometric, mechanical, and construction parameters.

The main objective is to achieve an optimal balance between:

- Material consumption
- Structural safety
- Serviceability performance
- Construction efficiency

Key design variables include:

- Cross-sectional geometry variation along the span
- Tendon layout and eccentricity profiles
- Prestressing sequence and staged construction effects

Optimization techniques aim to minimize structural weight while ensuring compliance with limit state requirements and construction constraints.

IX. Advantages and Limitations of External Prestressing:

External prestressing systems provide significant engineering advantages:

- Improved durability and corrosion resistance
- Facility of inspection and maintenance
- Capability for strengthening and re-tensioning
- Enhanced construction flexibility

However, they also introduce certain challenges:

- High stress concentration at anchorages and deviators
- Requirement for precise geometric control
- Sensitivity to global structural deformations
- Complex nonlinear analysis requirements

Proper detailing, execution quality, and advanced analysis methods are therefore essential to ensure long-term structural performance.

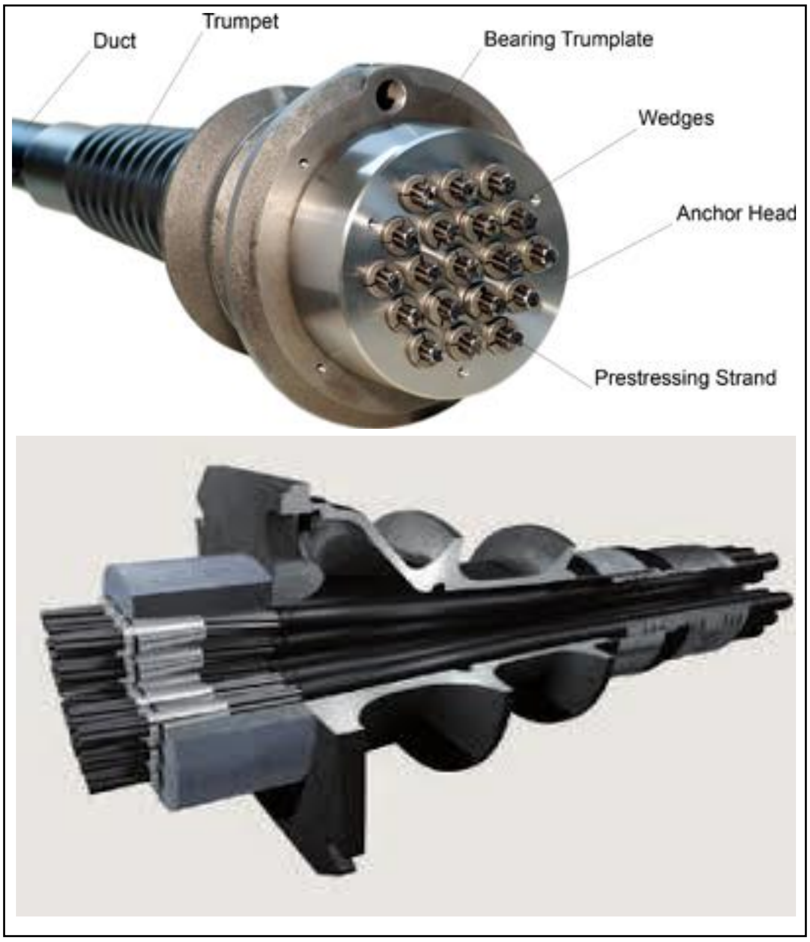

Figure 3: ***Detailed view of a mechanical anchorage assembly for external prestressing tendons: (a) External components and labels; (b) Internal longitudinal cross-section.***

Sources: *(a) Bridgeweb.com, 2011; (b) Freyssinet, 2020*

Conclusion:

This chapter has presented a comprehensive theoretical and historical overview of external continuity tendons in long-span bridge systems. Their integration into balanced cantilever construction represents a significant advancement in modern bridge engineering, offering improved structural efficiency, durability, and adaptability.

The following chapter introduces the structural system adopted for the case study bridge and presents the general design philosophy of balanced cantilever construction.

CHAPTER B: GENERAL INTRODUCTION

I. The Strategic Purpose of Modern-Day Infrastructure:

Bridges are more than just structural steel frameworks. Instead, bridges play the role of being the main channels through which modern-day civilization operates by overcoming the barriers posed by geography to spur economic growth and maintain territorial integrity while at the same time representing the attempt by humans to connect different communities. With urbanization and the emergence of efficient transport routes, the building of bridges of greater spans is an essential feature of modern-day civil engineering.

II. The Transition in Engineering Practice: From Passive to Active

With the growth of span lengths in the engineering sector, the reinforced concrete faces its natural limitations from both perspectives of strength and economics. Beyond a 30-meter span length, it is the self-weight of the structure itself that acts as the main antagonist. Self-weight, along with issues related to the performance of the structure like cracking and deflections, makes it necessary to consider an alternative option.

The need for innovation and development led to the advent of Prestressed Concrete. Through transition from passivity to activity, prestressing allows for preloading the structure against the stress caused by gravity. In this regard, External Prestressing System became one such innovation.

III. Structural System:

A structural system in bridges made using the balanced cantilever process could be grouped into rigid-frame bridge and supported bridge.

3.1. Rigid Frame Bridge Type:

Rigid frame bridge consists of a fixed connection between the bridge's superstructure and substructure and could be classified as: Partially articulated bridge, articulated bridge, and continuous bridge. The key benefit offered by the rigid frame bridge lies in the absence of intermediate support due to the rigid link between the two components of the bridge. Furthermore, in continuous bridges, the decrease in expansion joints results in improved flexibility and easier maintenance of the structure. Another positive attribute in multi-span designs is the ability of lateral forces to be transferred to individual piers in seismic activity while reducing the amount of bending moment at the base of piers, which might lead to better seismic behavior.

Another benefit provided by the rigid frame bridge lies in its nature as a statically indeterminate structure that allows for the redistribution of stress following partial yielding of members within the structure, resulting in avoiding collapse of the entire system. Moreover, temporary supports are not required in order to eliminate unbalanced moments in construction processes. However, the statically indeterminate feature makes it more sensitive to changes in prestress, temperature, shrinkage of concrete, and differential settlement of foundations. [8]

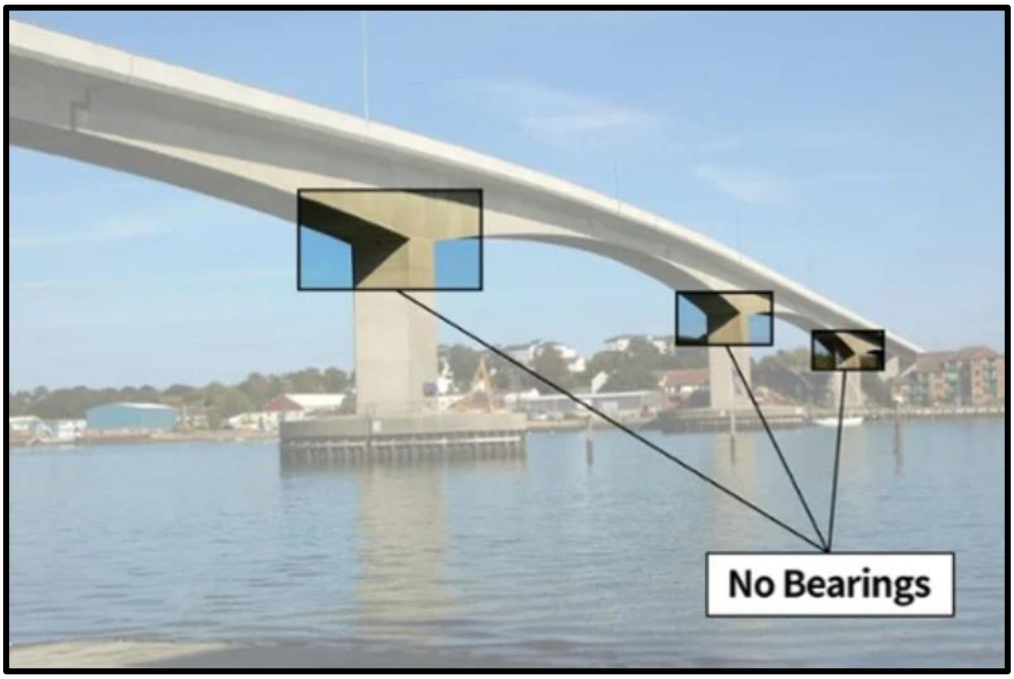

***Figure 4:* Rigid frame bridge type**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

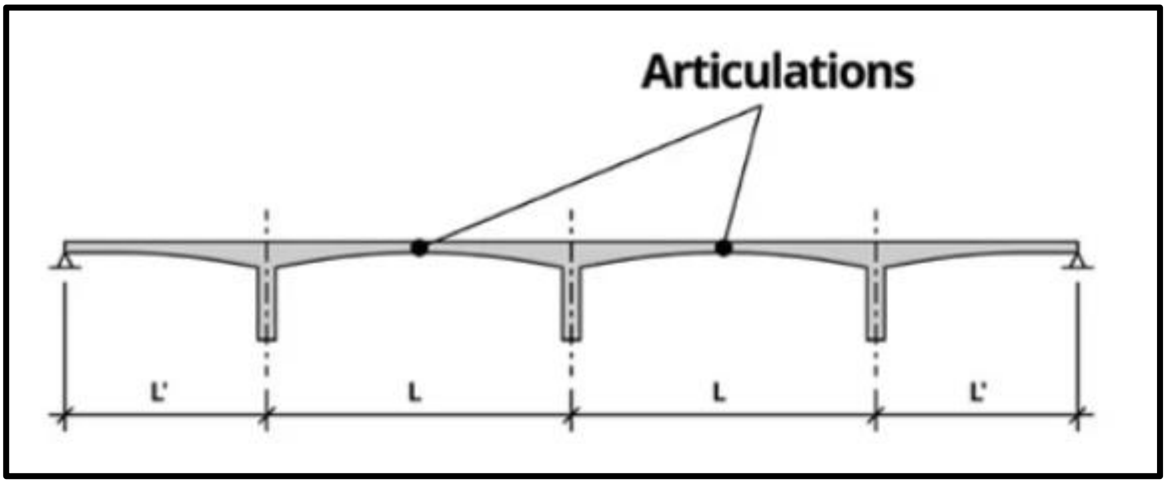

***Figure 5:* Rigid frame bridge type**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

3.1.1. Partially Articulated Balanced Cantilever Bridge:

In partially articulated balanced cantilever bridges, cantilever construction techniques are used where the middle joint acts as a hinge. This approach's main benefit is that it is statically determinate, making calculations easy. In addition, since the bending moment at the time of construction equals the bending moment after construction, the tendon layout is easy and needs fewer tendons. Additionally, it accommodates differential settlements, thus becoming useful in soft soil due to smaller settlements compared to continuous girder bridges. The disadvantages include the inability of this approach to redistribute moments since it is statically determinate, resulting in a weaker structure compared to continuous structures. The central hinge creates design challenges that might be problematic, and in the future, different problems might arise from it, raising maintenance costs. Lastly, there might be uplift forces acting on the girder attached to the abutment. Also, the hinge offers limited resistance against the bending moment caused by concrete creep. Thus, the approach became obsolete. [8]

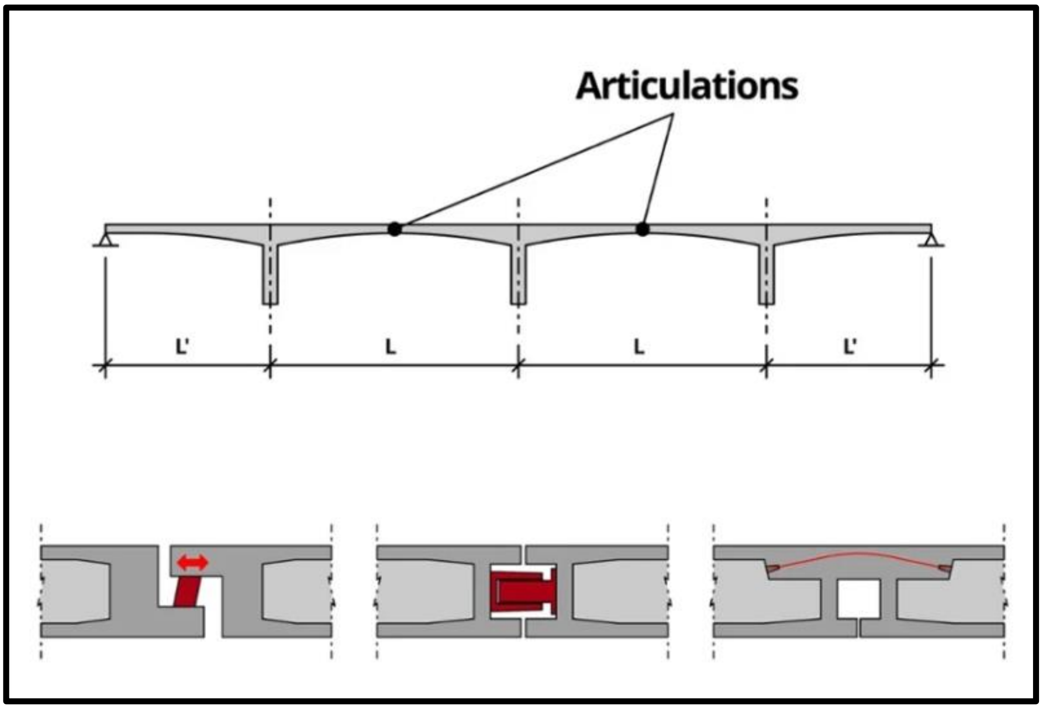

Figure 6: Partially articulated balanced cantilever bridge: central hinge type

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

3.1.2. Balanced Cantilever Bridge with Articulation:

Continuity in bridge designs is not feasible where the different settlement between two locations could take place due to settling down of foundations or other geological considerations. Under such topographical constraints, a solution would be presented where the slope and deflection difference will be minimized between the fully articulated bridge through suspension of an independent Gerber girder at the center of the span. Since of its characteristics, the design will be mostly suitable for a three-span bridge construction with the shorter spans beside the abutments. Gerber girder being suspended at the center will have only positive bending moment without the need for box-girder construction. Also, dead loads can be decreased by using I- or T-shaped cross section with the same number of cantilevers and webs. The design can be made into a steel bridge with substitution of steel for concrete construction. However, similar problems with a partially articulated bridge are faced by this bridge. [8]

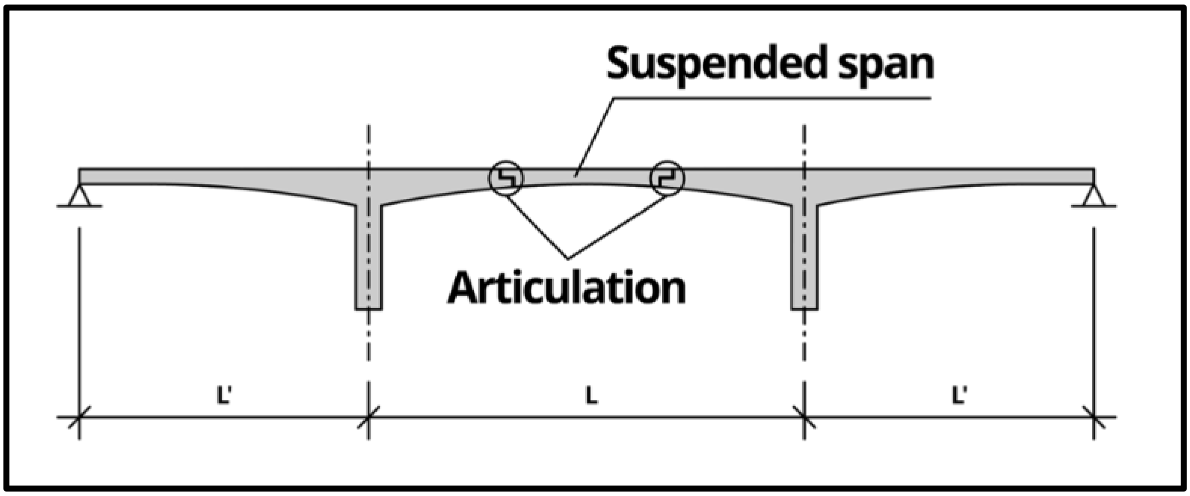

***Figure 7:* Articulated balanced cantilever bridge**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

3.1.3. Continuous Balanced Cantilever Bridge:

The continuous balanced cantilever bridge entails the installation of the closure span following the installation of the cantilever, whereby the connection occurs between the closure span and the cast-in-place or precast segment. The structure presents desirable maintenance features as it requires no support for the bridge piers and increases mobility due to less expansion joints. As a high-order statically indeterminate structure, it provides excellent resistance against earthquakes and strong winds. Compared to conventional continuous girder bridges, the system is significantly affected by concrete shrinkage and creep and different settlement on the foundation of the bridge due to its high order static indeterminacy. The primary disadvantage of this bridge system is that it continues to expose the foundations of the bridge to lateral loads. In terms of economics, this concept may be preferred over the continuous beam girders bridge as the design of the piers distributes the load, such as the earthquake force, evenly on the piers, leading to cheaper support. Multi-span continuous balanced cantilever bridge is ideal for structures with plastic piers (high ductility piers) because of girder expansion and additional pier deformation resulting from concrete creep, shrinkage, prestress, and temperature. [8]

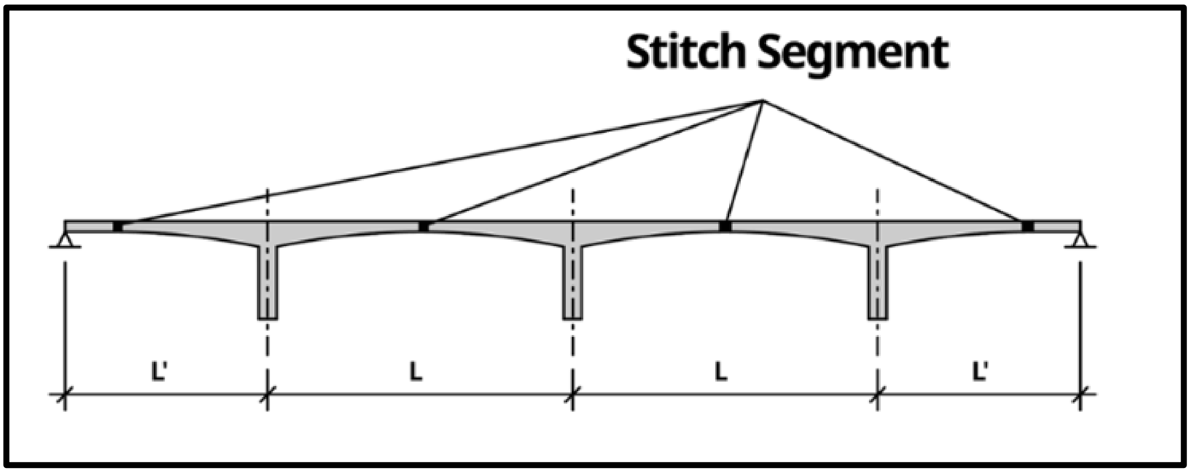

***Figure 8:* Conceptual drawing**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

***Figure 9:* Construction of a closure segment**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

***Figure 10:* Continuous balanced cantilever bridge**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

3.2. Supported Bridge Type:

The supported bridge represents an example of a category where the box girder spans over two or more segments while being supported by the supports located on the piers and abutments. In case of the supported bridge that is constructed through the cantilever method, there are some disadvantages associated with this process, such as the need for the installation of temporary supporting elements to counteract the effects of unbalanced moment caused by the construction process. According to the type of support involved, the classification is as follows: [8]

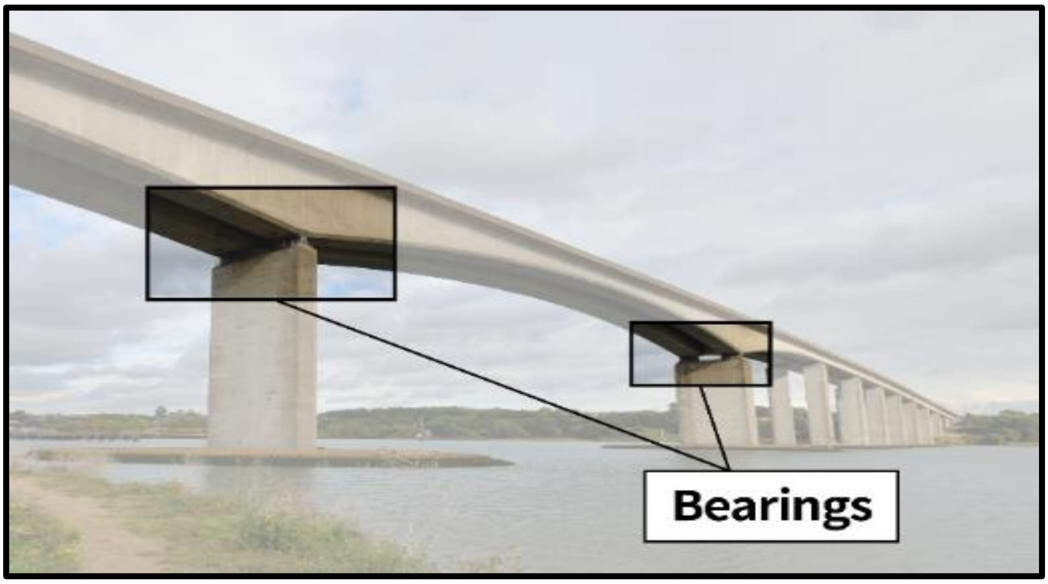

***Figure 11:* Supported bridge type**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

3.2.1. Single Fixed Support:

The structure made up of the single fixed support forms a continuously supported girder where there is only one support that is fixed, whereas other supports are movable supports. As the lateral loads applied to the superstructure due to changes in temperature, earthquakes, and the like concentrate at the pier having the fixed support, it is better to use a larger cross-section of piers. Also, no statically indeterminate load acts in response to longitudinal temperature changes and creep and shrinkage. [8]

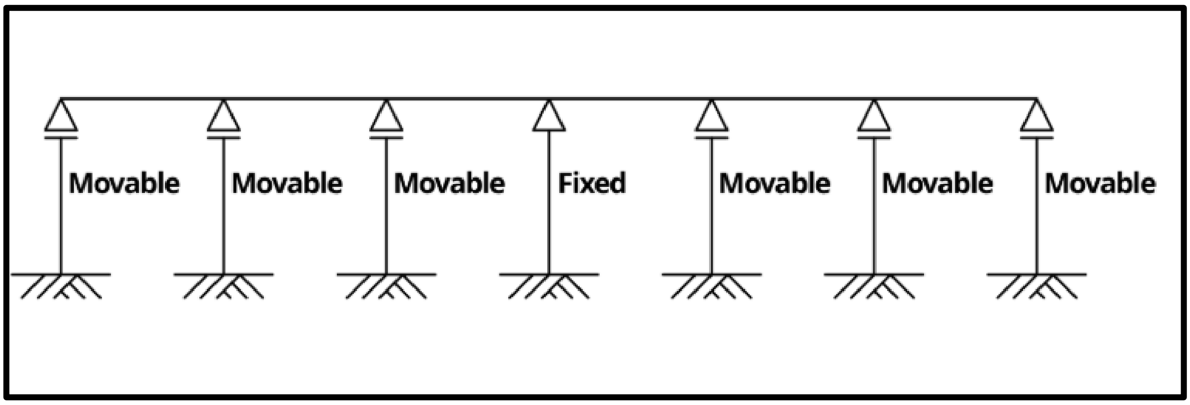

***Figure 12:* Conceptual diagram of single fixed supported balanced cantilever bridge**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

3.2.2. Multi-Fixed Supports:

The technique referred to as "multi-fixed-support" is one kind of continuous girder bridge where the force acting horizontally due to earthquakes is transferred to many piers using several fixed supports. In such an arrangement, the cross-sectional area of the pier where the fixed support exists can be less than the cross-sectional area of the pier where the fixed support exists under single-fixed-support design. However, in this design, there are forces acting on the structure due to static indeterminacy caused by temperature changes, creep, and shrinkage. [8]

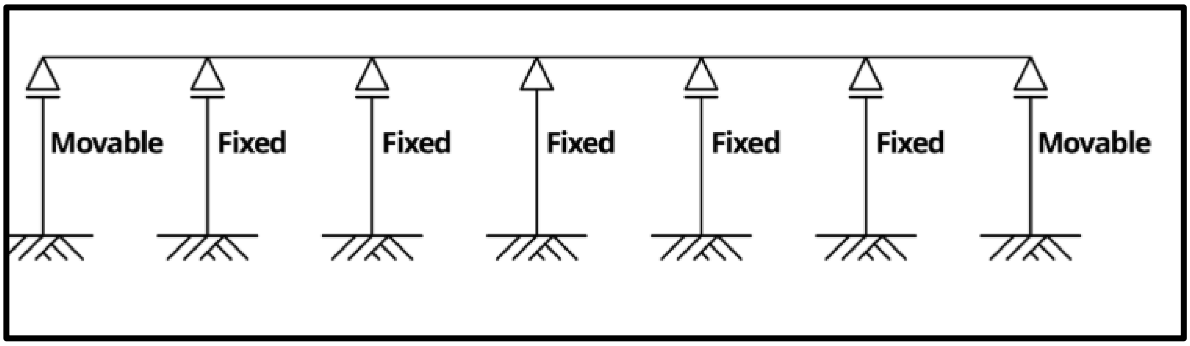

***Figure 13:* Conceptual drawing of a multi fixed supported balanced cantilever bridge**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

3.2.3. Stopper System:

Stopper system refers to the category of continuous girder bridges, where either one or several piers with rigid support systems are erected, together with other stoppers or dampers to piers fitted with moving supports. Stopper systems transfer any additional horizontal force generated due to the effect of loads such as earthquake to piers, without hindering the thermal effects on the box girders. Three main types of stoppers have been used, including liquid stoppers, leaf spring stoppers, and mechanical stoppers using hydraulic cylinders. In the absence of seismic designs, the horizontal forces will be generated mainly by temperature and vehicle braking effects, and the bridge piers could easily withstand the same. But when the bridge piers are designed for earthquake load, the horizontal forces generated might greatly exceed that of the support system, a challenge that can be addressed using stopper or seismic isolation supports. [8]

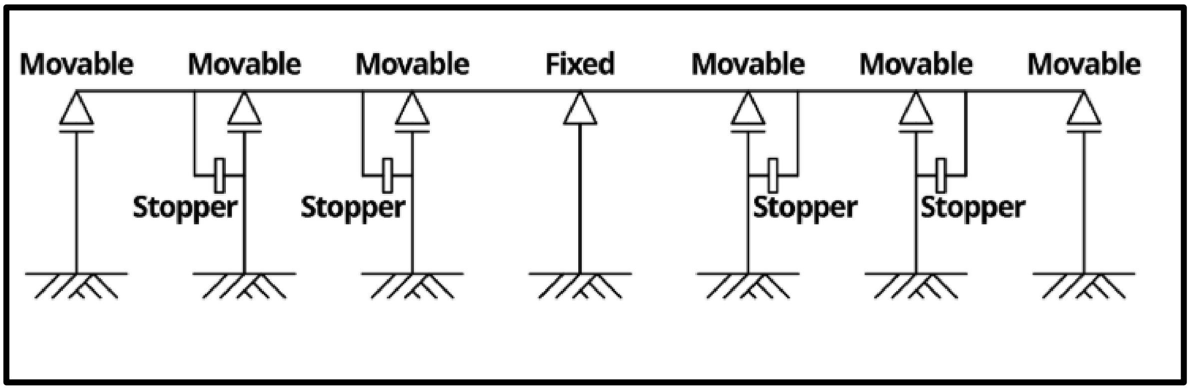

***Figure 14:* Stopper method**

***Source:** Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

IV. Superstructure:

The superstructure of a balanced cantilever bridge consists of the prestressed concrete girder. The next part gives the description of the properties and features concerning the prestressed concrete girder used in bridges. [8]

4.1. Girder and Fabrication Method:

The main load-bearing element of a balanced cantilever bridge is the box girder. The current paragraph provides a historical perspective and technical justification for box girders. Box girders made of concrete are commonly found in bridges in the western parts of the United States. At the outset, such girders were built from reinforced concrete alone, using conventional cross-sectional forms such as I-beams or T-beams. The stiffness in cross-section of box girders is greater than that in other forms, allowing the application of box girders for long-span bridges. Moreover, the geometrical shape of box girders allows them to possess good torsional resistance qualities. With the development of prestressed concrete, the technology was applied to box girders because the process of installing the tendons in box girders is relatively easy due to the presence of both upper and lower slab surfaces. Box girders can be single cell or multiple cell elements. For single cell boxes, the web can be thickened for the installation of tendons. Multiple cells allow for widening of bridges. Each option comes with its strengths and weaknesses; thus, the choice depends on various factors. In balanced cantilever bridges, the construction of girders happens in sections, and the segments are successively assembled starting from the pier. Girder manufacturing takes place in situ or in a prefabricated manner at specialized facilities for making girders. [8]

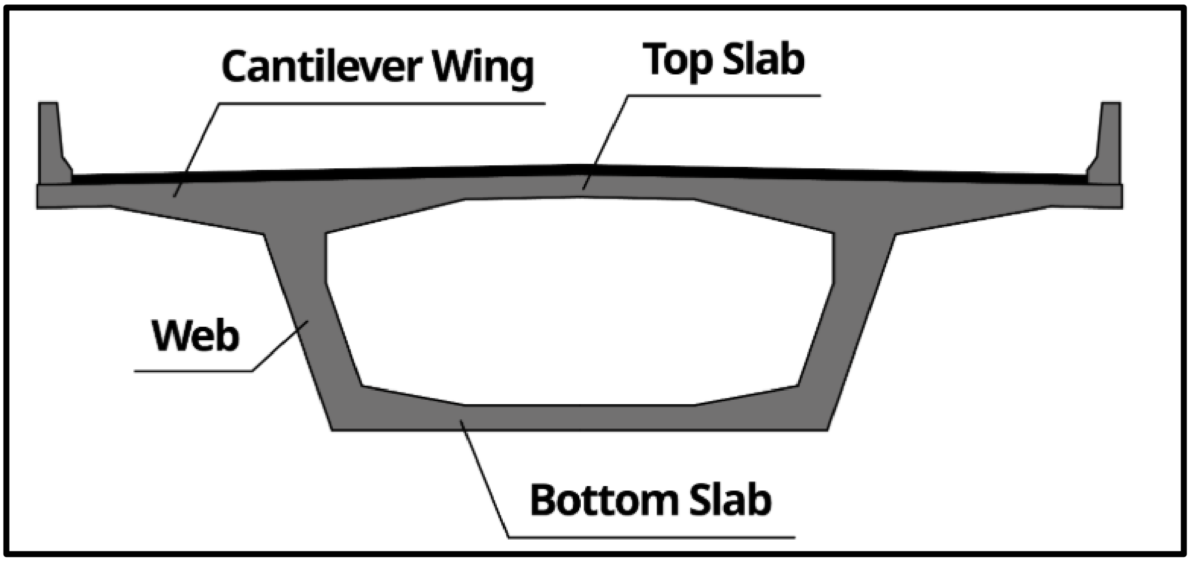

***Figure 15**: Cross-section of box girders*

***Source:** Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

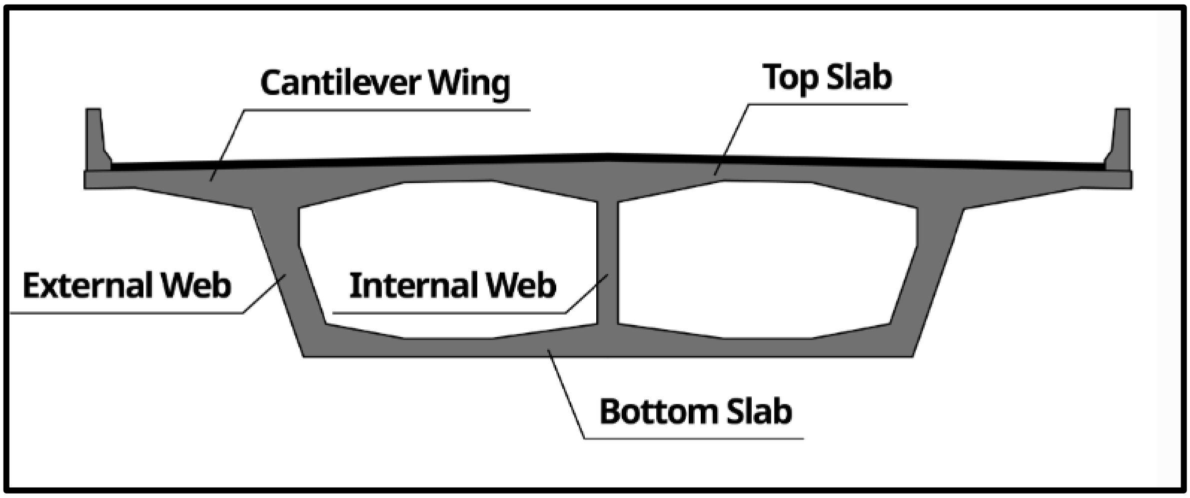

***Figure 16:** Cross-section of box girders double cell*

***Source:** Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

4.1.1. Method of In-Situ Concrete:

Construction Method of in-situ concrete construction refers to an approach used in constructing girders on the project site using form work. Incremental construction of girders in sections is done with the help of a form traveler. The entire construction procedure becomes efficient, as the same processes are repeated in an identical way. Additionally, making corrections during each step of the construction makes the entire construction process accurate. [8]

Figure 17: ***In situ concrete method***

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

4.1.2. Precast Concrete Method:

The precast concrete technique involves the casting of the girders offsite in a controlled environment and their subsequent transportation to the construction site for erection. The controlled casting environment, similar to that of a factory environment, allows better management control, which can lead to significant improvement in girder quality. Besides, simultaneous casting of girder parts and substructures compared to onsite casting can reduce the duration of construction. In superstructure construction, the concrete will have aged considerably, leading to small values of creep and drying shrinkage; hence there will be less loss of prestress. However, one factor of great importance in the precast construction method is transportation, which is dependent on the construction site. Careful management is needed in the process, while any mistakes cannot be easily corrected as opposed to traditional onsite construction. [8]

Figure 18: ***Precast Method***

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

4.2. Division into Segments:

In the Balance Cantilever technique, the size of the segment will play a role not only in the construction time but also in work efficiency. The size of the segment is defined based on the ability of the erection equipment. The lengths of the segment range between 3m to 5m. When dividing segments, it is crucial to keep consistent segment lengths and weights. With consistent segment weight, the erection equipment ability will also remain consistent, and thus erection equipment cost will be reduced. However, with varying segment lengths, there will be a decrease in work efficiency when erecting rebars. Moreover, an increase in segment numbers may also negatively impact construction time. With consistent segment lengths, the problem becomes the higher demand for erection equipment ability due to heavy adjacent pier segments. [8]

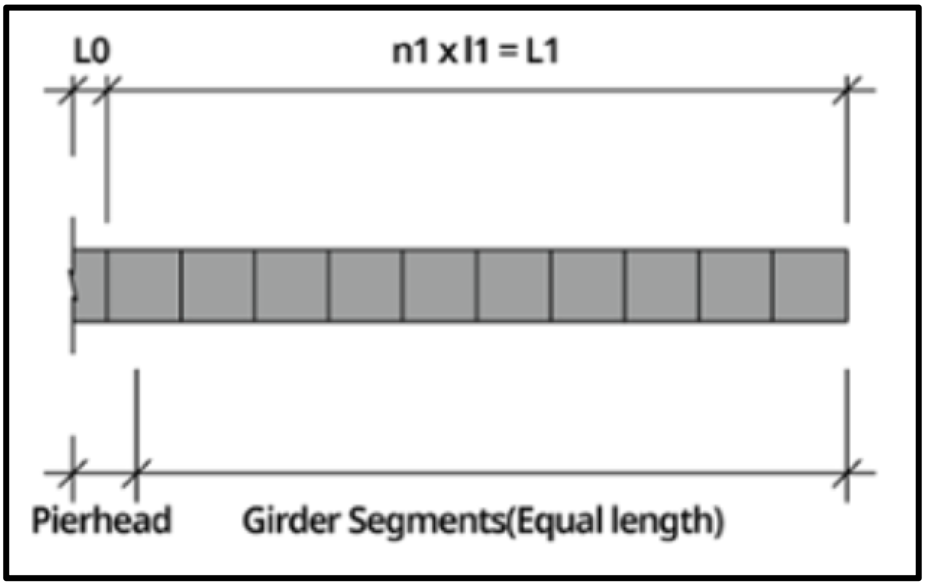

Figure 19: ***Equal Section Type***

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

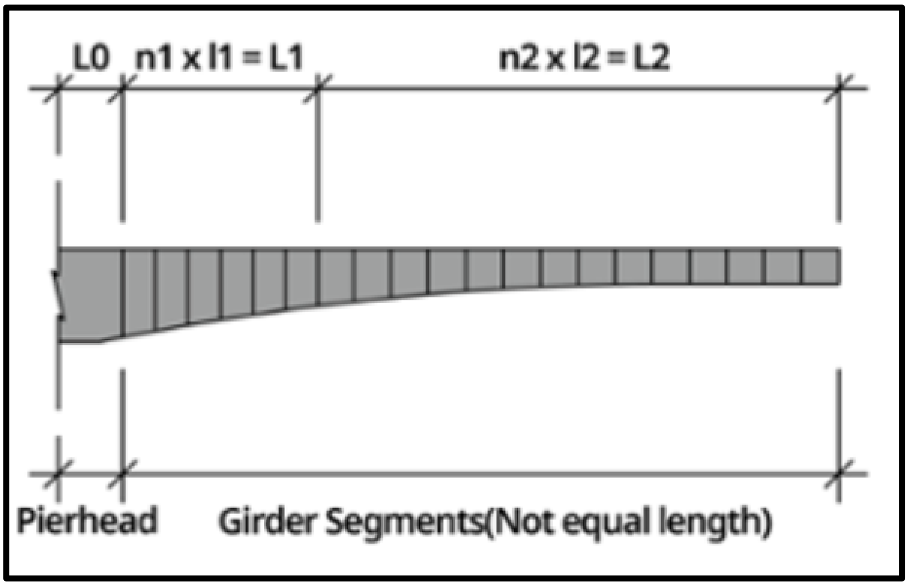

Figure 20: ***Tapered Section Type***

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

***Figure 21:* Tapered Section Type**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

4.3. Tendons in Box Girder:

Tendon arrangements can be provided longitudinally as well as transversely in the balanced cantilever bridge structure. In the longitudinal direction, tendons are placed in the top and bottom slabs of the box girder. Tendons in the top slab provide resistance against negative bending, while tendons in the bottom slab offer resistance against positive bending. In the transverse direction, tendon arrangement is done to resist moments due to cantilever beams of the box girder. If there is no inclination of longitudinal tendons, vertical web tendons can be used for resisting shear. [8]

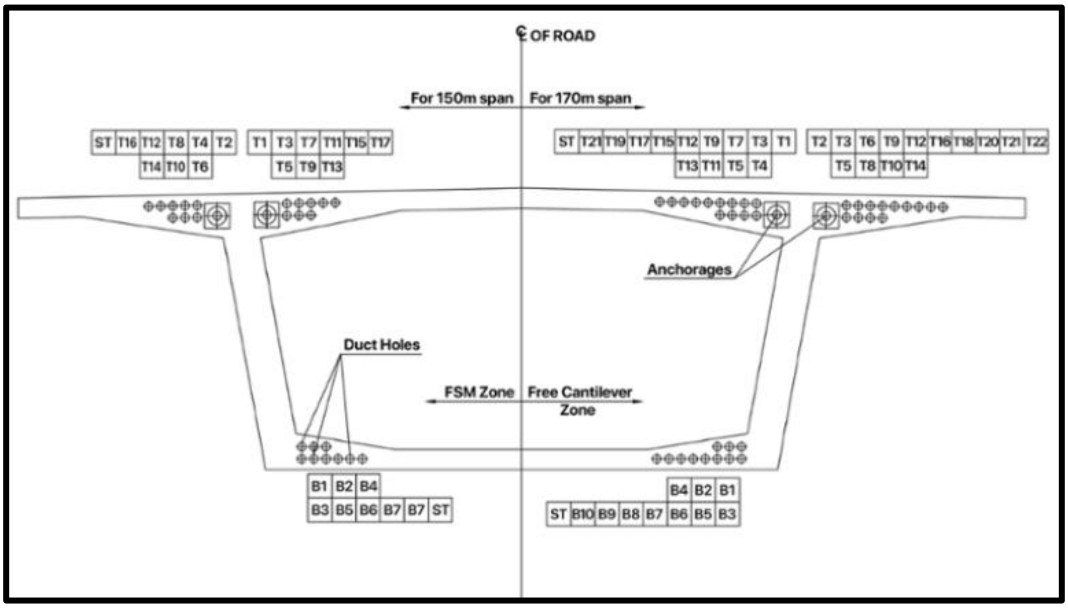

Figure 22: ***Tendon arrangement cross-section view***

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

V. Substructure:

Three kinds of piers are utilized in balanced cantilever bridges: short stocky columns, long slender columns, and twin-leaf twin-columns. Short column is considered relatively easy to design considering that there is more emphasis on concrete strength and reinforcement yielding with little consideration given to the column length effect or the tendency to buckle. Long column provides more benefits when it comes to dealing with environmental conditions because it can provide enough clearance under the bridge or traverse through difficult terrain. The design of long columns will be more involved than the short column because it is longer and subject to secondary moments brought about by p-delta effects and buckling. Finally, twin-leaf columns make use of two columns for the pier-head girder. This design makes it more stable because it allows for load distribution from the vertical load between columns and because of the higher lateral flexibility of the structure, it enables expansion characteristics similar to that of a continuous bridge. [8]

***Figure 23:* Short/Stocky column**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

***Figure 24:* Long/Slender column**

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

Figure 25*: *Twin leaf/Twin column

Source: *Adapted from "Balanced Cantilever Bridges," MIDAS Information Technology.*

VI. External Continuity Tendons and the Balanced Cantilever Technique:

The Balanced Cantilever Technique is a very efficient technique of constructing long-span bridges over large valleys. In this technique, External Continuity Tendons play an important role. Unlike the bonded tendons, external tendons are much more accessible for maintenance purposes, such as inspection, re-stressing or repair work. Accessibility, nonetheless, presents an engineering challenge.

The fact that the tendons are unbonded makes their behavior nonlinear and causes second-order effects, because the tendon deforms differently from the concrete cross-section. While the design of structures using codes like the Eurocode 2 (EN 1992-2) guarantees structural integrity, there is still a need for improvement in developing automatic tendon layout techniques.

VII. Objectives of the Study: Quest for Optimization and Sustainability

The modern engineer is no longer tasked simply with "making it stand," but with "making it optimal." In an era of resource scarcity, the goal of this thesis is to move beyond standard design protocols toward a rigorous Optimization framework.

This research focuses on the analysis and layout optimization of external continuity tendons to minimize material consumption. By refining these profiles, we not only reduce costs but also directly lower the carbon footprint of the infrastructure, aligning structural performance with environmental sustainability.

VIII. Case Study: A 660 m Engineering Challenge

The theoretical principles explored in this work are applied to a high-scale engineering project: a 5-span, 660 m long-span bridge. This case study serves as the proving ground for our optimization methodologies. By integrating advanced structural modeling software, specifically Midas CIVIL, we demonstrate how the careful calibration of external prestressing can ensure the stability and longevity of a massive infrastructure link while meeting the stringent requirements of modern design codes. Through this investigation, we aim to contribute to a future where bridges are not only longer and stronger but also smarter and more adaptable.

CHAPTER C:
STRUCTURAL DESIGNS

I. Introduction:

This chapter discusses the general aspects of designing a bridge built using the balanced cantilever method (BCM) as part of a highway bridge construction project in Algeria's rough terrain. The project site has complicated geological and geotechnical conditions, such as soils that can be compressed and a lot of seismic activity.

II. Scope of the chapter:

This project is part of a national plan to improve infrastructure that includes the highway transportation network. The goal is to make it easier for people and goods to get around, especially in remote or hard-to-reach areas. This infrastructure is strategically important because it can improve access for remote communities, boost economic growth, and make it easier for people to get around in the region.

This chapter talks about the most important things to think about when designing this kind of building, such as:

- **Span arrangement and support layout** figuring out the best way to arrange the spans and place the piers and abutments, taking into account the site's topographical, hydraulic, and geotechnical limits.
- **Choosing the right cross-sectional shape** means picking the right structural type (single-cell or multi-cell box girder) based on how it works, how it looks, and how it fits together.
- **Preliminary sizing of the box girder components** dimensioning of the webs, top slab, bottom slab, and cantilever flanges according to the requirements of the balanced cantilever construction method and EN 1992-2:2005.

Laws for depth variation and related geometric parameters define the deck's longitudinal profile, which includes the parabolic or linear change in structural depth between the pier sections and the mid-span closure segments. This is done to meet structural efficiency and aesthetic standards.

III. Significance of Initial Design:

It is important to stress that preliminary sizing is a key factor in deciding:

1) The bridge's structural behavior under both permanent and temporary loads, such as live loads from trains, thermal effects, seismic actions, and loads from the construction stage.
2) The project's economic efficiency comes from getting the most out of the materials, the time it takes to build, and the costs over the life of the project.
3) The structure's aesthetic quality, which means that it looks good with the landscape around it and follows architectural rules.

So, finishing this first phase of design successfully is a key factor in making sure that the bridge's overall quality and performance are good during the next phases of detailed design, construction, and service life.

IV. Project overview:

The present project involves the design of highway with a total deck width of 13.20m. The alignment extends over a total length of 660m, crossing a valley characterized by complex topographical conditions.

Table 1: *Summary of Design and Technical Parameters*

Parameter	Value/ Description
Total Bridge length	660 m
Construction method	Balanced cantilever (cast in place segments)
Structure type	Prestressed concrete box girder
Location	Algeria-mountainous region with rugged terrain
Seismic zone	Zone of significant seismic activity (RPA 2024)
Geotechnical conditions	Compressible soils-complex geology requiring deep foundations
Functionality	Highway bridge
Design standards	EN 1992-2, EN 1991-2

The structure consists of 5 spans distributed as shown in the longitudinal elevation:

Table 2: Configuration and Length Distribution of the Bridge Structure

Span type	Number of spans	Length per span	Total length
Central spans	3	160 m	480 m
Side spans	2	90 m	180 m
Total	5	160 m	660 m

4.1.Span Arrangement and Distribution:

The span configuration is as follows:

$$90\text{ m} + 160\text{ m} + 160\text{ m} + 160\text{ m} + 90\text{ m} = 660\text{ m}$$

This arrangement can be represented schematically as:

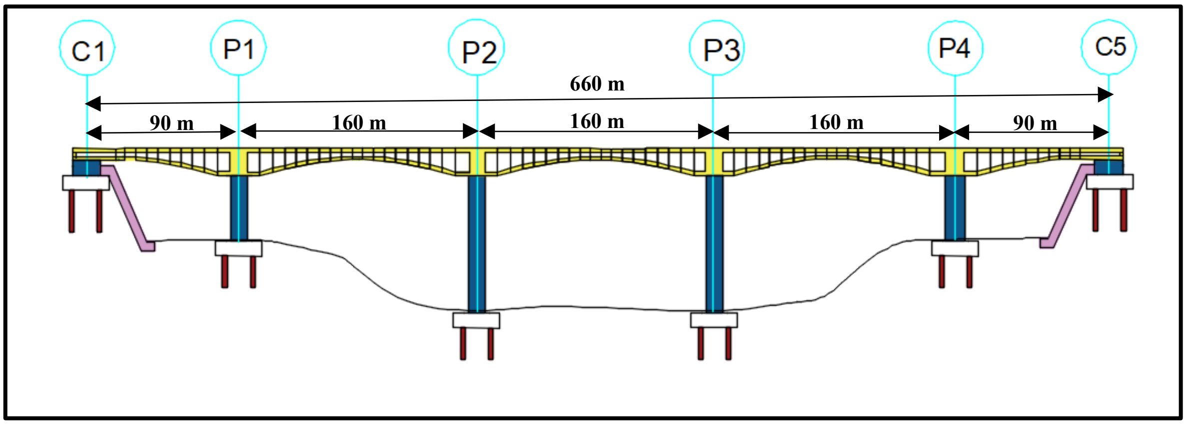

Figure 26:Longitudinal Elevation of the Five-Span Prestressed Concrete

4.2.Design Criterion:

According to established design guidelines, the side span length Lr should satisfy [9]:

$$0.5L_i \leq L_r \leq 0.7L_i$$

Where:

- L_r : side span length
- L_i : interior span length

4.3.Span Length Determination:

Give that the structure comprises 2 side spans and 3 interior spans,

The total length constraint is:

$$2L_r + 3L_i = 660\text{ m}$$

Adopted interior span length L_i=160 m

Allowable range for side span:

$$0.5\times 160 \leq L_r \leq 0.7\times 160$$

$$80 \text{ m} \leq L_r \leq 112 \text{ m}$$

Selected side span length L_r= 90 m

Verification

- ✓ $\frac{Lr}{Li}=\frac{90}{160}$= 0.5625 ≈0.6 (within acceptable range)

4.4.Selection of Cross-Sectional Typology:

4.4.1. Justification for box girder selection:

People who build balanced cantilever bridges often use box girder sections for the following reasons:

- Closed sections have a lot of torsional rigidity, which keeps the structure stable during the cantilever construction phase and makes it unnecessary to use intermediate diaphragms.
- A variable depth profile lets you lower the self-weight while keeping the structure efficient.
- No intermediate cross-beams make building easier and cheaper.

4.4.2. Cros-Section selection criteria:

The total deck width *B* is the main factor that determines the cross-sectional configuration. The most common arrangements are as follows:

***Table 3:** Recommended Box Girder Cross-Section Configuration Based on Deck Width*

Deck Width	Recommended Configuration
B ≤13 m	Single box girder “2 webs”
13 ≤B≤ 18 m	Single box girder “3 webs”
18≤ B ≤25 m	3 web box girder, or twin 2 web box girder with transverse prestressing
B >25 m	Multi-cell box girder

4.4.3. Adopted Cross-Section:

The total width of the deck in this project is:

$$B = 13.20 \text{ m}$$

According to the criteria above, the chosen cross-section is:

- Single-cell box girder with 2 webs

4.5. Transverse Design of the Box Girder:

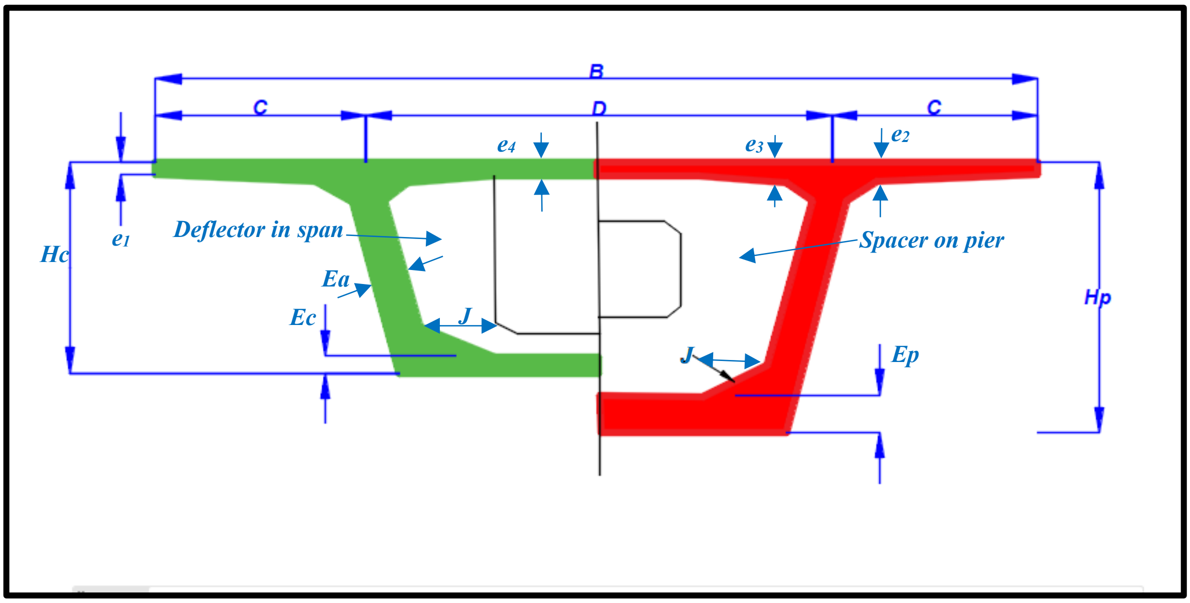

Figure 27:Transverse Cross-Section of the Box Girder

4.6. Deck Depth and Web Design:

4.6.1. Deck Depth:

Adopted Depths:

According to the first sizing calculations, the following depths have been chosen:

Table 4: Adopted Deck Depth at the Pier Mid-Span

Location	Symbol	Adopted Value
At pier	H_p	9.00 m
At mid-span (Key segment)	H_c	5.00 m

- **Depth Variation Law:**

The depth of the deck changes in a parabolic way along the bridge axis, going from

H_p = 9.00 m at the pier to H_c = 5.00 m at the mid-span closure joint, as shown below:

$$H(x) = H_c + (H_p - H_c)\times\left(\frac{x}{L/2}\right)^2$$

Where:

- x: distance measured from the mid-span closure joint
- L: span length
- $H_p - H_c = 9.00 - 5.00 = 4.00$ m

4.6.2. Web Design:

Web Inclination:

The webs of the box girder are slanted, like this:

Makes it easier for the form traveler to remove (strip) the formwork;

Helps make the pier head narrower;

- Makes the building look better.
- The usual inclination is between 5% and 25%. The following inclination is used in this project:

4.6.3. Web Thickness:

From a longitudinal perspective, the web thickness is typically maintained at a consistent level in variable-depth bridges. Nevertheless, in spans that exceed approximately 160 m as is relevant to the current project it is common practice to increase the thickness of the webs near the top slab through the implementation of haunches for the following justifications:

- The shear forces are maximized in this specific area.
- Cantilever tendons are anchored within the upper haunch, necessitating an augmented thickness to effectively counteract the resultant local stresses.

In light of the aforementioned factors, a uniform web thickness is established for both the pier and mid-span sections.

$$E_{ap} = E_{ac} = 0.50\ m$$

Web Parameter:

***Table 5:* Box Girder Web Design Parameters**

Parameter	Value
Web inclination	10%
Web thickness at pier	0.50 m
We thickness at mid-span	0.50 m
Upper haunch	Provided for spans 160 m
Tendon anchorage zone	Upper haunch region

4.7. Top Slab Design:

4.7.1. General Behavior:

The upper slab is firmly integrated with the box girder webs, functioning as a continuous slab supported at the locations of the webs. Its design is chiefly influenced by transverse bending due to the impact of highway live loads (dynamic loads).

The connection between the upper slab and the webs is accomplished through the use of haunches, which offer the following benefits:

- Facilitate the placement of concrete during the casting process.
- Provide sufficient space for the anchorage of cantilever tendons.

4.7.2. Top Slab Thickness:

4.7.2.1. Minimum Thickness at Mid-Span (e_1):

The minimum thickness of the top slab at the crown (mid-span) is determined as follows [10]:

$$e_1 = 25 \text{ cm}$$

- This minimum value ensures:
- ➢ Adequate cover for reinforcement and prestressing ducts
- ➢ Sufficient resistance to local punching under concentrated loads
- ➢ Compliance with EN 1992-2 minimum thickness requirements

4.7.2.2. Thickness at Web Root (e_2):

At the cantilever root, the thickness of the slab, denoted as e_2, is augmented to withstand the elevated bending moments present at the web connection. This thickness is contingent upon [10]:

- The functional cross-sectional profile that has been selected.
- The equipment and installations supported by the deck (such as ballast, tracks, walkways, and drainage systems).

$$e_2 = \left(\frac{1}{7} \text{ or } \frac{1}{8}\right) \times C$$

e_2: Corbel thickness at the root

C: Cantilever length (projecting part beyond the support)

➔ $C = \frac{B}{4} = \frac{13.2}{4} = 3.3$ m
Where ***B*** is the deck width.

$$e_2 = \left(\frac{1}{7}\right) \times 3.3 = 0.47 \text{ m} = 47 \text{ cm}$$

4.7.2.3. Minimum thickness at crown (e_4):

The minimum thickness of the top slab at the crown is determined from the following empirical formula [10]:

$$e_4 = \frac{L}{30} \text{ to } \frac{L}{25}$$

- $e_4 = 25$ cm

4.7.2.4. Thickness at the Cantilever Root (e_3):

The thickness at the cantilever root e_3 is determined using the following three criteria, and the most conservative value is retained [10]:

Criterion 1 "Based on root thickness e_2":

$$e_3 > (e_2 - 0.10) \rightarrow e_3 = 37 \text{ cm}$$

Criterion 2 "Based on deck width":

$$e_3 = 0.10 + \frac{D}{25} \quad \text{where } D = \frac{B}{2} = \frac{13.20}{2} = 6.60 \text{ m}$$

$$e_3 = 0.10 + \frac{6.60}{25} = 0.364 \text{ m} = 36.4 \text{ cm}$$

Criterion 3 "Based on minimum crown thickness":

$$e_3 = 1.5 \times e_4 = 1.5 \times 25 = 37.5 \text{ cm}$$

Adopted Value:

Taking the most conservative result among the three criteria:

$$e_3 = 40 \text{ cm}$$

Important remark (Section at pier):

This segment serves as the critical support and origin point for all cantilever tendons. As such, it experiences the most elevated stress concentrations within the overall structure, which encompass:

- Intense compressive stresses arising from the total prestressing force.
- Considerable shear forces due to the influences of dead loads and live loads.
- Localized bearing stresses at the tendon anchorage locations.

This underscores the imperative of implementing a substantial transverse cross-section at the pier to accommodate these substantial forces, achieved through:

- Enhanced web thickness
- Augmented bottom slab
- Inclusion of a solid diaphragm at the pier head
- Sufficient reinforcement at anchorage locations

4.8. Bottom Slab Design:

4.8.1. Design Governing Criteria:

The bottom slab thickness depends primarily on the limitation due to the stress effect in the bottom fiber both during construction and in service. The parameters that affect the design include the following:

- Size of the compressive force during construction and service.
- Concreting operations.
- Anchorages for the prestress tendons and resulting force distribution.
- Control of the bursting forces caused by the curvature of the tendons.
- Provision for continuity tendons.

4.8.2. Thickness Conditions:

4.8.2.1. Thickness at Mid-Span (Key Segment “E_{ic}”):

The minimum thickness at the crown must satisfy [10]:

$$E_{ic} \geq \max\left(18\text{ cm};\ 3\emptyset;\ \frac{ea}{3}\right)$$

Where:

- $\emptyset$: maximum reinforcement bar diameter
- e_a : duct spacing

Adopted value:

$$E_{ic} = 25 \text{ cm}$$

Verification:

- ✓ 25 cm ≥ 18 cm

4.8.2.2. Thickness at Pier "Ep"

The bottom slab thickness at the pier is generally 2 to 2.5 times the thickness at mid-span [10]:

$$2.0 \le \frac{Ep}{Eic} \le 2.5$$

Adopted value:

$$E_p = 60 \text{ cm}$$

Verification :

- ✓ $\frac{Ep}{Eic} = \frac{60}{25} = 2.4$ (within recommended range)

4.8.3. Thickness Variation Law:

Since $E_p \neq E_{ic}$, the bottom slab thickness varies along the span. Two variation laws are considered [10]:

***Table 6:* Bottom Slab Thickness Variation Laws**

Law	Expression	Advantage
Linear	$E(x) = E_{ic} + (E_p - E_{ic}) \times \left(\frac{x}{L/2}\right)$	Simple to construct
Parabolic	$E(x) = E_{ic} + (E_p - E_{ic}) \times \left(\frac{x}{L/2}\right)^2$	Better adaptation to moment distribution

- The parabolic law is preferred as it provides:
  - Better adaptation to the bending moment envelope
  - Optimized material distribution
  - Smooth transition between pier and mid-span

4.8.4. Determination of Minimum Thickness "Geometric Condition":

The minimum thickness condition $E_{ic} \geq \max(18\ cm;\ 3\emptyset;\ \frac{ea}{3})$ is illustrated as follows:

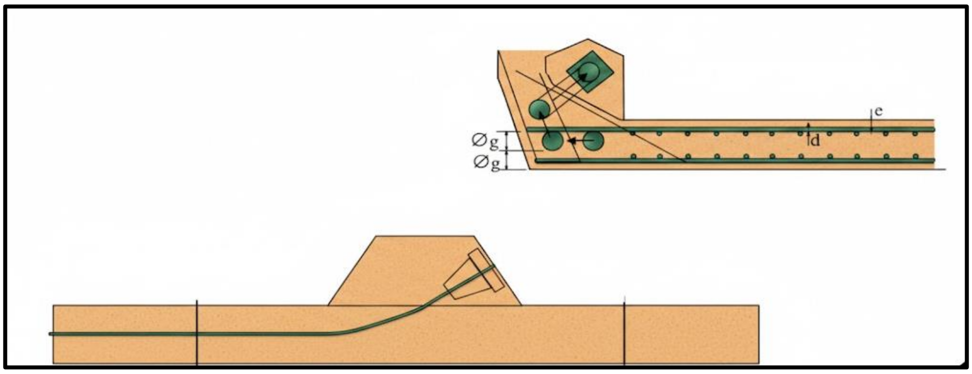

Figure 28:Geometric Condition for Minimum Bottom Slab Thickness

Where:

- 18 cm: absolute minimum for concrete placement
- $3\emptyset$: minimum cover related to bar diameter
- $\frac{ea}{3}$: minimum related to duct spacing

$$E_{ic} = 25\ cm \geq 18\ cm$$

4.9. Haunch Design:

4.9.1. Upper Haunches:

4.9.1.1. Structural Functions:

Several functions influence the design of the upper haunches:

- Increased thickness of the upper slab for areas with high concentrations of shear stress.
- Shape resembling a funnel that facilitates concrete filling in the web area.
- Incorporation of cantilever tendons and adequate cover for concrete.
- Tendon deflection before anchoring.

4.9.1.2. Governing Parameters:

The size of the upper haunch is established based on: (a) number and size of plates used for tendon anchoring, (b) location and diameter of the tendons in the cantilever, and (c) design of the transverse reinforcement.

4.9.1.3. Adopted Dimensions:

Based on cross-section drawings, the following dimensions are adopted:

Table 7: Adopted Dimensions of the Upper Haunch

Parameter	Symbol	Value
Height	*H*	65 cm
Base length	*V*	170 cm

4.9.1.4. Actual Angle Verification:

$$\tan(\propto) = \frac{v}{h} = \frac{170}{65} \approx 2.615$$

$$\propto = \arctan(2.615) \approx 69°$$

Noteworthy: Although the calculated angle is 69°, which is beyond the acceptable limits of 25°to 45°, the dimensions given for the structural drawings (170cm x 65cm) are structurally sound. The accepted angle is more of a guide for building and can be disregarded in case there is a need to do otherwise structurally.

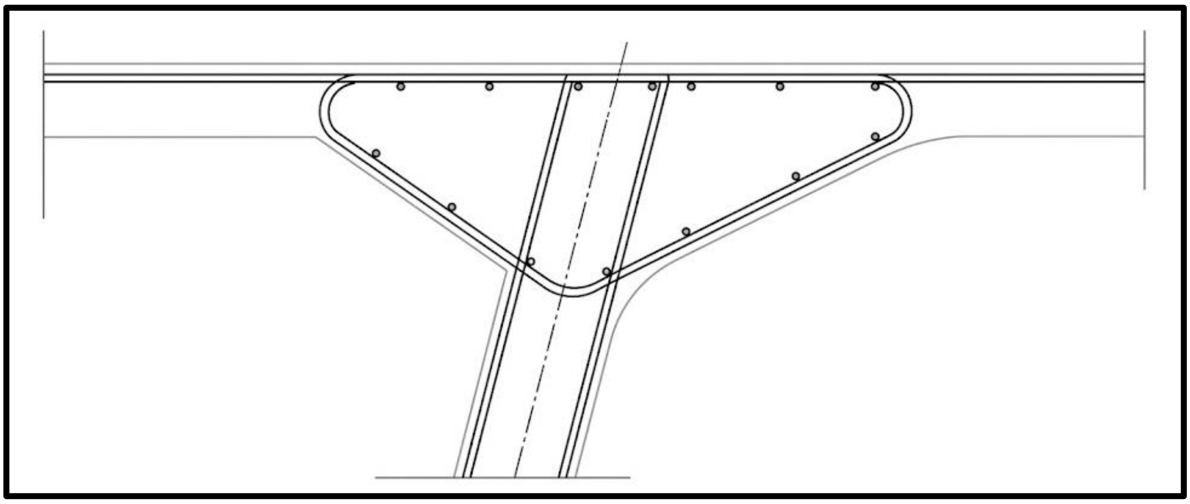

Figure 29: Cross-Section Detail of the upper Haunch at Pier

4.9.2. Lower Haunches:

4.9.2.1. Structural Functions:

Besides their mechanical requirement of providing the interface between the webs and the bottom slab, the functions of the lower haunches are as follows:

- Housing of internal continuity tendons.
- Providing anchor bosses at the web-bottom slab junction for the tendon anchorages.
- Assisting in the movement of concrete while casting.

4.9.2.2. Adopted Dimensions:

The following dimensions are adopted for the lower haunch:

Table 8: Adopted Dimensions of the Lower Haunch

Parameter	Symbol	Value
Angle	$\propto$	62°
Height	H	40 cm
Base length	V	75 cm

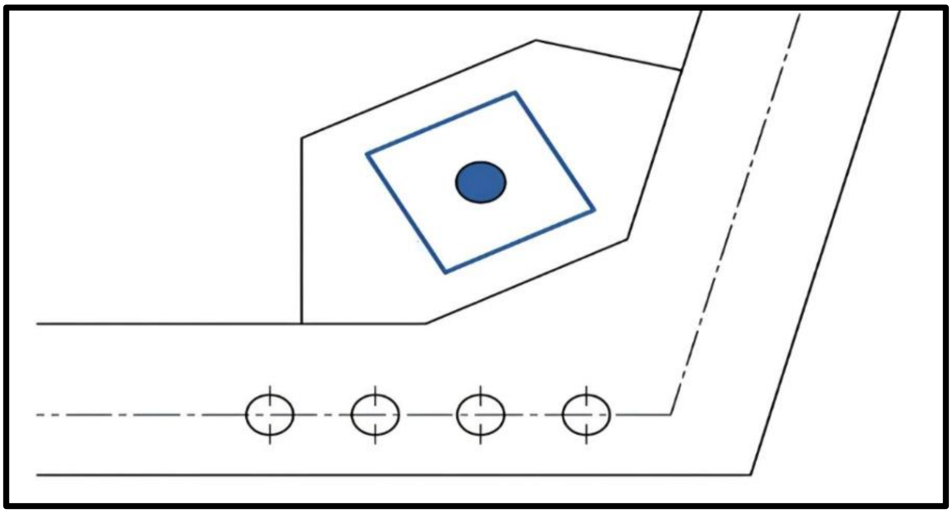

Figure 30:Cross-Section Detail for the Lower Haunch

4.10. Diaphragms and Deviators:

4.10.1. Pier Diaphragms:

Diaphragms play a very important role in bridges that are constructed using the balanced cantilever system of construction. Diaphragms in the piers are especially important since

- they help in fixing external pre-stress tendons and distributing their loads on the cross-section.
- they transfer the vertical load of external deviated pre-stress tendons in the pier segment to the bearings and piers.
- they transfer the shear flow in the webs and slab caused.

4.10.2. Span Deviators:

Deviation beams that are placed within each span are used to divert the tendons of continuity. The following parts make up the deviator:

- Rectangular lower beam.
- Two thin trapezoidal webs.

The use of this configuration helps to deflect the tendon without contributing to much self-weight from the beams.

Figure 31: ***Interior View of a Span deviator Within the Box Girder***

Source: *Viaduct over the Trois Basins Ravine in France*

4.11. Segmental Division:

4.11.1. General Construction Principle:

In the present case, the voussoir units have been prefabricated on-site using form travelers, which have been symmetrically placed about the vertical axis of each pier to ensure uniformity in cantilevering throughout the process. Since the length of the main span is 160 m, the bridge can be segmented into four arms units.

4.11.2. Segment Composition:

4.11.2.1. Each Cantilever Arm:

Each cantilever arm comprises the following segment:

Table 9: Segment Composition of Each Cantilever Arm

Segment Type	Length	Quantity	Total Length
Standard segments	3 m	9	27 m
Standard segments	4 m	5	20 m
Standard segments	5 m	5	25 m
Total standard segments	-	19	72 m

Special Segments:

Table 10: Special Segment Types and Quantities per Span

Segment Type	Length	Quantity per Span
Pier segment	14 m	1
Closure segment	2 m	1
Cast-on-falsework segments	-	2

4.11.2.2. Cantilever Arm Geometry:

Half-span composition (one arm cantilever):

$$L_{arm}=\frac{Lps}{2}+(9\times3)+(5\times4)+(5\times5)+\frac{Lkey}{2}=80\text{ m}$$

Full span verification:

$$L_{span}=2\times L_{arm}=2\times80=160\text{ m}$$

4.12. Segment Length Distribution:

The lengths of the segments increase gradually from the pier to mid-span using the conventional method of balanced cantilevers:

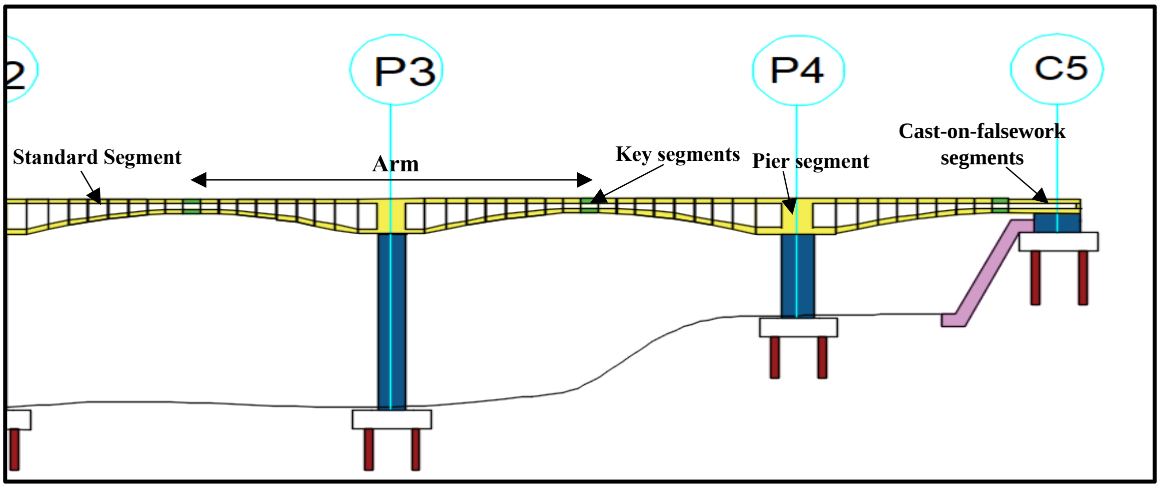

Figure 32: Segment Length Distribution Along the Bridge Spans

4.13. Side-Span Arrangement:

Each Side-span measuring 90 m in length, is composed of:

- The half-length of the pier portion of the adjacent cantilevered span.
- Conventional cantilever spans built using the balanced cantilever construction technique.
- Segments formed on falsework at the abutment end, which offer the required link to the abutment itself. The segments formed on falsework at the abutment.

4.14. Transverse Design of Pier Segment:

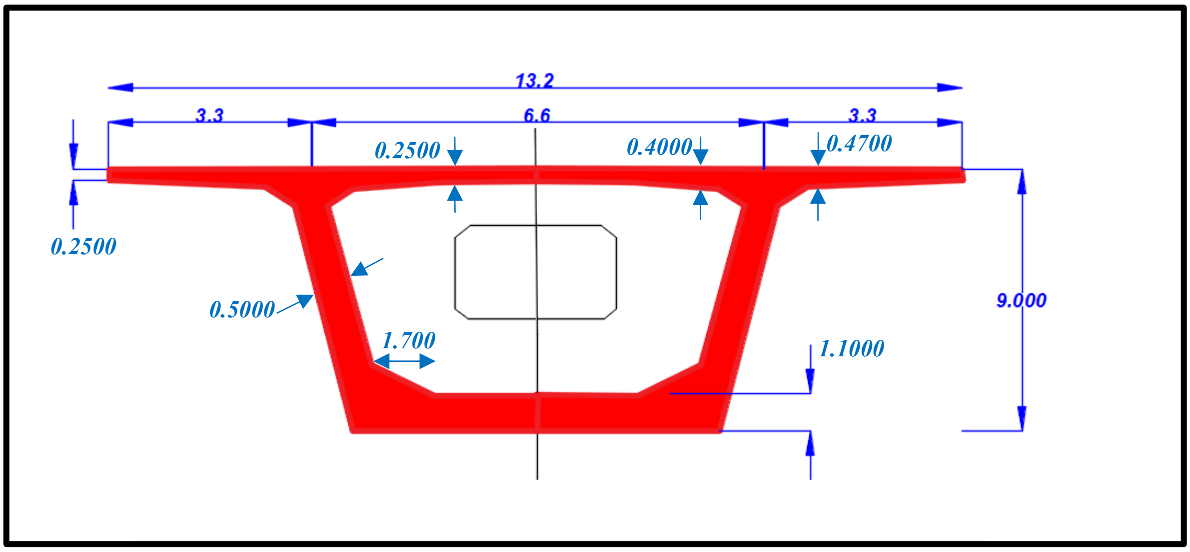

Figure 33: Transverse Cross-Section of the Pier Segment with Dimensions (matter)

4.15. Transverse Design of Key Segments:

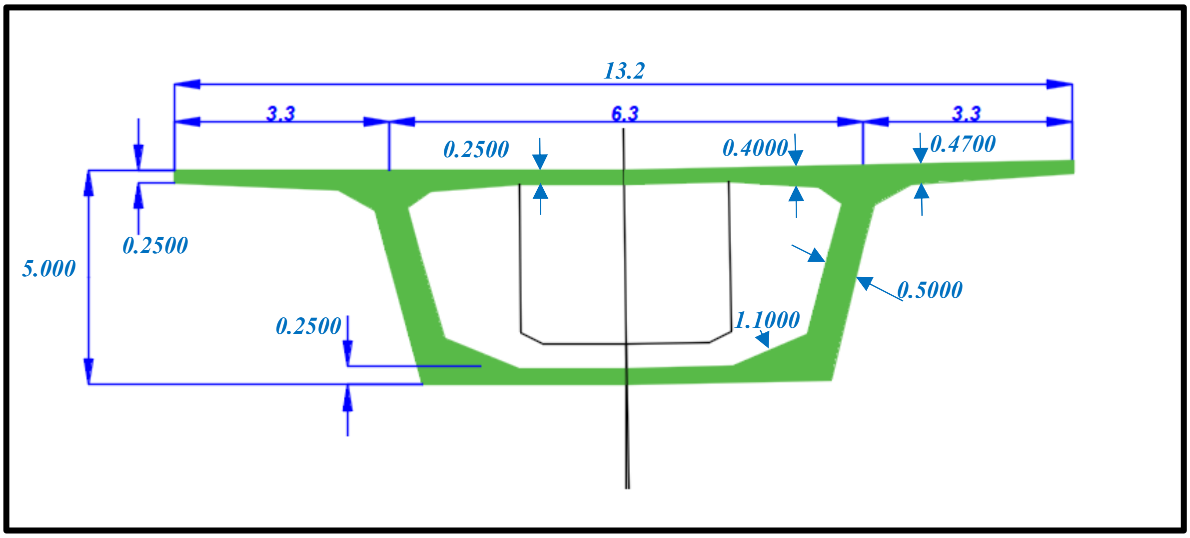

Figure 34: Transverse Cross-Section of the Key Segment with Dimensions (matter)

V. Prestressing Layout:

5.1. Top Slab Tendons:

Top slab cantilever tendons are designed to fulfil the following functions:

During construction:

- Assembly of subsequent segments.
- Stiffness against the action of negative bending moment caused by the weight of cantilever arms and load.

In service:

- The combined effect of the top slab tendons and the external tendons continuity will provide resistance to negative moments for permanent and live loads.

5.1.1. Tendon Layout:

These tendons are situated close to the top fiber of the deck to provide effective resistance against hogging actions. The anchorage detail of these tendons can be described in the following way:

- Deflection of tendons downwards towards the web location around the point of anchorage.
- Anchorages of tendons on the surface of segments in the upper haunch or webs wherever necessary.
- Complete embedding of tendons inside concrete for their entire length.

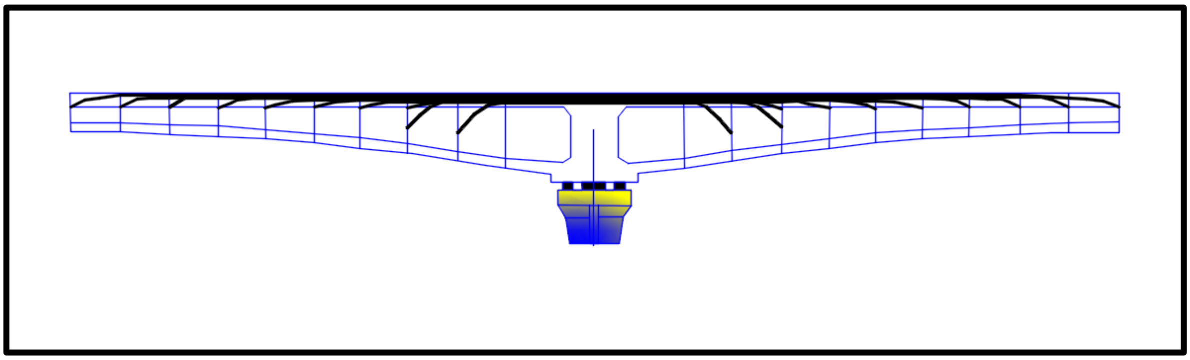

Figure 35: Design of Top Slab Continuity Tendons in Box-Girder Bridge

5.2. Bottom Slab Continuity Tendons:

The internal continuity tendons, which are referred to as, run longitudinally through the center of the spans as well as in the end zones of the side spans. The salient features of the tendons include:

- Designed to take positive (sagging) bending moments.
- Placed in the bottom haunches of the box-girder section.
- Connected at the points where the web and the bottom slab meet.

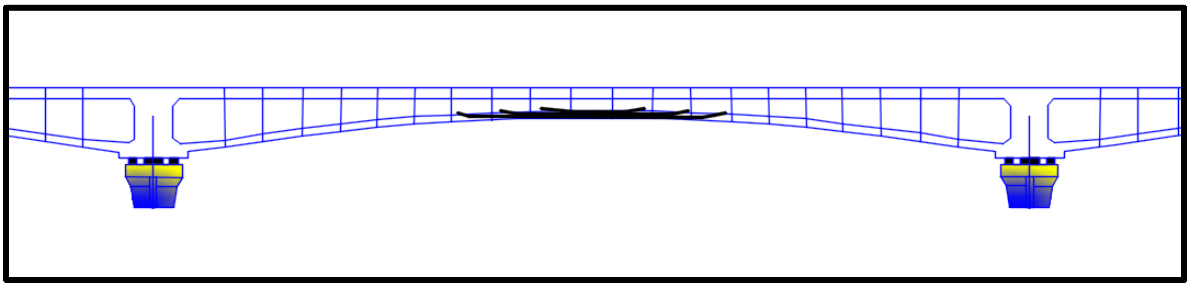

Figure 36: Design of Bottom Slab Continuity Tendons in Box-Girder Bridge

5.3. External Continuity Tendons:

External continuity tendons are designed in the service phase in order to cope with the loads not covered by the two previous categories of tendons. There should be two primary requirements for the layout of tendons:

- Allowable shear stress: it can be obtained through the reduction of shear loads on the webs.
- Allowable normal stress: compression and tension in all cross-sections should not exceed allowable levels.

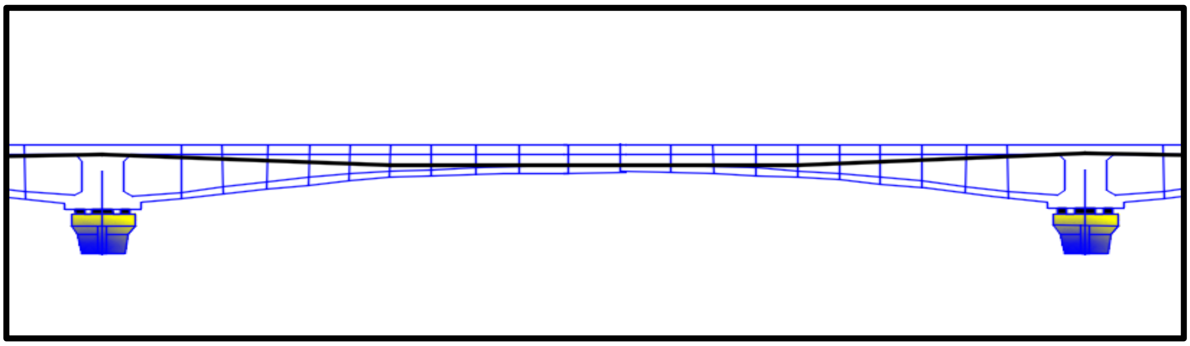

***Figure 37:* Design of External Continuity Tendons in Box-Girder Bridge**

VI. Preliminary of Supports and Foundations:

6.1. General Structure Behavior:

Most balanced cantilever bridges utilize simple bearings in their normal operation, but while constructing these bridges, they remain temporarily fixed to the piers through the use of cantilever stability tendons.

6.2. Pier Head Design Requirements:

Pier heads should incorporate the following elements:

- Fixed supports.
- Provisional stabilizing blocks or temporary supports.
- Fixing tendons.
- Cantilever adjusting jacking supports and possible bearing replacements.
- Facilities for inspection and maintenance.
- Earthquake stops or ship impact dampers as necessary.

6.3. Foundation System:

A deep foundation with pile structures is to be used for all support systems.

Table 11: *Deep Foundation System Parameters for Piers and Abutments*

Support	Pile Configuration	Pile Diameter	Pile Length	Pile Cap Dimensions
Piers (134 m)	12 piles: (3 × 4) arrangement	Ø 1200 mm	30 m	**15.2 m × 16.1 m**
Pier (85 m)	12 piles: (3 × 4) arrangement	Ø 1200 mm	30 m	**12.63 m × 14.1 m**
Pier (46 m)	12 piles: (3 × 4) arrangement	Ø 1200 mm	30 m	**12.2 m × 12.54 m**
Pier (31 m)	12 piles: (3 × 4) arrangement	Ø 1200 mm	30 m	**12.0 m × 11.94 m**
Abutments	9 piles: 3×3 arrangement	Ø 1200 mm	25 m	**12.0 m × 12.0 m**

6.3.1. Pile Cap Dimensions Calculation:

2.1.1. Code-Based Design Criteria:

According to AASHTO LRFD Article 10.7.3.2 [11] , the minimum center-to-center pile spacing and edge distance requirements for pile cap design are as follows:

- Pile Diameter: D = 1200 mm = 1.2 m
- Minimum Pile Spacing: S_{min} = 3.0 × D = 3.0 × 1.2 = 3.6 m
- Adopted Pile Spacing: S = 3.5 × D = 3.5 × 1.2 = 4.2 m
- Minimum Edge Distance: e = 1.5 × D = 1.5 × 1.2 = 1.8 m
- Total Edge Allowance (both sides): 2e = 2 × 1.8 = 3.6 m

2.1.2. General Formula:

The pile cap dimension in each direction is calculated by adding the total edge distance to the corresponding column base dimension:

Pile Cap Dimension = Column Base Dimension + 2 × Edge Distance

Pile Cap Dimension = Column Base Dimension + 3.6 m

2.1.3. Calculations per Pier Height:

1) **Piers (h = 134 m)**: Column Base: 11.6 m × 12.5 m

Longitudinal: Pile Cap = 11.6 + 3.6 = 15.2 m

Transverse: Pile Cap = 12.5 + 3.6 = 16.1 m

→ Pile Cap Dimensions: 15.2 m × 16.1 m

2) **Pier (h = 85 m)**: Column Base: 9.03 m × 10.5 m

Longitudinal: Pile Cap = 9.03 + 3.6 = 12.63 m

Transverse: Pile Cap = 10.5 + 3.6 = 14.1 m

→ Pile Cap Dimensions: 12.63 m × 14.1 m

3) **Pier (h = 46 m)**: Column Base: 8.6 m × 8.94 m

Longitudinal: Pile Cap = 8.6 + 3.6 = 12.2 m

Transverse: Pile Cap = 8.94 + 3.6 = 12.54 m

→ Pile Cap Dimensions: 12.2 m × 12.54 m

4) **Pier (h = 31 m):** Column Base: 8.4 m × 8.34 m

Longitudinal: Pile Cap = 8.4 + 3.6 = 12.0 m

Transverse: Pile Cap = 8.34 + 3.6 = 11.94 m

→ Pile Cap Dimensions: 12.0 m × 11.94 m

5) **Abutments**: Column Base: 8.4 m × 8.4 m (3×3 pile arrangement)

Longitudinal: Pile Cap = 2× (3.5×1.2) + 2× (1.5×1.2) = 8.4 + 3.6 = 12.0 m

Transverse: Pile Cap = 2× (3.5×1.2) + 2× (1.5×1.2) = 8.4 + 3.6 = 12.0 m

→ Pile Cap Dimensions: 12.0 m × 12.0 m ✓

Note: All pile cap dimensions are calculated in accordance with AASHTO LRFD Article 10.7.3.2. The column base dimensions were provided by the supervising engineer and are used as the governing interior dimensions of the pile cap. A minimum edge distance of 1.5D = 1.8 m is applied on each side, resulting in a total addition of 3.6 m per direction.

Based on the pile group configuration and load distribution, larger pile caps were adopted for the central piers (15.2 × 16.1 m) due to higher loads and a 12-pile arrangement (3×4).

In contrast, smaller pile caps (12 × 12 m) were sufficient for the abutments, which are subjected to lower loads and supported by a 9-pile arrangement (3×3). This configuration also ensures that the abutment foundation width is consistent with the total bridge deck width.

6.4. Abutment Configuration:

Each abutment consists of:

- A crossbeam of 1.5 m depth.
- Supported on a pile cap of dimensions 12 m × 12 m.
- Founded on 9 bored piles of 1200 mm diameter and 25 m length, arranged in a 3 × 3 grid.

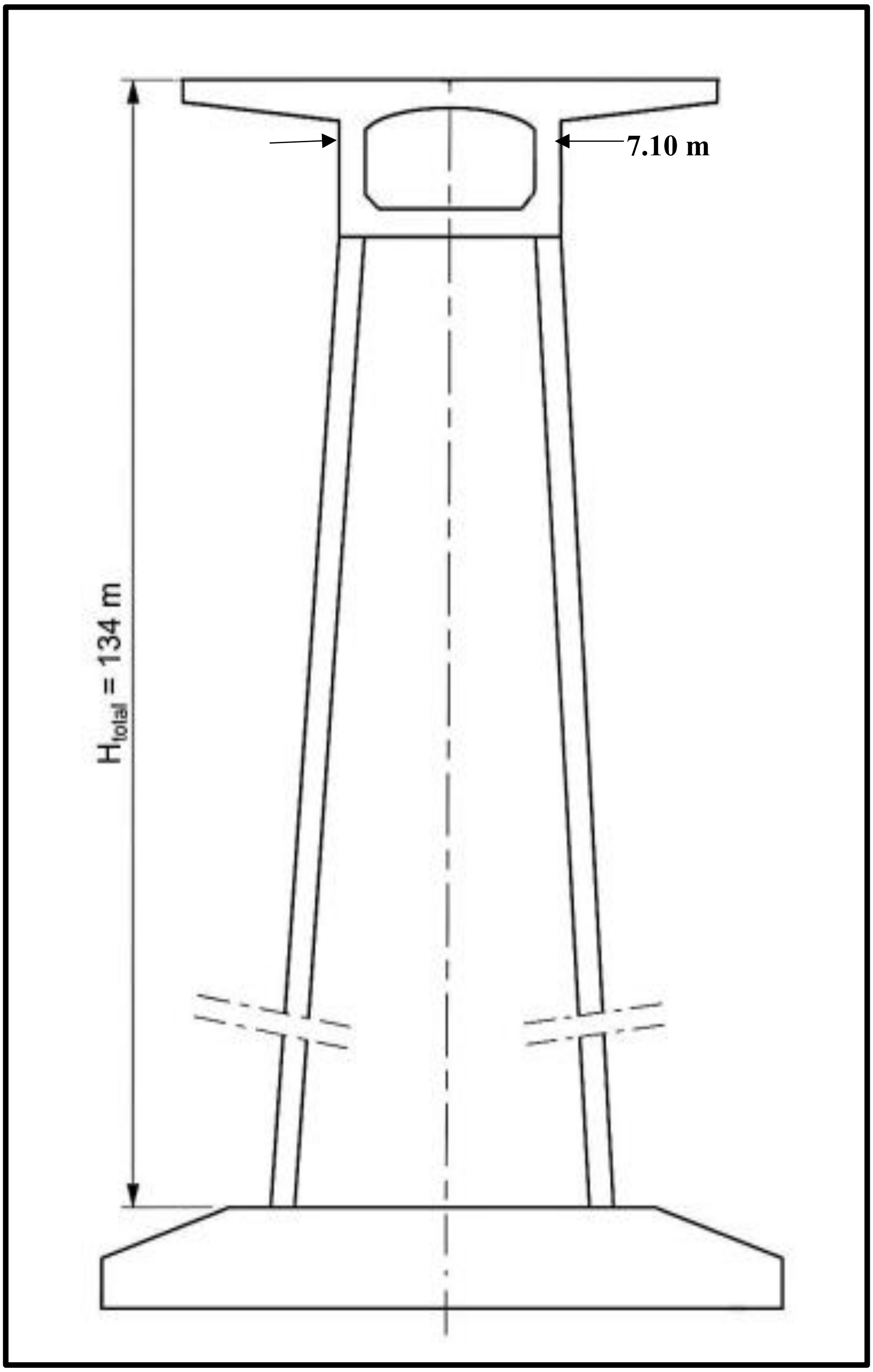

Figure 38:Structural Elevation od Bridge Pier and Foundation Details

VII. Material Properties:

7.1. Concrete:

7.1. General Properties:

The following general properties are adopted for reinforced and prestressed concrete throughout all structural components:

Table 12: General Physical and Thermal Properties of Concrete

Property	Symbol	Value	Reference
Unit weight	γ_c	25 kN/m^3	EN 1991-1-1
Thermal expansion coefficient	$\propto_T$	10×10-6 /C	EN 1991-1-5, Annex C

The concrete used for the bridge deck is a class 45/55. The corresponding mechanical characteristics of this concrete, according to EN 1992-1-1, Table 3.1, presented below:

- **Compressive Strength:**

$$f_{ck}= 45 \text{ MPA} \quad ; \quad f_{ck,cube}=55 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 =45+8=53\text{MPa}$$

$$f_{cd}= \frac{\propto cc \times fck}{\gamma c}= \frac{1.0\times 45}{1.5}= 30 \text{ MPa}$$

- **Tensile strength:**

$$f_{ctm} = 2.12\times \ln\left(1+\frac{fcm}{10}\right)= 2.12\times \ln(6.3) = 3.80 \text{ MPa}$$

$$f_{ctk,0.06}= 0.7\times f_{ctm}=0.7\times 3.80= 2.66 \text{ MPa}$$

$$f_{ctk,0.95}=1.3\times f_{ctm}=1.3\times 3.80=4.94 \text{ MPa}$$

- **Modulus of Elasticity:**

$$E_{cm} = 22 \left(\frac{fcm}{10}\right)^{0.3} = 22 \times (5.3)^{0.3} = 36.283 \text{ MPa}$$

- **Deformation Properties:**

$$\epsilon_{c1}= 0.23\% \quad ; \quad \epsilon_{c1}=0.35\%$$

$$\epsilon_{c2}=0.20\% \quad ; \quad \epsilon_{cu2}=0.35\%$$

$$n=2.0 \quad ; \quad \epsilon_{c3}=0.175\% \quad ; \quad \epsilon_{cu3}=0.35\%$$

7.2. Concrete Class for piers and foundations (35/45):

As for the pier shafts and piles, the concrete material used will be C35/45 concrete. The corresponding values for mechanical properties of this concrete type are according to EN 1992-1-1 Table 3.1:

- **Compressive Strength:**

$$f_{ck}= 35 \text{ MPA} \quad ; \quad f_{ck,cube}=45 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 45+8=43\text{MPa}$$

$$f_{cd}= \frac{\propto cc\times fck}{\gamma c} = \frac{1.0\times 45}{1.5} = 23.33 \text{ MPa}$$

- **Tensile strength:**

$$f_{ctm} = 2.12\times \ln\left(1+\frac{fcm}{10}\right)= 2.12\times \ln(5.3) = 3.51 \text{ MPa}$$

$$f_{ctk,0.06}= 0.7\times f_{ctm}=0.7\times 3.51= 2.46 \text{ MPa}$$

$$f_{ctk,0.95}=1.3\times f_{ctm}=1.3\times 3.51=4.56 \text{ MPa}$$

- **Modulus of Elasticity:**

$$E_{cm} = 22 \left(\frac{fcm}{10}\right)^{0.3} = 22 \times (4.3)^{0.3} = 34.077 \text{ MPa}$$

7.3. Concrete Class for Piles:

The concrete class for the foundation piles is C30/37. The following are the mechanical properties of the material as per EN 1992-1-1, Table 3.1:

- **Compressive Strength:**

$$f_{ck}= 30 \text{ MPA} \quad ; \quad f_{ck,cube}=37 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 45+8=38\text{MPa}$$

$$f_{cd}= \frac{\propto cc\times fck}{\gamma c} = \frac{1.0\times 45}{1.5} = 20.00 \text{ MPa}$$

- **Tensile strength:**

$$f_{ctm} = 2.12\times\ln\left(1+\frac{fcm}{10}\right)= 2.12\times\ln(4.8) = 3.28 \text{ MPa}$$

$$f_{ctk,0.06}= 0.7\times f_{ctm}=0.7\times3.28= 2.30 \text{ MPa}$$

$$f_{ctk,0.95}=1.3\times f_{ctm}=1.3\times3.28=4.26 \text{ MPa}$$

- **Modulus of Elasticity:**

$$E_{cm} = 22\left(\frac{fcm}{10}\right)^{0.3} = 22 \times (3.8)^{0.3} = 32.836 \text{ MPa}$$

7.4. Poisson's Ratio:

In accordance with EN 1992-1-1, Section 3.1.3, the Poisson's ratio is adopted as follows:

- $v = 0.2$ for uncracked concrete (elastic deformation): The substance behaves as an isotropic elastic solid material. And the lateral strain produced by the axial strain is significant enough to be considered.
- $v = 0$ for cracked concrete: After cracking happens, the tensile strength of concrete is reduced, and there will be no interaction between lateral strain, therefore, Poisson's ration is taken as zero.

7.5. Stress and deformation Limits:

7.1.1. Ultimate Limit State (ULS):

- **Design Compressive Strength:**

In accordance with EN 1992-1-1, Sec 3.1.6, the design compressive strength of concrete is defined as:

$$f_{cd}= \frac{\propto cc\times fck}{\gamma c}$$

- **Design Tensile Strength:**

$$f_{ctd}= \frac{\propto ct\times fctk{,}0.05}{\gamma c}$$

Where:

$\propto_{cc}$=1.00 (Coefficient for long-term effects-compression)

$\propto_{ct}$=1.00 (Coefficient for long-term effects-tension)

γ_c= 1.50 (Partial safety factor for concrete)

7.1.2. Special Case "Deep Foundation Piles":

For piles, which are considered deep foundations, the design strength equations take into consideration the quality control coefficient k_3 based on the degree of quality control adopted during site construction activities:

$$f_{cd,pieux} = k_3 \times \frac{\propto cc \times fck}{\gamma c}$$

$$f_{ctd,pieux} = k_3 \times \frac{\propto ct \times fctk,0.05}{\gamma c}$$

Note: The factor k_3=0.67 stands for the natural variability associated with cast-in-place piles, where achieving thorough quality control and inspection in much more difficult than for exposed structural elements.

Table 13: ***Summary of Concrete Design Strengths (fcd and fctd) for ULS***

	f_{cd} (MPa)	f_{ctd} (MPa)
C45/55	30.00	1.77
C35/45	23.33	1.50
C30/37	20.00	1.35

7.1.3. Serviceability Limit State (SLS) "Stress Limitations":

In accordance with EN 1992-1-1, Sec 7.2 and EN 1992-2, Sec7.2, the following stress limitations are applied at the Serviceability Limit State:

The compressive stress in concrete must not exceed the following limits under the respective load combinations:

- $\sigma c \leq 0.45 f_{ck}$: under quasi-permanent load combination
- $\sigma c \leq 0.60 f_{ck}$: under characteristic load combination
- $\sigma c \leq 0.60 f_{ck}$: under frequent load combination

- **Deck Concrete-C45/55(f_{ck}= 45 MPa):**

Table 14: ***SLS Stress Limitations for Deck Concrete (C45/55)***

Load Combination	Limit Expression	Allowable Stress
Quasi-permanent	0.45 × 45	20.25 MPa
Characteristic	0.60 × 45	27.00 MPa
Frequent	0.60 × 45	27.00 MPA

- **Pier & Pile Cap Concrete-C35/45(f_{ck}= 35 MPa):**

Table 15: SLS Stress Limitations for Pier and Pile Cap Concrete (C35/45)

Load Combination	Limit Expression	Allowable Stress
Quasi-permanent	0.45×35	15.75 MPa
Characteristic	0.60×35	21.00 MPa
Frequent	0.60×35	21.00 MPA

7.2. Passive Reinforcement:

7.1. Mechanical Characteristics:

In accordance with EN 1992-1-1 Sec 3.2.7, the main mechanical properties of high-yield reinforcing steel type B500B are as follows:

Table 16: Mechanical Characteristics of B500B Passive Reinforcement

Property	Symbol	Value
Modulus of elasticity	E_E	200,000 MPa
Characteristic yield strength	f_{yk}	500 MPa
Characteristic tensile strength	$f_t = k \times f_{yk}$	540 MPa (k= 1.08)
Ultimate strain under maximum load	ϵ_{uk}	5.0%
Design Ultimate strain	$\epsilon_{ud} = 0.9 \times \epsilon_{uk}$	4.5%
Unit weight	P	77 kN/m^3 (7,850 kg/m^3)
Smooth bars yield strength	f_{yk}	235 MPa

7.2. ULS Design Strength:

The design yield strength of reinforcing steel is defined as: EN1992-1-1 and EN 1992-1-1/NA, Sec2.4.2.4.

$$f_{yd} = \frac{fyk}{\gamma s} = \frac{500}{1.15} = 434.78 \text{ MPa}$$

where the partial safety factor for steel is taken as:

γ_s= 1.15 (persistent and transient situations-fundamental ULS)

7.3. SLS Stress Limitations and Crack Control:

A. Stress Limitation Under Characteristic Combination:

To prevent unacceptable cracking levels, the stress in passive reinforcement is limited under the characteristic load combination to: [12]

$$\sigma_s \leq 0.80 \times f_{yk} = 0.80 \times 500 = 400 \text{ MPa}$$

B. Crack Width Control:

The maximum allowable crack width is limited to:

$$w_{max}=0.30 \text{ mm}$$

This corresponds to a limitation of tensile stress in the reinforcement under the quasi-permanent load combination:

$$\sigma_s \leq 300 \text{ MPa}$$

7.3. Prestressing System:

7.1. Overview of the Selected Prestressing System Technology:

The selected prestressed concrete technology for the proposed bridge consists of bonded post-tensioned tendons consisting of 27 strands of prestressing steel type T15, referred to herein as 27T15. This choice was made to meet the requirements of the applied load conditions and comply with the common practice of post-tensioned bridges.

The nominal diameter of each strand is 15.70 mm and the area is 150 mm². Hence, the steel area in one strand is equal to 2,850 mm². The relevant properties of the used prestressing steel are listed below:

***Table 17:* Mechanical Properties of 27T15 Prestressing Steel Strands**

Mechanical Property	Symbol	Value
Total Tendon Area	A_p	4,050 mm^2 (27×150 mm^2)
Number of Strands	N	27
Characteristic tensile strength	f_{pk}	1860 MPa
Characteristic 0.1% proof stress	$f_{p0.1k}$	1650 MPa
Modulus of elasticity	E_p	190,000 MPa
Relaxation class	-	Low relaxation

7.2. Permissible Stress at Jacking:

In order to overload the tendons in their process of tensioning, the stress on the anchorage must not exceed the upper limit given below [13]:

$$\sigma_{p,max} = \min (k_1 \times f_{pk} \; ; k_2 \times f_{p0.1k})$$

Applying the prescribed coefficients $k_1 = 0.80$ and $k_2 = 0.90$:

$$\sigma_{p,max} = \min (0.80 \times 1860 \; ; 0.9 \times 1650) = \min (1488; 11485)= 1485$$

This limiting value governs the maximum force that may be applied to each tendon during stressing operations.

CHAPTER D:

STRUCTURAL ASSESSMENT AND DESIGN OF BOTTOM CONTINUITY TENDONS DURING CONSTRUCTION STAGE

I. Design of Bottom Continuity Tendons:

This chapter is dedicated to the calculation of the bottom slab continuity tendons, which are designed to withstand all supplementary actions applied to the structure after the completion of the cantilevers.

The design of bottom continuity tendons constitutes one of the most critical stages in the structural development of balanced cantilever bridges. Unlike cantilever tendons, which mainly ensure equilibrium during segmental erection, continuity tendons are introduced to guarantee the mechanical continuity of the final bridge system after closure operations. Their contribution becomes particularly significant during the transition from the statically determinate cantilever configuration to the final hyperstatic structural behavior of the completed bridge.

The introduction of continuity prestressing significantly modifies the internal force distribution within the structure. Once the closure segments are completed, the bridge begins to behave as a continuous multi-span system, leading to moment redistribution effects caused by creep, shrinkage, and long-term prestress losses. Consequently, the accurate sizing and positioning of continuity tendons become essential to limit tensile stresses in the bottom slab and maintain serviceability requirements under permanent and variable actions.

The combined use of internal and external continuity tendons allows the designer to optimize both structural efficiency and construction feasibility. Internal bonded tendons provide an efficient local stress transfer mechanism and contribute to crack control near closure joints, whereas external tendons offer easier inspection, simplified replacement operations, and improved adaptability during future strengthening interventions. This hybrid prestressing strategy therefore combines the advantages of bonded and unbonded systems while enhancing the global structural redundancy of long-span box girder bridges.

The internal continuity tendons, often referred to as **"** bottom continuity tendons**"**, are located in the central part of the spans (key segments) and at the ends of the side spans (sections cast on falsework).

➢ Key Segment Closure & Tensioning Sequence:

During the execution, the key segment closures and the tensioning of the bottom continuity tendons must follow this specific chronological order to ensure structural stability:

- **Closure of Abutment C0 – Pier P1:** Key segment execution of the C0-P1 span and tensioning of the **C0-P1 bottom continuity tendons**.
- **Closure of Pier P4 – Abutment C5:** Key segment execution of the P4-C5 span and tensioning of the **P4-C5 bottom continuity tendons**.

- **Closure of Pier P1 – Pier P2:** Key segment execution of the P1-P2 span and tensioning of the **P1-P2 bottom continuity tendons**.
- **Closure of Pier P3 – Pier P4:** Key segment execution of the P3-P4 span and tensioning of the **P3-P4 bottom continuity tendons**.
- **Closure of Pier P2 – Pier P3:** Key segment execution of the P2-P3 central span and tensioning of the **P2-P3 bottom continuity tendons**.

The selected closure sequence directly governs the evolution of internal stresses during the final stages of construction. Since each closure operation progressively transforms the structural system, the chronology adopted for tendon tensioning must ensure the preservation of structural equilibrium while minimizing undesirable secondary stresses generated by restrained deformations.

From a structural mechanics perspective, the final central closure generally represents the most sensitive stage of the construction process. At this stage, the bridge experiences the highest redistribution of bending moments due to continuity establishment and time-dependent effects. Any inappropriate sequencing may therefore induce excessive tensile stresses in the bottom slab or lead to undesirable stress concentrations near deviators and anchorage zones.

The internal continuity tendons are the primary reinforcement used to transform the independent cantilevers into a continuous structural system.

a) Functional Definition:

- **Purpose:** These tendons are designed to resist the positive bending moments (tension in the bottom fiber) caused by superimposed dead loads (pavement, safety barriers) and live traffic loads after the bridge is completed.

  The positive bending moments generated after bridge completion differ fundamentally from those encountered during cantilever erection. During construction, the deck mainly experiences negative moments above the piers due to cantilever action. However, after continuity establishment and traffic loading, significant positive moments develop at mid-span regions, creating tensile stresses in the lower fibers of the box girder. The continuity tendons are therefore specifically introduced to compensate for this inversion of structural behavior and maintain the stress state within admissible service limits.
- **Mechanical Role:** They "clamp" the key segments together and manage the redistribution of stresses caused by concrete creep as the structure transitions from its construction state to its hyperstatic final state.

Beyond their direct contribution to flexural resistance, continuity tendons also improve the global stiffness of the bridge deck. Their presence reduces long-term deflections and contributes to limiting crack propagation in regions subjected to tensile stresses. In addition, the redistribution capability provided by prestressing continuity enhances the overall durability of the structure by maintaining concrete compression within favorable ranges throughout the service life of the bridge.

b) *Geometric Layout and Positioning:*

- **Location:** Unlike cantilever tendons that remain in the top slab, continuity tendons are primarily placed in the bottom slab and the lower haunches of the box girder.
  The eccentricity of continuity tendons relative to the centroidal axis of the box girder strongly influences the efficiency of prestressing actions. Increasing tendon eccentricity enhances the generated counteracting moment and consequently improves tensile stress reduction in the bottom slab. However, excessive eccentricity may lead to local stress concentrations near deviators and anchorage zones, requiring careful geometric optimization to ensure both structural efficiency and constructability.
  For this reason, the determination of tendon profiles cannot be limited to simplified analytical calculations alone. Numerical modelling becomes necessary to accurately capture the interaction between tendon geometry, staged construction effects, creep redistribution, and global bridge behavior.
- **Anchorage:** They are anchored in concrete blocks known in French as bossages, typically located at the junction where the webs meet the bottom slab.
  Anchorage zones constitute highly sensitive regions due to the localized transfer of concentrated prestressing forces into the concrete section. The resulting bursting and spalling stresses may generate local cracking if insufficient transverse reinforcement is provided. Consequently, the design of bossage regions requires particular attention to ensure an adequate diffusion of prestressing forces and preserve the durability of the bridge deck.
- **Symmetry:** For a balanced design, tendons are typically tensioned in symmetrical pairs across the mid-span closure joint to maintain structural equilibrium and ensure a uniform distribution of the prestressing forces across the section.

c) *Tensioning Sequence:*

The tensioning must follow a strict order of closure to manage the internal stresses effectively:

- **End Spans:** Tensioning of Bottom tendons in the sections cast on scaffolding at the abutments.
- **Central Spans:** Tensioning across the key segments between piers (e.g., P1-P2, P2-P3) only after the concrete of the key segment has reached sufficient strength.

II. Construction Phases:

The analysis confirms that construction stages remain a governing parameter in the design of continuity prestressing systems. Since stresses evolve continuously during segment erection and closure operations, staged construction analysis becomes essential to accurately evaluate the final structural behavior of balanced cantilever bridges.

For 660 m bridge with 5 spans (90m + 160m + 160m + 160m + 90m), the construction follows these five critical stages:

Phase 0: Sub-Structure and Pier Table:

- **Piers and Abutments:** Vertical columns and foundations are constructed using reinforced concrete.
- **Pier Table Setup:** The first section is set up on top of the piers, which acts as a table to support the remaining sections of the balanced cantilever bridge.

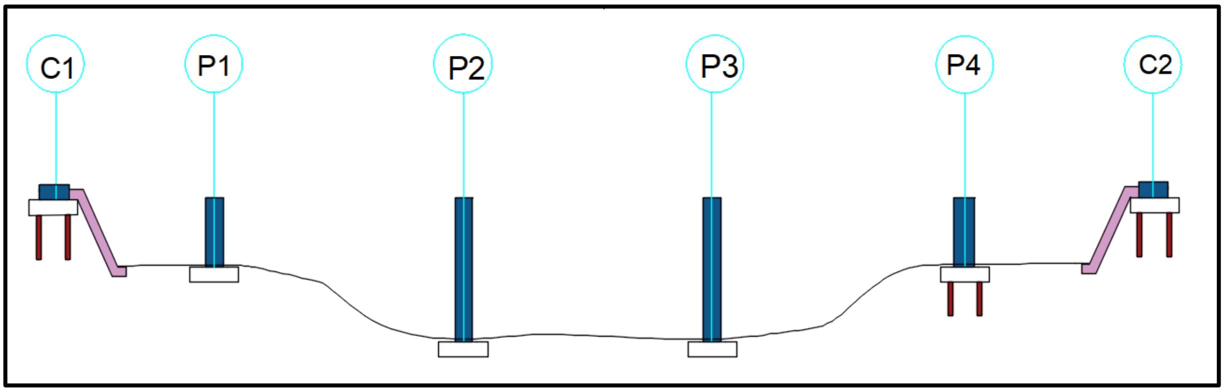

*Figure 39: **Sub-Structure Layout: Piers and Abutments Configuration***

Phase 1: Balanced Cantilever Segmental Construction:

- **Symmetrical Segment Casting:** Gradually casting the segments of the deck on both sides of the pier to keep the structure balanced.

- **Installation of Internal Tendons:** Installing prestressing tendons within the segments to balance the cantilever effect.

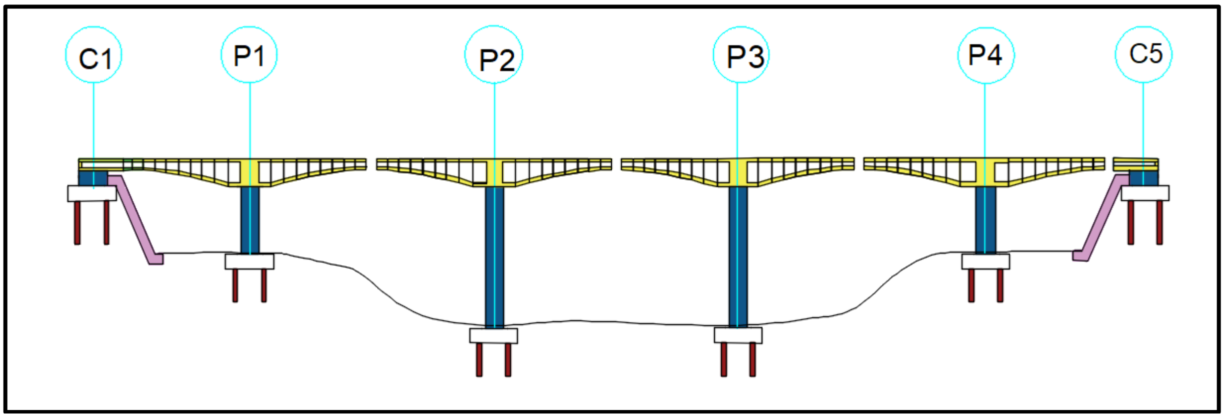

Figure 40: ***Balanced Cantilever Segmental Construction with Internal Tendons Installation***

Phase 2: Transition and Continuity (Key Segments):

- Closing the side spans to connect the end cantilevers with the abutments (C1/P1 and P4/C5).
- **Bottom tendon tensioning**: post-tensioning the bottom to ensure structural continuity.

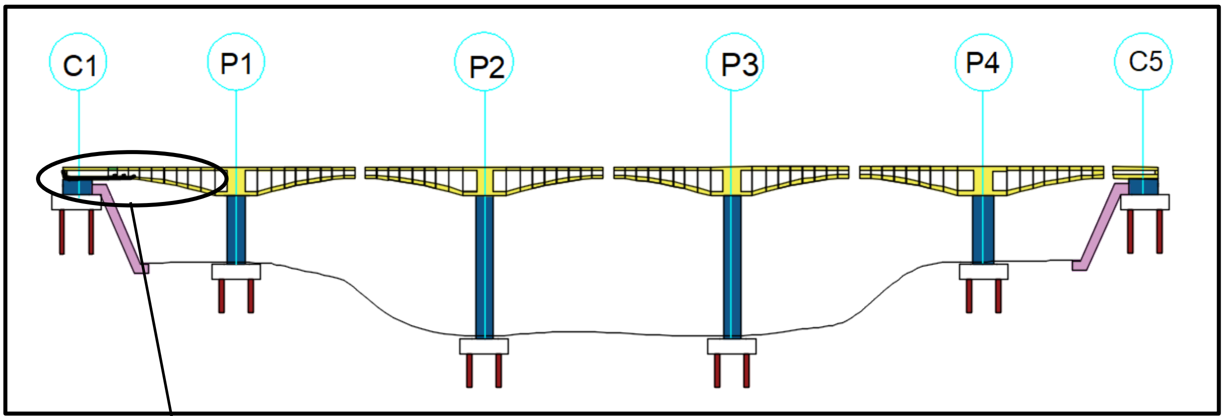

Figure 41: ***Bottom Slab Continuity Tendons Layout at Abutment Zone C1-P1***

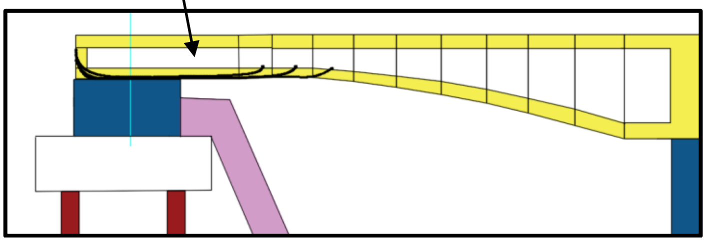

Figure 42: ***Detail View: Bottom Slab Tendon Profile at Abutment C1-P1***

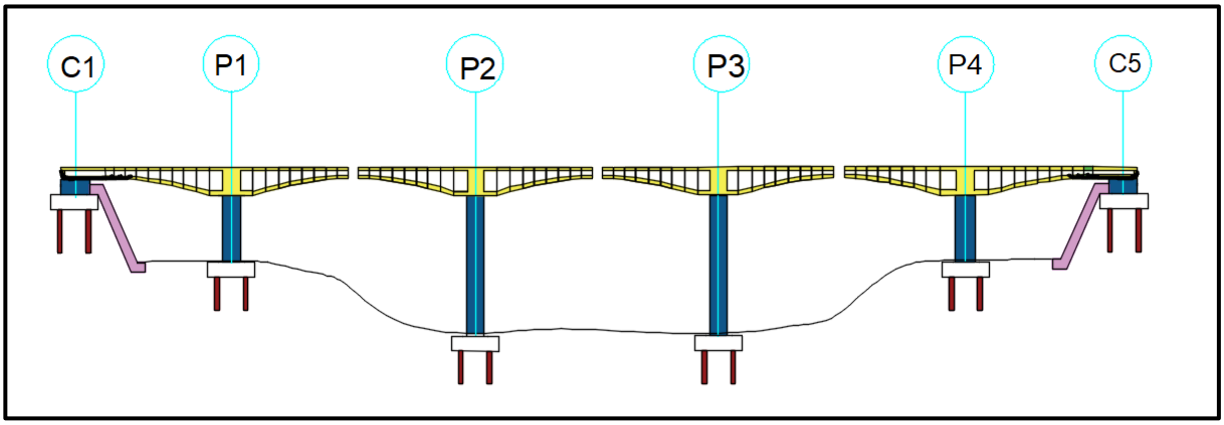

*Figure 43: **Completed Bridge Deck After Closure of Side Spans and Continuity (C1-P1)***

Phase 3: Linking the First Interior Span (P1-P2):

- **Mid-span Stitching (P1/P2):** Completion of the fundamental closure joint between the cantilever arm of Pier P1 and the cantilever arm of Pier P2.
- **Continuity Tendon Installation:** Installation of the bottom continuity tendons to control the positive bending moments at mid-span.
- **Structural Transformation:** Two cantilever structures merge into a single beam with transformed internal stresses.

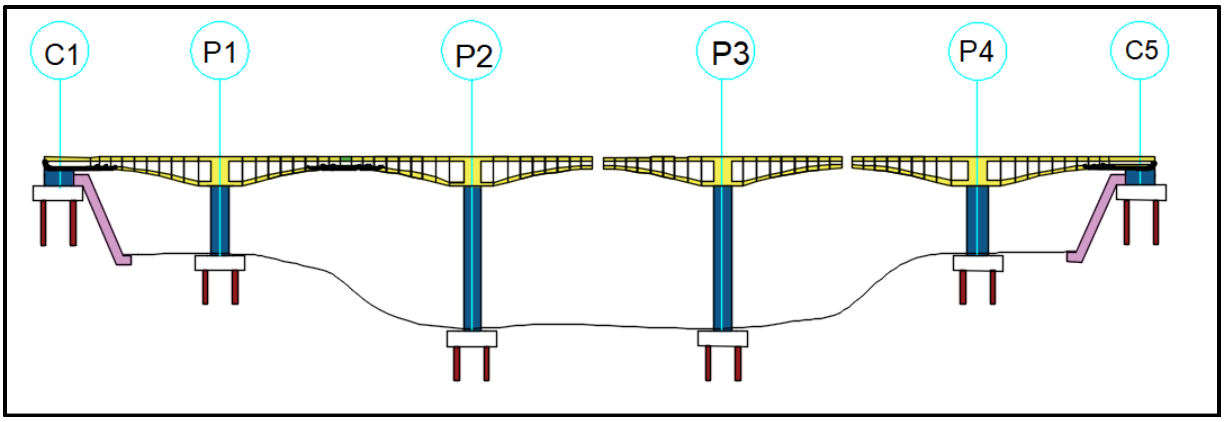

*Figure 44: **Structural Continuity After Interior Span Stitching (P1-P2)***

Phase 4: Linking the Second Interior Span:

- **Closure Segment Execution (P3/P4):** Building the last essential segment that connects the cantilever arms of Pier P3 and Pier P4.
- **Post-Tensioning Sequence:** Gradually tensioning the continuity prestressing tendons to produce a monolithic behavior of the deck.
- **Load Redistribution:** Refining the structure to account for the full dead load as well as the effects of creep and shrinkage.

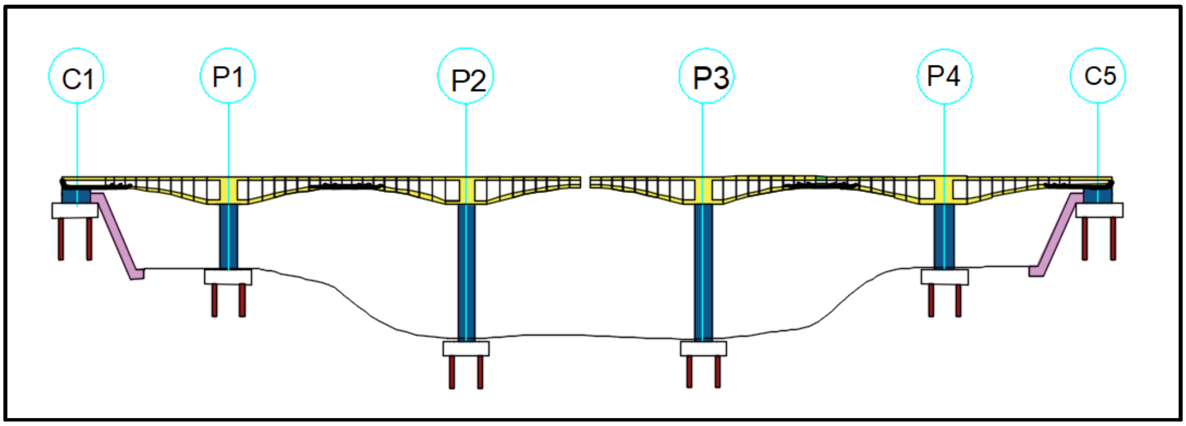

Figure 45: Full Structural After Closure of Second Interior Span (P3-P4)

Phase 5: Final Closure and Completion:

- **Central Span Key Segment:** Complete the central 160-meter span connection between piers P2 and P3.
- **Establishment of Final Deck Continuity:** Complete the last tensioning step to achieve the intended long-term structural behavior.

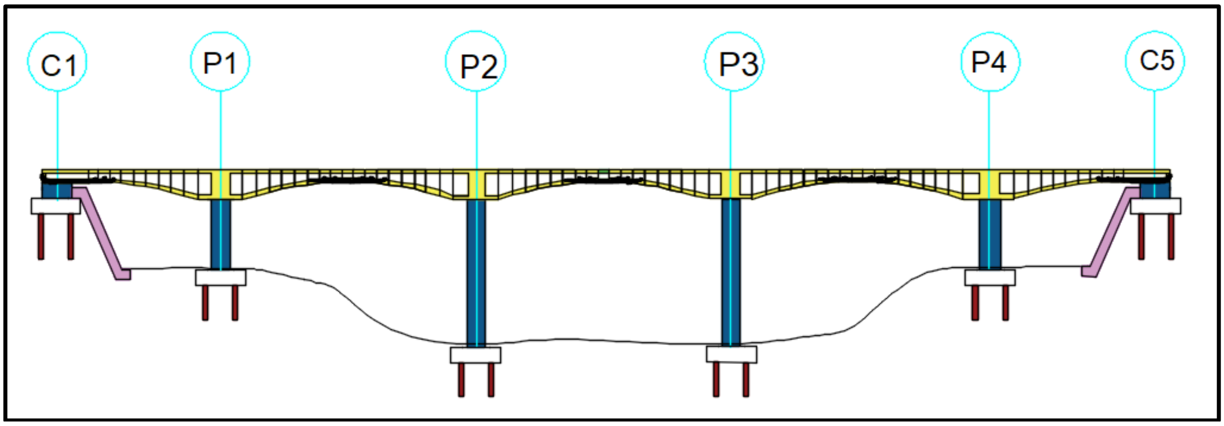

Figure 46: Final Closure of Central Span (P2-P3) and Full Deck Continuity

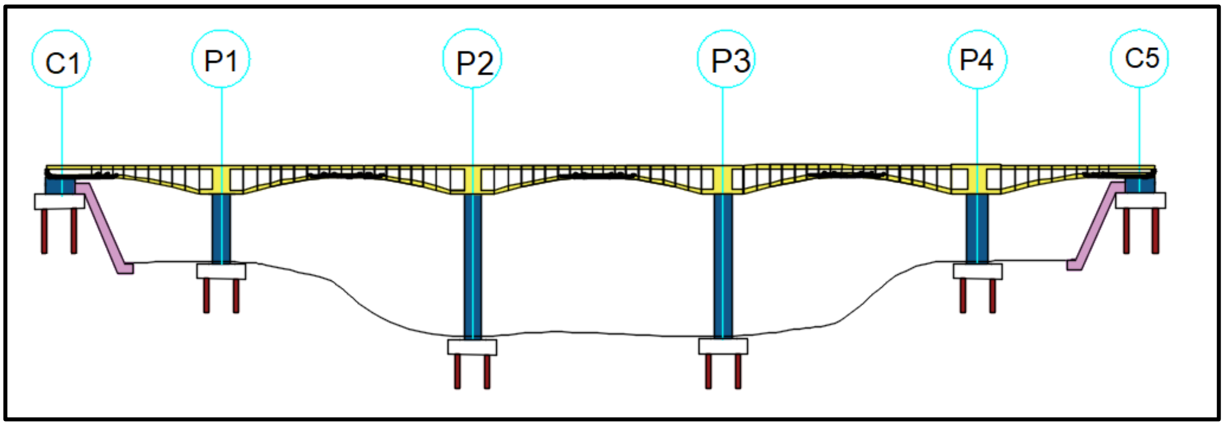

Figure 47 : Completed Bridge-Final Structural Configuration (C1 to C5)

➔ *Following the definition of the structural role and geometric arrangement of continuity tendons, the next stage consists of determining the required prestressing force capable of satisfying serviceability criteria during the final bridge configuration. The calculation procedure presented in the following sections is based on the verification of tensile stress limitations in the bottom slab under the combined effects of permanent loads, prestressing actions, and live traffic loads.*

III. Calculation Of the Bottom Continuity Tendons for The Side Span (C1–P1) Constructed by Casting on Falsework:

The bottom continuity tendons incorporated in the side-span segment cast on falsework are intended to resist the following loading effects:

- the self-weight of the closure segment.
- the effects of the removal of the form traveler.
- the effects of the thermal gradient.

In contrast to the other bottom continuity tendons, which are used throughout the structure, the bottom continuity tendons incorporated in the side-span segment are intended to create both isostatic primary moments and hyperstatic secondary moments simultaneously, owing to the static configuration adopted by the structure in this particular stage of the construction process.

To calculate the number of tendons required to close the side span, the self-weight of the segment cast on falsework is applied to the continuous structure model, which is justified because the effects of the self-weight become active after the removal of the falsework, i.e., after the full establishment of the deck's mechanical continuity.

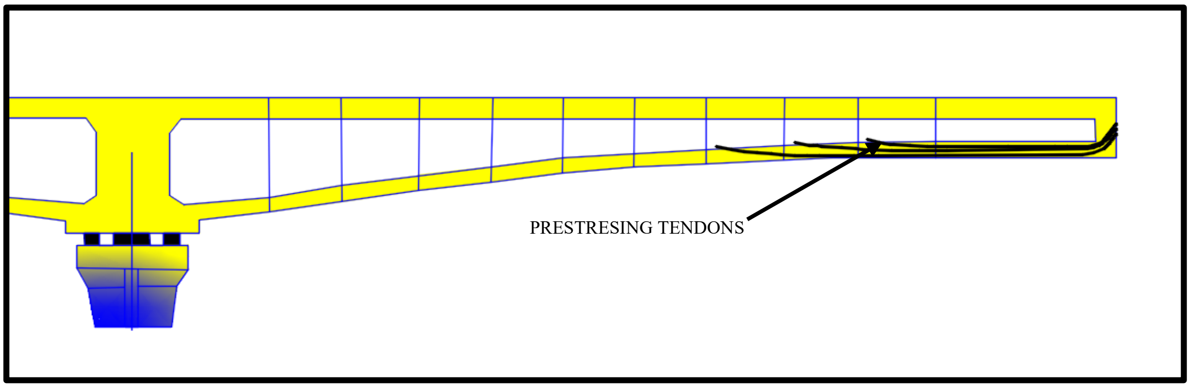

Figure 48: Side span continuity tendons

Data & Parameters:

- M_{DL} **(Dead Load Moment):** $+14711.18225$ kN.m at 11 m (Moment due to self-weight after continuity).

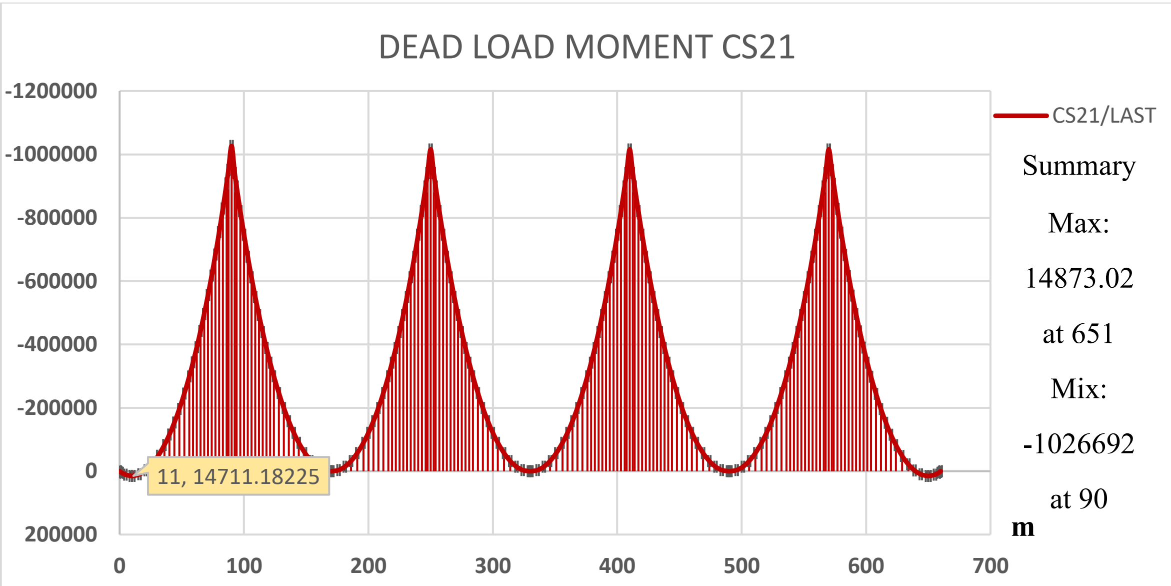

Figure 49: Dead Load Moment Diagram

- M_{iso} **(Tendon Primary Moment):** -66643.64477 kN.m at 11 m (Moment due to long-term concrete deformation).

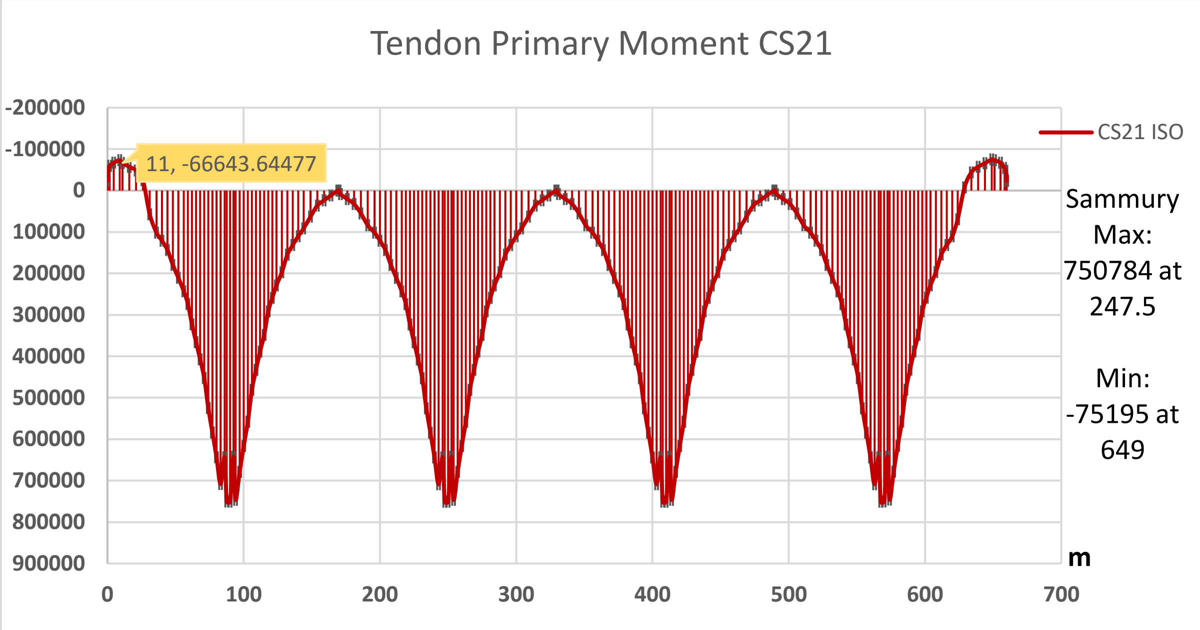

Figure 50: Tendon Primary Load Moment Diagram

- M_{hyp} **(Tendon Secondary Moment):** +3129.7639 kN.m at 11 m (Moment due to long-term concrete deformation).

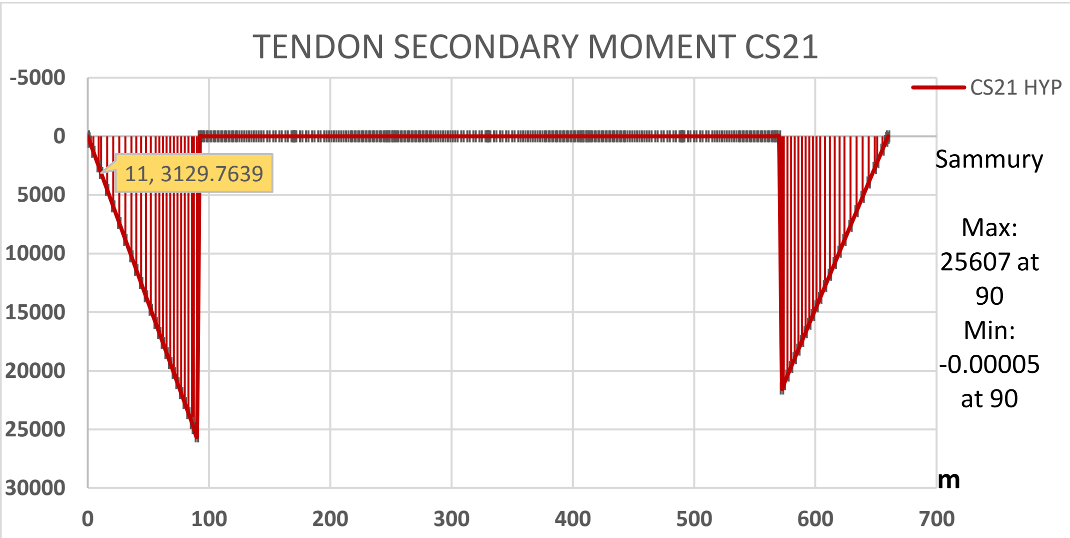

Figure 51: Tendon Secondary Load Moment Diagram

3.1. Geometric & Cross-Sectional Properties:

The side-span box girder cross-sectional dimensions and properties were verified from the finite element model (FEM). To preserve exact dimensional compatible with the cross-sectional second moment of area (I= 42.48 m^4), the physical concrete cross-sectional area (A_c) is established as 11.496 m2.

Table 18: Geometric and Sectional Properties of the Box Girder Case Study

Parameter descriptions	Symbol	Value	Unit
Second Moment of Area (Cross-Section)	I	42.48	m^4
Distance from Neutral Axis to Extreme Fiber	V_{bot}	2.96	m
Section Modulus (Bottom Fiber)	Z_b	14.35	m^3
Concrete Cross-Sectional Area	A_c	11.496	m^2
Tendon Eccentricity at Side-Span	e_c	2.75	m

3.2. Numerical Verification of Bottom Continuity Tendons:

In terms of designing structural continuity tendons at the bottom, the initial structure included a number of tendons of type 27T15, totaling to 6 tendons, during construction process.

The safety of such a structure had to be assessed through analysis using MIDAS Civil software. It was found that the capacity provided by 6 tendons chosen met all the necessary conditions, preventing secondary hyperstatic effects within the acceptable structural limit. which showed in the diagram follows:

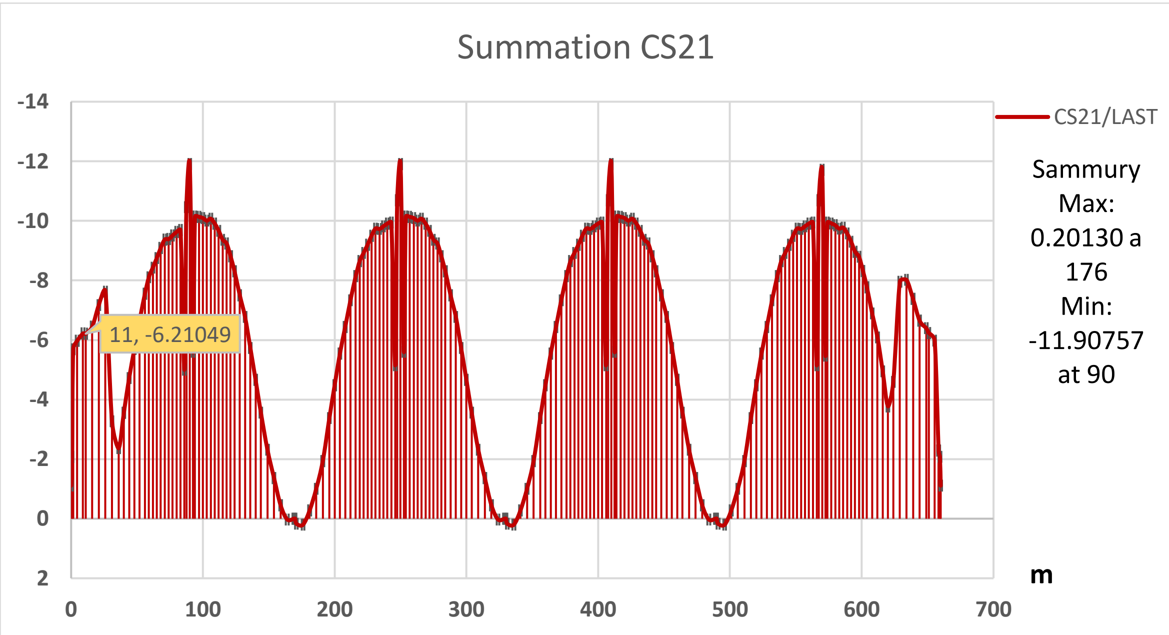

Figure 52: Summation Diagram

It is clear that the constraints have been verified, so the selection of the number and type of bottom continuity tendons has been validated.

- The designed bottom continuity tendons 6×27T15 are satisfied the tension stress

IV. Analysis and Calculation of the Internal Prestressing Employed During the Closure Segment Construction:

After determining the bottom continuity tendons of the cast-in-place segment on the falsework, the internal continuity prestress of the in-span closure between P1 and P2 can be calculated.

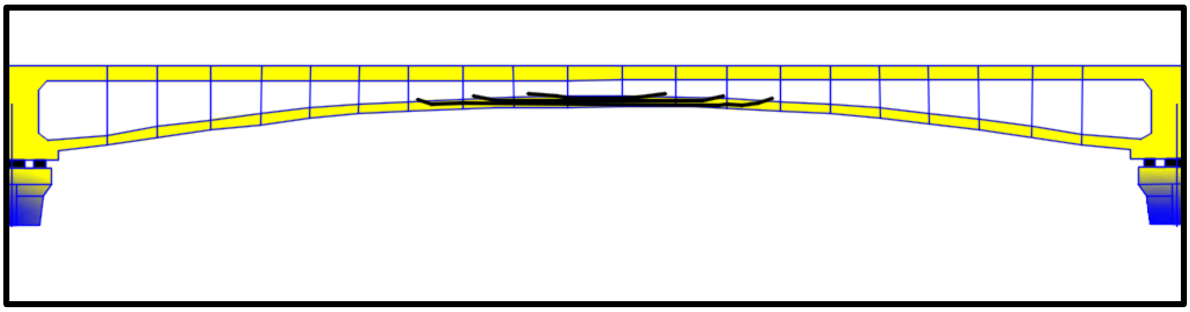

***Figure 30:** Second-Span continuity tendons*

4.1 Determination of Moments Induced by the Applied Effects on the Closure Segment:

- **Overview of Loading Considerations:**

The bottom continuity tendons, also known as continuity tendons, in the P1-P2 segment are required to withstand the following loading effects:

- Self-weight of the closure segment/key segment
- Removal of the form traveler/launching gantry
- Thermal gradient effects

To find out the required number of prestressing tendons required for the closing(keying) of the center span, each construction stage in the closing process needs to be analyzed separately, along with the internal forces and the corresponding bending moments applied to the structure in each stage.

➢ <u>Effect Of Dead Load:</u>

- M_{DL} **(Dead Load Moment):** 8.561 × 10^9 kN.m at key segment (Moment due to self-weight after continuity).

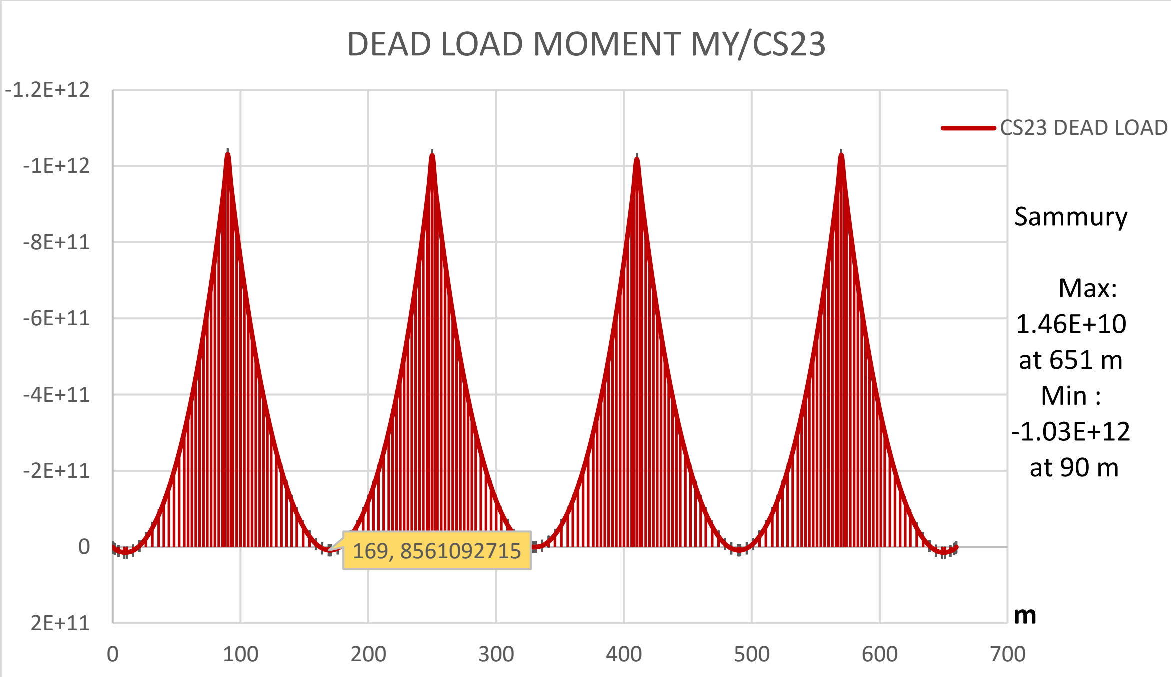

***Figure 53:* Dead Load Moment Diagram**

➢ Effect of Thermal Gradient:

a. General Considerations:

During the closure phase, the structure is subjected to non-uniform temperatures across the depth of the cross-section. This non-uniformity in temperature is termed the thermal gradient and results in internal stresses and secondary moments in the statically indeterminate structure.

b. Design Thermal Gradient:

A linear thermal gradient is applied across the depth of the section, between the top and the bottom fiber of the box girder section, with ΔT = +10°C. This is in accordance with the design code and is due to the differing amounts of solar radiation received by the upper slab and the soffit of the lower slab.

The thermal gradient generates:

- Self-equilibrating stresses, which occur in statically determinate and indeterminate structures; and
- Secondary hyperstatic moments, which occur in statically indeterminate structures due to the restraint against free thermal expansion at the supports.

c. **Structural Behavior:**

At this stage, the concrete has cured in the closure segment, and the structure is in a state of continuity. Hence, the structure is in the statically indeterminate state, and the effects of the thermal gradient must be taken into account in the design of the continuity tendons.

- M_{tg}:8.216×10^4 KN.m (Effect of thermal Gradient Moment)

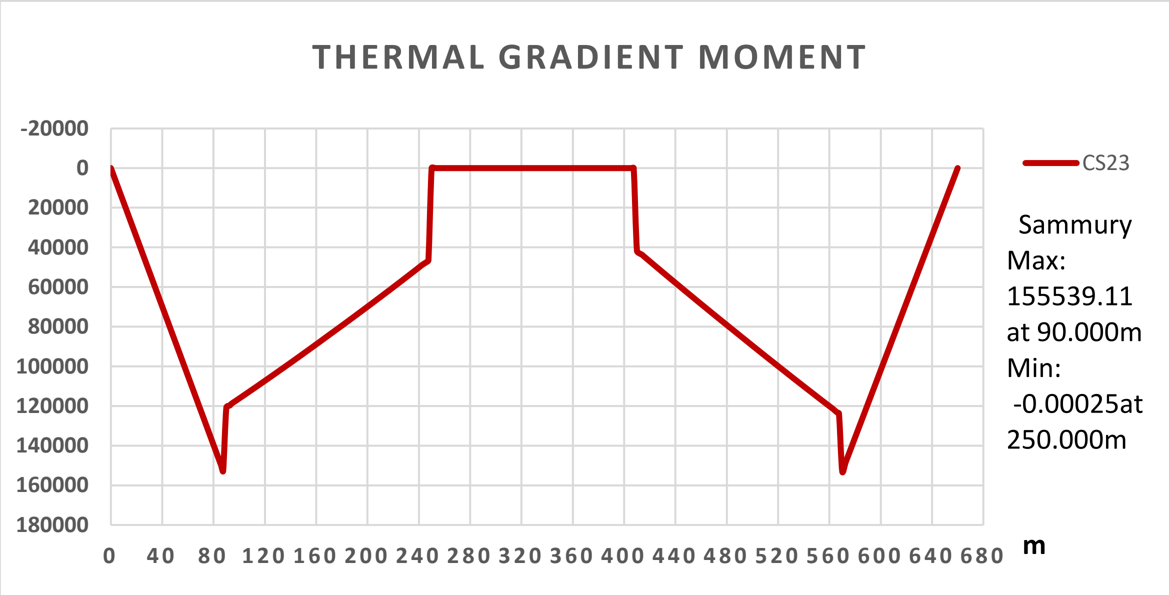

Figure 54: Thermal Gradient Moment Diagram

➢ **Tendon Primary (Isostatic) Moment:**

The primary prestress moment is the direct bending effect produced by the tendon eccentricity below the centroid. It would exist even in a simply supported (statically determinate) structure:

- Miso: -1.157×10^4 KN.m at the key segment

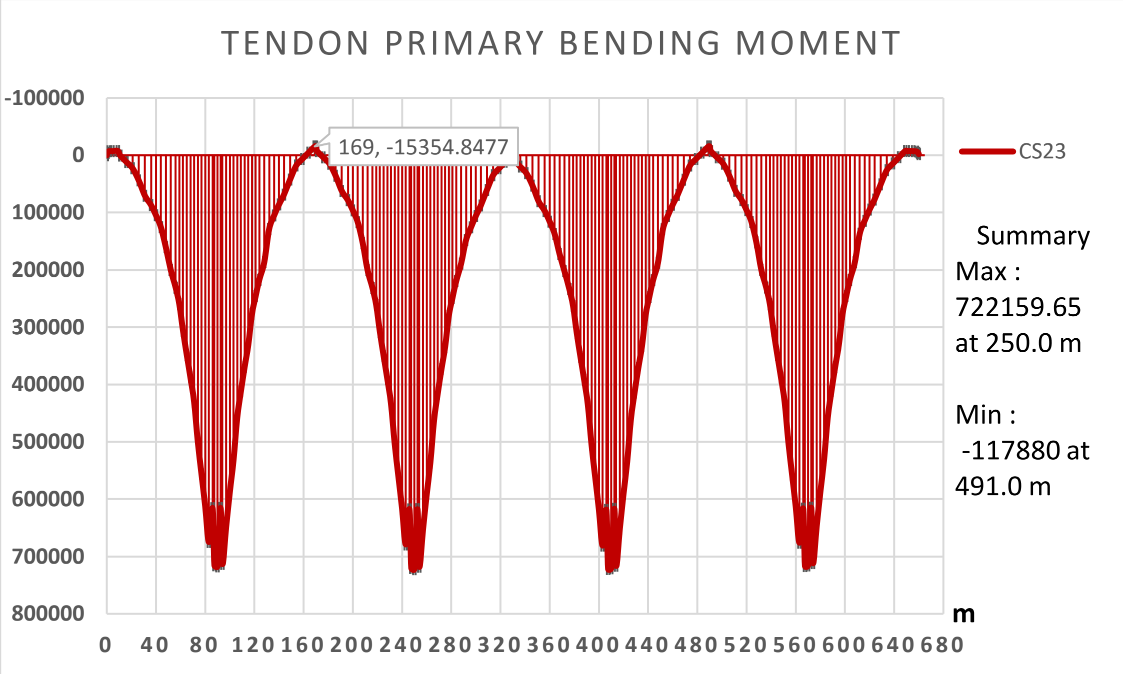

Figure 55: ***Tendon Primary Bending Moment Diagram***

➢ Tendon Secondary (Hyperstatic) Moment:

The secondary (hyperstatic) moment arises from the support reactions induced by the prestress in the continuous structure. It partially reduces the beneficial hogging effect of the primary moment:

- M_{hyp}:6.311×10^4 KN.m (Effect of Tendon Secondary Bending Moment)

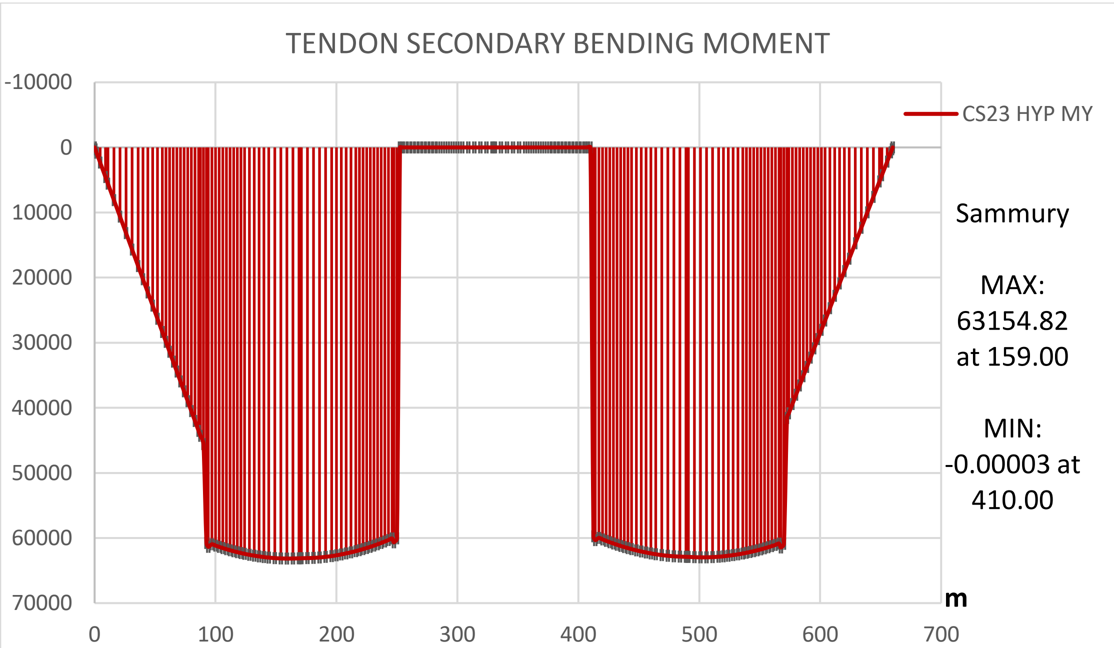

Figure 56: *Tendon Secondary Bending Moment*

4.2. Geometric & Cross-Sectional Properties:

The side-span box girder cross-sectional dimensions and properties were verified from the finite element model (FEM). To preserve exact dimensional compatible with the cross-sectional second moment of area (I= 42.48 m^4), the physical concrete cross-sectional area (A_c) is established as 11.496 m2.

Table 19: Cross-Section Properties

Parameter descriptions	Symbol	Value	Unit
Second Moment of Area (Cross-Section)	I	42.48	m^4
Distance from Neutral Axis to Extreme Fiber	V_{bot}	2.96	m
Section Modulus (Bottom Fiber)	Z_b	14.35	m^3
Concrete Cross-Sectional Area	A_c	11.496	m^2
Tendon Eccentricity at Side-Span	e_c	2.75	m

4.3. Numerical Verification of Bottom Continuity Tendons:

The design of the bottom continuity tendons involved an initial design of 10 tendons of type 27T15. Safety analysis for the design of 10 tendons was performed using MIDAS Civil software to verify whether the capacity provided by these 10 tendons met the required criteria, thus avoiding any hyperstatic effects within structural limits, as shown below:

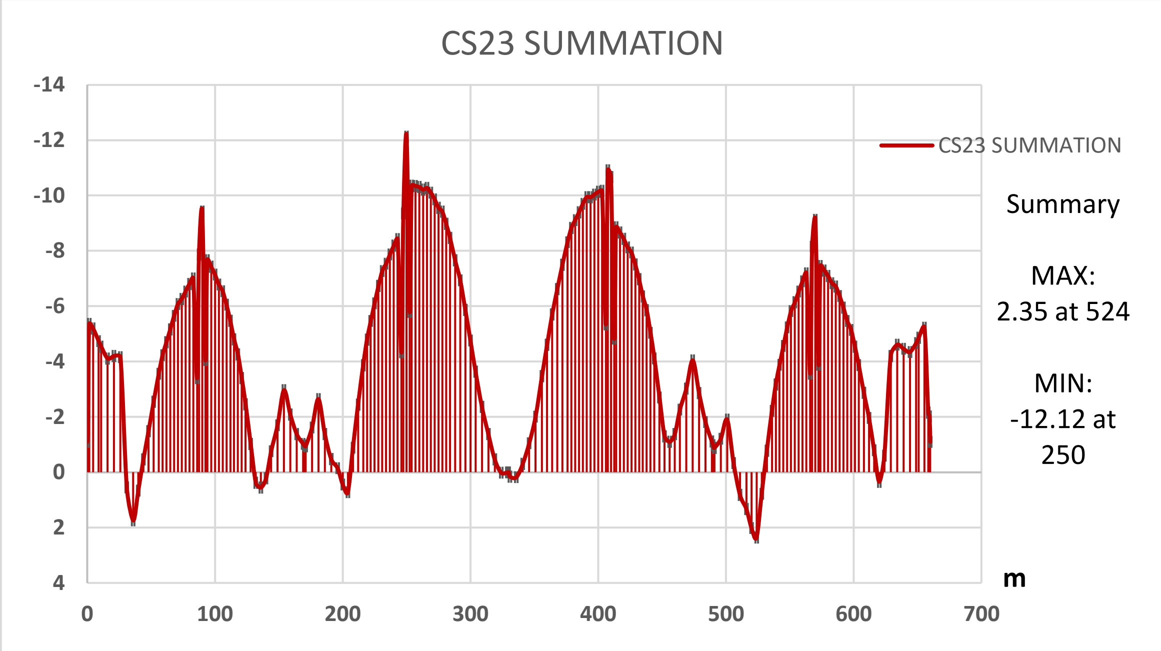

Figure 57: *Stress Summation Diagram*

It is clear that the constraints have been verified, so the selection of the number and type of bottom continuity tendons has been validated.

- The designed bottom continuity tendons 10×27T15 are satisfied the tension stress

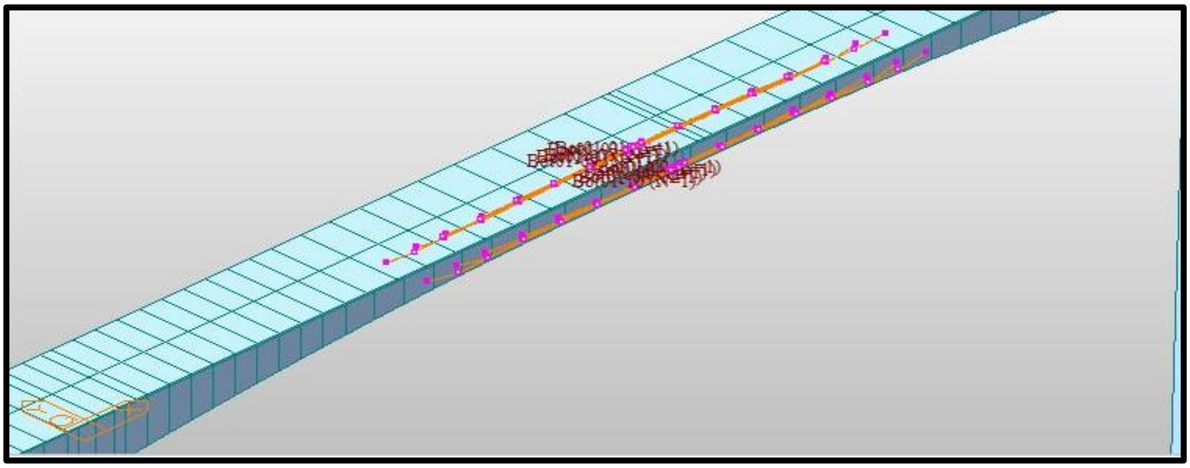

Figure 58: *Continuity Tendons Design of the Second-Span*

V. Analysis and Evaluation of the Internal Prestressing Adopted for the Construction of the Closure Span (P2-P3):

After calculating the bottom continuity tendons in the cast-in-place Second-Span, the internal continuity prestress in the closure span existing between P2 and P3 can be determined.

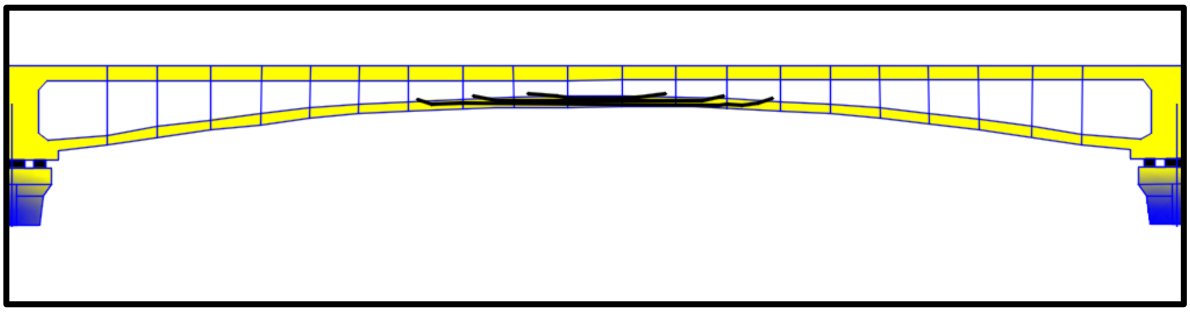

Figure 59: Mid-Span continuity tendons

5.1. Estimation of the Moments Acting on the Closure Segment Caused by Loading Effects Introduction to the Analysis of the Load Effects:

On the P2–P3 section, the continuity tendons must take into account the following load effects:

- Weight of the closure segment and the key segment
- Decommissioning of the form traveler and launching system

For estimating the number of prestressing tendons required to close (key) the central span, the analysis of each construction phase of the closure should be conducted individually, taking into account the internal forces and the moment effect produced by them on the bridge.

➢ Effect Of Dead Load:

- M_{DL} **(Dead Load Moment):** 8.674×10^3 kN.m at key segment 330 m (Moment due to self-weight after continuity).

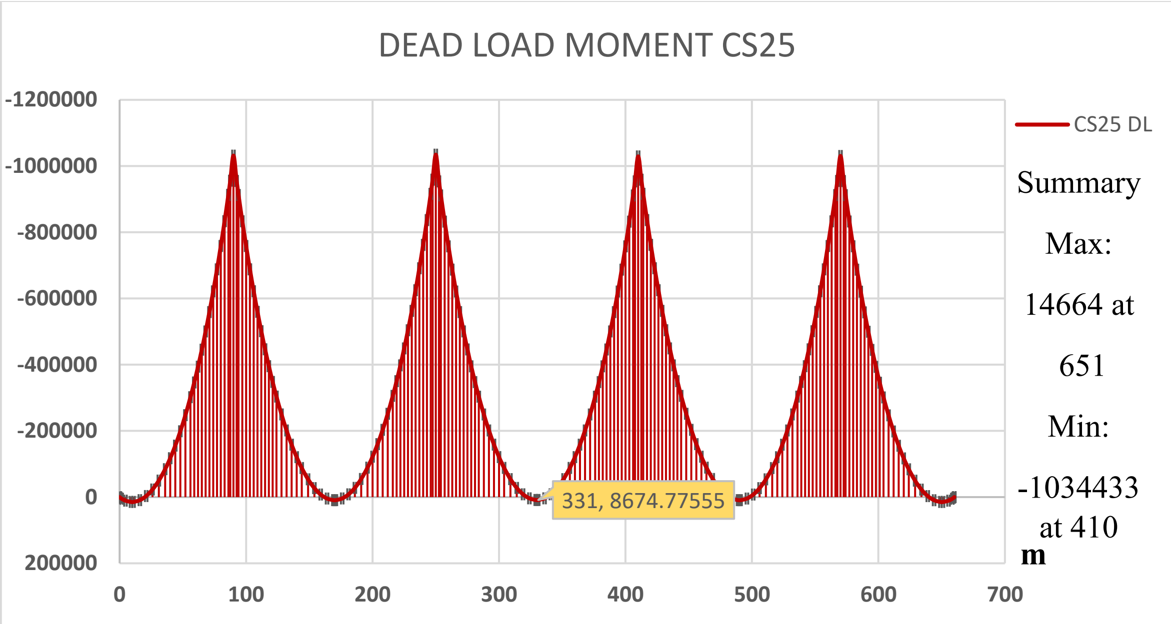

***Figure 60:* Dead Load Diagram**

- M_{TG} **(Thermal Gradient Moment):** 8.0862×10^4 KN.m at key segment 330m

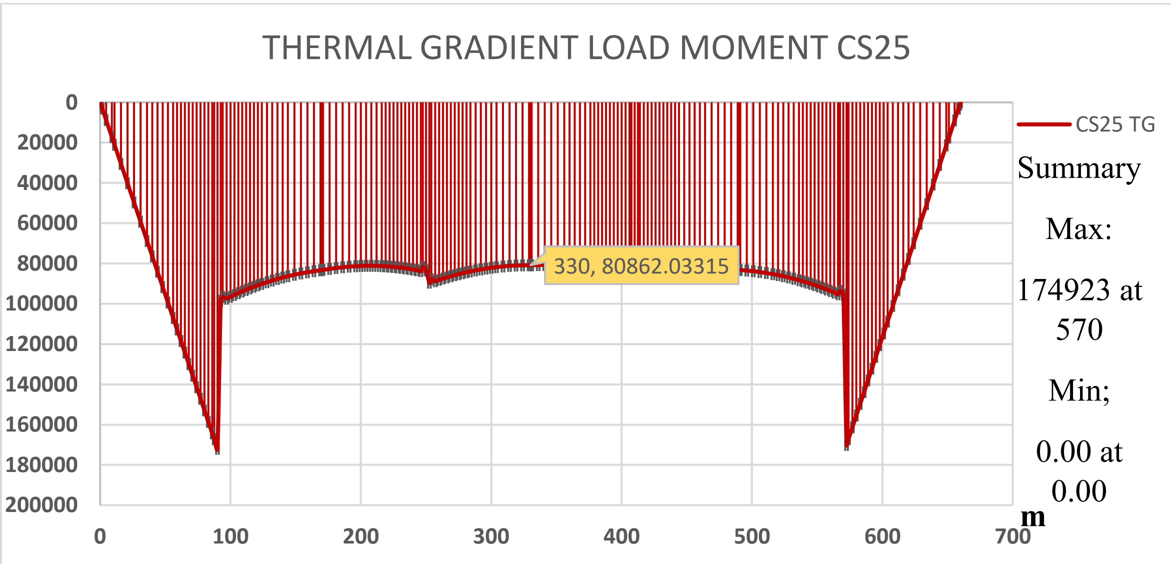

***Figure 61:* Thermal Gradient Load Diagram**

- M_{iso} **(Tendon Primary Moment):** -1.09656×10⁵ KN.m at key segment 330m

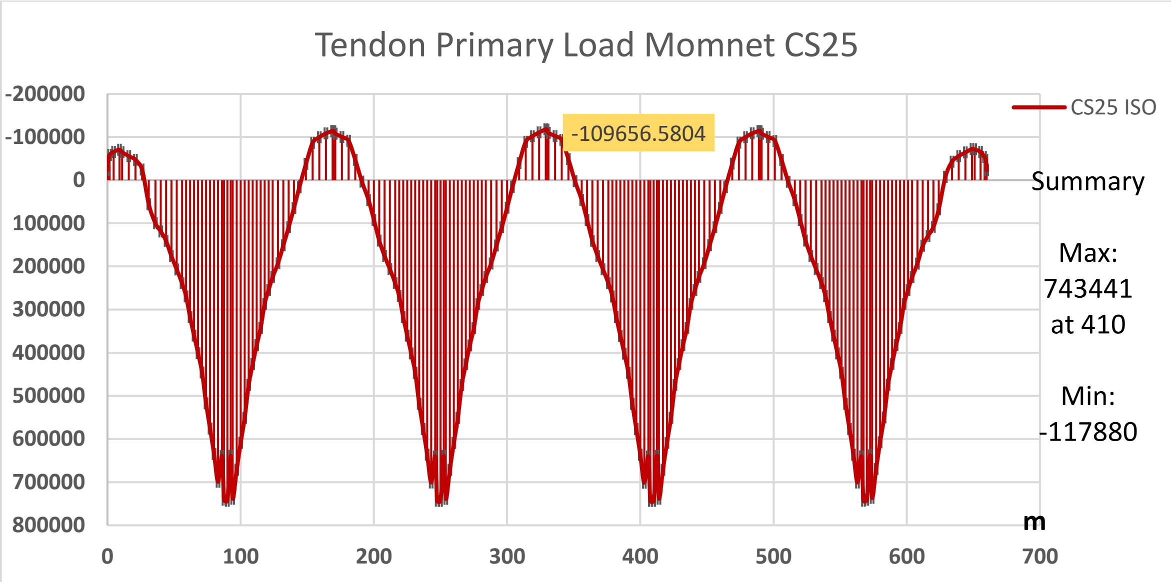

Figure 62: Tendon Primary Moment Diagram

- M_{hyp} **(Tendon Secondary Moment):** 6.2474×10⁴ KN.m at key segment 330m

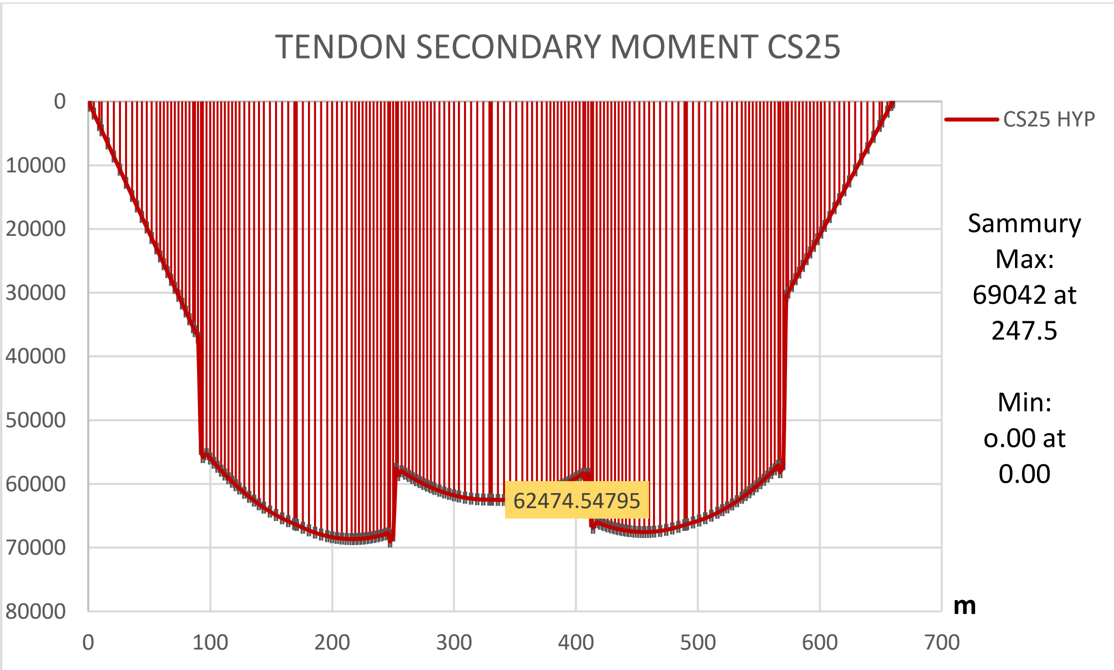

Figure 63: Tendon Secondary Moment Diagram

4.2. Geometric & Cross-Sectional Properties:

The side-span box girder cross-sectional dimensions and properties were verified from the finite element model (FEM). To preserve exact dimensional compatible with the cross-sectional second moment of area (I= 42.48 m^4), the physical concrete cross-sectional area (A_c) is established as 11.496 m2.

Table 20: Cross-Section Properties

Parameter descriptions	Symbol	Value	Unit
Second Moment of Area (Cross-Section)	I	42.48	m^4
Distance from Neutral Axis to Extreme Fiber	V_{bot}	2.96	M
Section Modulus (Bottom Fiber)	Z_b	14.35	m^3
Concrete Cross-Sectional Area	A_c	11.496	m^2
Tendon Eccentricity at Side-Span	e_c	2.75	M

4.3. Numerical Verification of Bottom Continuity Tendons:

The tendons used in the structural continuity at the foundation base were first made up of 10×27T15 tendons when building. Structural safety for the configuration was determined by carrying out a test using the MIDAS Civil software. From the test results, the strength provided by the tendons met all the necessary criteria, thus avoiding any secondary hyperstatic effect on the structure, as shown in the figure below.

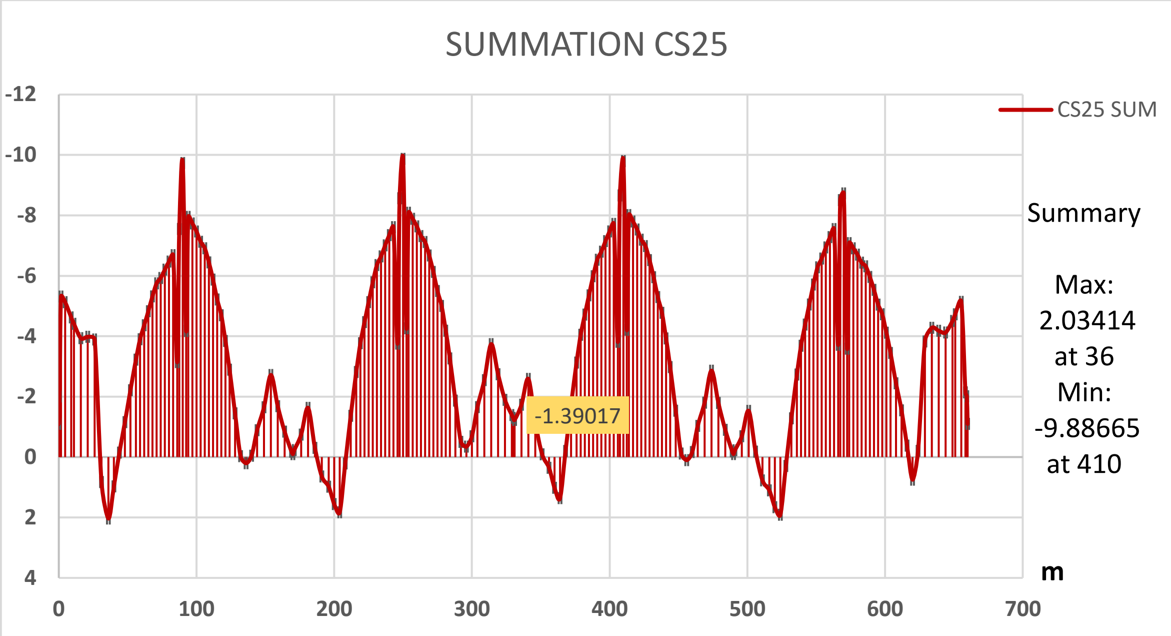

***Figure 64:* Summation Diagram**

It is clear that the constraints have been verified, so the selection of the number and type of bottom continuity tendons has been validated.

- The designed bottom continuity tendons 10×27T15 are satisfied the tension stress

Remark on Structural Symmetry: Taking advantage of the symmetrical configuration of the cantilever box girder bridge, the analytical verification of the tendon layout was conducted solely for the left half of the structure. Due to this bilateral symmetry, the computed tendon requirements and structural responses are identically mirrored and applicable to the right half.

VI. Geometric Layout and Spatial Configuration of Bottom Continuity Tendons:

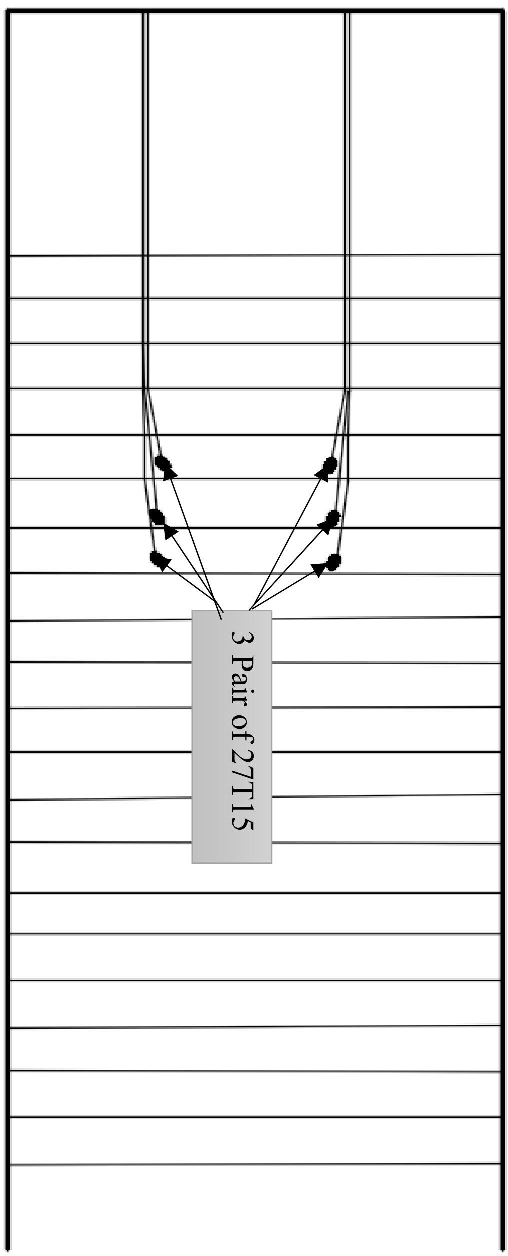

Figure 65: ***Bottom Continuity Tendons -Plan View-Side-Span***

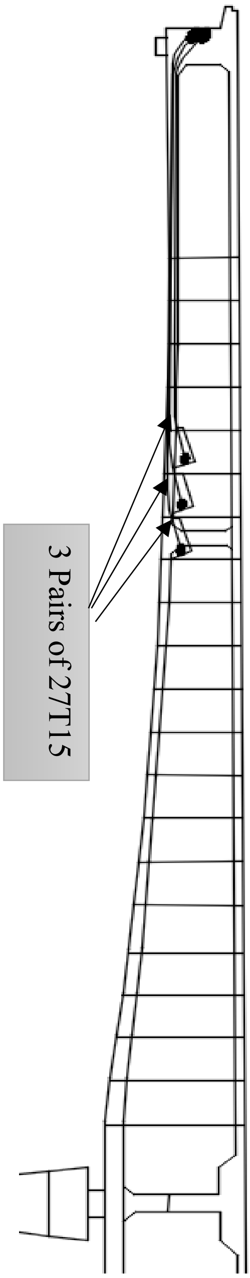

Figure 66: ***Bottom Continuity Tendons -Longitudinal Profile- Side-Span***

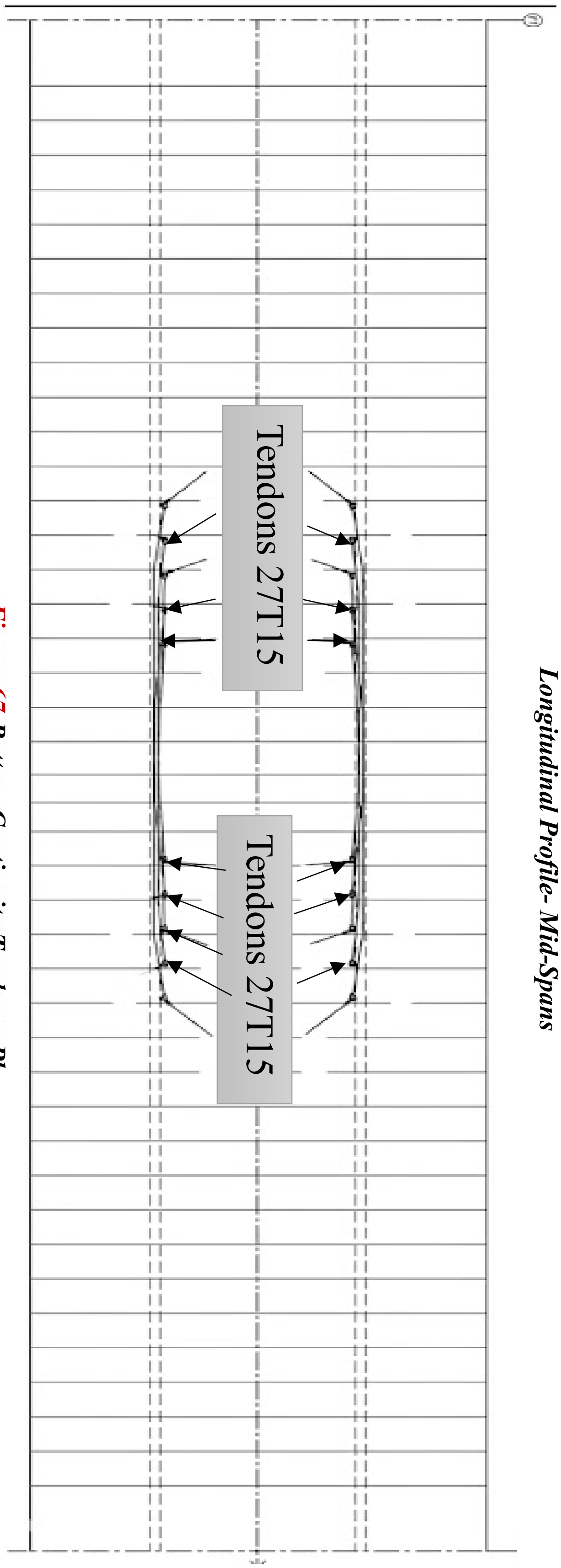

Figure 67: ***Bottom Continuity Tendons -Plan View-Mid-Spans***

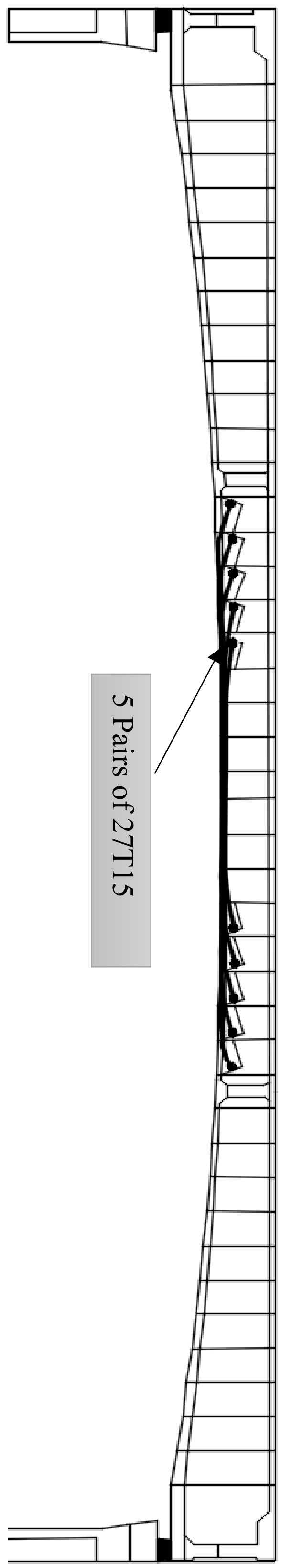

Figure 68: ***Bottom Continuity Tendons - Longitudinal Profile- Mid-Spans***

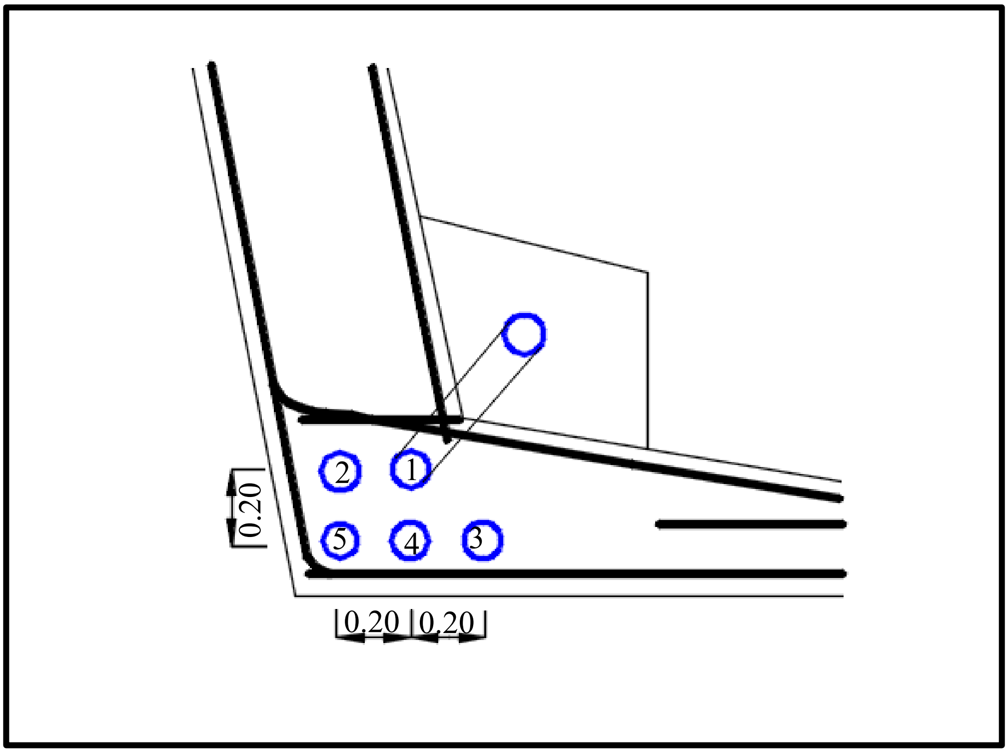

Figure 69: ***Bottom Continuity Tendons*** *cable layout details – intermediate spans*

CHAPTER E:
DESIGN OF EXTERNAL TENDONS WITHIN SERVICE STAGE

I. Introduction:

In this chapter, we will examine the longitudinal flexural properties of the bridge, which represent an essential component of structural validation within the viaduct as a whole.

The organization of this chapter is structured as follows:

1. Actions and Internal Forces Description:

Firstly, all actions and internal forces affecting the structure during its operation must be identified and quantified, including those caused by external prestressed tendons. The respective load combinations are determined according to the SLS rules of EN 1990 and EN 1992-2.

Modelling a model is developed within the MIDAS CIVIL finite element modelling program with special focus paid to:

- The geometry of the external prestressing tendons
- The determination of the most unfavorable cases in terms of load from traffic and exploitation

a. Normal Stress Validation at SLS:

The validation of normal stresses at the top and bottom of the beam is performed at the most critical sections of the deck, adhering to allowable values prescribed by EN 1992-2.

b. Ultimate Limit State (ULS) Validation:

Validation is carried out in relation to the longitudinal flexural properties of the bridge in accordance with the ULS requirements.

c. Shear Stress and Shear Force Validation:

Lastly, tangential stresses and shear forces at the supports are validated to confirm the compliance of the structure with the shear resistance rules according to EN 1992-2.

II. Action acting on bridge:

In operation, bridges are exposed to various actions having specific properties that need to be defined clearly and synthesized. According to EN 1990:2002, actions are classified based on their time dependence into three types [14, p. Clause 4.1.1]:

- Actions of permanent nature (G): these are actions that either remain constant or change monotonically over time.
- Actions of variable nature (Q): these are actions that have temporal variability.
- Action of accidental nature (A) rare phenomena with significant impact.

2.1. Permanent Actions:

Permanent actions can be described as actions which are anticipated to be acting during the entire duration of a specified design situation, where the change in magnitude over time is considered insignificant or where the magnitude of the action changes monotonically until it achieves a certain limiting value. There are three types:

2.1.1. Dead Load (DL):

Dead load (DL) refers to the weight of all the structural materials making up the structural system. In a concrete box girder bridge, the dead load consists of the deck slab, webs, bottom flange, cross beams, and structural concrete barrier wall. The unit weight of structural materials according to EN 1991-1-1:2002 is listed below [15, p. Table A.1 & A.4]:

- **Normal concrete:** $\gamma = 25$ kN/m³
- **Structural steel:** $\gamma = 77.0$ -78.5 kN/m³
- **Light concrete:** $\gamma = 9$ -20 kN/m³ depending on the density class

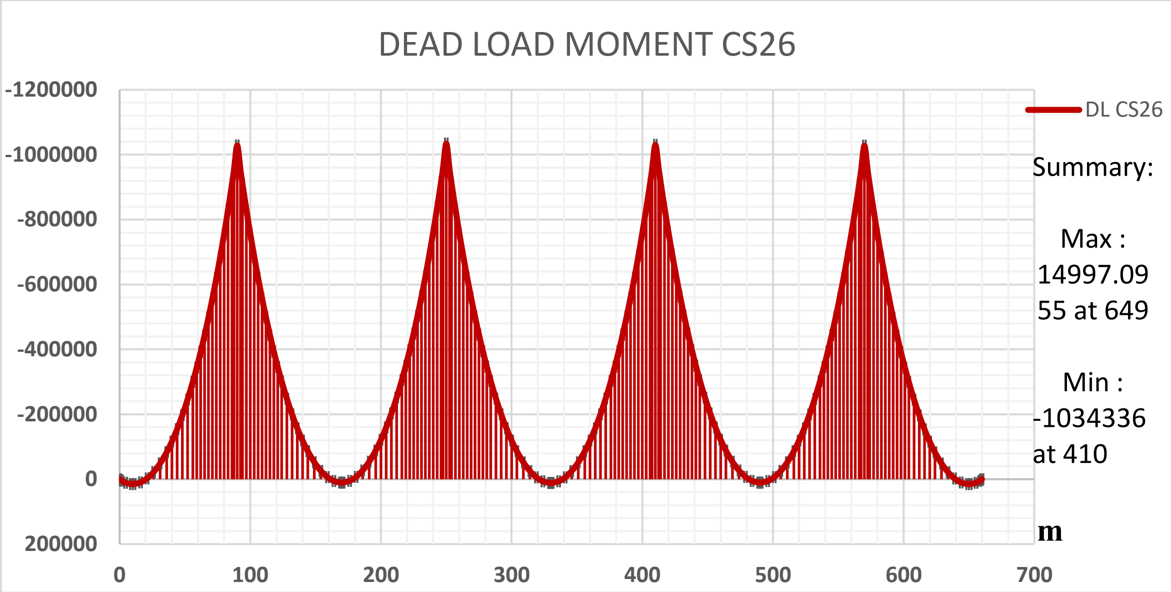

Figure 70: ***Dead Load Moment Diagram of Construction Stage 26***

2.1.2. Superstructure Load (SDL):

Superimposed dead load (SDL) are the dead load of secondary (non-structural) elements permanently attached to the bridge deck. Although constant after construction they are treated separately as they may be replaced during the service life of the bridge. In standard practice and in EN 1991-1-1:2002 and the Designer's Guide the SDL components for a road bridge are specified as [16, pp. Annex A & Section 5.2 (EN 1991-1-1:2002)]:

- Bituminous layer: γ= 23.5 kN/m^3
- Waterproofing membrane: γ= 23.5 kN/m^3
- Kerbs: γ= 26 kN/m^3
- Safety barriers/ guard rail: mass per unit length from supplier data
- Walk side: γ= 23.5 kN/m^3
- Street lighting poles and utility pipes: as per detailed design

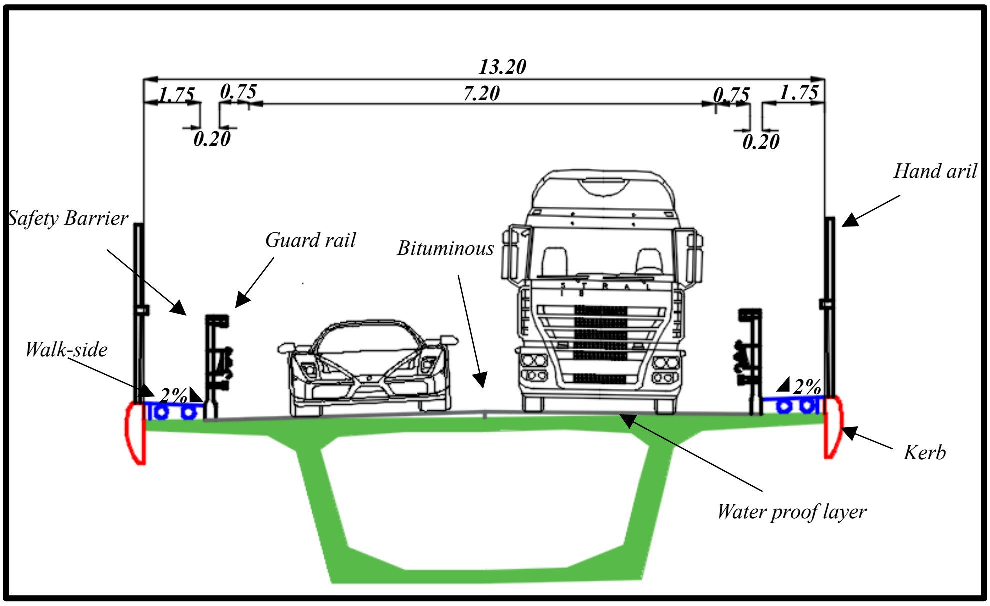

Figure 71: Superstructure Equipment

Table 21: Detailed Calculation of Superimposed Dead Loads (SDL)

Equipment	Unit Weight (kN/m^3)	Width (m)	Thick (cm)	Linear Load (kN/m)	Coef. Amplif. Ψ	Coef. Reduction	Nom Load (kN/m)	Max Load (kN/m)	Min Load (kN/m)
Bituminous layer	23.5	8.70	8	16.356	1.4	0.8	16.35	22.90	13.08
Waterproofing layer	23.5	8.70	1	2.0445	1.2	0.8	2.0445	2.45	1.64
Krebs	26.00	-	-	9.15	1.0	1.0	9.15	9.15	9.15
Handrails	-	-	-	0.5	1.0	1.0	0.95	0.95	0.95
Guard rails	26	0.2	90	9.36	1.0	1.0	9.36	9.36	9.36
Walk-side	23.5	4.75 ×2	15	12.34	1.0	1.0	12.34	12.34	12.34
Total SDL (kN/m)	**56.7**	**46.07**							

Result: The characteristic maximum SDL = 56.7 kN/m

Minimum SDL = 46.07 kN/m for the given bridge section.

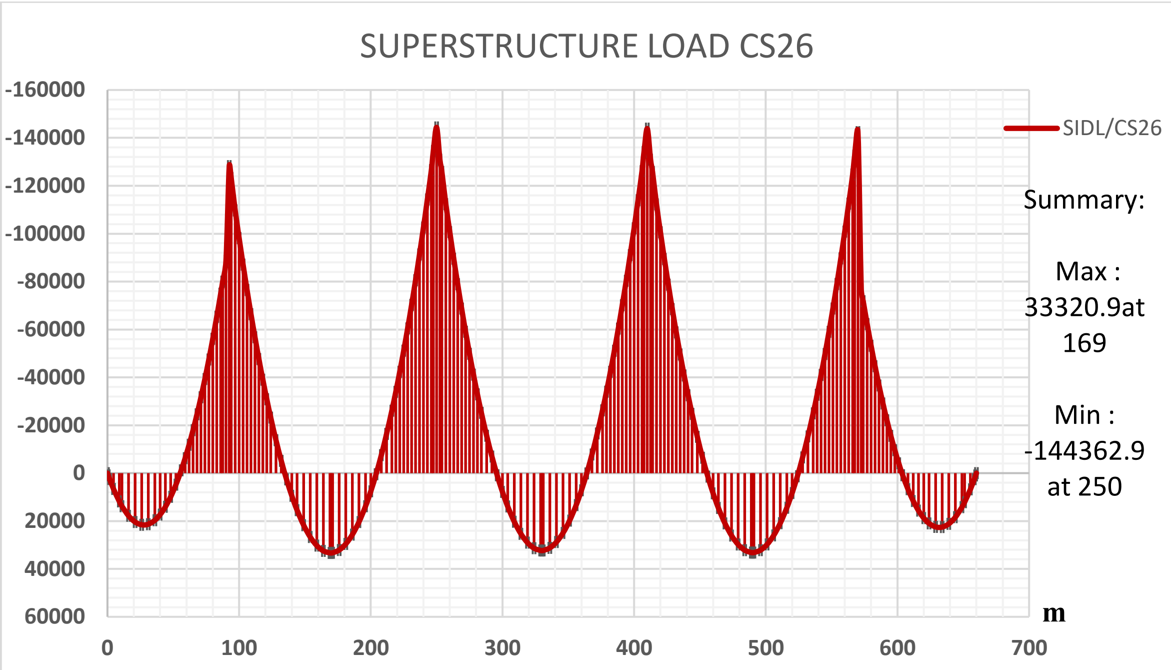

Figure 72: Superstructure Load Moment at CS26

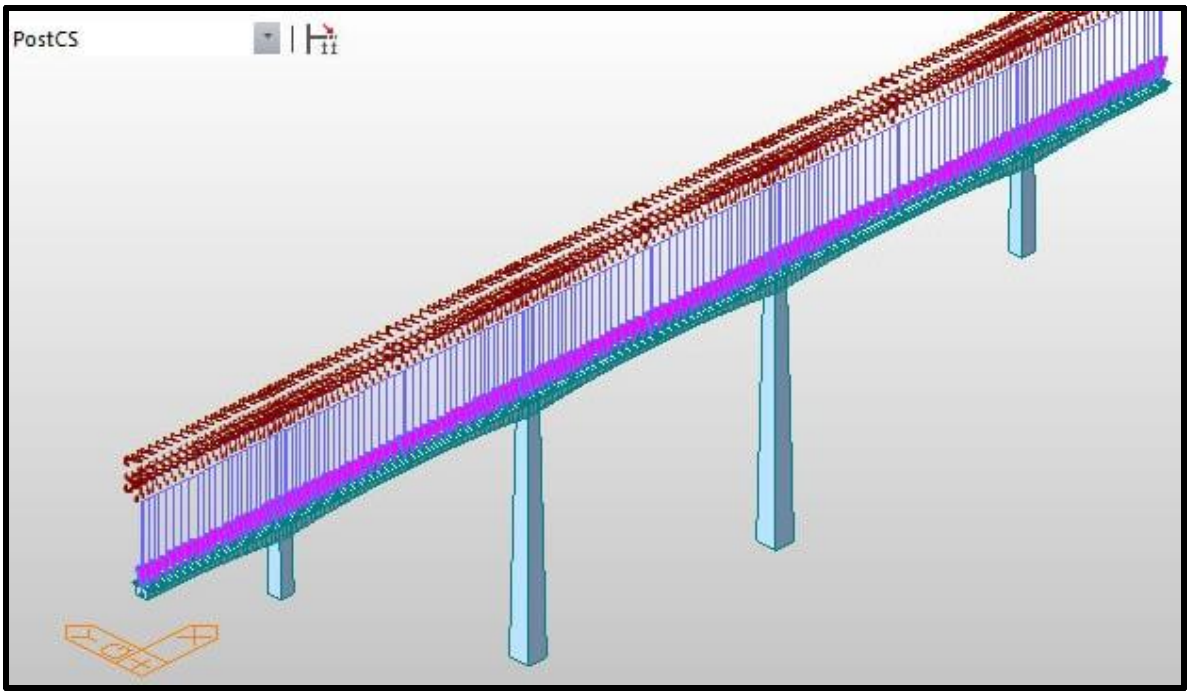

Figure 73: SIDL-Super imposed dead load=-56.70 kN/m

2.1.3. Permanent Action Effects:

Besides gravitation-induced permanent actions, other permanent action effects have to be considered in the design of service stage [17, p. Chapter 5.10 & 3.1.4] [18]:

- Prestress action (P): Loads equivalent to those caused by post-tensioning tendons or prestressed strands, as defined in EN 1992-1-1:2004, Chapter 5.10. The prestress action can be seen as a load but also as a structural characteristic.
- Imposed deformation: Creep and shrinkage of concrete that result in deformation and curvature over time as described in EN 1992-1-1:2004, Chapter 3.1.4.
- Differential settlement: Inequality in foundation settlement leading to increased bending moment and shear forces; determined based on EN 1997-1:2004 (Eurocode 7).

2.2. Variable Actions:

Variable actions are dynamic or temporary loads that have considerable variations regarding magnitude, location, or direction over time, unlike fixed actions. As described in the EN 1990:2002 code, in Clause 4.1.1, the examples of variable actions include [19]:

- Traffic actions (vehicles, trains, pedestrians)
- Wind actions (EN 1991-1-4) [20]
- Snow actions (EN 1991-1-3) [21]
- Thermal actions (EN 1991-1-5) [22]

Accidental actions: Impact, earthquake actions (EN 1998), braking actions, and many more.

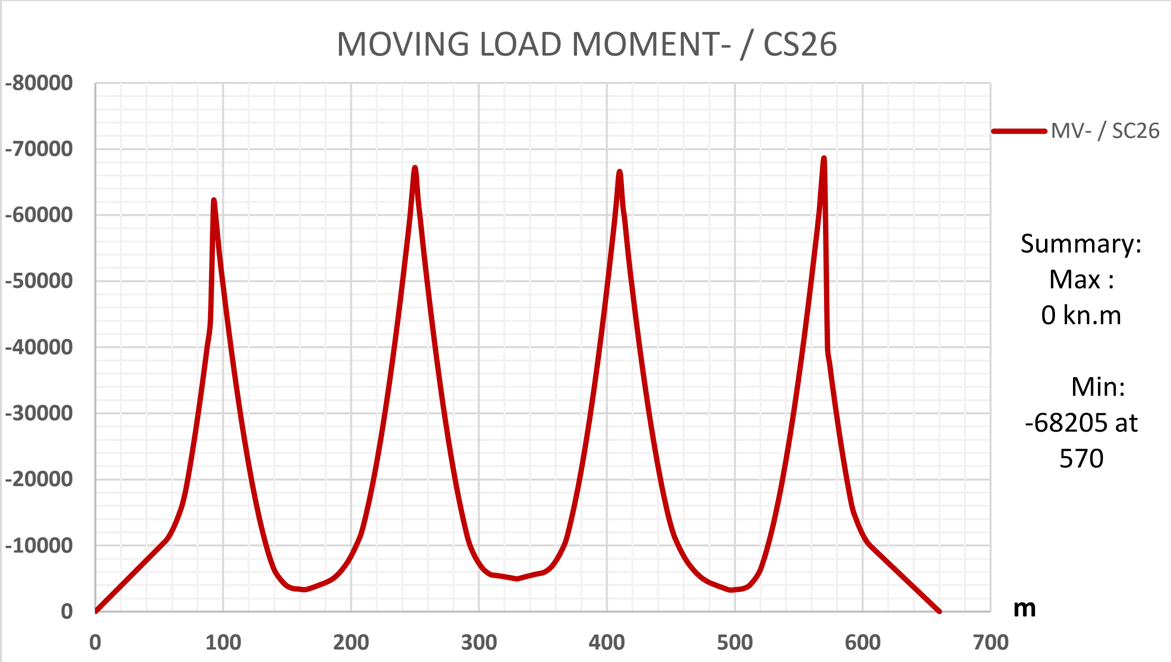

Figure 74: Negative Moving Load Moment at CS26

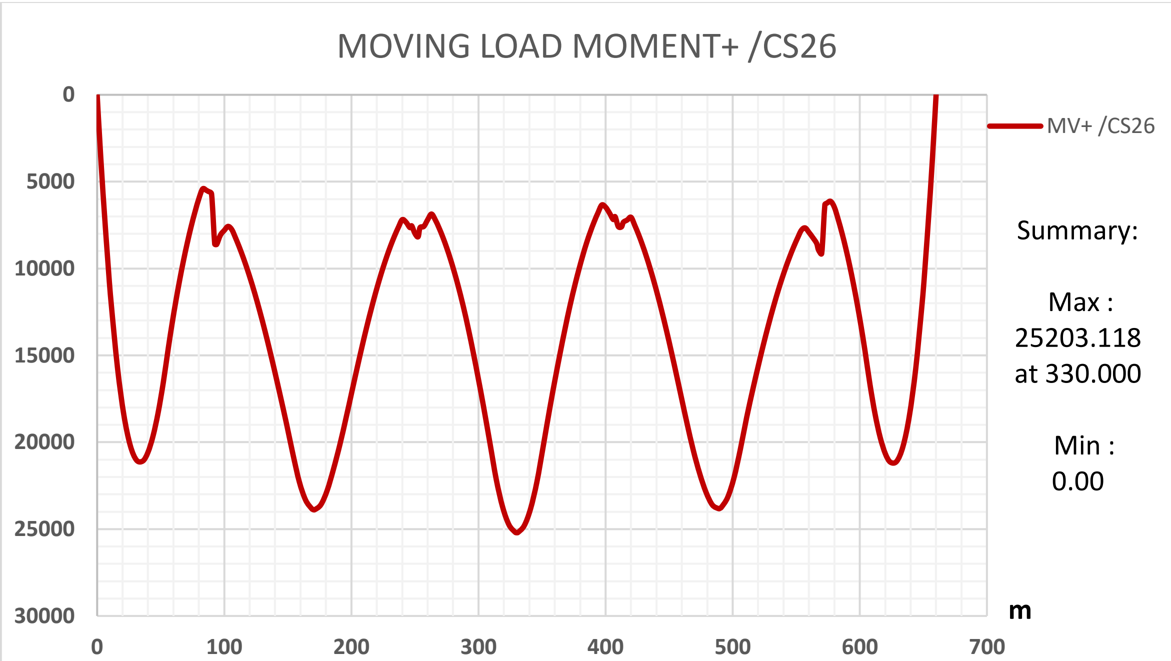

Figure 75: Positive Moving Load Moment at CS26

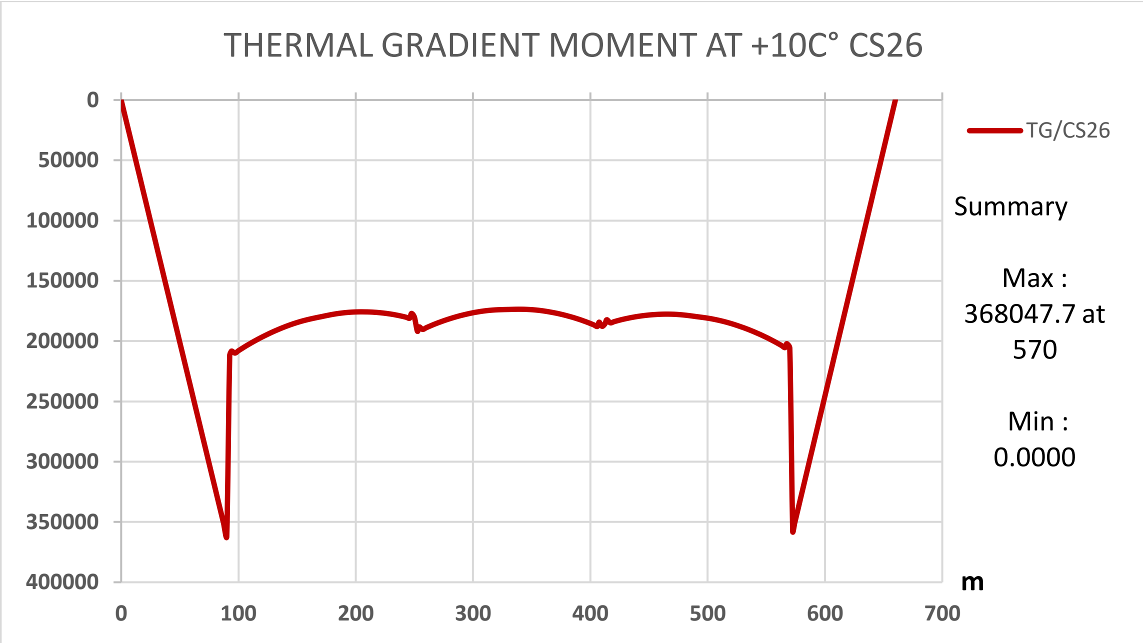

Figure 76:Thermal Gradient Moment at +10C° CS26

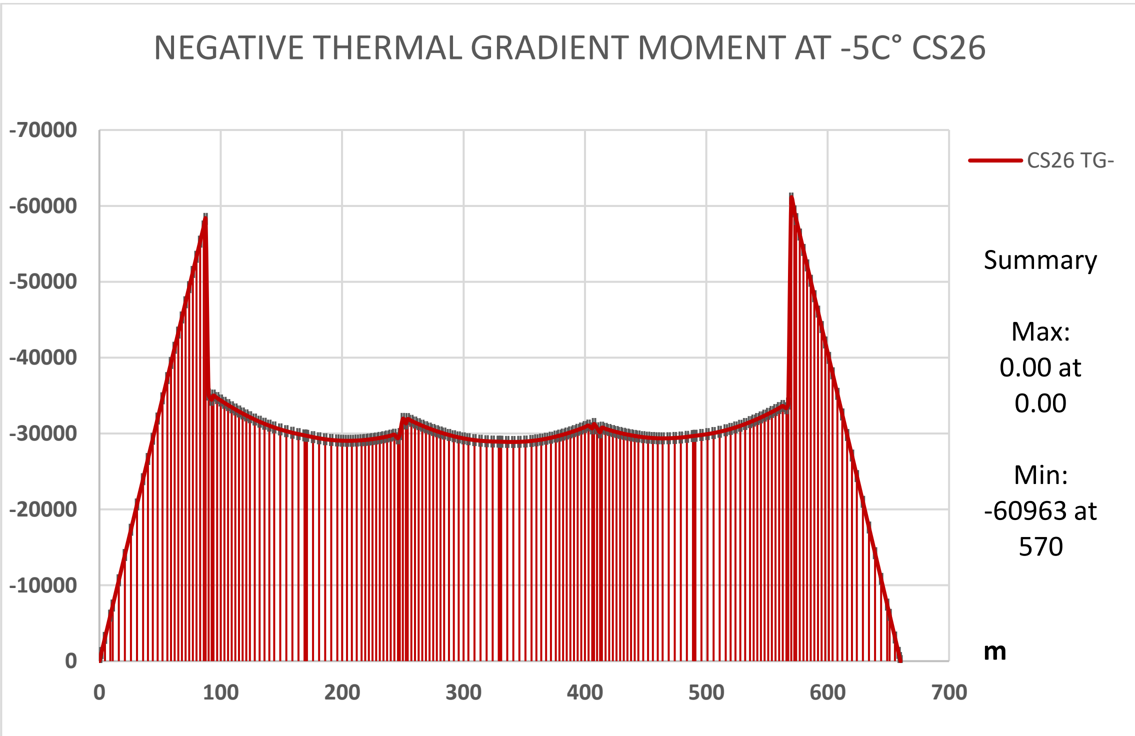

Figure 77: Negative Thermal Gradient Moment at -5C° CS26

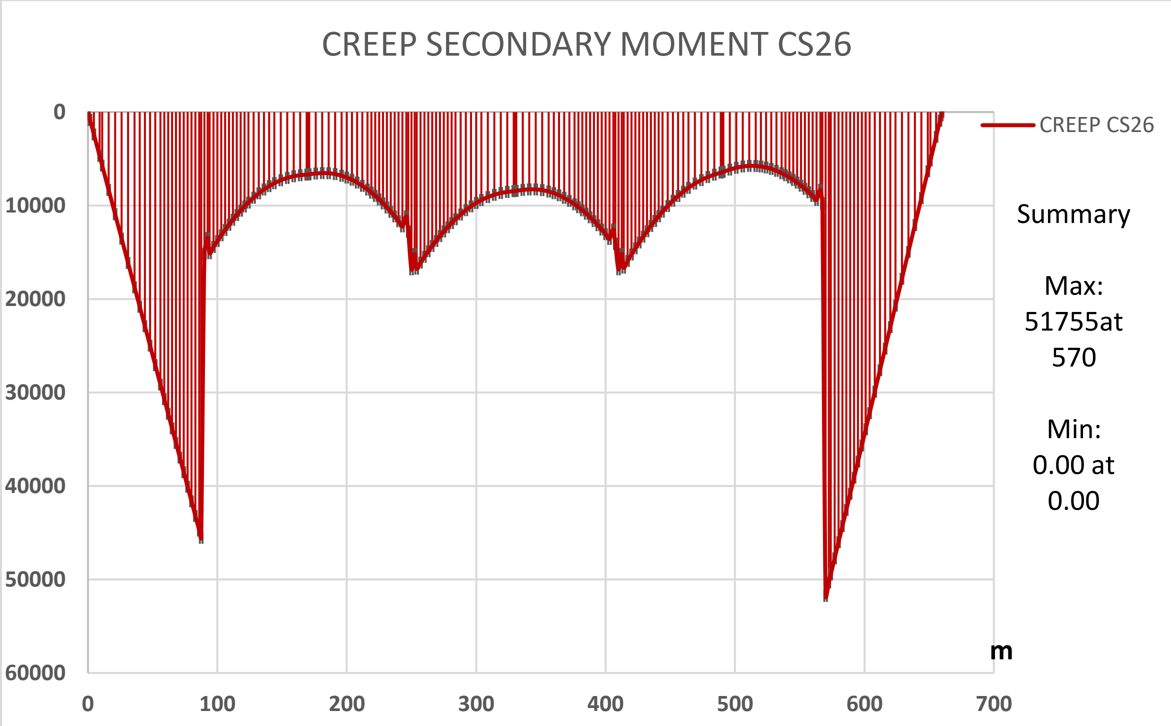

Figure 78: ***Creep Secondary Moment Diagram***

2.2.1. Highway Traffic Loadings (EN 1991-2):

Highway traffic loadings are specified in EN 1991-2:2003. The traffic loadings consist of static vertical loadings and horizontal loadings, which are [23]:

- Static Vertical Loadings: Load Models LM1, LM2, LM3, LM4
- Horizontal Loadings: braking/acceleration loadings, centrifugal loadings, and aerodynamic effects of wind on vehicle

Step 1: Determination of Notional Lane. As per EN 1991-2:2003, Para 4.2.3,

traffic loadings are considered along the carriageway both longitudinally and laterally in the most critical arrangement. Carriageway is divided into notional lanes, the widths of which being 3.0 m. Width of the carriageway w is the distance between kerbs or the width between safety barriers on both sides of the carriageway:

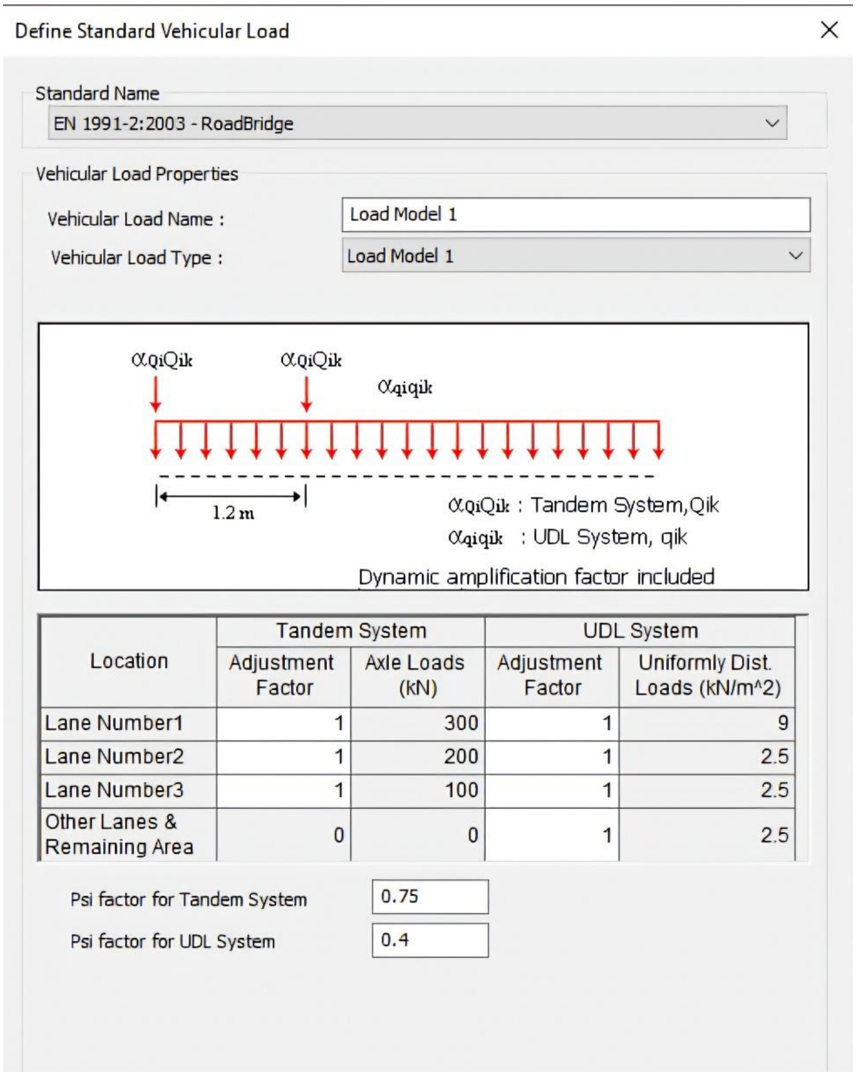

***Figure 79:* Load Model 1 (LMD1) Parameters**

***Table 22:* Determination of Notional Lanes Based on Carriageway Width**

Carriageway width w (m)	Number of notional lanes (n_l)	Width of each lane (m)	Width of remaining area w_r (m)
W<5.4	1	3.0	w-3
5.4≤w<6	2	w/2	0
w>6	Int[w/3]	3.0	w-3×n_l

Application: For the reference bridge, the carriageway width is w = 8.70 m (clear distance between guard rails). Since w ≥ 6 m [24]:

$$n_l = \text{Int}[\frac{w}{3}] = \text{Int}\ [\frac{8.70}{3}] = 2 \text{ notional lanes}$$

$$w_r = w - n_l \times 3 = 8.70 - 2 \times 3 = 2.70 \text{ m (remaining area)}$$

This lane arrangement is illustrated in Figure 4 of the course notes: Notional Lane 1 (3.0 m) + Notional Lane 2 (3.0 m) + Remaining area (2.70 m) = 8.70 m.

2.2.2. Road Bridge Loading Models:

As per EN 1991-2:2003, Section 4.3, there are four loading models that can be defined as follows:

a. Loading Model 1 (LM1):

Double Tandem Axles with Uniformly Distributed Load (UDL) The fundamental loading pattern is one of double tandem axles (TS) together with a uniformly distributed load. The loading pattern is made up of double TS, which is constituted by concentrated loads, along with UDL covering all lanes and remaining regions [25, p. Clause 4.3.2 & Table 4.2].

***Table 23:** Load Model 1 (LM1) Parameters for Notional Lanes*

Location	Tandem axel load Q_{ik} (kN)	UDL load q_{ik} (kN/m^2)	Adjustment factor αQ_i / αq_i
Lane No. 1	300 kN	9.0 kN/m^2	$\alpha Q_1/\alpha q_1$
Lane No. 2	200 kN	2.5 kN/m^2	$\alpha Q_2/\alpha q_2$
Lane No. 3	100 kN	2.5 kN/m^2	$\alpha Q_3/\alpha q_3$
Remaining area	0	2.5 kN/m^2	αq_r

Note: The αQi and αqi coefficients facilitate this adjustment. For situations involving road bridges where there is no particular National Annex, EN 1991-2:2003, Table 4.2 recommends that all values of α be set at 1.0.

III. Stress Check of Bridge Deck Within Service Stage:

3.1. Top fiber stress check of Bridge deck:

- ***Dead Load:***

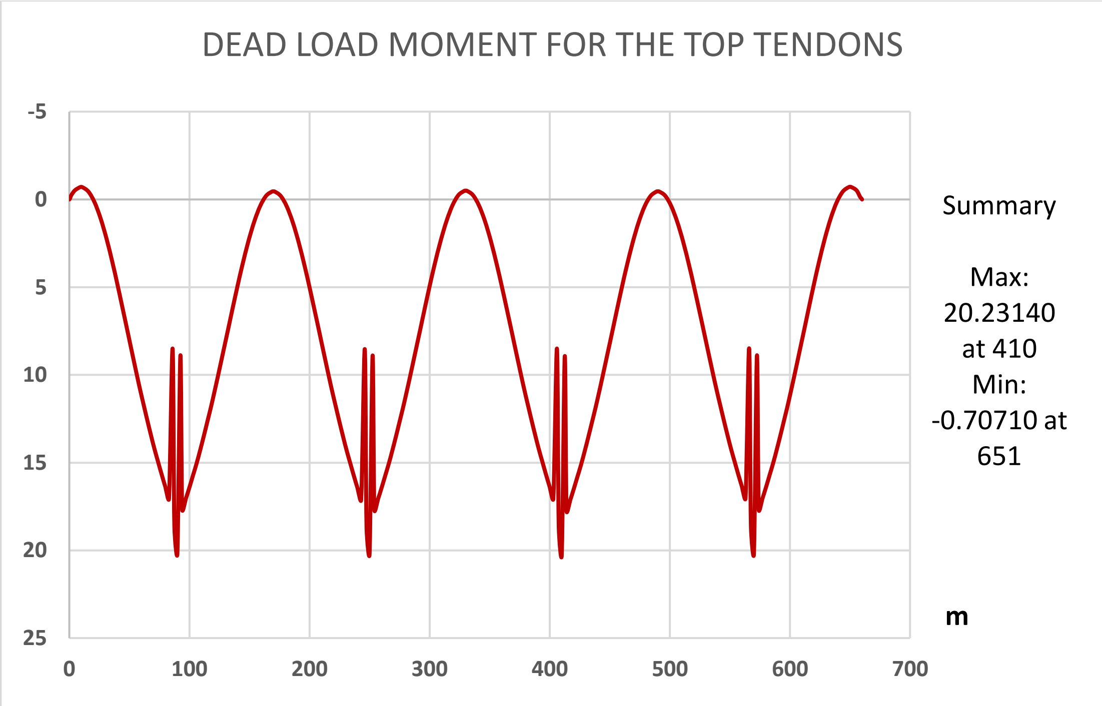

Figure 80: ***Dead Load Moment Diagram***

- ***Positive & Negative Thermal Gradient:***

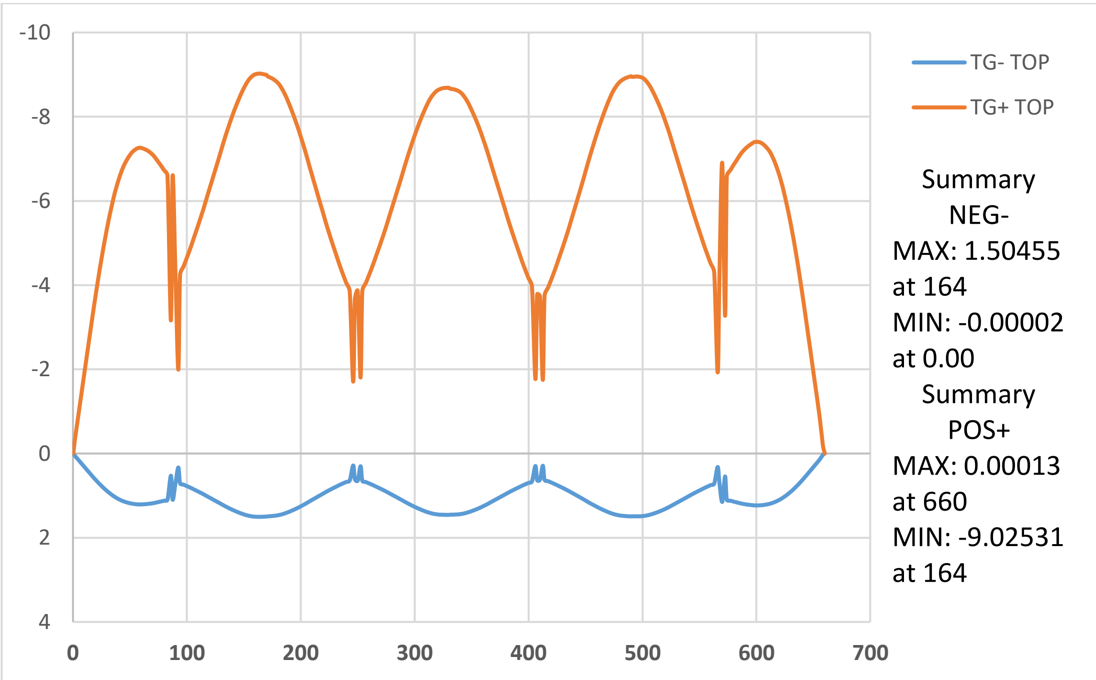

Figure 81: ***Positive and Negative Thermal Gradient Diagram***

- ***Creep Secondary Moment:***

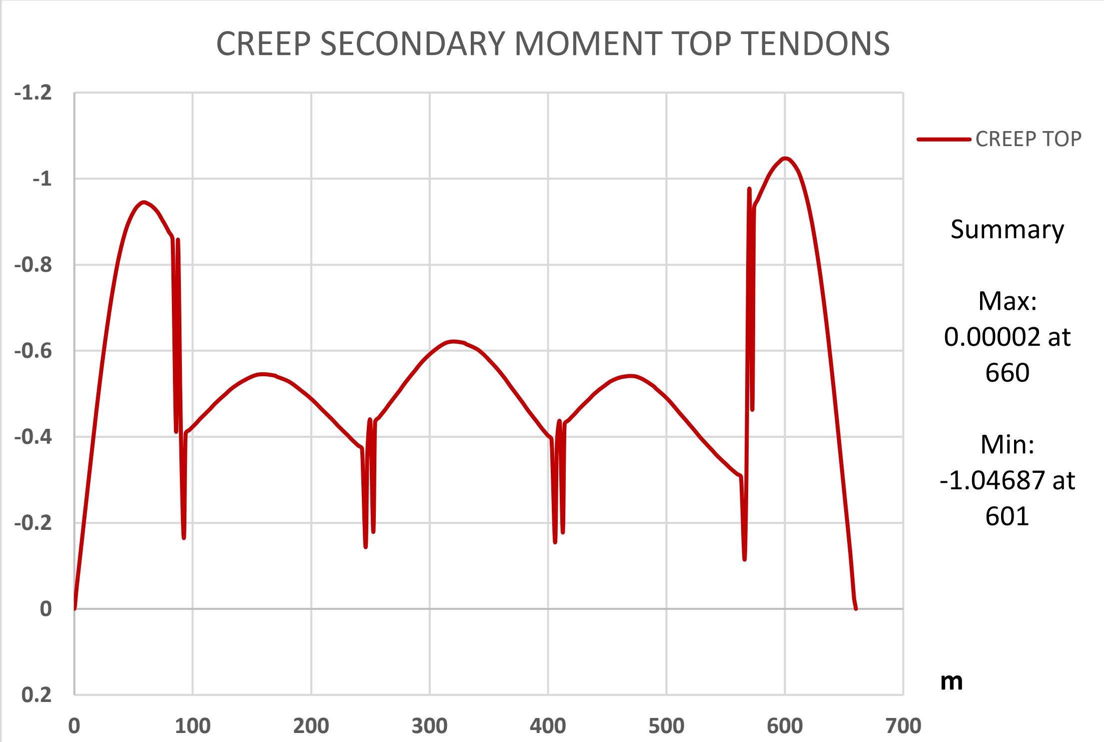

Figure 82: ***Creep Secondary Moment Diagram***

- ***Moving Load (LM1):***

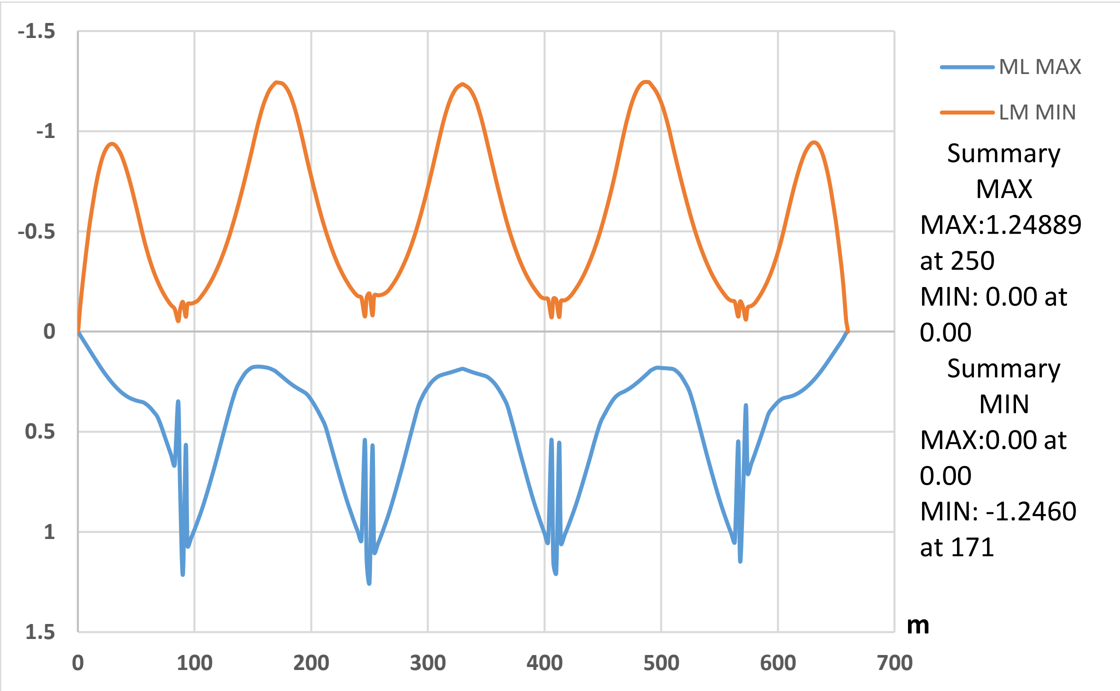

Figure 83: ***Load Model 1 Moment Diagram***

- ***Tendon Primary & Secondary Moment:***

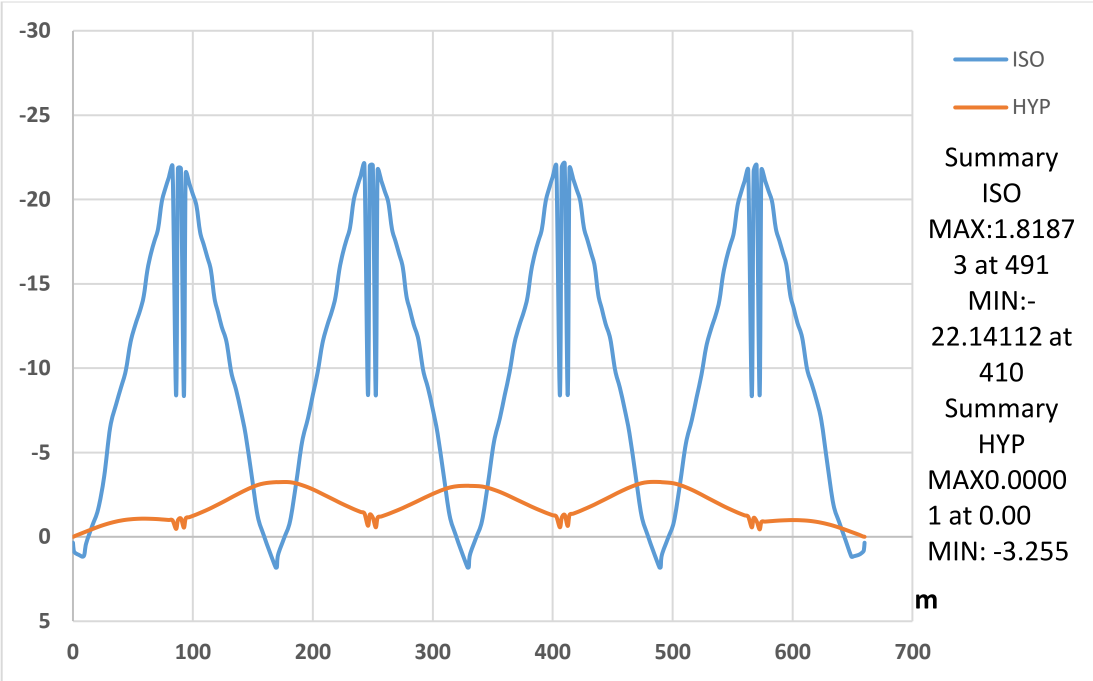

***Figure 84:* Tendon Primary & Secondary Moment Diagram**

- ***Superstructure Load:***

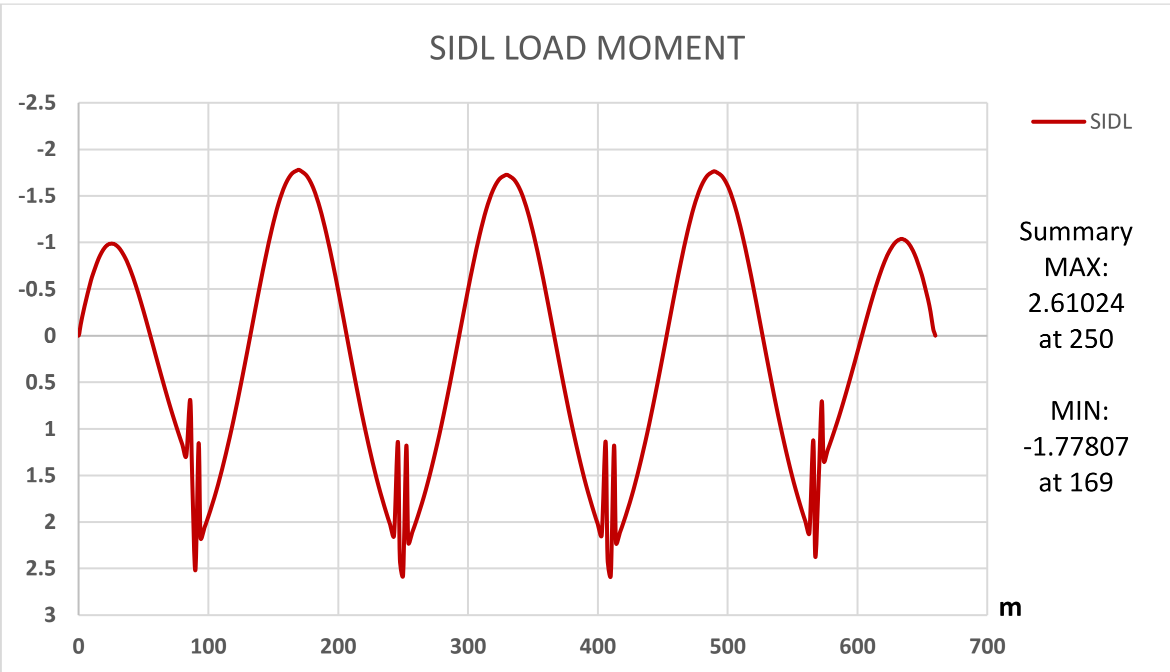

***Figure 85:* Superstructure Load Moment Diagram**

- ***Shrinkage Secondary Moment:***

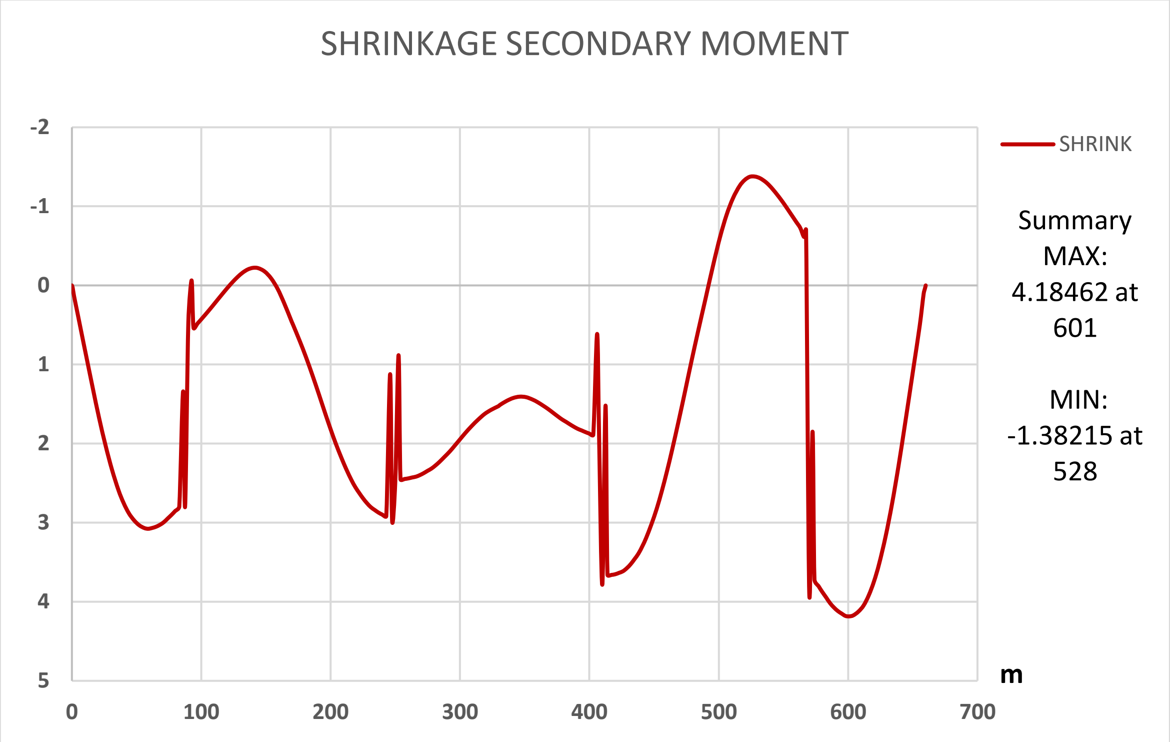

Figure 86: ***Shrinkage Secondary Moment Diagram***

- ***Characteristic load combination:***

According to Eurocode

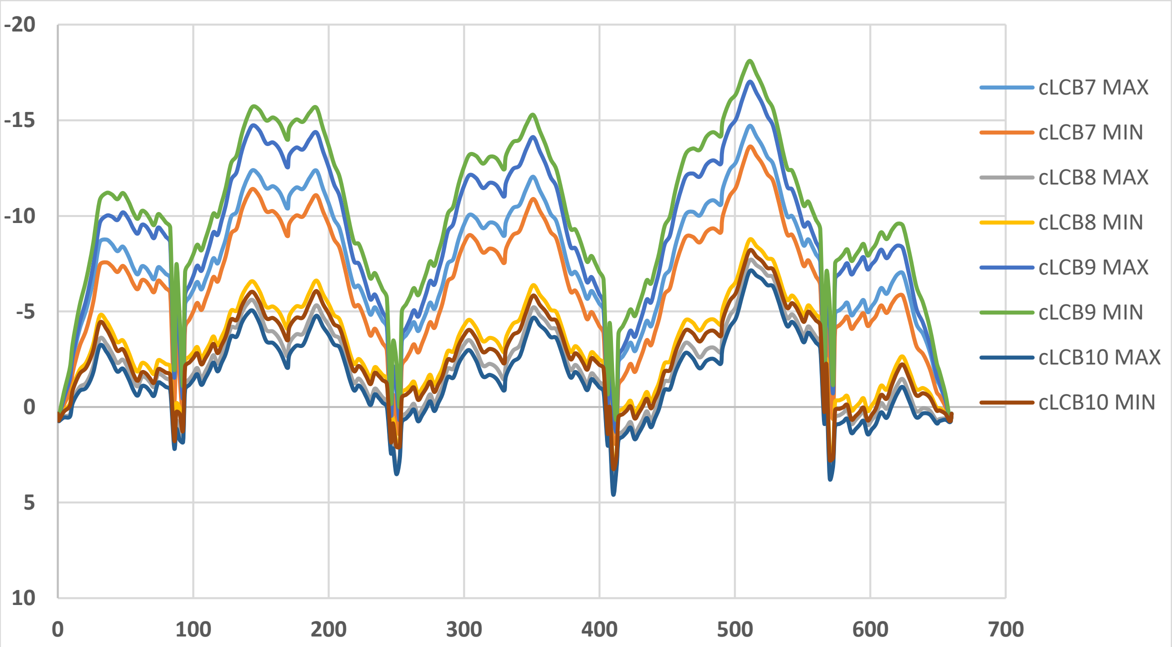

Figure 87: ***Combinations Load Moment Diagram***

- ***ENV MOMENT:***

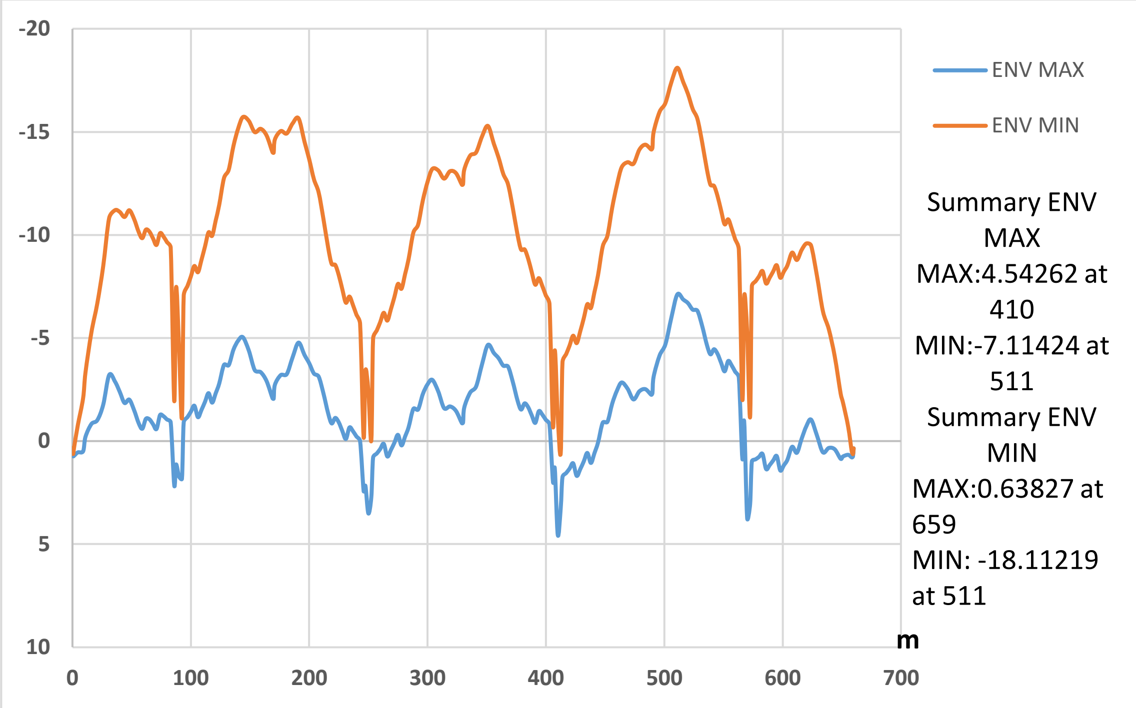

***Figure 88:* ENV Stress Diagram**

Note: Top fiber stress of Bridge Deck-Capacity check Before adopting external tendons is:

- Compression limit=26 MPa
- Tension limit=3.5 MPa

3.2. Bottom fiber stress check of Bridge deck:

- ***Dead Load:***

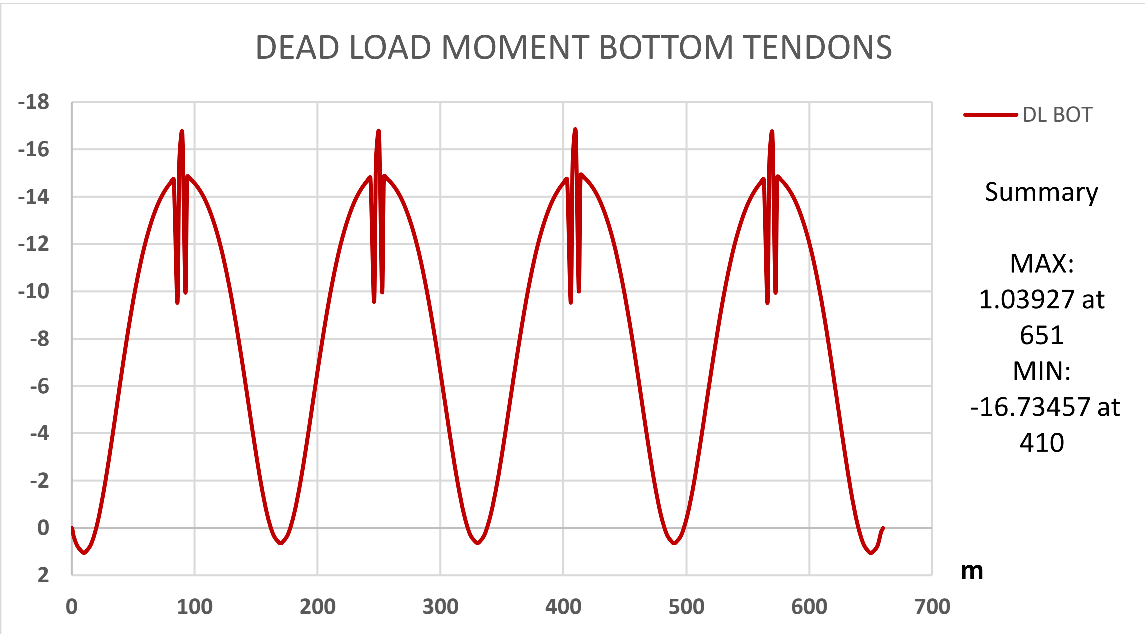

***Figure 89:* Dead Load Diagram**

- ***Creep Secondary Moment:***

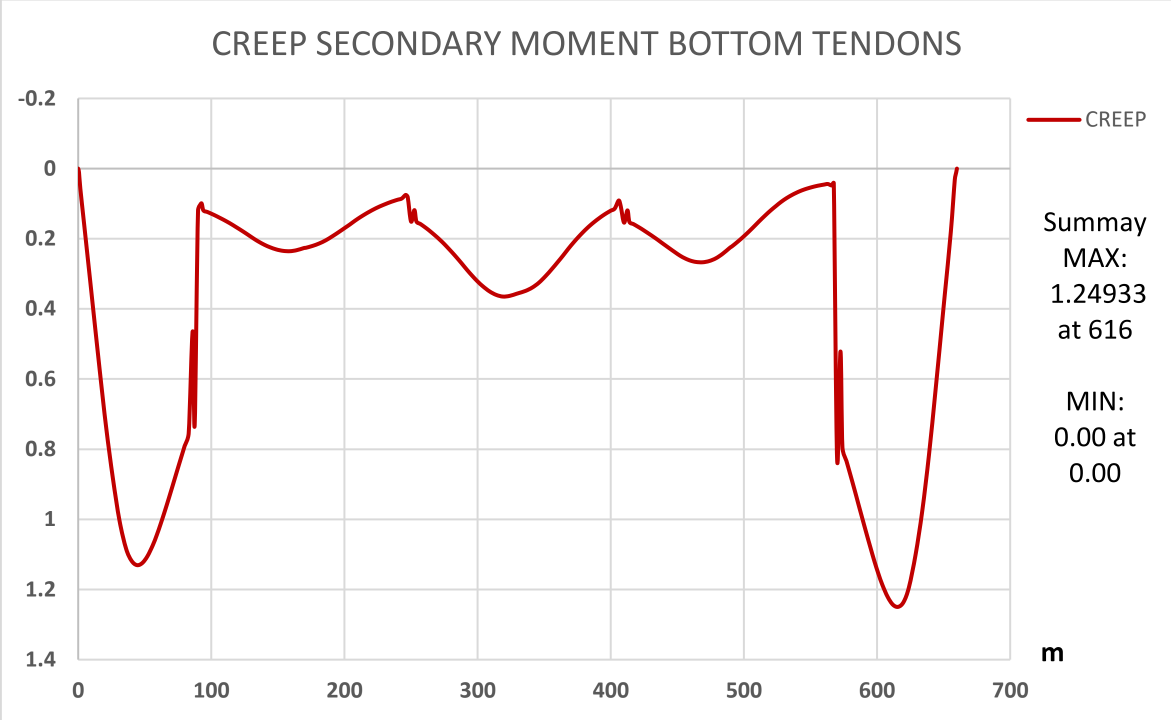

Figure 90: ***Creep Secondary Moment Diagram***

- ***Positive & Negative Thermal Gradient:***

Positive Thermal Gradient (+10’C):

MAX: 11.65541 at 164 ; MIN: 0.00 at 0.00

Negative Thermal Gradient (-5’C):

MAX: 0.00 at 0.00 ; MIN: -1.94310 at 164

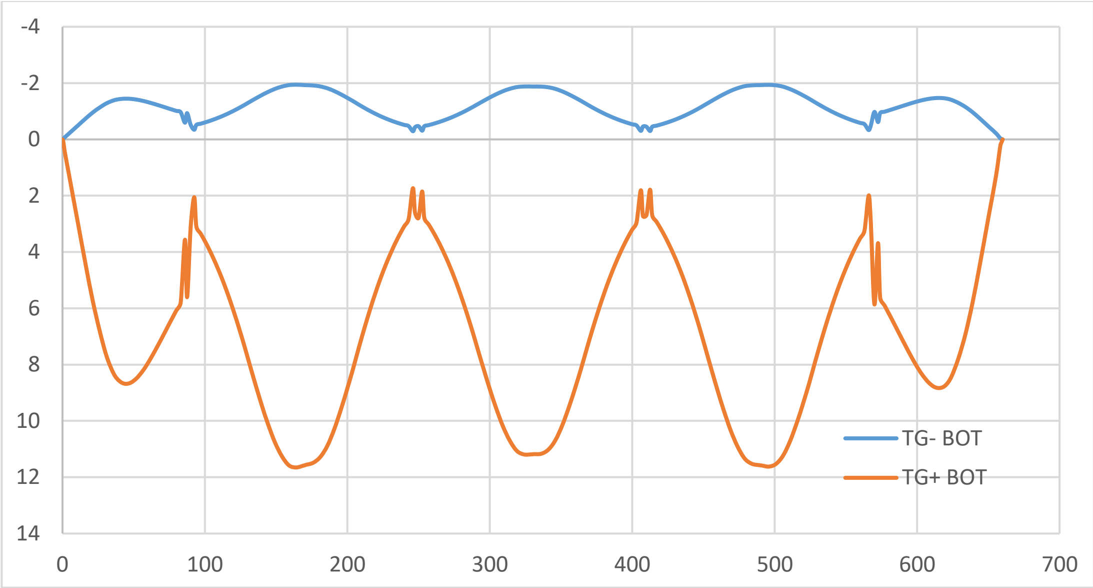

***Figure 91:* Positive & Negative Thermal Gradient Diagram**

- ***Moving Load (LM1):***

MAX:

MAX: 1.70252 at 330 ; MIN: 0.00 at 0.00

MIN:

MAX:0.00 at 0.00 ; MIN: -1.15250 at 570

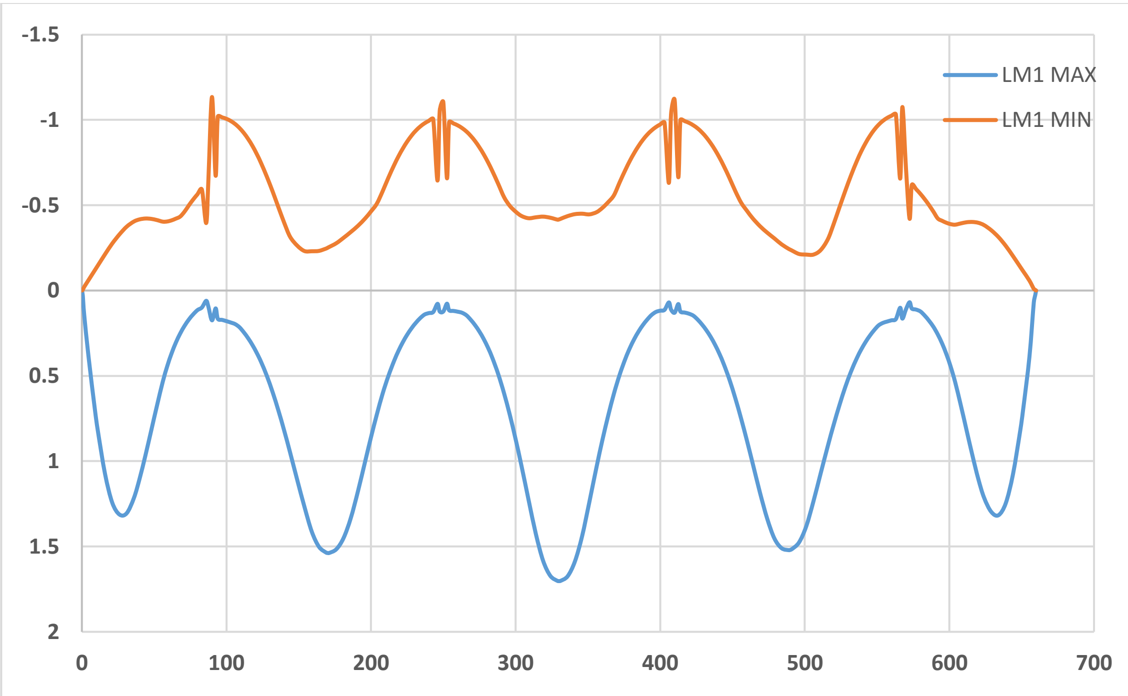

***Figure 92:* Load Model 1 Moment Diagram**

- ***Shrinkage Secondary Moment:***

Max: 5.84392 at 511 ; Min: -4.99386 at 616

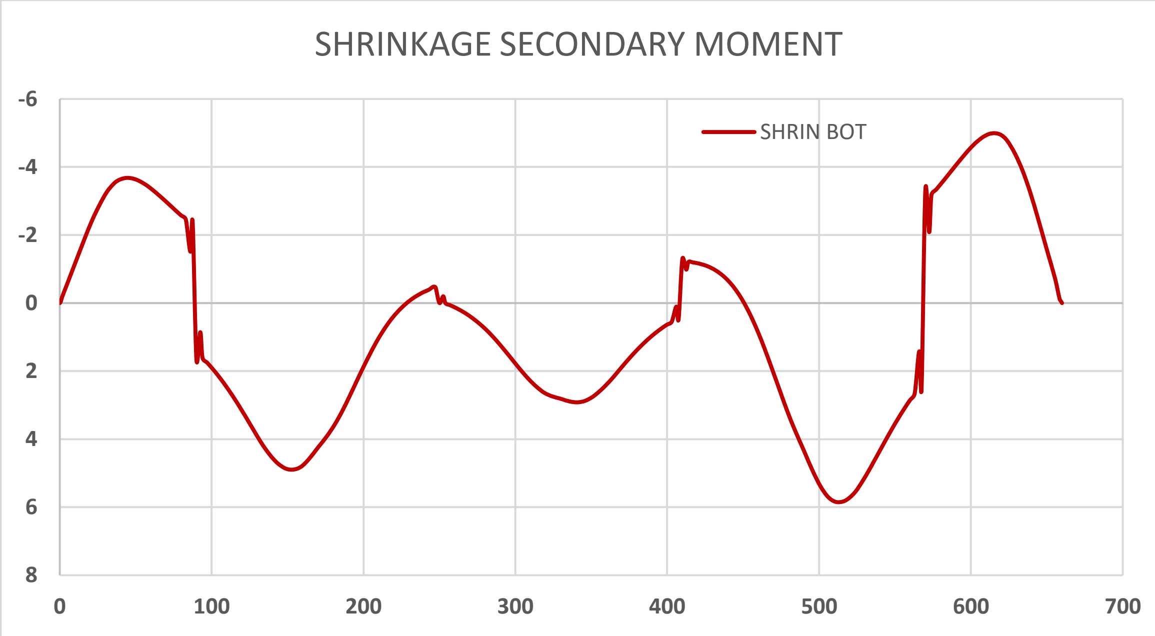

Figure 93: ***Shrinkage Secondary Moment Diagram***

- ***Superstructure Load:***

Max :2.05126 at 169 ; Min :-2.41873 at 250

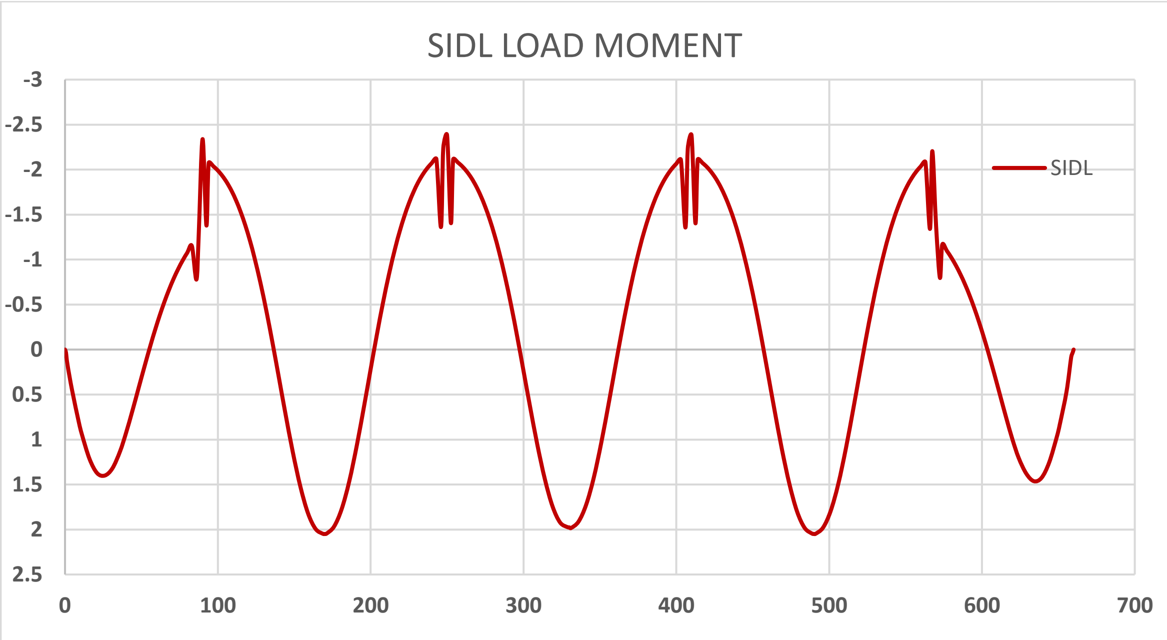

Figure 94: ***Superstructure Load Moment Diagram***

- ***Tendon Primary Moment:***

Max :4.40247 at 410 ; Min :-11.52748 at 496

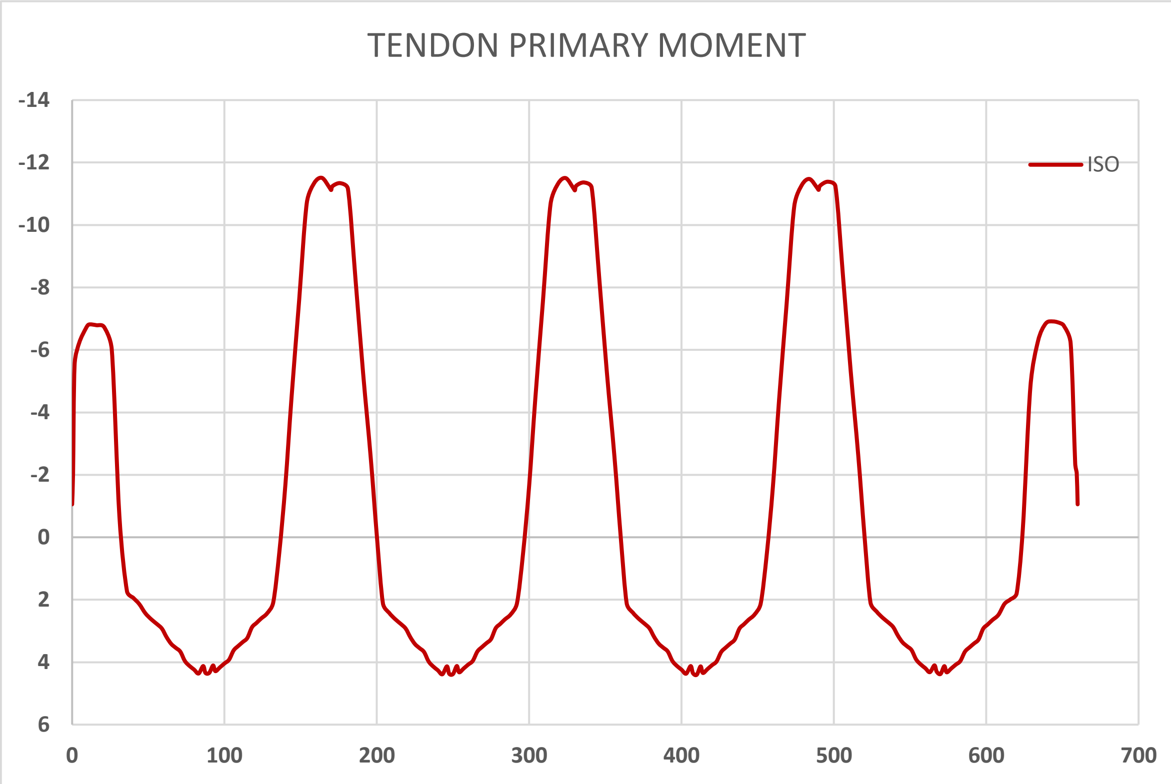

Figure 95: ***Tendon Primary Moment Diagram***

- ***Tendons Secondary Moment:***

Max :5.01972 at 176 ; Min :0.00 at 0.000

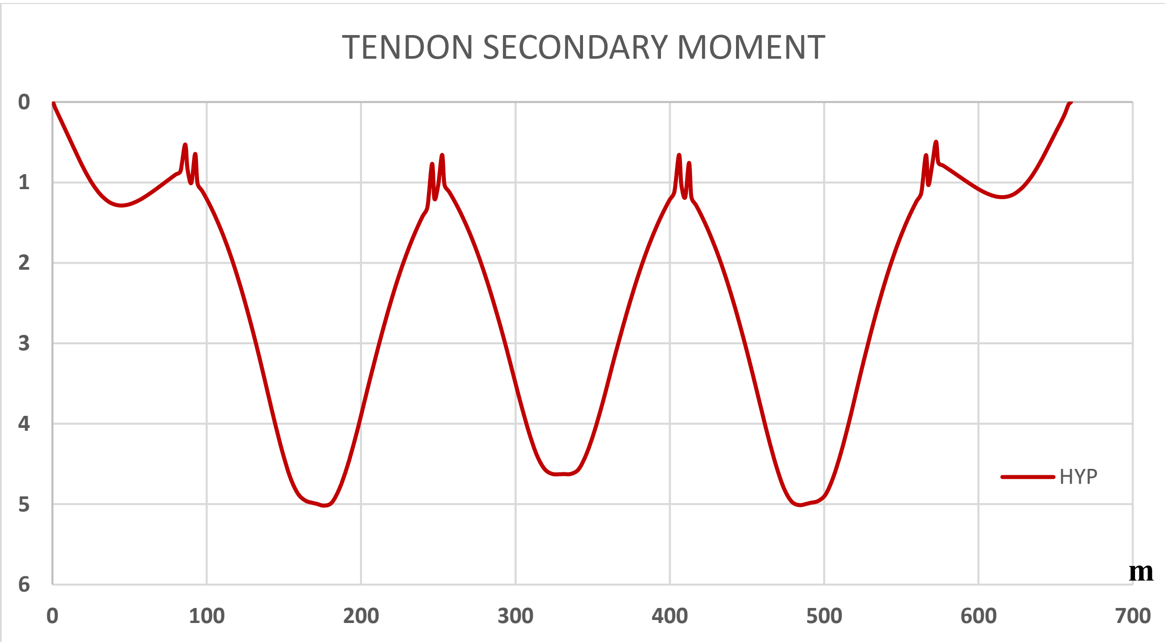

Figure 96: ***Tendon Secondary Moment Diagram***

- ***Characteristic Load Combinations:***

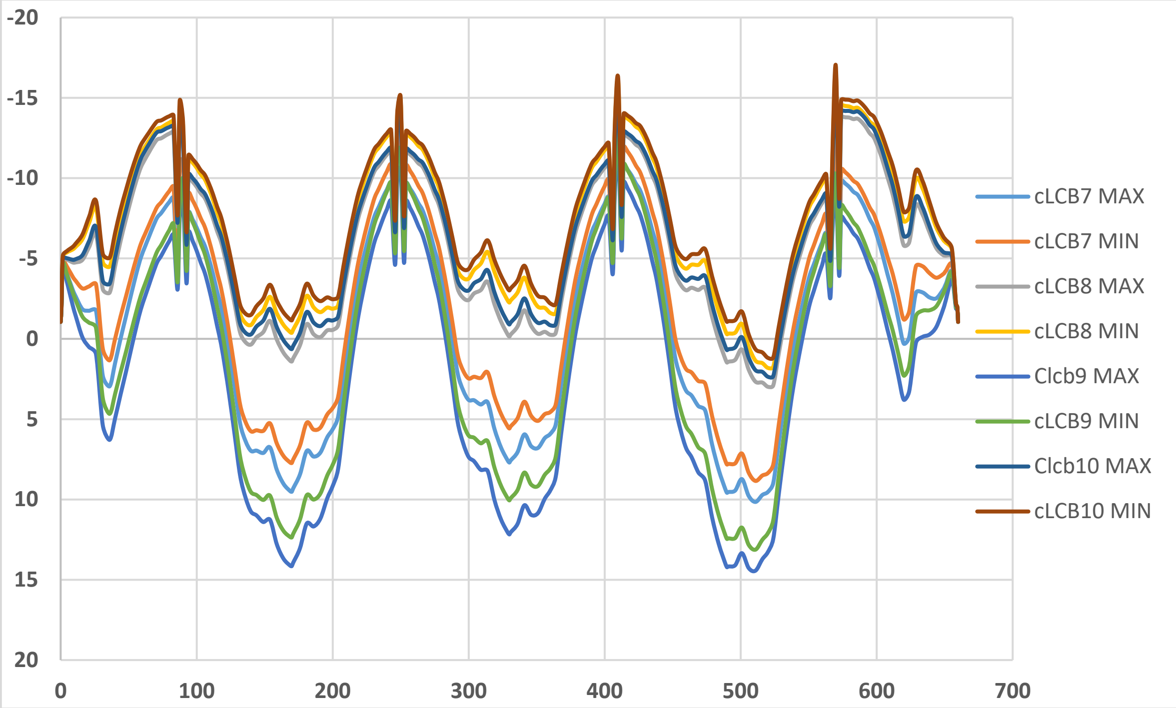

***Figure 97:* Combinations Load Diagram**

- ***ENV Moment:***

ENV max

Max : 14.43029 at 511 ; Min : -11.85460 at 410

ENV min:

Max : 1.18050 at 520 ; Min : -17.03281 at 570

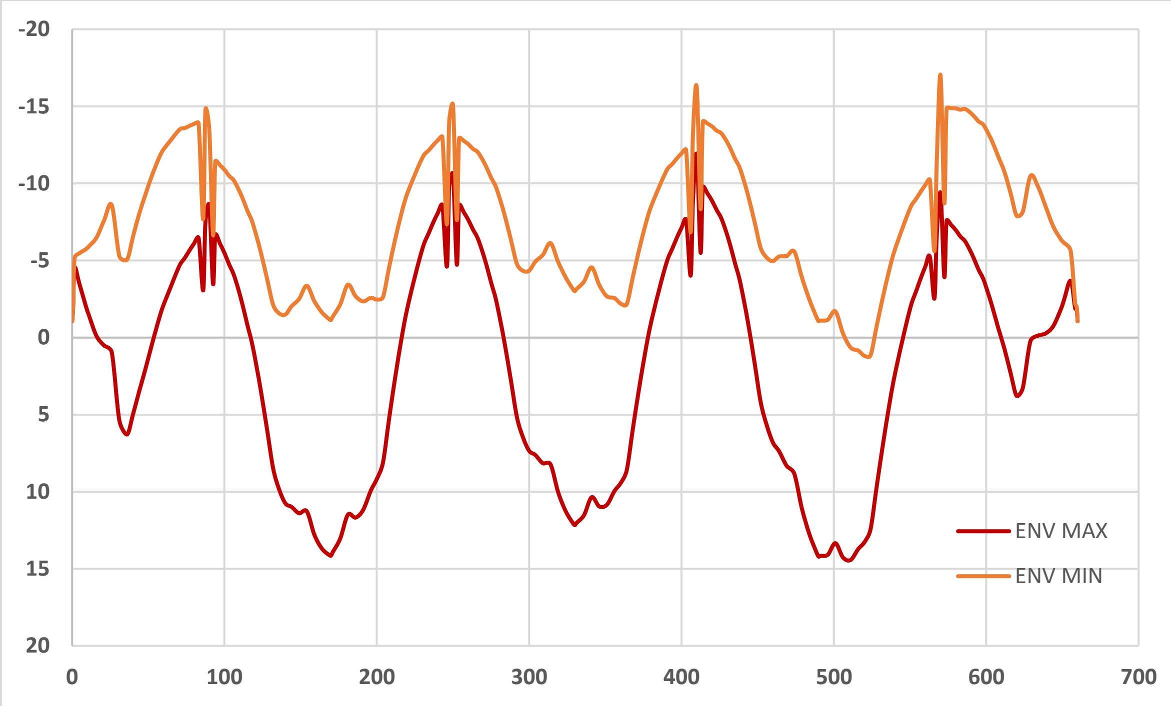

Figure 98: ***ENV Stress Diagram***

Note:

Characteristic load combinations (according to Eurocode):

- cLCB7→ 1.0MVL + 0.6POS.TPG + 1.0DL + 1.0SIDL + 1.0CR + 1.0SR 1.0PS
- cLCB8 → 1.0MVL + 0.6NEG.TPG + 1.0DL + 1.0SIDL + 1.0CR + 1.0SR 1.0PS
- cLCB9 → 1.0MVL + 0.6POS.TPG + 1.0DL + 1.0SIDL + 1.0CR + 1.0SR 1.0PS
- cLCB10→1.0MVL + 0.6POS.TPG + 1.0DL + 1.0SIDL + 1.0CR + 1.0SR 1.0PS

Each of them has two combinations:

- MAX Combination
- MIN Combination

IV. Design of External Tendons:

The fundamental goal of external prestressing tendons is the counteraction of the cumulative structural actions resulting from the permanent equipment weight, traffic load, temperature variations, and creep effects. The optimization of tendon number is achieved through a systematic, step-by-step process of prestress design:

- Geometry of Tendon Profile: Establishing the shape of the tendon layout according to the standards.
- Load Envelope Analysis: Calculation of the key loads and force envelopes.
- Hyperstatic Effect Analysis: Estimation of the additional parasitic loads due to the presence of external prestressing.
- Prestress Design: Calculation of the dimensions of the external prestress.

4.1. Choice of Layout and Definition of Design Sections:

a) **Vertical Profile (Elevation):** The vertical layout of the external continuity tendons is such that their location should be close to the bottom slab in each span and should rise toward the top slab at the pier supports. Taking into account allowable tolerances in construction work, there needs to be a minimum clearance of about 5 cm between the external surface of the tendon duct and the concrete surface of the segment slabs. A similar clearance is needed for the tendon duct above the anchorage blisters of the bottom slab continuity tendons.

b) **Horizontal Alignment (Plan View):** In plan, the external continuity tendons are located next to the webs. To provide for the necessary clearance allowance for tolerance, it should be equal to 5 cm from the outside of the tendon duct to the concrete surfaces of the webs and/or anchorage blisters.

c) **Deviation and Continuity:** The tendon profile is deflected at pier diaphragms and intermediate span deviators, thus producing a piece-wise linear (polygonal) shape of tendon path. The location of intermediate span deviators is typically 1/4 – 1/3 span away from the supports. Due to the long length of the superstructure and a multi-span construction, the use of continuously laid single tendons running through to the other deck end is not practical. Therefore, an overlapping design is used, with each tendon passing through 2 or 3 adjacent spans.

4.2. Span Deviators Positioning:

The positioning of the deviation blocks (deviators) is guided by two key structural considerations [26]:

- Segment Alignment: Every deviator should be placed at the middle point of each precast/cast-in-place segment.
- Side Spans (End Spans): Only one deviator is used per span, and this is placed at a distance of $0.4L_{side}$ up to $0.5L_{side}$ away from the abutment, where L_{side} stands for the total length of the side spans (L_{side} = 90m).
- Intermediate Spans: There are two deviators used per span, and these are placed at a distance of 2/5 L_{int} away from the piers, where L_{int} stands for the total length of the intermediate spans (L_{int} = 160m) (see figure below).

Given the described design parameters, the optimized solution will involve:

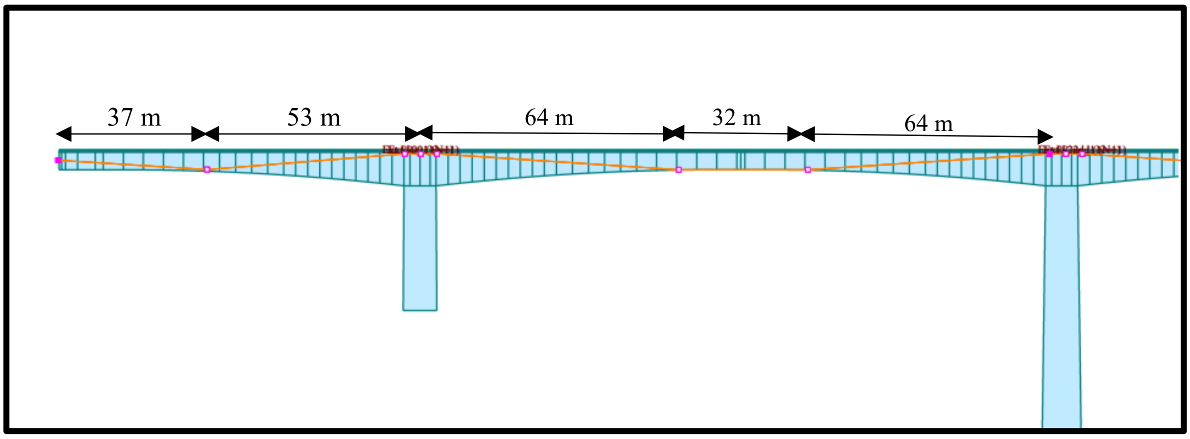

Figure 99: Spans Deviators Position

4.3. Tendon Positioning in Cross-Sections and Tendon Layout:

The vertical positioning of tendons in the cross-sectional area is based on the following geometrical rules:

- Pier Section: The minimum distance between the center of gravity of the tendon section and the top fibers equals the thickness of the top slab, which is 0.35 m.
- Middle Span Section (Critical Section): The minimum distance between the center of gravity of the tendon section and the bottom fibers equals the thickness of the bottom slab, added to 0.15 m. This value allows for a structural clearance of 0.10 m between the external prestressing ducts and the top surface of the bottom slab.

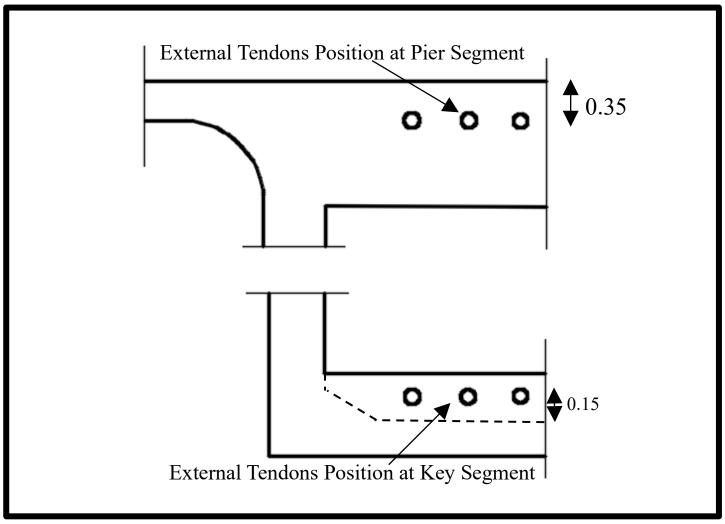

Figure 100: Cross-Sectional of External Tendons

4.3. Design and Checking the Tension of External Tendons:

We selected tendons of type 27T15 arranged as follows:

- Span C0-P1: 12×27T15 Tendons
- Span P1-P2: 12×27T15 Tendons
- Span P2-P3: 10×27T15 Tendons
- Span P3-P4: 12×27T15 Tendons
- Span P4-C2: 12×27T15 Tendons

- ***External Tendon Primary Moment:***

The external tendon primary moment diagram after using 27T15 Tendons:

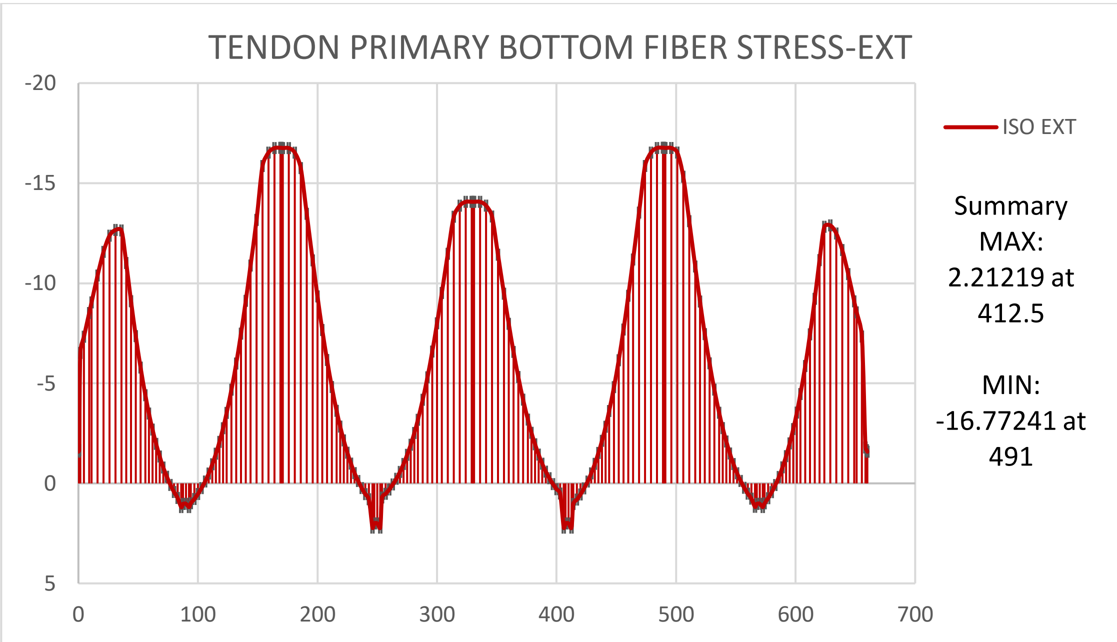

Figure 101: External Tendons Primary Moment Diagram

- ***External Tendon Secondary Moment:***

The external tendon secondary moment diagram after using 27T15 Tendons:

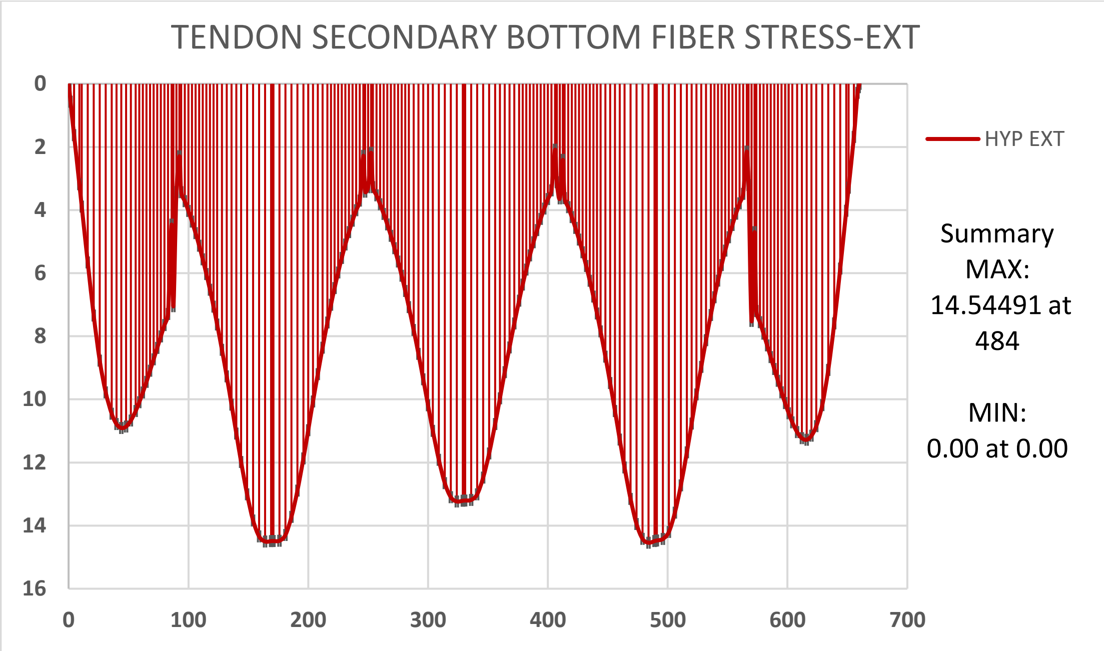

Figure 102: ***External Tendons Secondary Moment Diagram***

Note:

- *As noted, the results favor primary tendons over secondary tendons, which is a positive outcome.*

4.3.1. Stress Checking of the bottom Fiber:

According to the study results:

- Despite the addition of tendons, the allowable tension limit has been exceeded, which is as follows:

- ***ENV Characteristic mix:***

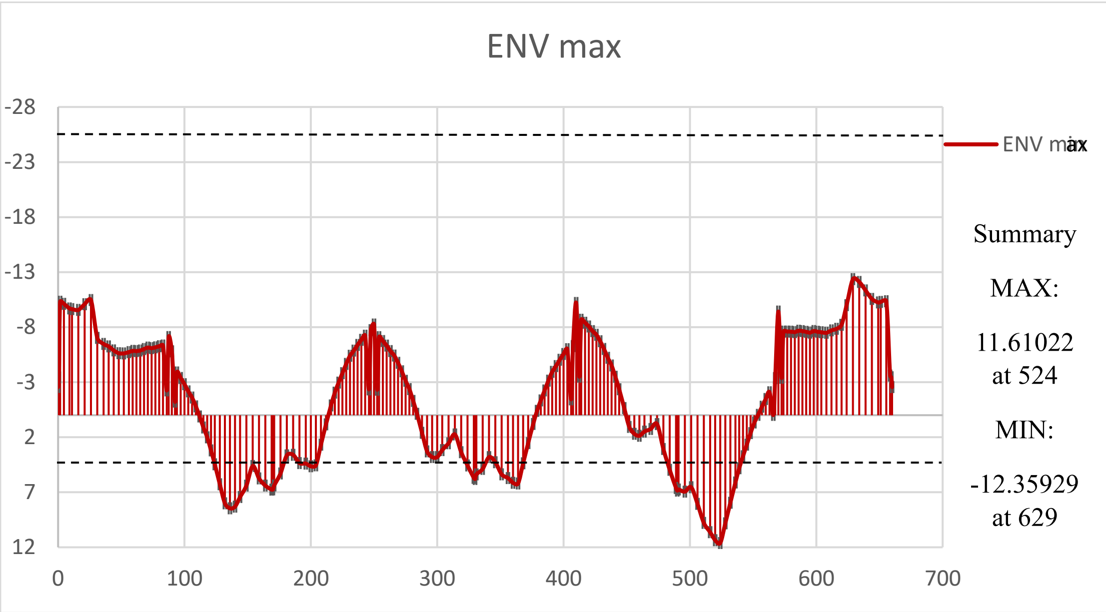

*Figure 103 : **ENV Characteristic max-bottom fiber stress diagram***

- ***ENV Characteristic min:***

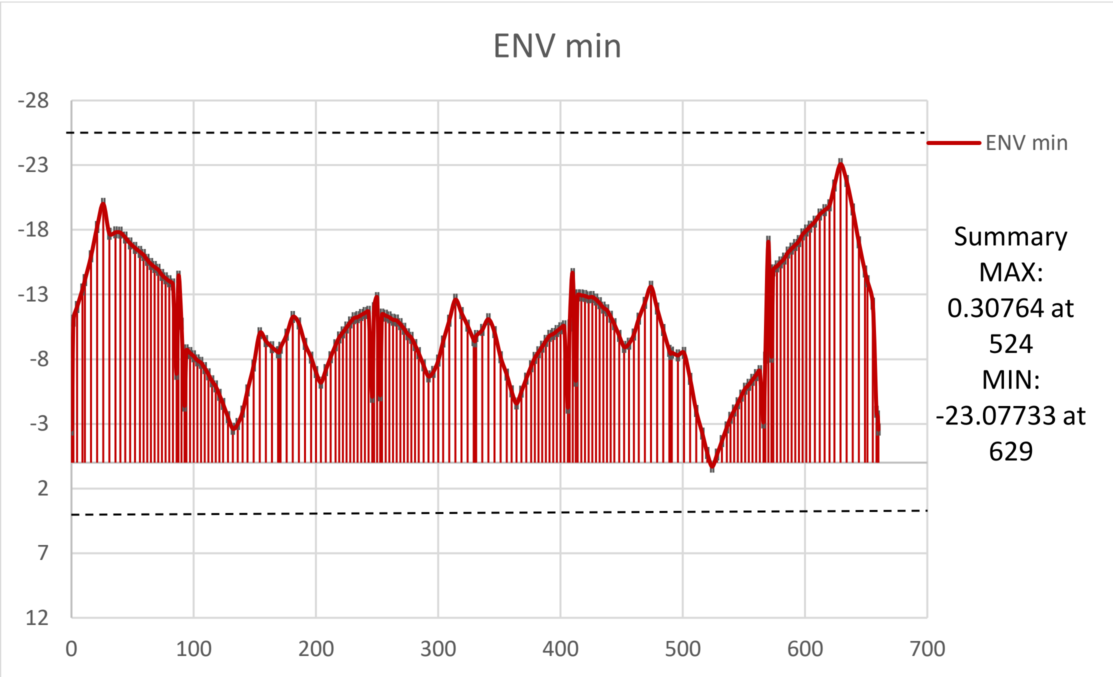

*Figure 104: **ENV Characteristic min-bottom fiber stress diagram***

Note:

- *Stress results for the bottom fiber according to the ENV Characteristic min specification have been achieved, but the bridge remain unsafe*

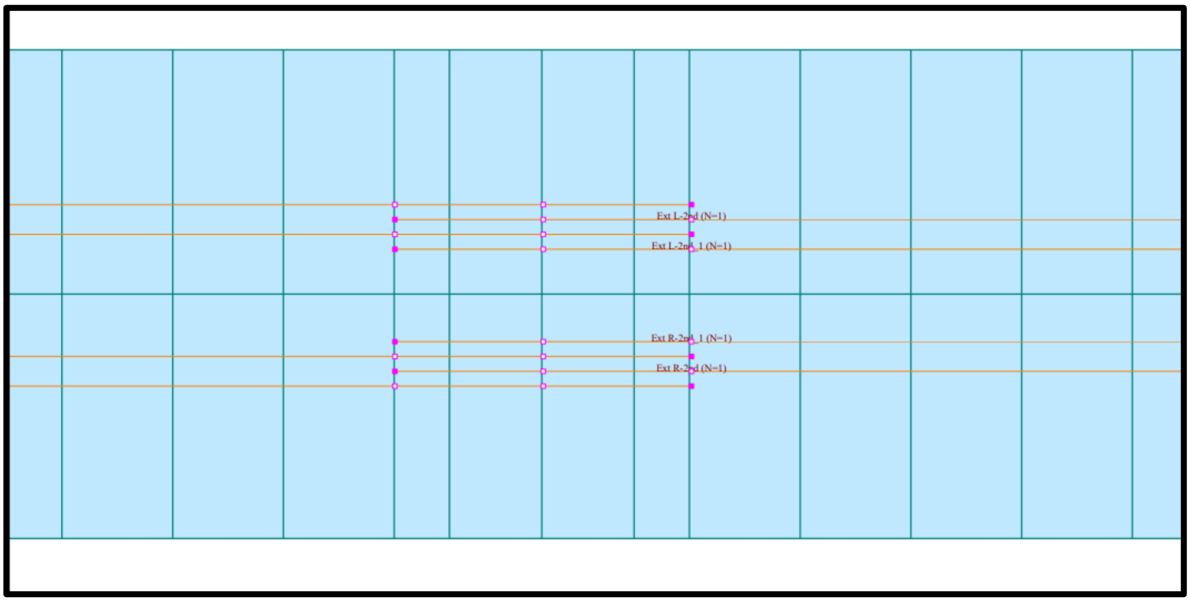

***Figure 105:* External Tendons Layout**

Note: It should be noted that the tendon does not span the entire length of the bridge, rather, it has been divided into sections, with some extending to the first and second spans, another section covering the third span, and yet another section supporting the fourth and fifth spans.

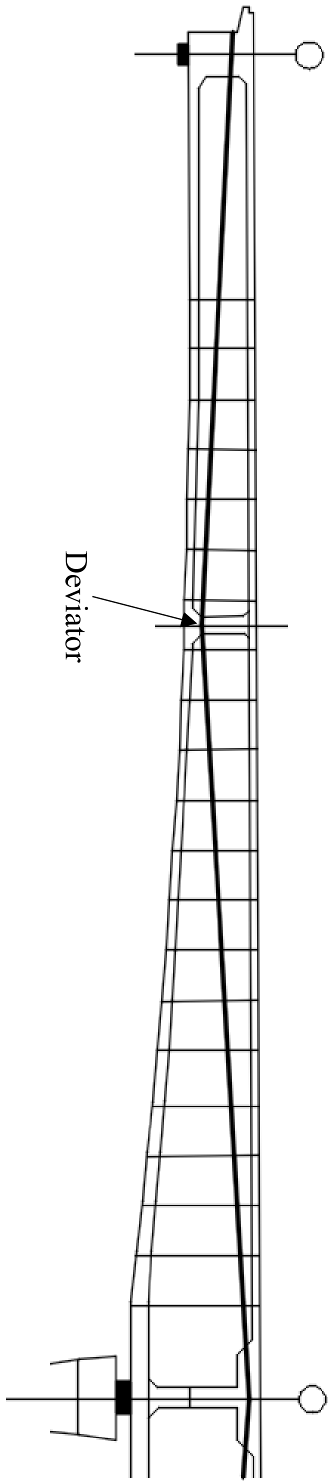

Figure 106: ***External Tendons Design Side-Span***

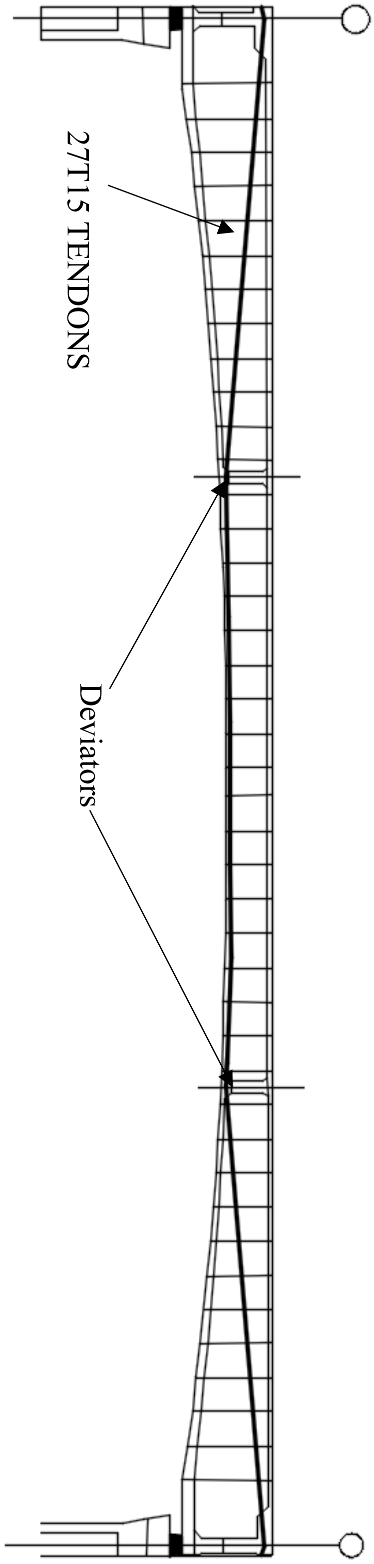

Figure 107: ***External Tendons Design Mid-Spans***

4.4. Serviceability Limit State (SLS) Verifications:

To accurately estimate the required number of external tendons, it is preferable to utilize a design methodology that explicitly accounts for the secondary effects of prestressing. Accordingly, the calculations tabulated below are performed in accordance with the technical note *'Sizing Continuity Tendons in Segmental Cantilever Bridges'*. [27] [28]

Table 24:External Tendon Primary-secondary stress at bot fiber

Elements	Part	SLS Bot Stress (Mpa)	Primary Bot Stress (Mpa)	Sec. Bot Stress (Mpa)	Tot. Ex. Tendons Bot Stress (Mpa)	Tension limit (3.5)-SLS Stress) (Mpa)	B=(Tension limit -SLS Stress) / (Tot.Ex.Tendons)	Primary Axial Force (MN)	Required Axial Force (MN)
5	I [5]	-2.0415	-8.5134	3.1808	-5.3326	5.5415		-60.13017907	
5	J [6]	-1.4953	-9.0508	3.8876	-5.1632	4.9953		-60.13017907	
11	I [11]	6.3003	-12.638	10.454	-2.184	-2.8003	1.282188645	-60.20914184	-77.19947797
11	J [12]	4.823	-11.009	10.776	-0.233	-1.323	5.678111588	-59.8269476	-339.7040844
Pier Seg. 27	I [27]	-7.4872	1.0255	7.046	8.0715	10.9872		-60.70752118	
Pier Seg. 27	J [28]	-8.7007	1.0252	7.2474	8.2726	12.2007		-60.70752118	
50	I [50]	14.148	-16.769	14.491	-2.278	-10.648	4.67427568	-61.38538154	-286.9321961
50	J [51]	14.164	-16.768	14.488	-2.28	-10.664	4.677192982	-61.38538154	-287.1112758
Pier Seg. 73	I [73]	-9.8157	2.0029	3.4174	5.4203	13.3157		-112.8281055	
Pier Seg. 73	J [74]	-11.242	2.0025	3.4171	5.4196	14.742		-112.8281055	
96	I [96]	12.1	-14.07	13.211	-0.859	-8.6	10.01164144	-50.88397544	-509.4321173
96	J [97]	12.191	-14.07	13.207	-0.863	-8.691	10.07068366	-50.88397544	-512.4364201
Pier Seg. 119	I [119]	-8.879	2.0033	3.1185	5.1218	12.379		-112.8534026	
Pier Seg. 119	J [120]	-10.259	2.0029	3.116	5.1189	13.759		-112.8534026	
142	I [142]	14.05	-16.772	14.489	-2.283	-10.55	4.621112571	-61.39959237	-283.7344282
142	J [143]	14.232	-16.772	14.479	-2.293	-10.732	4.680331444	-61.39959237	-287.3704428
Pier Seg. 165	I [165]	-6.5704	1.023	3.2037	4.2267	10.0704		-60.63483539	
Pier Seg. 165	J [166]	-7.893	1.0226	3.197	4.2196	11.393		-60.63483539	
180	I [180]	0.74628	-7.4461	11.229	3.7829	2.75372	0.727938883	-59.74817413	-43.49301913
180	J [181]	2.4671	-9.2264	11.285	2.0586	1.0329	0.501748761	-59.74817413	-29.97857236
188	I [188]	-1.8407	-9.0957	4.0218	-5.0739	5.3407		-60.00852602	
188	J [189]	-2.324	-8.5479	3.2905	-5.2574	5.824		-60.00852602	

Note: The table above indicates that the allowable tensile stress limits are not satisfied across the majority of the bridge spans regarding bottom fiber stress. Furthermore, the existing axial force generated by the primary effect of the external tendons must be amplified by factor B. For instance, element 96, which corresponds to the bridge mid-span, requires ten times the existing primary axial force (equivalent to ten times the current number of external tendons) to satisfy the allowable tension limit. This requirement is clearly unfeasible and impractical.

***Table 25:**Design of Bottom Internal Tendons at the Service stage*

Elements	Part	SLS Bot Stress (Mpa)	Primary Bot Stress (Mpa)	Sec. Bot Stress (Mpa)	Tot. Ex. Tendons Bot Stress (Mpa)	Tension limit (3.5)-SLS Stress) (Mpa)	B=(Tension limit -SLS Stress) / (Tot.Ex.Tendons)	Primary Axial Force (MN)	Required Axial Force (MN)
5	I [5]	-3.88	-7.21	2.11	-5.10	7.38		-24.81	
5	J [6]	-3.72	-7.41	2.58	-4.83	7.22		-25.46	
11	I [11]	-6.99	-0.03	6.94	6.91	10.49		-48.73	
11	J [12]	-6.82	-0.03	7.16	7.13	10.32		-49.90	
Pier Seg. 27	I [27]	-7.35	-0.25	4.76	4.51	10.85		-157.06	
Pier Seg. 27	J [28]	-8.59	-0.25	4.90	4.65	12.09		-157.05	
50	I [50]	12.45	-11.70	4.65	-7.05	-8.95	1.27	-40.69	-51.68
50	J [51]	12.36	-11.60	4.66	-6.94	-8.86	1.28	-40.43	-51.63
Pier Seg. 73	I [73]	-8.56	-0.25	1.14	0.89	12.06		-157.32	
Pier Seg. 73	J [74]	-10.01	-0.25	1.14	0.89	13.51		-157.36	
96	I [96]	11.51	-12.10	4.42	-7.68	-8.01	1.04	-42.41	-44.24
96	J [97]	11.54	-12.10	4.42	-7.68	-8.04	1.05	-42.12	-44.08
Pier Seg. 119	I [119]	-7.28	-0.25	0.99	0.74	10.78		-157.40	
Pier Seg. 119	J [120]	-8.66	-0.25	0.99	0.74	12.16		-157.42	
142	I [142]	12.30	-11.70	4.66	-7.04	-8.80	1.25	-40.70	-50.85
142	J [143]	12.47	-11.60	4.65	-6.95	-8.97	1.29	-40.43	-52.16
Pier Seg. 165	I [165]	-4.67	-0.25	0.97	0.72	8.17		-157.35	
Pier Seg. 165	J [166]	-5.97	-0.25	0.97	0.72	9.47		-157.38	
180	I [180]	-7.90	-0.05	6.22	6.17	11.40		-62.44	0.00
180	J [181]	-7.89	-0.05	6.24	6.19	11.39		-61.51	0.00
188	I [188]	-4.28	-7.40	2.22	-5.18	7.78		-25.46	
188	J [189]	-4.34	-7.20	1.81	-5.39	7.84		-24.80	

➢ OTHER STRATEGIES:

According to the table 25, here is an extra attempt to satisfy the allowable tension limit:

❖ **<u>THE 1ST STRATEGY:</u>** Max Bot fiber stress according to the characteristic combination load after adding another 4 bot slab tendons type 27T15 to the 2nd, 3rd and the 4th spans:

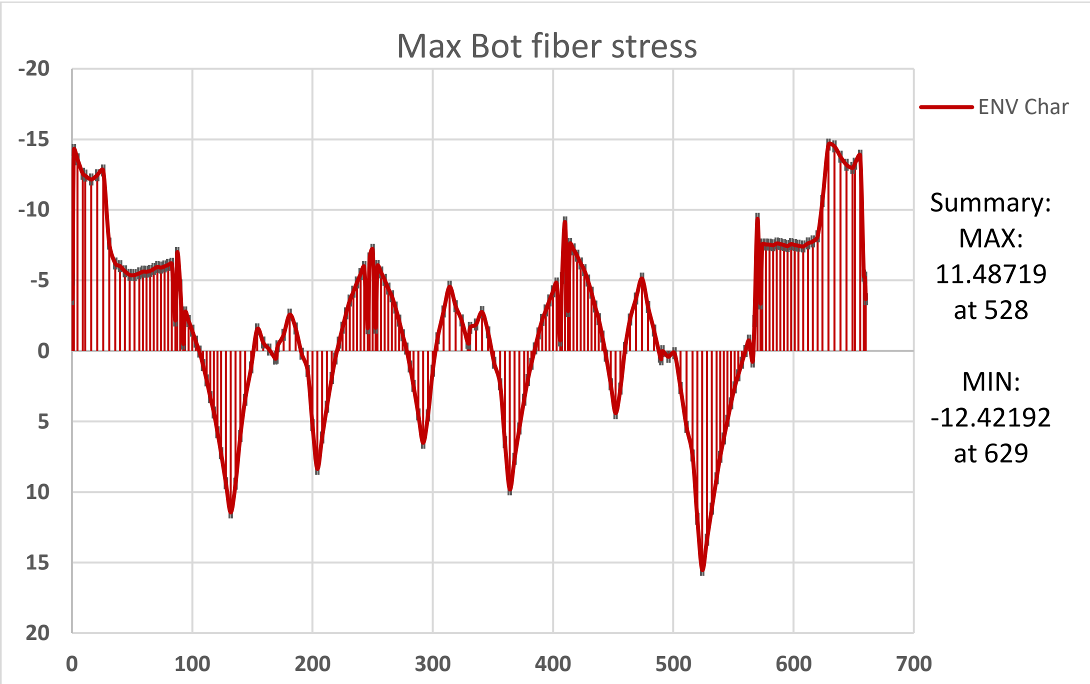

Figure 108: ***Max Bot fiber stress Diagram***

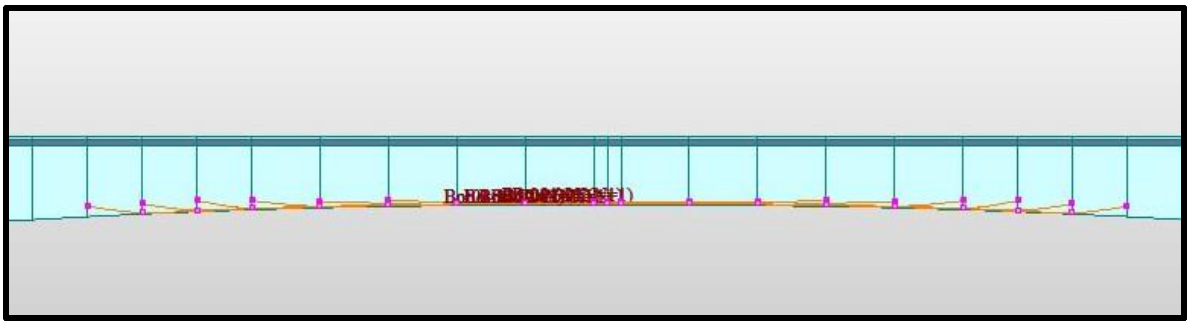

Figure 109: ***Plan of View of the Extra Tendons***

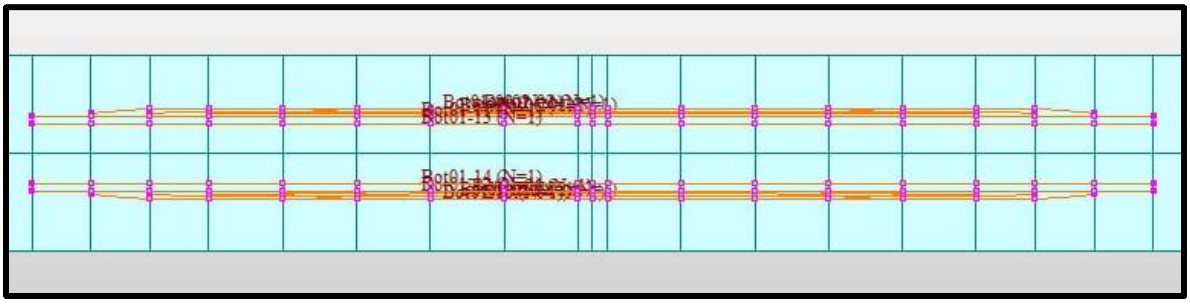

Figure 110: Plan of View of the Extra Tendons Length

Note: Max Bot Stress after adding 4 tendons at service stage, Not Satisfied.

- **THE 2ND STRATEGY:** Max Bot Stress after modifying the 4 extra internal bottom tendons at service stage:

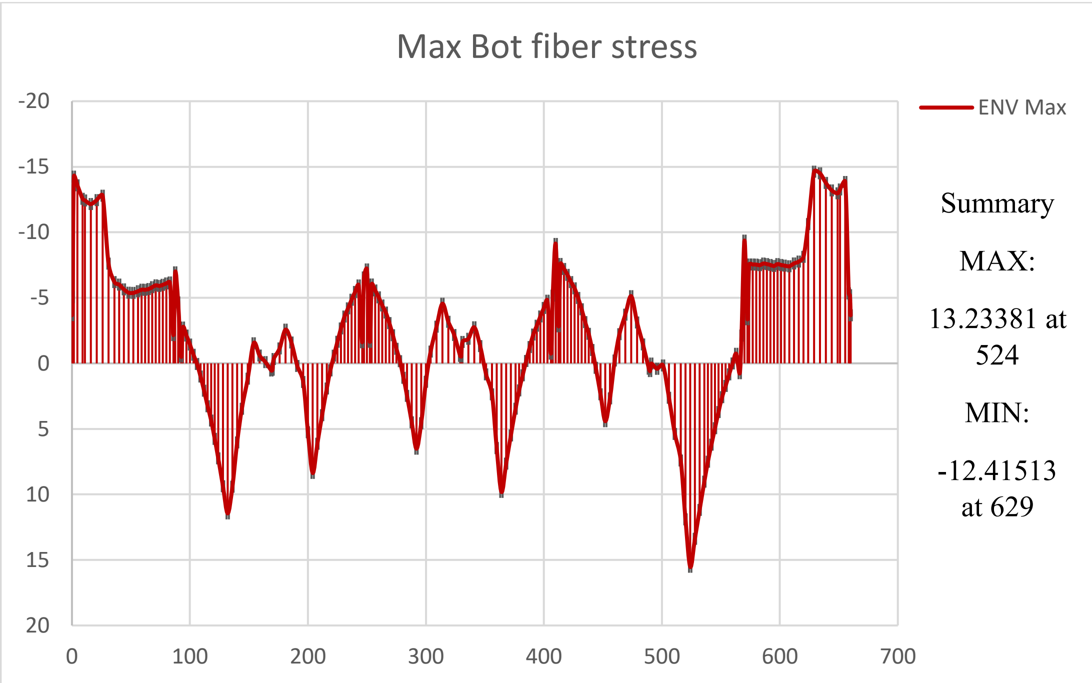

Figure 111: Max Bot fiber stress Diagram After Modifying

- Plan view of the 3rd configuration of internal tendons (06-14-14-14-06) with length modification of one of the intermediate-Span:

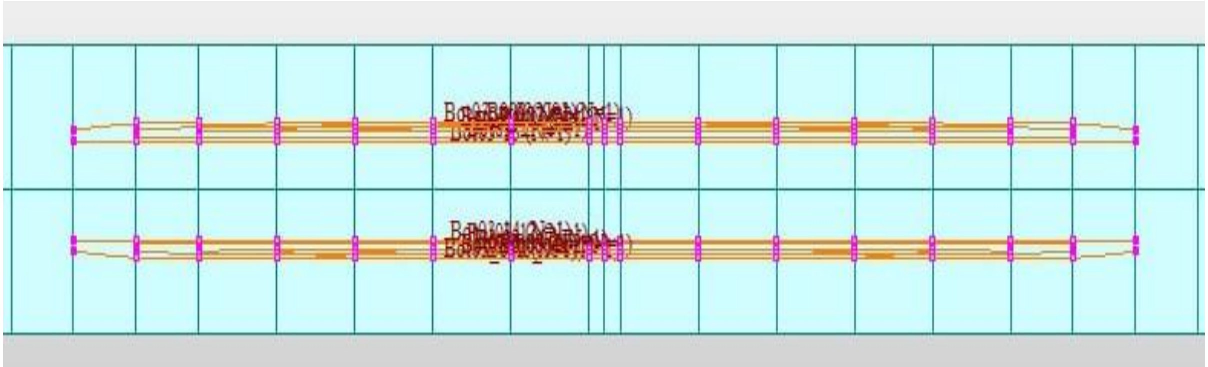

Figure 112: ***Plan of View of the Mid-Spans Tendons***

Note: *Despite the adjustment on the length of the 4 extra tendons, the result was still unsatisfactory*

- **THE 3RD STRATEGY:** Adopting 37T15 for all internal bottom tendons-Bot stress according to max characteristic comb.

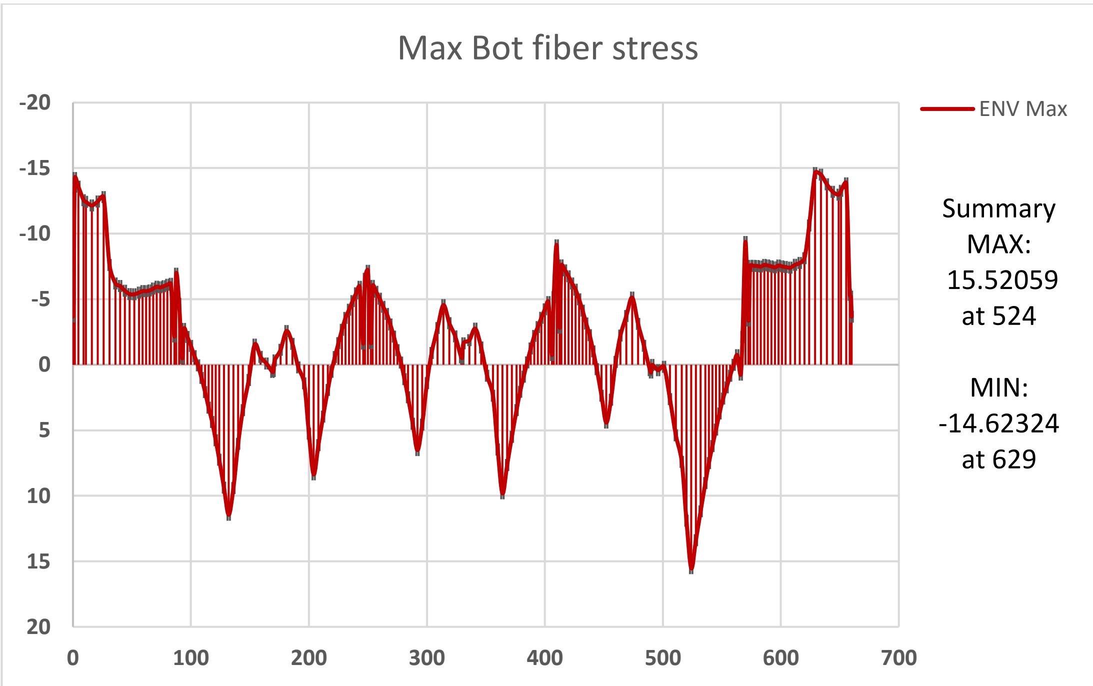

Figure 113: ***Max Bot fiber stress Diagram After Adding 37T15***

Note: *Even after adding tendons of type 37T15, the result remains not satisfactory.*

- Despite all these design measures, the allowable tensile stress limit of 3.5MPa has not been met.

CHAPTER F: TECHNICAL AND ECONOMIC OPTIMIZATION STUDY

Technical and Economic Limitations of the Optimization Study:

The technical and economic optimization study for the external tendons could not be completed. This is due to the non-compliance with the bottom fiber normal stress requirement of the bridge deck slab. Even after introducing additional internal bottom tendons, adjusting their layout lengths, and upgrading the prestressing steel capacity to the 37T15 configuration, massive and unacceptable tensile stresses persist in the concrete section.

Furthermore, the approach used to determine the internal tendons based on the methodology outlined in the research *'Sizing Continuity Tendons in Segmental Cantilever Bridges'* proved unsuccessful in this specific case, as clearly demonstrated by the results detailed in the previous chapter.

Based on the structural analysis, it is established that no margin remains for optimizing the external tendons, nor is it feasible to introduce additional internal prestressing to satisfy the serviceability limit state (SLS) requirements. The persistence of severe tensile stresses in the bottom fiber of the deck slab even after applying the maximum code-allowed prestressing force renders the current box girder typology structurally inadequate.

Consequently, this study concludes that conventional prestressing has reached its physical and technical limits for this span configuration. Adopting an extradosed or tendon stayed bridge system emerges as the most viable and structurally sound alternative to effectively control stresses, optimize material consumption, and ensure long-term structural integrity.

GENERAL CONCLUSION:

The continuous evolution of modern infrastructure demands engineering solutions that transcend traditional design boundaries, requiring a meticulous balance between structural integrity, economic viability, and environmental sustainability. This thesis embarked on a comprehensive investigation into the analysis and optimization of external continuity tendons in long-span prestressed concrete bridges constructed using the balanced cantilever method. By utilizing a high-scale 660-meter, five-span highway bridge as a practical case study, this research bridged the gap between theoretical optimization frameworks and real-world structural applications.

Throughout the study, a rigorous, multi-phased methodology was applied in accordance with the Eurocode 2 (EN 1992-2) design standards. The initial phases of the research established a robust preliminary design, carefully selecting a single-cell box girder cross-section with a parabolic depth variation to optimize structural weight while maintaining high torsional rigidity. Subsequently, the structural assessment of the construction stages highlighted the critical nature of internal bottom continuity tendons. The staged construction analysis demonstrated how the precise chronological sequencing of key segment closures and tendon tensioning is paramount to safely managing the transition from a statically determinate cantilever system to a continuous, hyperstatic final structure.

The core focus of this research was the numerical implementation and attempted structural optimization of the external prestressing system during the service stage, executed via the Midas Civil structural analysis software. Extensive comparative analyses were conducted by modifying the geometric variables specifically increasing the total number of tendons, adjusting the longitudinal positioning of span deviators, and modifying anchorage levels in strict accordance with established technical recommendations.

However, contrary to initial theoretical expectations, the findings demonstrated the profound limitations and overall ineffectiveness of external continuity tendons in sufficiently mitigating tensile stresses for this specific high-scale bridge configuration. The results of the analytical framework revealed several critical structural realities:

- **Ineffectiveness in Stress Reduction:** Despite systematically modifying the external tendon profiles and significantly increasing their quantity, the tensile stresses at critical cross-sections were not sufficiently reduced to satisfy the strict requirements of the Serviceability Limit State (SLS).
- **Impact of Secondary Effects:** The structural response indicated that the secondary moments and parasitic effects generated by the geometry of the external tendons often counteracted the intended beneficial compressive forces, thereby neutralizing their overall efficiency in stress control.
- **Economic and Environmental Inviability:** Attempting to force the design to meet code limits purely by adding more external tendons proved structurally unfeasible and economically counterproductive. It led to an excessive demand for prestressing steel mass, fundamentally negating the desired material optimization and sustainability goals.

In conclusion, while the integration of unbonded external tendons offers well-known advantages regarding future inspect ability and maintenance, this thesis proves their structural inadequacy as a standalone solution for controlling tensile stresses in certain long-span balanced cantilever bridges. The study clearly establishes that for spans of this magnitude, manipulating the external prestressing layout cannot compensate for excessive tensile demands. Consequently, future engineering approaches for similar structures must look beyond external tendons, prioritizing either a significantly reinforced internal prestressing layout, an iterative modification of the concrete cross-section dimensions, or the adoption of fundamentally different hybrid structural systems to ensure absolute safety and long-term serviceability.

BIBLIOGRAPHY

[1] R. M. Barker and J. A. Puckett, Design of Highway Bridges: An LRFD Approach, Hoboken: John Wiley & Sons, 2013.

[2] E. G. nawy, Prestressed Concrete: A Fundamental Approach, Upper Saddle River: Prentice Hall, 2010.

[3] W.-F. Chen and Lian, Bridge Engineering Handbook, Boca Raton: CRC Press, 2014.

[4] M. P. Virlogeux, "External prestressing: From construction history to modern technique and technology," in *External Prestressing in Bridges*, American Concrete Institute, 1990, pp. 1-60.

[5] E. Freyssinet, Une révolution dans l'art de bâtir: les bétons précontraints, Paris: Éditions Science et Industrie, 1936.

[6] J. Muller, "Construction of Long Key Bridge," *PCI Journal,* vol. 25, no. 6, pp. 97-111, 1980.

[7] Fédération internationale du béton (fib), "External prestressing in bridges," Fédération internationale du béton (fib), Lausanne, 2002.

[8] MIDAS Civil, "Structural Systems and Design Considerations for Balanced Cantilever Bridges," 2026.

[9] Gimsing, Niels J, Georgakis and Christos T, Cable Supported Bridges: Concepts and Design, Chichester: John Wiley & Sons, 2012.

[10] SERTA, Prestressed Concrete Bridges Builts Using the Cantilever Method, Toulouse: SERTA, 2003.

[11] A. A. o. S. H. a. T. O. (AASHTO), AASHTO LRFD Bridge Design Specifications, 4th Edition, SI Units ed., vol. Single volume, Washington, Washington, DC: American Association of State Highway and Transportation Officials, 2007, p. 1930.

[12] European Committee for Standardization, "EN 1992-1-1: Eurocode 2: Design of concrete structures," CEN, Brussels, 2004.

[13] Europien Committee for Standardization , "EN 1992-1-1: Eurocode 2: Design of concrete structures," CEN, Brussels, 2004.

[14] CEN , "Eurocode - Basis os structural design (EN 1990:2002)," CEN, Brussels, 2002.

[15] CEN, "Eurocode 1: Actions on structures- Par 1-1 : General actions Desinities, self-weight, imposed loads for buildins (EN 1991-1-1:2002)," CEN, Brussels, 2002.

[16] C. e. al., Designer's Guide to Eurocode 1: Actions on Bridges (EN 1991, 1-3 to 1-7 and 3), London: Thomas Telford, 2010.

[17] CEN, "Eurocode 2: Design of concret structures Part 1-1: General rules and rules for building (EN 1991-1-1:2004)," CEN, Brussels, 2004.

[18] CEN, "Eurocode 7: Geotechnical design Part 1: General rules (EN 1997-1:2004)," CEN, Brussels, 2004.

[19] CEN, "Eurocode Basis of structural design (EN 1990:2002)," CEN, Brussels, 2002.

[20] CEN, "Eurocode 1: Actions on structures Part 1-4:General actions-Wind loads," CEN, Brussels, 2005.

[21] CEN, "Eurocode 1: Actions on structures Part 1-3:General actions-Snow loads," CEN, Brussels, 2003.

[22] CEN, "Eurocode 1: Actions on structures Part 1-5:General actions-Thermal actions," CEN, Brussels, 2003.

[23] J.-A. Calgaro, Designer's to Eurocode : Actions on Bridges (EN 1991-2), London: Thomas Telford, 2010.

[24] CEN, "Eurocode 1: Actions on structures- part 2: Traffic loads on bridges (EN 1991-2:2003)," CEN, Brussels, 2003.

[25] J.-A. Calagaro, Designer's to Eurocode : Actions on Bridges (EN 1991-2), London: Thomas Telford, 2010.

[26] SERTA , "Cours de ponts: Conception-calcul des ponts en beton precontraint par precontrainte exterieure," SERTA, Bagneux, France, 1990.

[27] A. Luis Bernardo Fargier Gabaldo´n and a. E. C. n-Valero, "Sizing Continuity Tendons in Segmental Cantilever Bridges," *ASCE Practice Periodical on Structural Design and Construction,* vol. 29, no. DOI: 10.1061/PPSCFX.SCENG-1351, 2024.

[28] L. B. Fargier Gabaldón, J. F. Rosales Pérez and J. Kingland Paredes, "Moment redistribution in segmental cantilever bridges: Simplified approach," *Pract. Period. Struct. Des. Constr.,* vol. 25, no. 3, p. 06020005, 2020.

[29] CEN, Eurocode 2: Design of Concrete Syructures Par 1-1 : General Rules and Rules for Buildings Section 3.1.2 (EN1992-1-1 Sec3.1.2), Brussels: European Committee for Standardization (CEN), 2004.

[30] CEN, Eurocode 2: Design of Concrete Syructures Par 1-1 : General Rules and Rules for Buildings, Section 3.1.3, Table 3.1, Brussels: CEN, 2004.

[31] CEN, Eurocode 2: EN1992-1-1, Section 5.10.2.

[32] CEN, Eurocode 2: EN1992-2, Section 7.3.1, Brussels: CEN, 2004.

[33] CEN, Eurocode 2: EN1992-1-1, Section 5.10.3, Brussels: CEN, 2004.

[34] CEN, Eurocode 2: EN1992-1-1, Section 3.1.6, Table 3.1, Brussels: CEN, 2004.

[35] CEN, Eurocode 2: EN1992-1-1, Section 7.3.2 & Section 2.4.2.2, Brussels: CEN, 2004.

[36] R. S. B. e. al, "R. S. Bursi et al., "External post-tensioning retrofit and high-and low-cycle fatigue study of connections and joints in steel-concrete box-girder bridges."".

[37] CEN, "Eurocode1: Actions on structures-part 1-1: General action - Densities, self-weight, impossed loads for buildings (EN 1991-1-1:2002)," CEN, Brussels, 2002.

[38] CEN, "Eurocode 1: Actions on structures Part 2: Traffic loads on bridges (En 1991-2:2003)," CEN, Brussels, 2003.

[39] A. O. a. C. Antoun, "Pont de Jinja en Ouganda : Pont en encorbellement successif," École des Ponts ParisTech, 2024.

Software:

- **MIDAS Civil Version 2022**
- **AutoCAD Version 2022**

Websites:

https://www.setra.fr/

https://www.eurocode2.fr/

https://AASHTO_LRFD_Standards