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THEME

Study of a prestressed concrete girder bridge crossing Oued Laadar at PK 289+700 on the RN 83

Étude d'un pont à poutres en béton précontraint franchissant Oued Laadar au PK 289+700 sur la RN 83

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Dedication

Above all, I thank God

I dedicate this modest work to my parents especially My Dear Father

To my supervisor: Houhou Nabil

To My familly and friends

And last but not least to all those who care for me

Thank you

Acknowledgements

We first and foremost thank God, who guided us along the right path throughout this journey and granted us the strenght and preservance to compmlete this work.

We would like to express our sincere gratitude to our supervisor: Pr. Houhou Nabil who helped us carry out this work.

We would like to thank all the teachers who contributed to our education, as well as all those who, from near or far, played a part in the completion of this work.

I would also like to thank the members of the jury who have done me the honor of reviewing this work.

Abstract

As part of the rehabilitation and modernazation efforts for national roads, the Directorate of Public Works in Biskra has initiated the duplication of the RN 83, a road that connects Biskra province to Aïn Naga, passing through the Oued LAAdaar . The primary objective of this work is to study a bridge design incorporating prestressed concrete girders, which have been constructed using the post-tensioning method.

It is worth emphasizing that this study was proposed by the Algerian company EPE/SPA-SAPTA, a company specializing in the design and realization of civil engineering structures. The bridge consists of two spans, each measuring 26.55 meters, supported by seven prestressed concrete girders, spaced 1.50 meters apart. The structure includes a 25 cm thick slab.

We proceeded with the design of this structure in order to ensure that the bridge would meet a broad range of requirements, including economic, technical, and environmental considerations. The results of these calculations demonstrate that the proposed design of a prestressed concrete girder bridge, utilizing the post-tensioning method, offers significant advantages in terms of cost-effectiveness, construction time, and aesthetic appeal.

ملخص

في إطار جهود إعادة تأهيل و عصرنة الطرق الوطنية، شرعت مديرية الأشغال العمومية ببسكرة في مضاعفة الطريق الوطني رقم 83، وهو طريق رئيسي يربط ولاية بسكرة بعين الناقة، مرورا بوادي عدار. الهدف الأساسي من هذا العمل هو دراسة تصميم جسر يضم عوارض خرسانية مسبقة الإجهاد، والتي تم بناؤها باستخدام طريقة الشد المسبق.

ومن الجدير بالذكر أن هذه الدراسة اقترحتها الشركة الجزائرية EPE/SPA-SAPTA و هي شركة متخصصة في تصميم وانجاز المنشأت الفنية. يتكون الجسر من امتدادين، يبلغ طول كل منهما 26.55 متًرا، مدعومين بسبعة عوارض خرسانية مسبقة الإجهاد، متباعدة بمسافة 1.04 متًرا. يتضمن الهيكل بلاطة من الخرسانة بسمك 20 سم

لقد شرعنا في تحديد أبعاد هذا الهيكل لضمان أن يلبي الجسر مجموعة واسعة من المتطلبات، بما في ذلك الاعتبارات الاقتصادية والفنية والبيئية. وتوضح نتائج هذه الحسابات أن التصميم المقترح لجسر العارضة الخرسانية المجهدة مسبقًا، يوفر مزايا كبيرة من حيث الفعالية من حيث التكلفة ووقت البناء والجاذبية الجمالية.

Résumé

Dans le cadre des efforts de réhabilitation et de modernisation des routes nationales, la Direction des Travaux Publics de Biskra a lancé le projet de dédoublement de la RN 83, une route reliant la wilaya de Biskra à Aïn Naga, en traversant l'oued LAADAAR. L'objectif principal de ce travail est d'étudier le dimensionnement d'un pont en poutres en béton précontraint, réalisé selon la méthode de post-tension.

Il convient de souligner que cette étude a été proposée par l'entreprise algérienne EPE/SPA-SAPTA, spécialisée dans la conception et la réalisation d'ouvrages de génie civil. Le pont se compose de deux travées, chacune de 26,55 mètres, reposant sur sept poutres en béton précontraint espacées de 1,50 mètre. L'ouvrage comprend également une dalle de 25 cm d'épaisseur.

Nous avons procédé au dimensionnement de cette structure afin de garantir que le pont réponde à un large éventail d'exigences, qu'elles soient économiques, techniques ou environnementales. Les résultats de ces calculs montrent que le choix d'un pont en poutres précontraintes, réalisé par post-tension, présente des avantages notables en termes de rentabilité, de temps de construction et d'esthétique.

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GENERAL INTRODUCTION

In general we call a bridge every stucture that is built over a natural or an artificial obstacle to allow people and vehicles to cross from one side to another. A thorough understanding of the main types of structures, their areas of application, and their pre-dimensioning methods is essential for undertaking the design studies of a bridge at a given site, depending on specific needs, different types of bridges can be distinguished, such as:

- Culvert: A structure with small dimensions used for small spans.
- Slab bridge (pont a dalle): A slab bridge is a type of bridge where the structure absorbs loads by resisting bending, with a solid, continuous concrete slab as the main load-bearing component.
- Girder bridges (Ponts a poutres): A girder bridge is a type of bridge in which the primary load-bearing elements are horizontal beams, known as girders; which absorb loads through bending resistance.
- Viaduct: A viaduct is a high-level bridge consisting of multiple spans, which can include suspension bridges, cable-stayed bridges, etc.

Modern bridge engineering has evolved significantly through the use of advanced materials and innovative construction methods such as pre-stressed concrete. These techniques allow for the creation of more durable, efficient, and longer-lasting structures. Our project focuses specifically on a two-span, simply supported girder bridge, constructed using prestressed concrete girders a prime example of how these modern advancements are applied in practice. By studying this structure, we aim to explore how pre-stressing enhances performance and efficiency in bridge design, while also considering the impact of these methods on both construction and long-term durability.

Chapter I

PROJECT PRESENTATION

I. Project presentation:

The project consists of designing and studying a prestressed concrete bridge with prefabricated girders to cross the Oued LAADAR. It will be an integral part of the construction of the doubling of the RN83 over 80 km connecting Ain Naga and Biskra at PK 289+700. Our structure is made up of a two-span identical isostatic bridge, with each span measuring 25.5 meters in length and 10.54 meters in width, a drivable width of 7.91 meters, and two sidewalks of 1.05 meters each. The superstructure consists of a deck made up of 7 girdrs, each 27meters in length and 1.3 meters in height, prefabricated and post-tensioned for prestressing, with a spacing of

1.50 meters between them.

Fig. I.1. Plan view of the structure on Google Earth

II. Site Survey:

The design engineer responsible for the study must have access to all relevant data of the crossing in order to proceed under optimal conditions.

II.1 Collection of natural data:

The engineer's site visit is essential to the project's development. The key information to be gathered includes:

a) The topography:

In the case of crossing a watercourse, the hydrological study provides the necessary data to determine the known high water levels. According to the hydrological report prepared by the Public Economic Enterprise, Center for Studies and Urban Planning Realizations (URBACO) in Constantine, the bridge height is set at 3 meters, with a maximum flow rate of 2489.25 m^3/s.

b) Climatic data:

- **Temperature:**

It is evident that temperature variations are taken into account when calculating the structures, as they impact the dimensions of the pavement joint and the bearing devices (Bearings). The structure is considered to be located in an area with variable temperatures.

- **Snow:**

Bridges calculation does not take into consediration the effects of snow, however they can occur in certain specific cases (the structure is still under construction). Our work is located in a very close to the coast site, so there is no snow.

- **Wind:**

The forces generated on structures by wind are set by the load regulations, R.C.P.R article 3.2.1:

2.00 KN/m2 for structures in service

1.25 KN/m2 for structures under construction

- **Seismological data:**

According to the Algerian paraseismic regulation R.P.O.A 2008, the country has been divided into five (05) seismic zones:

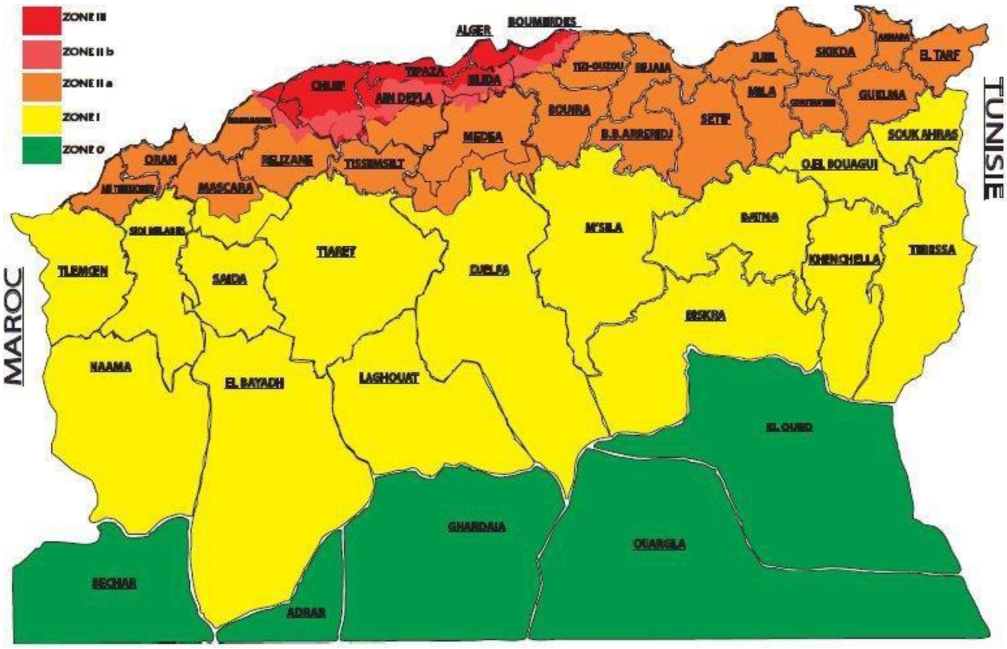


Fig I.2 seismic zones in Algeria

zone 0: negligible

zone 1: low

zone 2a: medium

zone 2b: high

zone 3: very high

The minimum level of seismic protection granted to a structure depends on its location and its importance with respect to the objectives set by the location.

Any structure exposed to the scope of application of the Algerian earthquake regulations, RPA 99 version 2003, must be classified in one of the four groups defined below:

group 1a: structures of vital importance

group 1b: structures of great importance

group 2: standard structures of medium importance

group 3: structures of low importance

- **Functional data:**

Which are set by the project owner, taking into concediration what is imposed or desired by the various departments concerned. It brings together all the characteristics allowing the bridge to ensure its function as a crossing structure when it is put into service. In general, the main elements to be considered are: the plan view, the longitudinal profile of the roadway, and the cross-sectional profile.

- **The plan view:**

The plan layout represents the line that defines the geometry of the supported roadway's axis, drawn on a site plan and identified by the coordinates of its key points. This axis follows a conventional standard.

The structure's geometry consists of a straight segment measuring 67.12 meters in length.

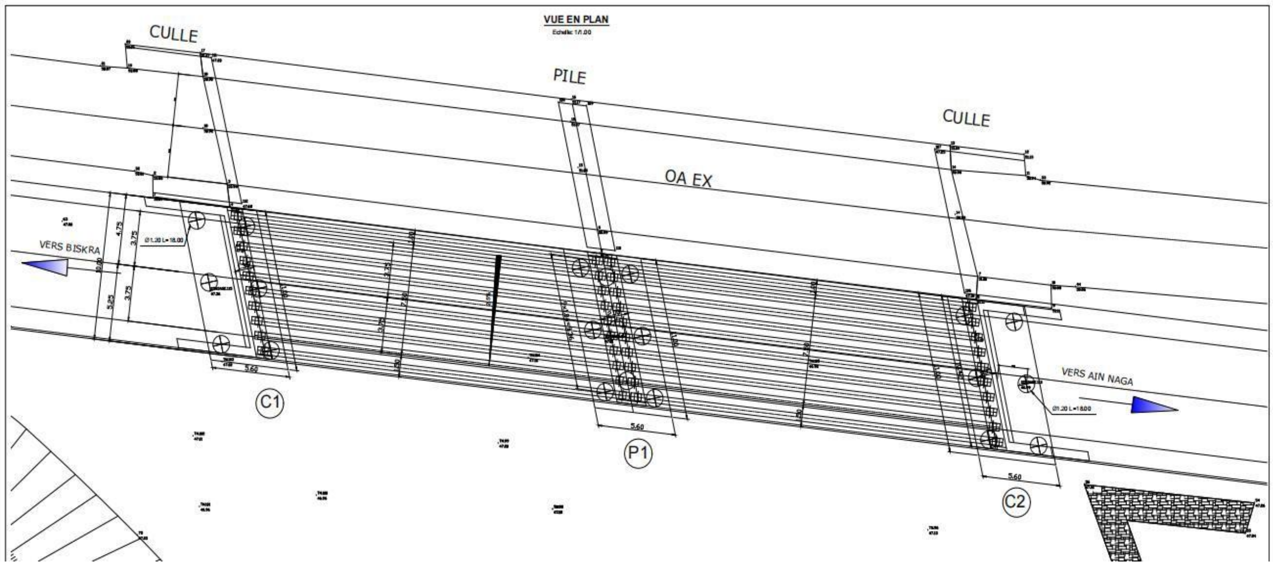


Fig I.3 plan layout

- **Longitudinal profile:**

The longitudinal profile follows the structure’s axis and represents its elevation in relation to the plan layout. Its design must take into account multiple factors, including the functional requirements of the obstacle it spans and the natural constraints of the site.

- **The cross-sectional profile:**

The cross-sectional profile encompasses all elements that define the geometry and infrastructure of the roadway in the transverse direction.

At the preliminary design stage, it is crucial to determine the carriageway width, as well as to establish the dimensions of sidewalks and traffic lanes. These specifications must be clearly defined before proceeding with the development of the detailed preliminary design.

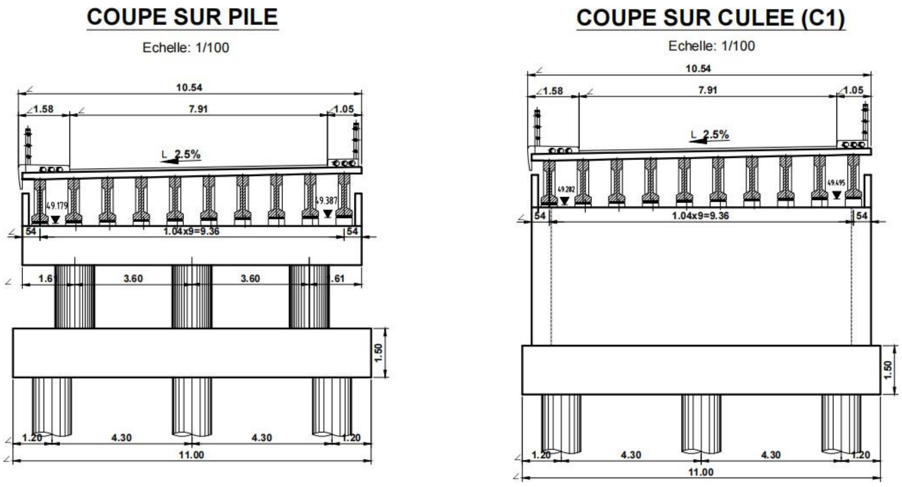


Fig I.4 cross-sectional profile

- Drivable width: 7.91m
- Number of traffic lanes: 2 lanes
- Two sidewalks, of 1.58, 1.05 meters wide respectively
- Slope of 2.5%

Note: in this thesis the used bridge is modefied keeping the same dimensions with different number of girders (7 girders)

d) Geological and Geotechnical Data:

A comprehensive understanding of the ground characteristics is crucial. These data significantly impact the selection of foundations and the structural design. They are typically obtained through pressuremeter tests and core drilling surveys. The geotechnical investigation program, established by EPE-SERSID SPA within the framework of the NR 83 widening project, concerning the proposed beam bridge at KP 289+700, includes:

- A 25-meter deep borehole survey
- A 25-meter deep pressiometric survey with pressiometric tests conducted every 1.5m

e) Main characteristics of the bridge:

The span of the bridge..26.5m

Length of the prestressed girders 27m

Height of the girders... 1.30m

Number of girders 7

The spacing between girders 1.50m

Width of the roadway...7.91m

Width of the sidewalks 1.05m, 1.58m

Total width of the bridge...10.54m

Thickness of the slab 0.25m

f) Constructive decisions:

This study focuses on the construction of a two-span structure, each spanning 27.5 meters. The deck is composed of seven prestressed concrete girders, spaced 1.50 meters apart, with an individual length of 27.5 meters. A transverse slope of 2.5% is achieved along the entire structure through variations in the thickness of the bituminous concrete. The abutments are of the 'wall abutment' type and are cast in place using reinforced concrete. The front wall has a height of 2.95 meters on the Biskra side and 3.45 meters on the Ain Naga side. The pier is designed as a 'column pier,' with shafts measuring 1.95 meters in height and founded on a deep footing.

To ensure structural stability, bored piles with a length of 18 meters and a diameter of 1.2 meters have been incorporated beneath the foundations of both the abutments and the pier.

Based on the soil analysis report, the use of sulfate-resistant cement (CRS) is required for the concrete used in the infrastructure to enhance durability and resistance to aggressive environmental conditions.

Chapter II

GENERAL DESIGN

I. Choosing the bridge type:

After gathering all relevant data, the designer evaluates feasible solutions based on technical, economic, and architectural criteria. Given the diversity of bridge types, various design options are considered to identify the most cost-effective and structurally efficient solution. This selection process involves analyzing constraints, balancing multiple parameters, and ultimately relying on the engineer's expertise to determine the most suitable approach for further study.

The selection of the structure type takes into consideration the following key elements:

The obstacle to be crossed.

The carried roadway: bridge for roads, railways, etc.

The primary construction materials.

The mechanical function: isostatic or hyperstatic.

The layout: straight, skewed, or curved.

The expected lifespan: permanent or temporary.

Several design proposals are considered, and an elimination process is carried out to exclude structures that do not meet the required conditions:

a) Composite bridges :

For relatively wide decks (typically over 13 meters), the most common approach is to connect a concrete slab longitudinally to two main girders and transversely to cross beams spaced approximately 4 meters apart, forming a composite bridge with cross beams.

For narrower decks, the slab is connected only to the two main girders, which are spaced about 0.55 times the slab width and linked by diaphragms. This configuration results in a composite bridge with diaphragms.

Advantages:

- Large spans, up to 80 meters.
- Easy fabrication of solid-web girders in the factory.
- Fast construction and easy installation.
- Reduced need for scaffolding.
- Lightweight structure.
- More cost-effective compared to metal bridges

Disadvantages:

- Risk of corrosion.
- Requires rigorous and costly maintenance.
- Requires highly skilled labor.

b) Pre-tensioned prestressed concrete girder bridges:

Decks composed of prefabricated prestressed girders with bonded wires or strands provide an effective solution for spans of up to approximately 30 meters. In this method, the prestressing cables are tensioned between two firmly anchored blocks before the concrete is poured. This technique is primarily used in precast production facilities to manufacture repetitive structural elements.

c) Post-tensioned prestressed concrete girder bridges:

The girders, designed as flanged girders , are structurally integrated through the deck slab beneath the roadway and diaphragms positioned at the ends of each span, aligned with the support lines. These diaphragms enhance the torsional rigidity of the girders and facilitate deck lifting using hydraulic jacks when necessary for the replacement of bearing devices.

This construction method involves tensioning the prestressing cables after the concrete has been poured and fully cured, utilizing the structural element itself as the reaction surface for compression. Primarily used for large-scale structures, this technique is typically implemented on-site.

d) Prestressed Concrete Slab Bridges:

This type of bridge is not suitable for our case, as it is best suited for span lengths between 15 and 23 meters, which represents its most economical range.

Compared to girder bridges, independent-span slab bridges are only considered for moderate openings and when achieving a high slenderness ratio is essential.

Advantages:

- Efficient material usage, as there is no unnecessary concrete.
- High-strength prestressing steel is more cost-effective than traditional reinforced concrete steel for the same load-bearing capacity.
- Prestressing forces counteract external loads, reducing deformations.
- Prefabricated elements can be assembled without scaffolding.
- Allows for longer spans compared to reinforced concrete structures.
- Minimizes cracking as a general principle.
- Requires minimal maintenance compared to other bridge types.

e) Reinforced Concrete Girder Bridges:

In this type of bridge, the deck consists of longitudinal girders with spans of up to 20 meters. However, this option is not suitable for our case due to our span, which exceeds the permissible limit for this type of structure. Additionally, the complexity of formwork further limits its feasibility.

f) Reinforced Concrete Slab Bridges:

- Reinforced concrete slab bridges are best suited for spans between 15 and 20 meters. However, this method is not feasible in our case for the following reasons:
- High material consumption: Requires significantly more concrete and steel for spans exceeding 20 meters.
- Best suited for skewed bridges, making it less ideal for our specific design requirements.

II Abutment Design:

Abutments are essential structural components of a bridge, providing end support for the deck. Their primary role is to ensure a seamless connection between the roadway and the bridge, maintaining continuity in the driving surface.

II.1 Types of Abutments:

Abutments are generally classified into two main types

1. Embedded Abutments: These are buried within the approach embankment and primarily serve a load-bearing function. Due to their positioning, they experience minimal horizontal forces and earth pressure.
2. Abutment with front wall (backfilled):

It serves both a load-bearing function and a retaining function for the embankment.

The selection of the abutment can be made progressively, based on an analysis that considers:

- The type and construction method of the deck.
- The natural constraints of the site.
- The functional constraints of the structure

II. 2 Abutement pre-sizing:

❖ strike-guard wall:

Wall Height

height of (girder + slab) + height of (bearing device + bearing plinth)= 1.30+0.25+0.04+0.31= 1.90m. Therfore H=1.90m

Thickness (e):

- for a height $H \leq 1$ m: e = 0.20 m
- or a height of $1 \text{ m} \leq H \leq 2$ m: e = 0.20m to 0.30
- for a height of $2 \text{ m} \leq H \leq 3$ m: e = 0.30m

in our bridge we consider e=0.32m

- Length corresponds to deck width: 10.54m

❖ Headwall

It is a thick structural wall (or thick web) with a typical thickness ranging from 0.8 m to 1.2 m depending on the height.

❖ The footing (or base slab):

- Thickness: 1.5 m
- Width: 5.6 m
- Length: 11 m
- Blinding concrete: 0.10 m

❖ Wing wall:

They are walls with either constant or variable thickness. Fixed to the strike-guard wall, the length of the free-standing part should not exceed 7 to 8 meters. The thickness of the wing walls is determined based on mechanical strength considerations and typically ranges between 30 cm and 80 cm.

❖ Transition slab:

The transition slab serves to provide a smooth passage between the flexible medium (roadway) and the rigid structure (bridge). The length of the slab is determined based on the slope of the material used for the backfill.

In our case the length L=5m, th e thikness e= 0.30 in general.

III. Design of Support Elements:

III.1 Piers:

Are an intermediate support which is responsible for transferring the forces from loads and surcharges to the foundation soil. The design of piers depends on numerous factors, including the construction method of the deck, whether the location is urban or rural, the height of the span to be crossed, the execution method of the foundations, and the connection with the deck.

❖ Selection of Piers:

There are two types of piers: wall-type piers and column piers.

A pier, as an intermediate support, plays a crucial role in transferring loads and overloads from the bridge deck to the foundation soil. It contributes to the overall structural resistance of the bridge. The design of piers is influenced by numerous factors, including:

- Environmental conditions: Aquatic or terrestrial.
- Deck construction method.
- Location context: Urban or rural.
- Height of the span to be crossed.
- Foundation construction method.
- Connection with the deck.

Pier Selection Criteria: the choice of pier type is based on four key criteria

- Geometric criteria
- Mechanical criteria
- Economic criteria
- Aesthetic criteria

For our case it is decided that:

Best bridge type solution: Post-tensioned prestressed concrete girder bridges

Type of abutement: Abutment with front wall (backfilled)

As for piers we opt for a column pier (with three shafts)

III.2 pre-sizing of the pier:

❖ Pier cap:

It is the element on which the girders rest and ensures the transmission of loads to the columns.

- Height: 1m < H < 1.6m, we take H = 1.2m.
- Width: 2m.
- Length: Equal to the length of the deck, so Lch = 10.54m.

❖ Pier columns:

Their role is to transfer the forces to the footing. They are cylindrical in shape and have geometric dimensions, with a generally constant diameter of e = 1.2m. L2 is the spacing between the centers of the two piers, so: L2 = 3.6m. Height: H = 1.95m.

Chapter III

MATERIALS CHARACTERISTICS

I Introduction:

The selection of construction materials plays a crucial role in shaping the bridge design. This section outlines the properties of concrete and both active and passive reinforcement steels, as they directly influence our structural calculations.

II Used Documents and Regulations:

B.A.E.L. 91 (modified in 1999): These are technical regulations for the design and calculation of reinforced concrete structures, based on the limit state method.

B.P.E.L. 91: These are technical regulations for the design and calculation of prestressed concrete structures, following the limit state method.

Fascicle 61, Title II of the CPC ("Common Specifications Document") for load considerations.

Fascicle 62, Title V: Technical regulations for the design and calculation of foundations for civil engineering structures.

III Concrete:

Concrete is one of the most widely used materials in the construction industry. It consists of a carefully proportioned mixture of cement, aggregates (sand and gravel), water, and, when required, admixtures (such as Sika) to enhance specific properties. The classification of concrete is primarily based on its compressive strength at 28 days, denoted as fc28 (or f 'c: f prime c).

Prestressed concrete exhibits similar characteristics to reinforced concrete but is designed to withstand higher stress levels. In this system, the cross-section is subjected to initial compressive stresses, effectively counterbalancing the tensile stresses induced by applied loads and overloads, thereby improving structural performance and durability.

For prestressed concrete applications, the cement dosage is set at 400 kg/m³ to meet design and performance requirements.

Density: The unit weight of reinforced concrete is $\gamma = 2.5$ t/m³.

The selection of concrete mix proportions is determined based on local material availability and must adhere to quality control standards to ensure compliance with structural and durability requirements.

- **The characteristic compressive strength:**

For concrete aged d days, we have:

$$Fcj = \begin{cases} 35 \text{ mpa, } 27 \text{ mpa} & \text{if } j \geq 28 \text{ d} \\ Fcj = \frac{j}{4,76+0,83j} Fcj & \text{if } < 28 \text{ d} \end{cases}$$

With:

$$Fc28 = \begin{cases} \text{35 MPA for the superstructure concrete} \\ \text{27 MPA For the concrete of supports and foundation} \end{cases}$$

➢ Characteristic tensile strength:

Tensile strength is related to compressive strength:

$$ft28 = \begin{cases} 0.6 + 0.06fcj = 0.6 + 0.06(27) = 2.2 \text{ MPa (for fc28= 27 MPa)} \\ 0.6 + 0.06fcj = 0.6 + 0.06(35) = 2.7 \text{ MPa (for fc28= 35 MPa)} \end{cases}$$

➢ Concrete longitudinal deformation:

A longitudinal deformation modulus for concrete, denoted as 'Eij,' is considered and defined according to B.P.E.L. regulations as follows:

- Instantaneous deformation modulus:

$Etj = 11000 \sqrt[3]{fc28}$ MPa

$$Etj = \begin{cases} \text{36000 MPa for girders} \\ \text{33000 MPa for fondation and supports} \end{cases}$$

- Long-term deformation modulus:

$E_{vj} = 3700 \sqrt[3]{f_{c28}}$ MPa

$$E_{vj} = \begin{cases} 12000 \text{ MPa for girders} \\ 11000 \text{ MPa for fondation and supports} \end{cases}$$

➢ **Transverse deformation of concrete:**

It is given by the following formula: $G = E / 2(1+\nu)$

➢ **Poisson's ratio:**

Poisson's ratio (ν) quantifies the relative transverse deformation of a material in response to longitudinal strain. For concrete under instantaneous loading, is typically around 0.3 but gradually decreases over time, stabilizing near 0.2. In the case of cracked concrete, Poisson's ratio drops to zero. In prestressed concrete design, a value of **ν=0,2** is adopted, for uncracked concrete under Serviceability Limit State (SLS); (ELS) conditions, while **ν=0** is used for cracked concrete under Ultimate Limit State (ULS); (ELU) conditions.

IV. Steel:

The steel used in prestressed concrete structures is of two different types:

- Prestressing steel (active reinforcement): the steel that generates and maintains the prestress.
- Non-prestressed reinforcement (passive reinforcement): the steel that handles additional loads such as shear and helps control cracking.

IV .1 Passive reinforcement:

passive reinforcement is similar to the reinforcement used in reinforced concrete. It is only subjected to tension under external loads.

➢ **Yield Strength:**

The steels used are high-bond reinforcing steels of class FeE500, with a yield strength of 500 MPa

➢ **Longitudinal Elastic Modulus of Steel:** $E_s = 2.1\ 10^5$ Mpa

➢ Ultimate Tensile Strength (UTS):

In the Ultimate Limit State (ULS) calculation, a safety factor **γs** is introduced

σs ≤ fe/γs with:

- **γs** =1.00 In accidental situations
- **γs** =1.15 In Permanent or temporary situations

In the Serviceability Limit State (SLS) calculation:

Cracking with limited impact (peu nuisible): σs ≤ fe/γs

Cracking with serviceability concerns (prejudiciable):

$\sigma s = \min\left(\frac{2}{3fe}, 110(nftj)^{1/2}\right)$

Cracking with structural implications (très prejudiciable):

$\sigma s = \min\left(\frac{2}{2fe}, 90(nftj)^{1/2}\right)$

With: n=1 for welded steel mesh and smooth (or plain) bars.

n= 1.6 for high-adhesion steels.

IV.2 Active reinforcement:

Active reinforcements are high-strength steel reinforcements utilized in prestressed concrete structures. These reinforcements remain under tension even without external loads. They are classified into three main types: wires, bars, and strands.

The initial prestress to be considered in the calculations is given by the following formula:

Po= (0,8fprg, 0,9fpeg).

fprg (fpu): The guaranteed ultimate strength of prestressing steel.

fpeg (fpu): The yield strength of prestressing steel.

➢ Yield Strength:

As these steels lack a distinct plasticity plateau, their yield strength is defined as the stress at which a residual strain of 0.1% occurs. Conventionally, the yield strength of prestressing steels is considered to be 89% of their guaranteed ultimate tensile strength.

➢ Young's modulus:

The longitudinal elastic modulus EP of prestressing steels is taken as:

EP= 200000 MPa for bars.

EP= 190000 MPa for strands.

Chapter IV

GIRDER CHARACTERISTICS

I. Introduction:

In prestressed girder bridges, the girders are prestressed using bonded cables or wires, either through post-tensioning or pre-tensioning, respectively.they are often highly cost-effective for spans ranging from 25 to 50 meters when using post-tensioning. Their on-site prefabrication allows for significant cost savings by reducing the need for expensive formwork.

For our bridge, we will consider I-section girders with post-tensioning, spaced at 1.04 meters apart.

II. Preliminary design:

Deck elements design:

- **The slab:**

1. Slab length equal to the girders' slenderness (Ld) LS= 27.5m .
2. The width of the slab is equal to the width of the roadway plus the sidewalk (ld) ls= 10.54m
3. slab thickness 20 < h< 30

- **Sidewalks:**

The role of sidewalks is to protect pedestrians by isolating them. The sidewalk width is taken as 1.05 m.

- **Deck surfacing:**

It primarily consists of a waterproofing layer and a wearing course. The total thickness of the surfacing is 8 cm.

Girders:

According to a SETRA document:

❖ *For Pre-tensioned Prestressed Girders:*

- **girders spacing:**

$0.9 \leq d \leq 1m$, d=1.04

- **Number of girders:**

N= la/d+1

la: the center-to-center spacing between the two edge girders la= 9.36m

N= (9.36/1.04)+1 =10 girders

- **Girders depth (ht) :**

0.8 ≤ hp ≤ 1.60m in our case hp= 1.30m

- **Web thickness:**

E ≥ 14cm in our case: E=15cm

- **Flange width:**

0,5 ≤ Lt ≤0,8m in our case: 0.5m

- **Thickness of the flange:**

In our case Et= 15cm

- **Slab Thickness:**

Eh ≥ 15cm verification or punching shear due to concentrated loads

❖ *For post-tensioned Prestressed Girders:*

- **Girders depth (ht) :**

L/22≤ Hp ≤ L/16

In our case L=27.1m, So 1,23≤ Hp≤1,69 we consider: Hp = 1.30

- **Number of girders:**

The number of girders is determined by the ratio between the deck width and the spacing:
N= la/d+1

With:la= m : Center-to-center distance between the two edge girders; La=9.36m

d : Center-to-center distance between the two girders 1,50 ≤d ≤ 2,50 we consider d=1.50m,
N=7 girders

- **Width of the compression flange (Lm):**

0,6Hp≤ Lm ≤ 0,7 Hp   0.9≤ Lm ≤1.05 we consider: Lm= 1m

- **Thickness of the compression flange (e):**

10≤ e ≤15cm we consider: e=11cm

- **Flange width (l_t):**

50≤Lt≤80   we consider: Lt= 44cm

- **Flange thikness (e_t):**

et≥14cm we consider: et= 18cm

- **web thickness in span (b0):**

18 ≤b0≤25   we consider: b0= 18cm

- **Web thickness at the support (b_0):**

40 ≤b0 ≤50 we consider: b0= 44cm

- **web thickness in the mid-span:**

25 ≤b0≤35 we consider: b0=32cm

- **Gussets:**

These are angled sections designed to improve the cross-section and accommodate the placement of steel reinforcements and prestressing cables,

(45° ≤ α ≤ 60°)

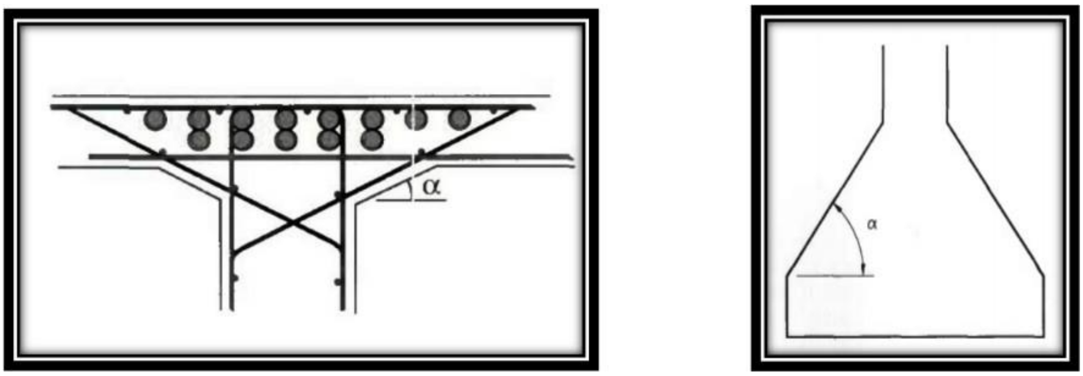


Fig IV 1. (a) compression flange gusset **(b)** Flange gusset

❖ *Flange gusset:*

In the span:α=56.97° e=20cm

At the intermediate support:α=56.3° e=9cm

❖ *Compression flange gusset:*

In the span: α=12° e=6cm

α=45° e=10cm

At the intermediate support: α=45° e=6cm

α=79.05° e=10cm

At the support: α=12° e=6cm

- **Slab thickness:**

$20 \leq Hh \leq 30$ Hh= 25 cm

III. Determining the girder's characteristics:

- **Notation:**

(Δ): The axis taken at the extreme bottom fiber.

I/Δ: Moment of inertia about the bottom fiber axis (I_Δ)

I/Δ =I0+Bi*Z^2

I_0: Moment of inertia about the centroidal axis.

For a rectangular section: : I0= $bh^3/12$

For a triangular section: : I0= $bh^3/36$

B: section of the girder.

B (net) = B (gross) - 5% B (gross)

S/Δ=B*Z

S/Δ: static moment about (Δ)

S/Δ (net)=S/Δ (gross) -5% S/Δ(gross)

V'= $\frac{S/\Delta}{B}$: Distance from the centroid to the extreme bottom fiber.

V=h-V': Distance from the centroid to the extreme top fiber.

IG=IΔ- S/Δ*V'

ρ= IG / BVV': Geometric efficiency of the cross-section.

I^2=IG / B: Gyration radius

A. Girder without slab (hourdis):

- **Characteristics of the end section:**

Table IV.1 End section characteristics

Designation	X	Y	B	Z	S/Δ=B.Z	I0 (cm4)	I/Δ=I0+B.Z2
1	44	130	5720	65	371800	8055666.667	32222666.67
2*2	28	11	308	124.5	38346	3105.666667	4777182.667
3*2	28	6	168	117	19656	168	2299920
B gross			6196				
B net			5886.2				
S/Δ gross					429802		
S/Δ net					408311.9		
I/Δ gross							39299769.33
I/Δ net							37334780.87

Table IV.2 End section characteristics

IG(cm4)	ρ%	v	V'	I ²(cm2)
9485411.81	0.36398493	60.6323434	69.3676566	1530.8928

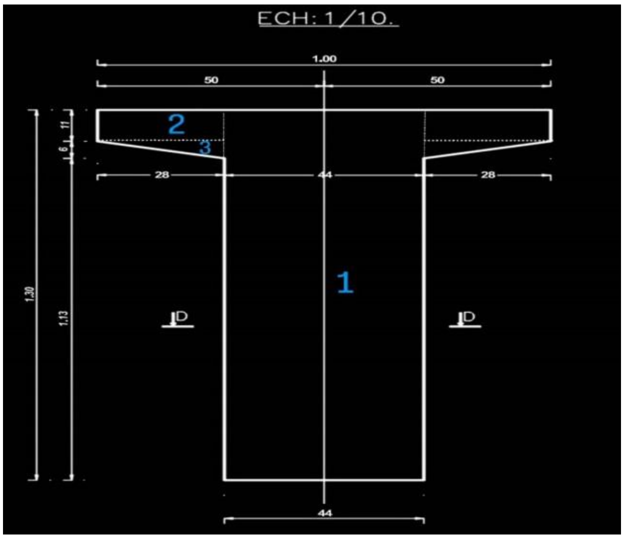


Fig IV 2. End section characteristics

- **Mid-span section:**

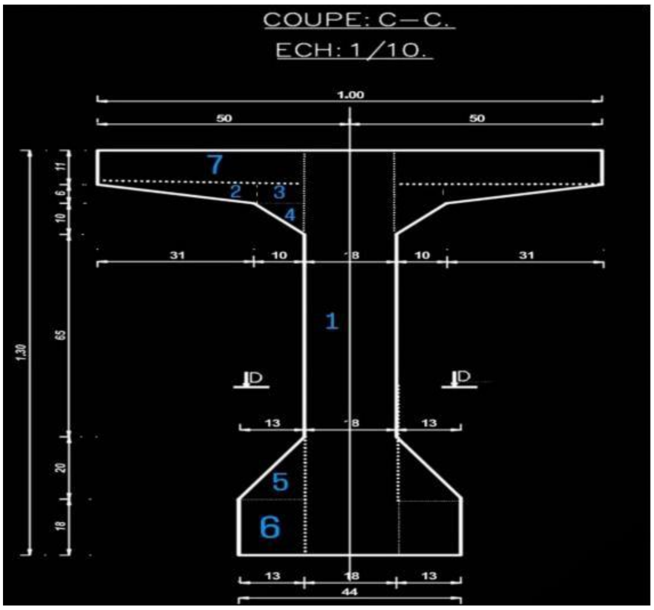


Fig IV 2 Mids-pan section characteristics

Table IV 3 Mid-span section characteristics

Designation	X	Y	B (cm2)	Z(cm)	S/Δ=B.Z	I0(cm4)	I/Δ=I0+B.Z²
1	18	130	2340	65	152100	3295500	13182000
2x2	31	6	186	117	21762	186	2546340
3x2	10	6	60	116	6960	180	807540
4x2	10	10	100	109.67	10967	277.7777778	1203028.668
5x2	13	20	260	24.67	6414.2	2888.888889	161127.2029
6x2	13	18	234	9	2106	6318	25272
7x2	41	11	451	124.5	56,150	4547.583333	6995160.333
B gross			3631				
B net			3449.45				
S/Δ gross					256,459		
S/Δ net					243635.765		
I /Δgross							24920468
I/Δ net							23674444.79

Table IV.4 Mid-span section characteristics

IG(cm4)	V'	v	ρ%	I 2(cm2)
6806707.586	70.6303222	59.3696778	0.44704883	1874.60964

❖ Characteristics of Midsection:

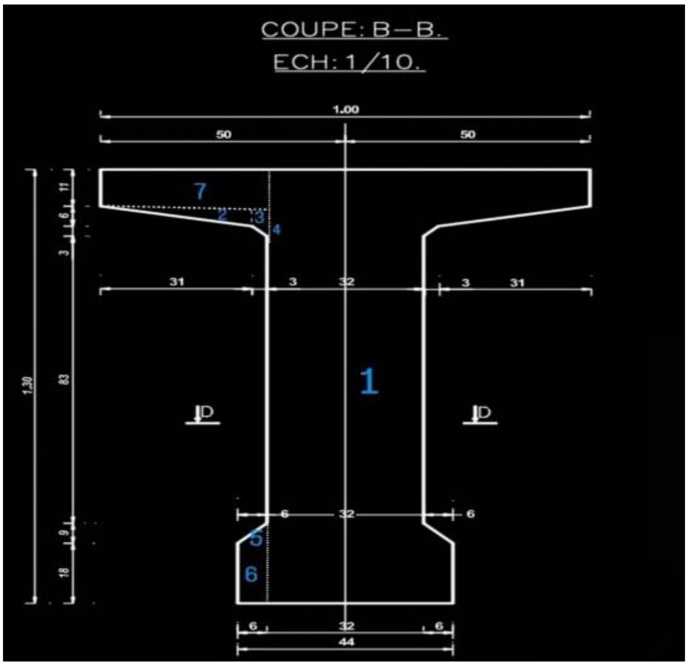


Fig IV 3 Midsection characteristics

Table IV 5 Midsection characteristics

Designation	X	Y	B (cm2)	Z(cm)	S/Δ=B.Z	I0(cm4)	I/Δ=I0+B.Z²
1	32	130	4160	65	270400	5858666.667	23434666.67
2x2	31	6	186	117	21762	186	2546340
3x2	10	6	60	116	6960	180	807540
4x2	10	3	30	112	3360	7.5	376327.5
5x2	6	9	54	21	1134	121.5	23935.5
6x2	6	18	108	9	972	2916	11664
7x2	41	11	451	124.5	56149.5	4547.583333	6995160.333
B gross			5049				
B net			4796.55				
S/Δ gross					360,738		
S/Δ net					342700.625		
I /Δgross							34,195,634
I/Δ net							32485852.3

Table IV.6 Midsection characteristics

IG(cm4)	V'	V	ρ%	I ²(cm2)
8421907.736	71.4473163	58.5526837	0.398724	1668.03481

B. Girder with slab (hourdis):

❖ Characteristics of the end section:

Table IV.7 End section characteristics

Designation	X	Y	B	Z	S/Δ=B.Z	I0 (cm4)	I/Δ=I0+B.Z2
Girder			6196		429802		39299769.3
Slab	150	25	3750	142.5	534375	195312.5	76343750
B gross			9946				
B net			9448.7				
S/Δ gross					964177		
S/Δ net					915968.1		
I/Δ gross							115643519.
I/Δ net							109861343.

Table IV.8 End section characteristics

IG(cm4)	V'	v	ρ%	I 2(cm2)
22175060.93	96.9411824	58.0588176	0.39613195	2229.54564

❖ Mid-span section:

Table IV.9 Mid-span section characteristics

Designation	X	Y	B	Z	S/Δ=B.Z	I0 (cm4)	I/Δ=I0+B.Z²
Girder			3631		256459		24920468
Slab	150	25	3750	142.5	534375	195312.5	76343750
B gross			7381				
B net			7011.9				
S/Δ gross					790834		
S/Δ net					751292.01		
I/Δ gross							101264218
I/Δ net							96201007.29

Table IV.9 Mid-span section characteristics

IG(cm4)	V'	v	ρ%	I 2(cm2)
16530721.24	107.14452	47.8554803	0.43679228	2239.63165

❖ **Characteristics of Midsection:**

Table IV.10 Midsection characteristics

Designation	X	Y	B	Z	S/Δ=B.Z	I0 (cm4)	I/Δ=I0+B.Z²
Girder			5049		360,738		34,195,634
Slab	150	25	3750	142,5	534375	195312.5	76343750
B gross			8799				
B net			8359.0				
S/Δ gross					895,113		
S/Δ net					850356.87		
I/Δ gross							110,539,384
I/Δ net							105012414.8

Table IV.11 Midsection characteristics

IG(cm4)	V'	v	ρ%	I 2(cm2)
19480583.27	101.72889	53.2711104	0.40853812	2213.95423

CHAPTER V

CALCULATION OF LOADS, LIVE LOADS, AND LONGITUDINAL FORCES

I. Calculation of loads:

Permanent loads include the self-weight of the supporting structure, non-load-bearing elements, and fixed installations.

- **Load-bearing elements:** these loads pertain to the deck alone (DL: dead loads).
- **Non-load-bearing elements:** the surfacing, screed, sidewalks, curbs, guardrails, and safety barriers (SCL: supplementary components of dead loads).

I.1 Calculation of permanent loads (G):

a)_Girders:

The length of the girder is 27 m (according to the plan).
So, we divided the length in half, **L = 13.5 m**, to simplify the calculation of **P**.

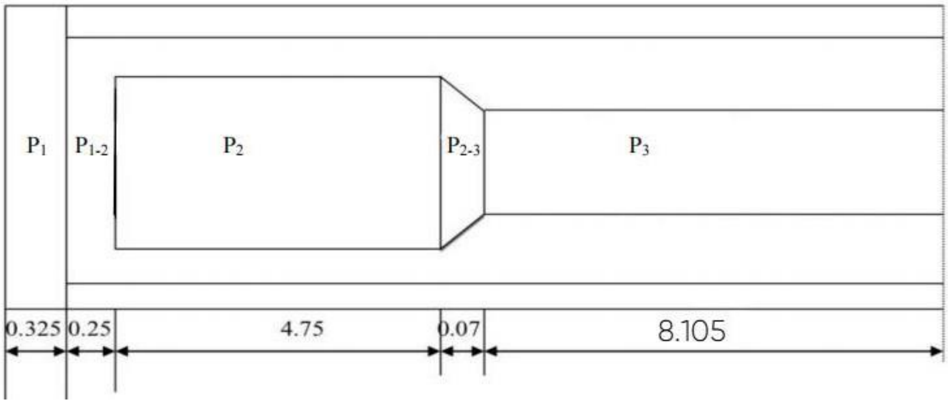


Fig. V.1 Girder's dimensions

Pt =(P1 + P2 + P3 + P12 + P23)× 2

P= γ × Li × Si

γ : unit weight (or specific weight)

Li : the length of the section
Si : the area

End section: S1 = 0.6196m²

Midsection section: S2 = 0. 5049 m²
Mid-span section: S3 = 0. 3631 m²

Therfore :

P1 = 2.5×0.325×0. 6196=0. 503t
P2 = 2.5×4.75×0. 5049 =5,996t
P3 = 2.5×8.105×0. 3631=7.357t

P1-2= $\gamma \times 1 \times \frac{S1+S2}{2} = 2.5 \times 0.25 \times \frac{0.6196+0.5049}{2} = 0{,}351t$

P1-2= $\gamma \times 1 \times \frac{S2+S3}{2} = 2.5 \times 0.07 \times \frac{0.5049+0.3631}{2} = 0{,}076m^2$

For the whole of the girder:

Pt= 2× (0. 503+5,996+7.357+0,351+0,076)= 28.566t

And per linear meter: : P(t/ml) =28.566/27=1.058t/ml. (WG)

The total weight of the girders is: 1.058×7= 7.406 t/lm.

b)_ The slab:

L=27m

e=0.25m

I=10.27m

Pd =0.25 × 10.24 × 2.5 =6.40 t/lm (WS)

C)_Diaphragms:

L=1.8m

e=0.34m

I=0.98m

n=18

PE=0.8 × 0.34 × 2.5 × 0.34= 1.95t

The total weight of the diphrams is: 1.95×18= 26.99 .

Therfore: permanent actions (charges permanentes)

$Cp = PD + Pp = 7.406 + 6.40 = 13.806$t/lm

Cp= 13.806t/lm

I.2 Calculation of the Additional Permanent Loads (CCP):

a) Pavement and Waterproofing:

Weight of the pavement and waterproofing screeds

L=27m

e=0.08m

I=7.90m

Pr = 0.08m × 2.2t/m3×7.90m= 1.400 t/ml

b) _ Sidewalk and parapet:

St (Sc) : cross-sectional area of the sidewalk

Sc (Sp) : cross-sectional area of the parapet

L=27m

e=0.25m

I=0.94m

St= 0.25×0.94= 0.235 m^2 .

Sc= 0,15m^2

S_{t+c} = 0.235+ 0,15= 0.385m^2

Therfore: P_{t+c} = 0.385×2.5×2= 1.925t/lm.

C)_ Guardrail:

0.3t/ml × 2 =0.60t/ml

Therfore:

CCP (APL)= 1.400 +1.925+0.60= 4.00t/lm

TabV.1 Results Table

Designation	Element	Weight (t/m.l.)	Weight (t)
CP (PL)	Girders	7.406	199.962
	Slab	6.40	172.80
	Diaphragms	/	26.99
CCP (APL)	Pavement and Waterproofing	1.400	37.800
	Sidewalk	1.175	31.725
	Guardrail	0.60	16.2
	parapet	0.75	20.25
TOTAL		19,9618	505.727

II. Calculation of Live Loads:

We will calculate the applicable live loads for road bridges supporting one or more carriageways (roadways) as defined by the bridge layout:

- Bridge classification:

Tab V.2 Bridge classification

The classification	Carriageway width (Lr)
1	$L_r \geq 7$ m
2	5,50 m< L_r< 7 m
3	L_r<5,50 m

- The carriageway width (Lr) of our bridge is 7.91 meters, therefore the bridge is classified as Class 1, since Lr > 7 m.
- Number of lanes: $N= \frac{Lc}{3} = \frac{7.91}{3} = 2.63$ therfore: N=2 lanes
- Lane width : $Lv= \frac{Lc}{N} = \frac{7.91}{2} = 3.955$m

We distinguish:

- A (L) live load type.
- B system : (BC, Bt, Br).
- Military load: MC 120.
- Exceptional convoy: D240.
- Sidewalk (footway) live loads.
- Loads due to wind and seismic actions**.**

II.1 Load System A (L):

The A system consists of a uniformly distributed load, whose intensity depends on the loaded length *L*, and is given by the following formula:

$$A\ (L) = a1 \times a2 \times A\ (L)$$

With:

$A(L) = 230 + \frac{36000}{L+12}$ (kg/m2), L : the span =26.55m

$A(L) = 230 + \frac{36000}{26.55+12}$ =1163.8521(kg/m2)

$A(L)$ =1.1638 t/m2 .

a_1: transverse load reduction coefficient, provided in the following table:

Tab.V. 3. transverse load reduction coefficient

	Number of loaded lanes				
Bridge classification	1 lane	2 lanes	3 lanes	4 lanes	≥ 5 lanes
1	1.00	1.00	0.9	0.95	0.7
2	1.00	0.9	-	-	-
3	0.9	0.8	-	-	-

Therefore: a_1= 1

Then the load A(L) is multiplied by the coefficient a2 which is defined by:

$a2 = \frac{V0}{V}$; V: the width of the lane V=3.955m

V_0: as specified in the following table:

Tab.V.4 V_0 table bridge classification

Bridge classification	V_0
1st	3.5m
2end	3m
3 rd	2.75m

Therefore: a_2 = 3.5/3.955 = 0.88

- For one charged lane n=1 :

➢ A(L) = 1×0.88× 1.1638 × 3.955= 4.0504t/ml.

- For two charged lanes n=2 :

➢ A(L) = 1×0.88× 1.1638 × 7.91= 8.1009t/ml.

Results are presented in the following table:

Tab.V. 5 A(L) values

N of lanes	a_1	a_2	A(L) **t/m2**	Lane width	A(L) **t/lm**
1 lane	1	0.88	1.1638	3.955	4.0504
2lanes	1	0.88	1.1638	7.91	8.1009

II.2 Load System B:

The load system B consists of the following three subsystems:

-System Bc: composed of standard truck models.

-System Br: composed of a single isolated wheel.

-System Bt: composed of sets of two axles, referred to as tandem axles.

- **Bc System:**

On the carriageway, a number of truck lanes or convoys is placed, not exceeding the number of traffic lanes.

-Transverse arrangement: The maximum number of truck lanes that can be placed transversely is equal to the number of traffic lanes.

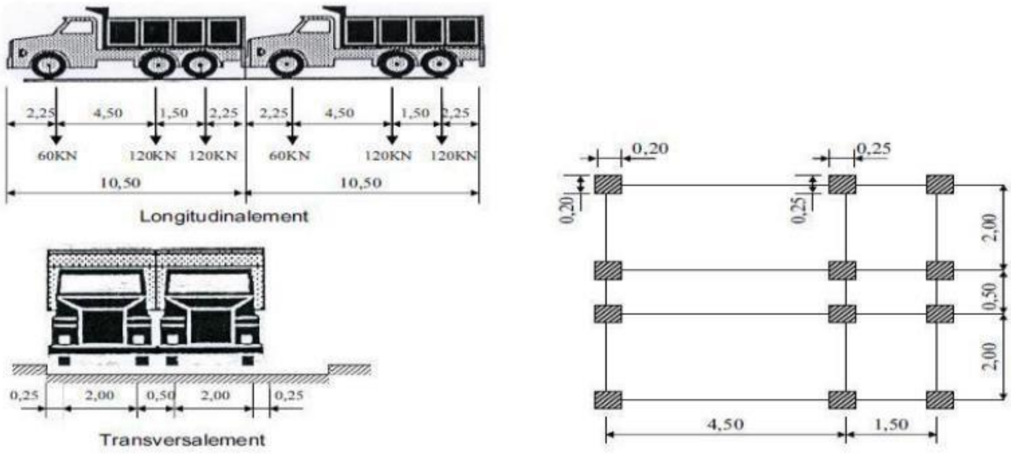


Fig.V.2 Bc system

Each truck has three single-tyre axles with a total mass of 30 tons, therefore:

- One front axle of 6 tons.
- Two rear axles of 12 tons each.

Depending on the bridge class and the number of considered lanes, the load value of the Bc system taken into account is multiplied by the coefficient bc, given in the following table:

Tab.V. 6. Table providing the coefficient bc

	Number of Considered lanes				
Bridge classification	1 lane	2 lanes	3 lanes	4 lanes	≥ 5 lanes
1	1.2	1.1	0.95	0.8	0.7
2	1	1	-	-	-
3	1	0.8	-	-	-

And multiplied by a dynamic amplification coefficient δc given by:

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

With: L : the span of the bridge = 26.55m

G: its permanent load G = 505,727 t.

S: maximum B-type load previously multiplied by the coefficient bc.

S = 2 × 30 × number of lanes × bc.

- For one charged lane:

bc =1.2 ⟶ S= 60×1.2 =72t

$$\delta c = 1+\frac{0.4}{1+0.2*26.55}+\frac{0.6}{1+4\frac{505.727}{72}} = 1.0840$$

- For two charged lanes:

bc =1.1 ⟶ S= 120×1.1 =132t

$$\delta c = 1+\frac{0.4}{1+0.2*26.55}+\frac{0.6}{1+4\frac{505.727}{132}} = 1.10$$

Tab V.7 Axle loads (t) of the Bc system

N of charged lanes	bc	Axle load	(t)
1	1.2	F. axle	6× 1.2 ×1.084=7.7904
		R. axle	12× 1.2 ×1.084=15.5808
2	1.1	F. axle	6× 1.1 ×1.10=7.26
		R. axle	12× 1.1 ×1.10=14.52

- **Br System:**

The Br system consists of a single isolated wheel carrying a load of 10 tons. Its impact area on the carriageway is a uniformly loaded rectangle, with a transverse side measuring 0.60 m and a longitudinal side measuring 0.30 m.

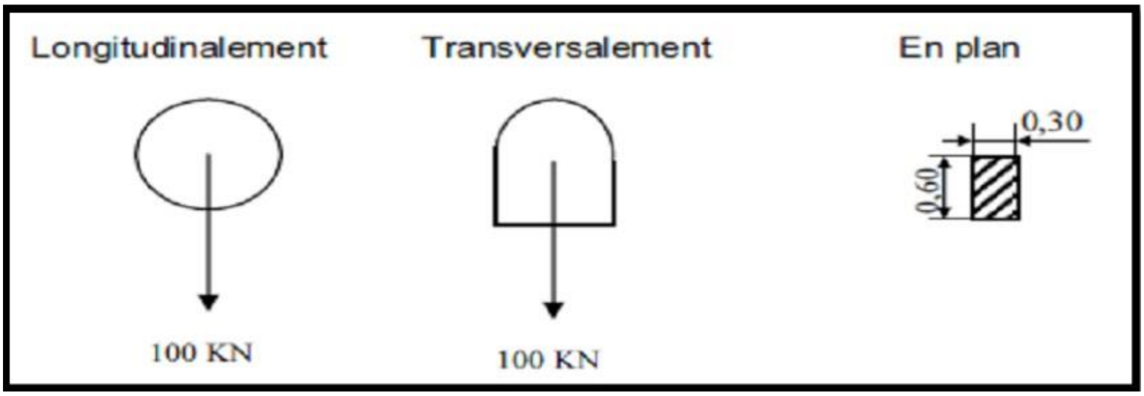


Fig V.3 Br System

The value of the Br system is multiplied by the dynamic amplification factor δc:

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

$$\delta c = 1 + \frac{0.4}{1+0.2*26.55} + \frac{0.6}{1+4\frac{505.27}{10}} = 1.066$$

Tab.V.8 Br load

Designation	S	δ_c	Wheel (10 tons)
Br	10	1.069	1.066× 10 =10.66

- **Bt System :**

A tandem of the Bt system consists of two axles, both equipped with single wheels, and has the following characteristics:

- Load carried by each axle: 16 t
- Distance between the two axles: 1.35 m
- Center-to-center distance between the two wheels of an axle: 2 m

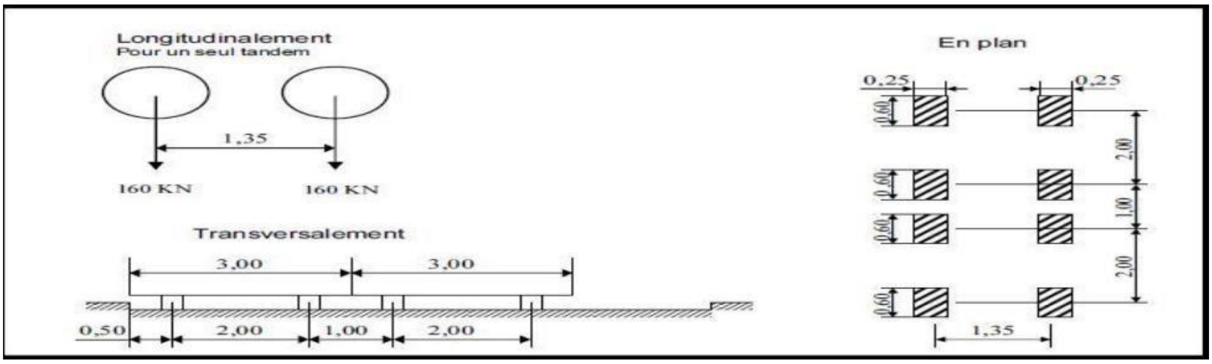


Fig V.4. Bt System

The Bt system must be multiplied by a coefficient bt, which depends on the bridge class.

The values of the bt coefficient are given in the following table:

TabV. 9. bt coefficient

Bridge classification	1rst	2end
bt	1.00	0.90

And by the dynamic amplification coefficient δc:

- For one charged lane:

S= s× bt =32 ×1 =32t

$\delta c = 1 + \frac{0.4}{1+0.2*26.55} + \frac{0.6}{1+4\frac{505.72}{32}} = 1.0721$

- For two charged lanes:

S = s× bt =32× 2 =64 t .

$\delta c = 1 + \frac{0.4}{1+0.2*26.55} + \frac{0.6}{1+4\frac{505.72}{64}} = 1.0807$

Tab V.10 δc coefficient

Designation	S	bt	δc	Axle (16t)
1 tandem	32	1.00	1.0721	17.153
2 tandem	64	1.00	1.0807	34.582

II.3. Military Load System MC 120:

Military vehicles often represent more critical loading conditions than Systems A and B for deck or superstructure elements.

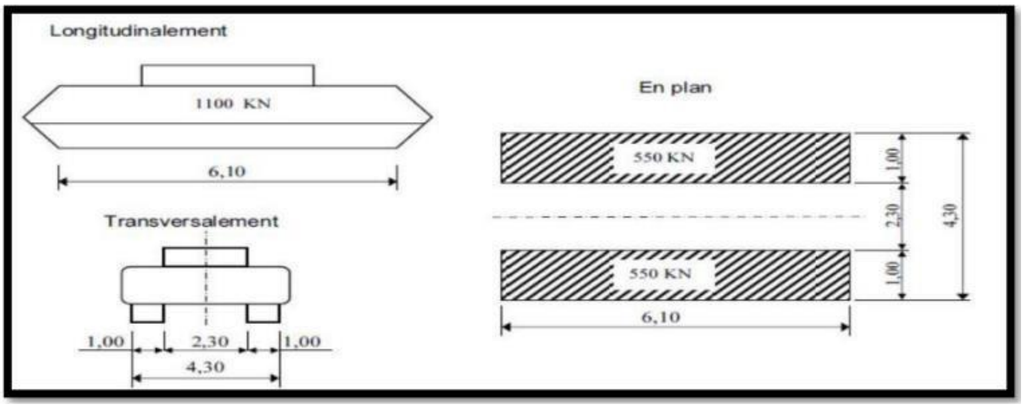


Fig.V.5 Mc120 System

Military loads are multiplied by a dynamic amplification factor.

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

L=26.55m. ,G=505.727t , S=110t

$$\delta c = 1+ \frac{0.4}{1+0.2*26.55} + \frac{0.6}{1+4\frac{505.72}{110}} =1.0923$$

Q =110×1.0923=120.153t

Q/ml= 120.153/6.1=19.697t/ml.

II.4 Exceptional Load System D240:

The D240 standard convoy consists of a trailer made up of three units with four axles each, carrying a total load of 2400 kN. This load is assumed to be distributed over the roadway on a uniformly loaded rectangle measuring 3.20 m in width and 18.60 m in length.

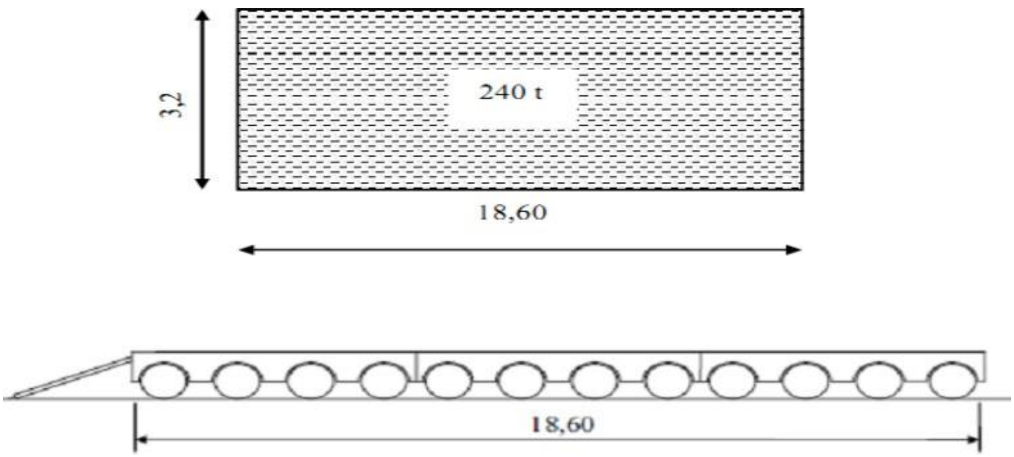


Fig V.6 D240 System

Q/ml =240/18.6 =12.903 t/ml.

II.5 Sidewalk Live Loads:

A uniform load of 150 kg/m² is applied to the sidewalk.

The width of the sidewalk is 1.05 m.

- For one loaded sidewalk:

P = 0.15 × 1.05 = 0.1575 t/m.l

- For two loaded sidewalks:

P = 2 × 0.15 × 1.05 = 0.315 t/m.l

II.6 Braking force:

Braking forces are taken into account for the stability of the supports and the strength of the bearing devices.

The braking force corresponding to load A(L) is equal to:

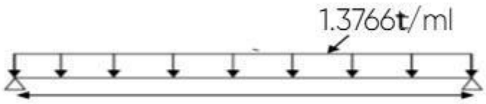
Ff =F× A(L), with F= $\frac{1}{20+0.0035\times S}$ and S = Lr × L (the loaded area (m^2))

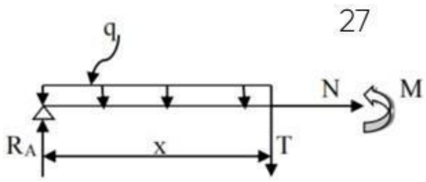
Tab V.11 Braking force

	A(L) (t)	S(m^2)	F	F_F(t)
1 lane	4.0504×26.55=107.538	3.955×26.55=105.005	0.04909	5.279
2lanes	8.1009×26.55=215.078	7.91×26.55=210.010	0.04822	10.126

III. Calculation of reduction factors due to loads:

III.1 Girder only: q=1.058t/ml





RA=RB = $\frac{q}{2}l$= 14.044t

M(x)= RA.x – q.x^2 /2

T(x) = RA – q.x

TabV.12 Girder only

Section (x)	(x)	M(t.m)	T(t)	R(t)
0.00L	0	0	14.044	
0.25L	6.75	68.166	6.902	14.044
0.5L	13.5	93.183	0	

III.2 The slab:

q=2,5 ×1,5× 0.25=0,938t/ml

$R_A=R_B=\frac{q}{2}l= 12.663t$

TabV.13 The slab

Section (x)	(x)	M(t.m)	T(t)	R(t)
0.00L	0	0	12.663	
0.25L	6.75	64.1063	6.3315	12.663
0.5L	13.5	85.4752	0	

III.3 Superstructure:

G=3.925 t/ml

Intermediate girder and edge girder: p = weight of the superstructure divided by the number of girders.

p= 3.925/7 = 0.5607t/m

$RA=RB=\frac{q}{2}l= 7.5696t$

TabV.13 Superstructure

Section (x)	(x)	M(t.m)	T(t)	R(t)
0.00L	0	0	7.5696	
0.25L	6.75	38.2763	3.7848	7.5696
0.5L	13.5	51.0958	0	

III.4 Summary table of M, T, and R values:

TabV.14 Summary table of M, T, and R values

Section (x)	(x)	M(t.m)	T(t)	R(t)
0.00L	0	0	34.2766	
0.25L	6.6375	170.5486	17.0183	34.2766
0.5L	13.275	229.754	0	

IV. Calculation of reduction elements due to live loads:

IV.1 Shear forces:

- Shear forces at x=0L :

  - A(l) and sidewalk live loads:

- A(l) live load:

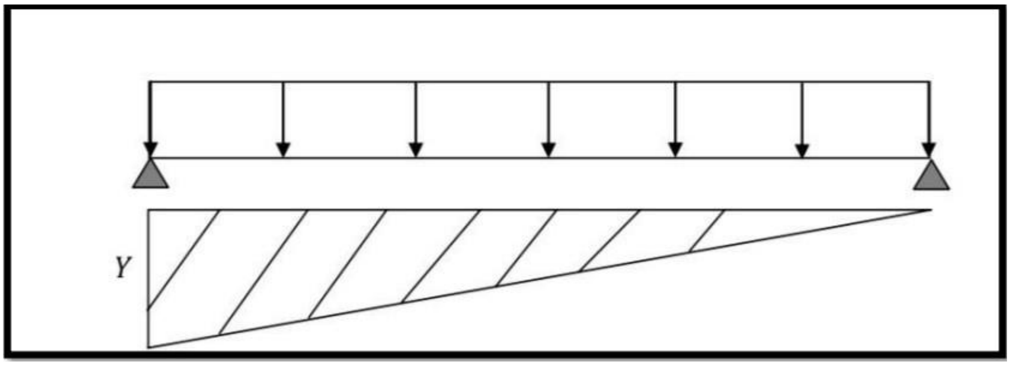


Fig.V.7 Influence line for A(L) at x = 0L

$Y=\frac{x}{L}=1$

$S=\sum Si=(\frac{1*26.55}{2})=13.275m^2$

$T_{max}=q\times S$

For one charged lane:

T=4.0504×13.275= 53.7690t

For two charged lanes:

T=8.1009×13.275= 107.5394t

- Sidewalk:

T=q× S

For one sidewalk:

T= 0.1575×13.275= 2.084t

For two sidewalks:

T= 0.318×13.275= 4.221t

- B system:

- Bc live load:

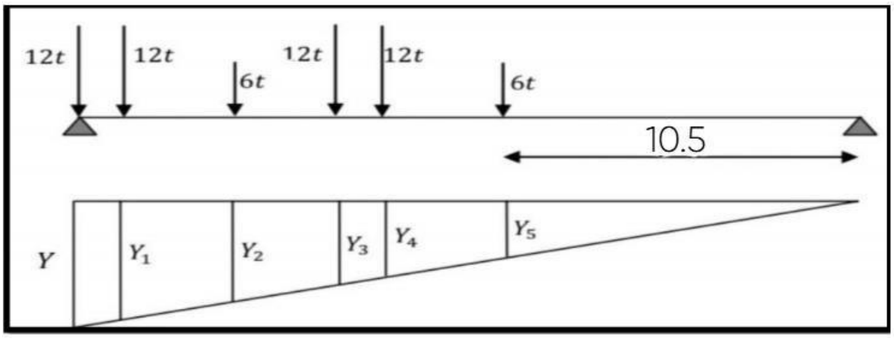


Fig.V.8 Influence line for Bc at x = 0L

Y=X/L=1 therfore :

TabV.14 Yi values of Bc at X = 0L

Y	Y1	Y2	Y3	Y4	Y5
1	0.9604	0.7909	0.6214	0.5649	0.3766

T = ∑(pi × Yi) = 44.7654t

Therfore we have : T = ∑ (pi ×Yi)× δc × bc

- One convoy:

T=44.7654×1.2×1.084=58.2308t

- Two convoys:

T=44.7654×1.1×1.10 ×2=108.3322t

- Bt live load:

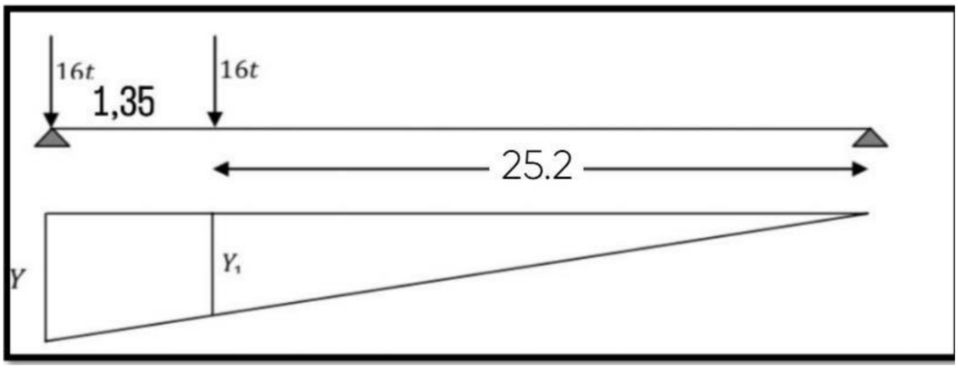


Fig. V.9 Influence line for Bt at x = 0L

Y=X/L=1 therfore : Y1= 0.9491

Tmax= (16×1+16×0.9491) =31.1856t

- One tandem:

T= (31.1856×1×1.0721) =33.4340t

- Two tandems:

T= (31.1856×1×1.0807×2) = 67.4045t

➢ Mc120 system:

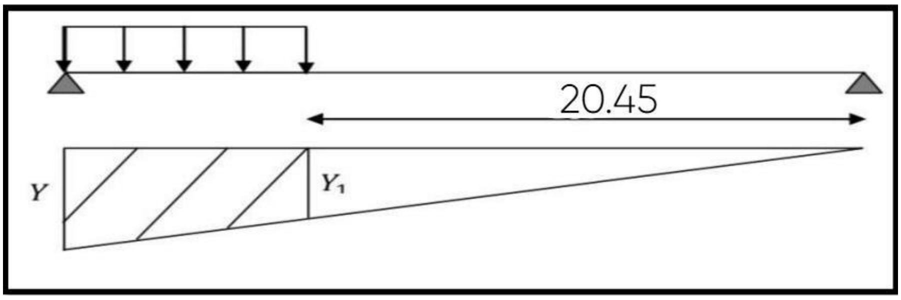


Fig. V.10 Influence line for Mc120 at x = 0L

Y=X/L=1 therfore : Y1= 0.7702

Tmax=∑ (qi ×Si) = 19.697×$(\frac{1+0.7702}{2})\times 6.1$= 106.3462t

➢ D240 system:

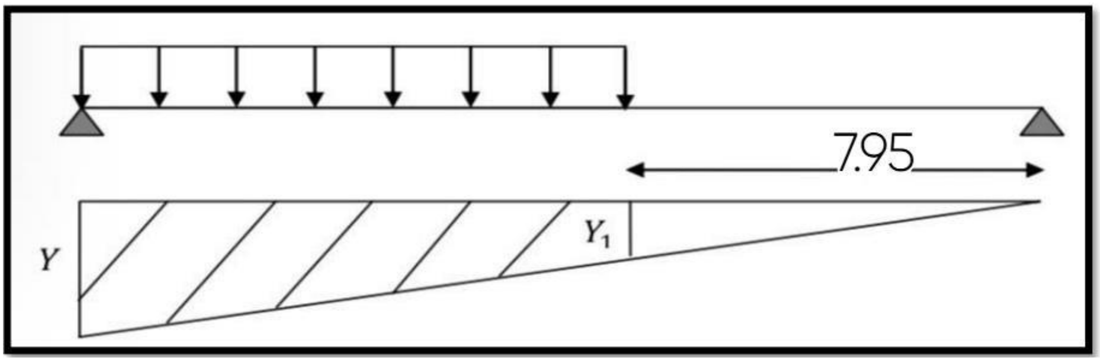


Fig. V.11 Influence line for D240 at x = 0L

Y=X/L=1 therfore : Y1= 0.2994

Tmax=∑ (qi ×Si) = 12.903 ×$(\frac{1+0.2994}{2})\times 18.6$= 155.9252t

- Shear forces at x=0.25L :

➢ A(l) and sidewalk live loads:

- A(l) live load:

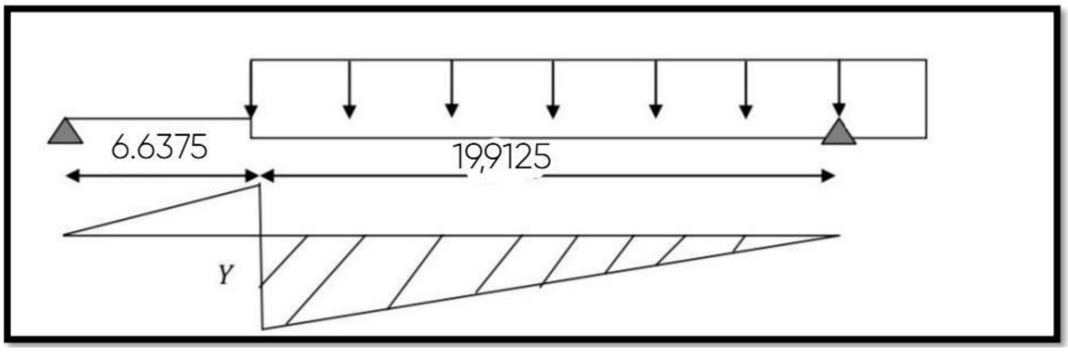


Fig. V.12 Influence line for A(L) at x = 0.25L

$Y = \frac{x}{L} = \frac{19.9125}{26.55} = 0.75$

$S = \sum Si = (0.75 \times \frac{19.9125}{2}) = 7.4671m2$

Tmax = q×S

For one charged lane:

T=4.0504×7.4671= 30.2450t

For two charged lanes:

T=8.1009×7.4671= 60.4902t

- Sidewalk:

T=q× S

For one sidewalk:

T= 0.157×7.4671= 1.172t

For two sidewalks:

T= 0.318×7.4671= 2. 374t

➢ B system:

- Bc live load:

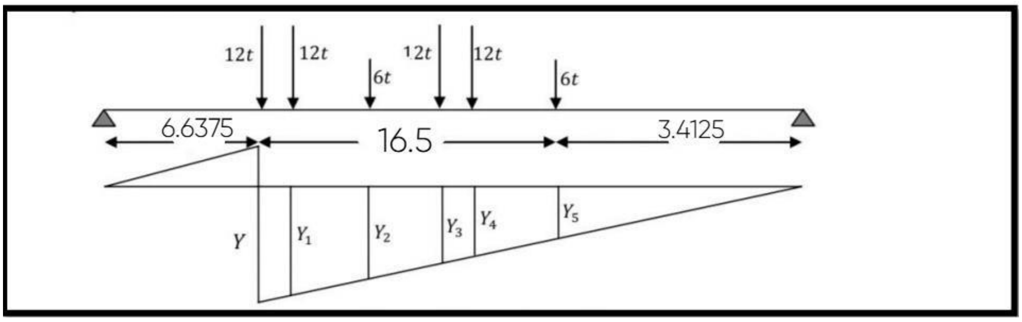


Fig.V.13 Influence line for Bc at x = 0.25L

$Y = \frac{x}{L} = \frac{19.9125}{26.55} = 0.75$

Tab. V.15 Yi values of Bc at X = 0.25L

Y	Y1	Y2	Y3	Y4	Y5
0.75	0.6935	0.5240	0.3545	0.2980	0.1285

T = ∑(pi × Yi) = 29.067t

Therfore we have : T = ∑ (pi ×Yi)× δc × bc

- One convoy:

T=29.067×1.2×1.084=37.8103t

- Two convoys:

T=29.067×1.1×1.10 ×2=70.3421t

- Bt live load:

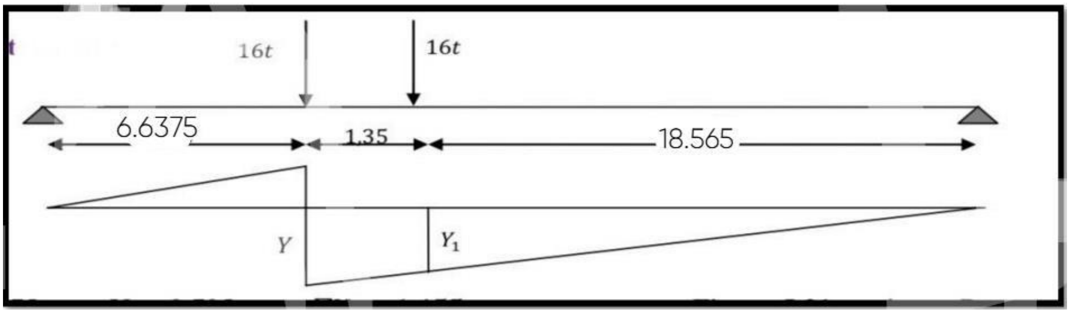


Fig. V.15 Influence line for Bt at x = 0.25L

Y=X/L=0.75 therfore : Y1= 0.6991

Tmax= (16×0.75+16× 0.6991) = 23.1865t

- One tandem:

T= (23.1865×1×1.0721) =24.8582t

- Two tandems:

T= (23.1865×1×1.0807×2) = 50.1153t

➢ Mc120 system:

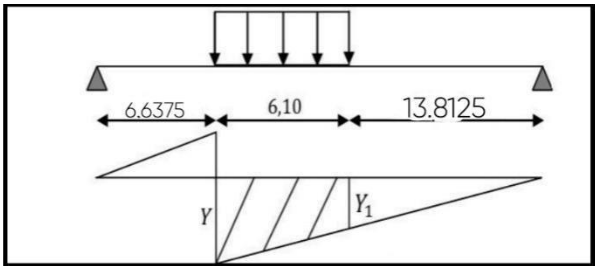


Fig. V.16 Influence line for Mc120 at x = 0.25L

Y=X/L=0.75 therfore : Y1= 0.5202

Tmax=∑ (qi ×Si) = 19.697×$(\frac{1+0.5202}{2})$ × 6.1= 91.3273t

➢ D240 system:

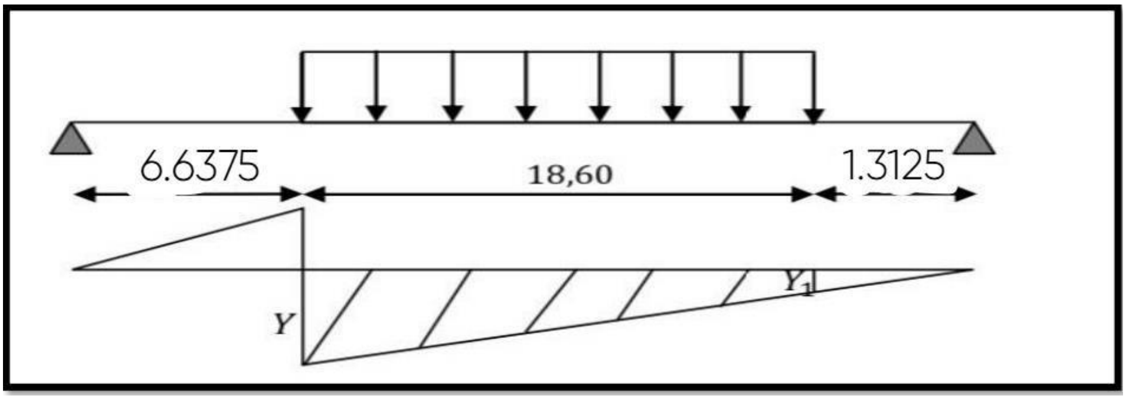


Fig. V.17 Influence line for D240 at x = 0.25L

Y=X/L=0.75 therfore : Y1= 0.0494

Tmax=∑ (qi ×Si) = 12.903 ×$(\frac{1+0.0494}{2})$ × 18.6= 125.9257t

❖ **Summary table of shear forces due to live loads:**

Tab. V.16 Summary table of shear forces due to live loads

Designation		ForX=0L		ForX=0.25L	
		Tmax (t)	T0 (T/7)	Tmax (t)	T0 (T/7)
Live load A(L)	1 charged lane	53.769	7.68128571	30.245	4.32071429
	2 charged lanes	107.5394	15.3627714	60.4902	8.64145714
Sidewalks	1 sidewalk	2.084	0.297	1.172	0.167
	2sidewalks	4.221	0.603	2.374	0.339
Bc system	1 charged lane	58.2308	8.31868571	37.8103	5.40147143
	2 charged lanes	108.3322	15.4760286	70.3421	10.0488714
Bt system	1 charged lane	33.434	4.77628571	24.8582	3.55117143
	2 charged lanes	67.4045	9.62921429	50.1153	7.15932857
Mc120		106.3462	15.1923143	91.3273	13.0467571
D240		155.9252	15.59252	125.9257	12.59257

IV.2 Bending moment:

- Bending moment x=0.5L :

  ➢ A(l) and sidewalk live loads:

• A(l) live load:

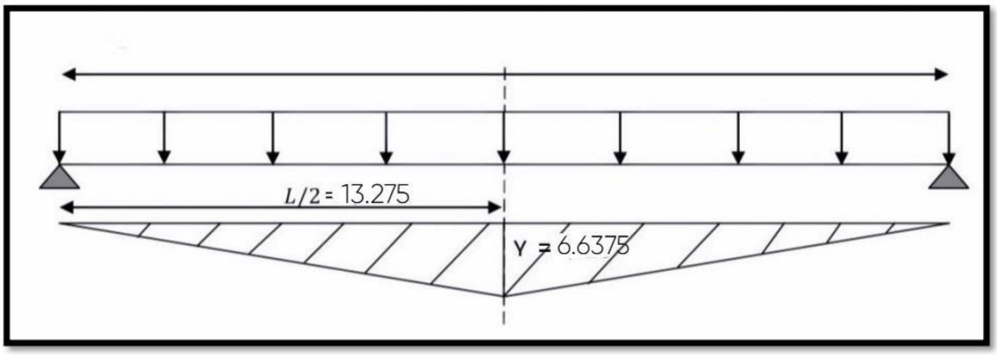


Fig. V.18 Influence line A(L)

$Y = 13.275(1 - \frac{13.275}{26.55}) = 6.6375$

$S = \sum Si = 2 \times (\frac{13.275 \times 6.6375}{2}) = 88.1128$

M= A(L) ×S

- For one loaded lane:

M=4.0504×88.1128= 356.8920t.m

- For two loaded lanes:

M=8.1008×88.1128= 713.7841t.m

- Sidewalk:

M=q× S

For one sidewalk:

M= 0.157×88.1128= 13.833t.m

For two sidewalks:

M= 0.315×88.1128= 27.667t.m

➢ B system:

APPLICATION OF BARRÉ'S THEOREM:

"The bending moment reaches its maximum directly under an axle when this axle corresponds to the resultant of the convoy positioned symmetrically with respect to the beam's axis."

- **Bc live load:**

✓ CASE No. 1: The resultant is located to the right of the girders's axis The resultant of the convoy: R = 2×6 +12 ×4 = 60 t

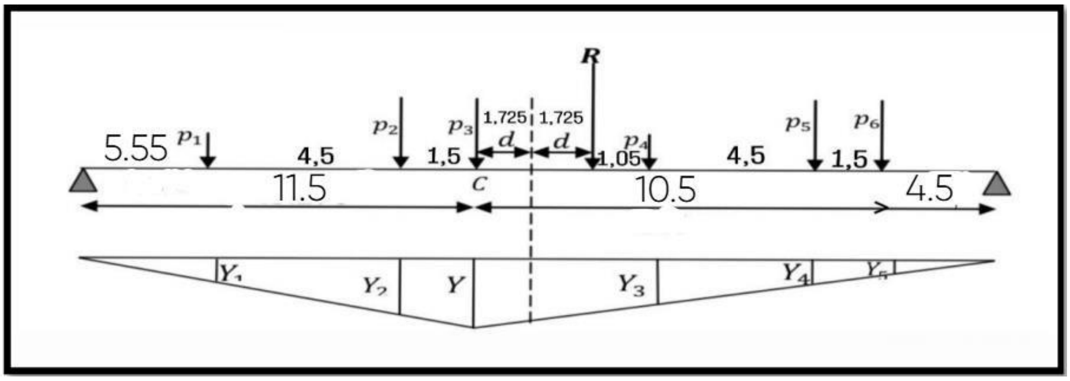


Fig. V.19 Influence line for the first case of Bc

$$dr = \frac{\sum Fi\ di}{\sum Fi} = \frac{(12\times4.5+12\times6+6\times10.5+12\times15+12\times16.5)}{60} = 9.45$$

a=9.45-6=3.45m

- The deflection equation is used to calculate the coordinates:

$$Y = x(1-\frac{x}{L})$$

$$Y = 11.50(1-\frac{11.50}{26.55}) = 6.5182m$$

Tab. V.16 Yi values, Case 1 of Bc at X = 0.5L

Y_1	Y_2	Y	Y_3	Y_4	Y_5
3.1457	5.6963	6.5182	4.5627	2.6072	1.9554

M = ∑(pi × Yi) = 247.5756t.m

✓ CASE No. 2: The resultant is located to the left of the girder axis: The resultant of the convoy: R = 2 × 6 + 12 × 4 = 60 t

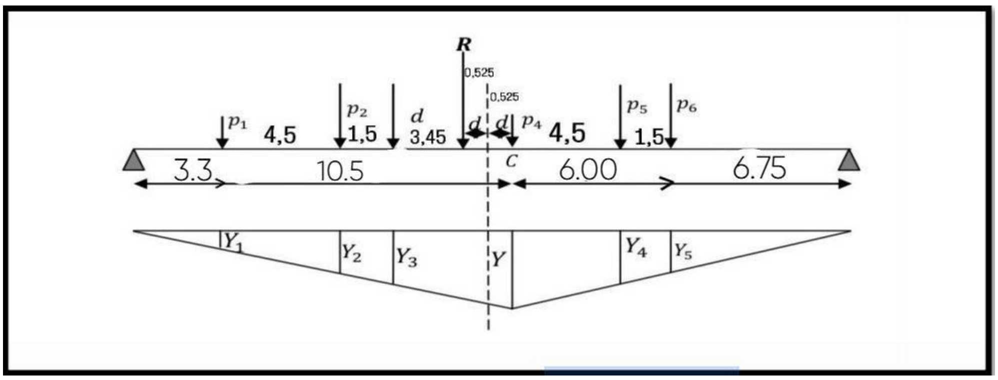


Fig. V.20 Influence line for the second case of Bc

$dr = \frac{\sum Fi\ di}{\sum Fi} = \frac{(12\times1.5+6\times6+12\times10.5+12\times12+12\times16.5)}{60} = 7.05$

a=7.05-6=1.05m

- The deflection equation is used to calculate the coordinates:

$Y = x(1-\frac{x}{L})$

$Y = 12.75(1-\frac{12.75}{26.55}) = 6.6271m$

Tab. V.17 Yi values, Case 2 of Bc at X = 0.5m

Y_1	Y_2	Y_3	Y	Y_4	Y_5
1.5847	3.745	4.4660	6.6271	4.2881	3.5084

$M = \sum(pi \times Yi) = 241.3608t.m$

– Therefore, the most unfavorable case is Case No. 1.

We have $M = \sum(pi \times Yi) \times \delta c \times bc$

-one convoy :
M=247.5756×1.2×1.084=322.0463t.m

-two convoys:
M=247.5756×1.1×1.10×2=599.1329t.m

- **Bt live load system:**

Only one case is considered for the position of R with respect to the girder's mid-axis. The system can take any position (to the right or to the left of the girder's axis) without affecting the result.

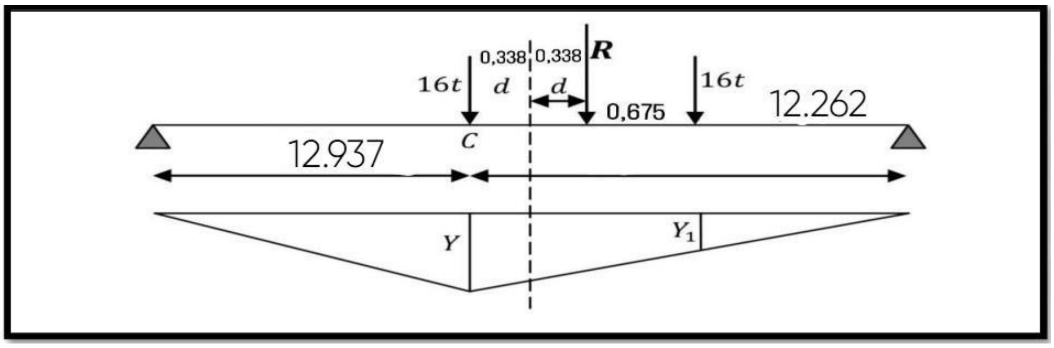


Fig. V.21 Influence line for Bt

$Y= x(1-\frac{x}{L})=12.937(1-\frac{12.937}{26.55}) = 6.6331m$

$Y_1= 5.9748$

Therfore : $M= 16(Y+Y1) \times \delta c \times bt$

- One tandem:

$M= 16(6.6331+5.9748)\times 1\times 1.0721 =216.2708t.m$

- Two tandems:

$M= 16(6.6331+5.9748)\times 1\times 1.0807\times 2 = 436.0114t.m$

- **Mc120 load system:**

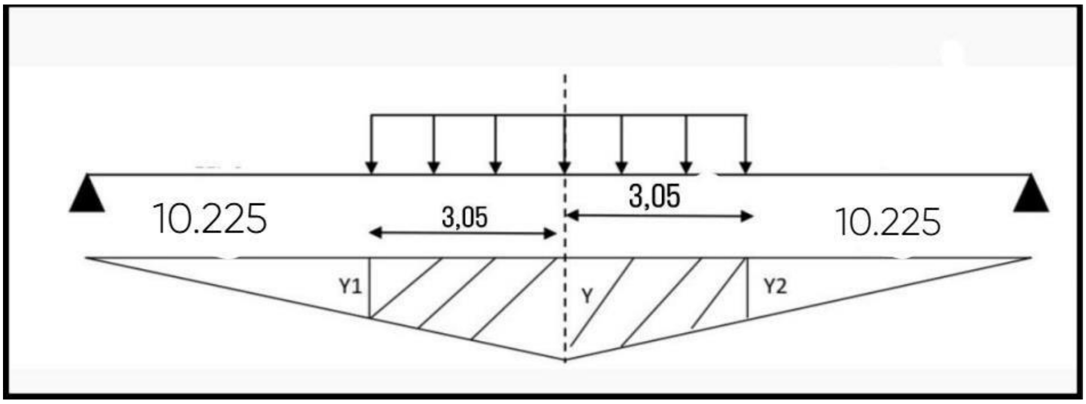


Fig. V.22 Influence line for Mc120

X=13.275 m
Y=6.6375m

Y1=Y2= 5.1125m

$S=2\times (\frac{5.1125+6.6375}{2}) \times 3.05 = 35.8375m^2$

Therfore : $M = q\times S \times \delta c =19.697\times 35.8375\times 1.0923=771.0449t.m$

- **Exceptional Load D240:**

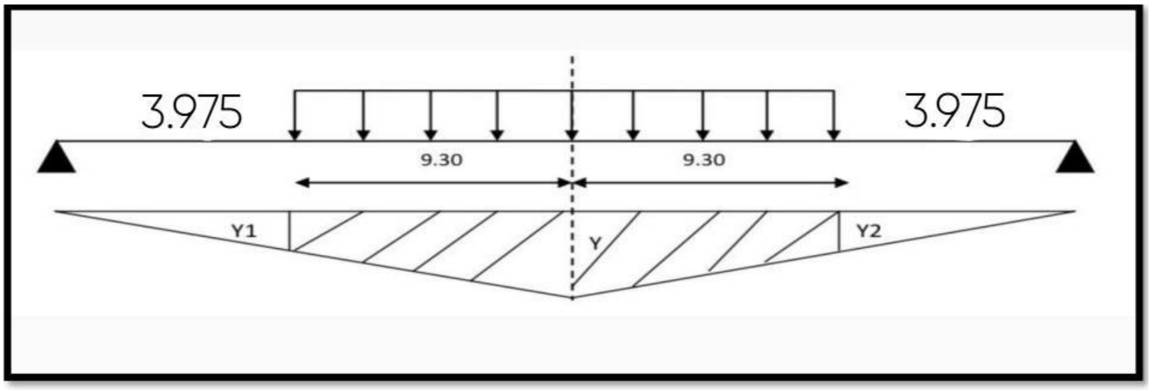


Fig. V.23 Influence line for D240

X=13.275 m
Y=6.6375m

Y1=Y2= 1.9875m

S=2× $(\frac{1.9875+6.6375}{2})$ × 9.30 = 80.2125m2

Therfore : M = q× S =12.903×80.2125=1034.9818t.m

- Bending moment x=0.25L :

  - A(l) and sidewalk live loads:

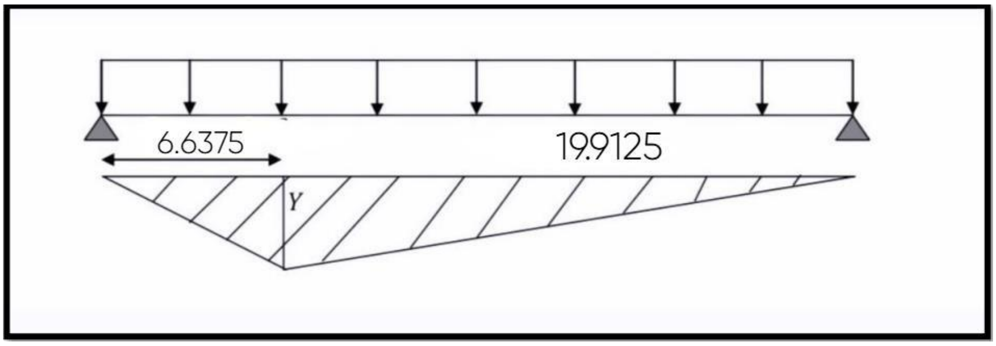


Fig.V.24 Influence line A(L) at x=0.25L

X= 6.6375

Y= 6.6375 $(1-\frac{6.6375}{26.55})$= 4.9781

S=∑Si=$(4.9781\times\frac{6.6375}{2})$ +$(4.9781\times\frac{19.9125}{2})$= 66.0842

- **A(l) live load:**

M= A(L) ×S

- For one loaded lane:

M=4.0504× 66.0842= 267.6674t.m

- For two loaded lanes:

M=8.1008× 66.0842= 535.3348t.m

- Sidewalk:

M=q× S

For one sidewalk:

M= 0.157×66.0842= 10.375t.m

For two sidewalks:

M= 0.318×66.0842= 21.014t.m

➢ **B system:**

- **Bc live load system:**

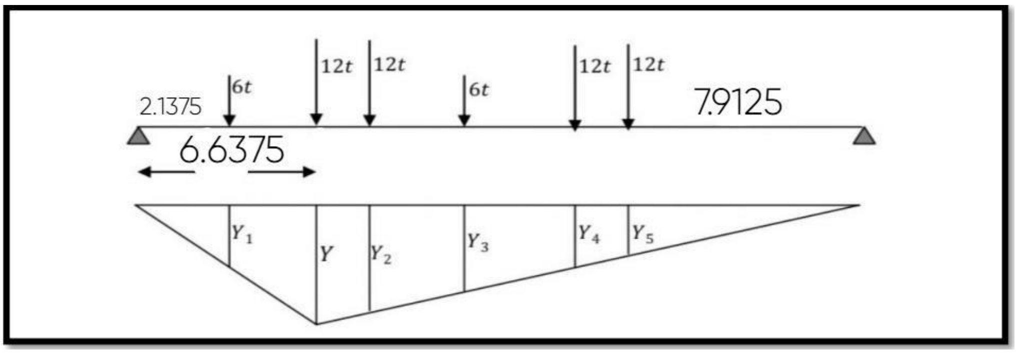


Fig. V.25 Influence line for Bc at x=0.25L

$Y = x(1-\frac{x}{L})$

$Y = 6.6375(1-\frac{6.6375}{26.55}) = 4.9781m$

Tab. V.18 Yi values of Bc at X = 0.25L

Y_1	Y	Y_2	Y_3	Y_4	Y_5
1.6031	4.9781	4.6031	3.4781	2.3531	1.9781

M = ∑(pi × Yi) = 197.436t.m

We have: M = ∑(pi × Yi) × δc × bc

-one convoy :

M= 197.436×1.2×1.084= 256.8247t.m

-two convoys:
M= 197.436×1.1×1.10×2= 477.7951t.m

- **Bt live load system:**

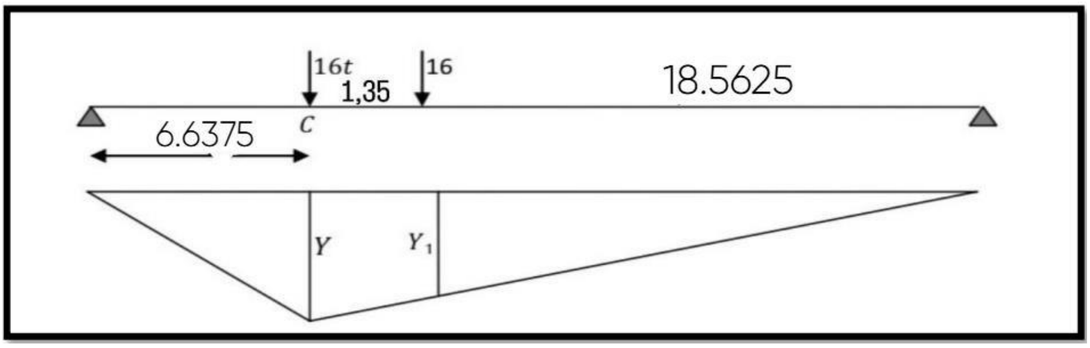


Fig. V.25 Influence line for Bt at x=0.25L

$Y = x(1-\frac{x}{L}) = 6.6375(1-\frac{6.6375}{26.55}) = 4.9781m$

Y1= 4.6406

Therfore : M= 16(Y+Y1) × δc × bt

- One tandem:

M= 16(4.9781+4.6406)×1×1.0721 =164.9953t.m

- Two tandems:

M= 16(4.9781+4.6406)×1×1.0807×2 = 332.6377t.m

- **Mc120 load system:**

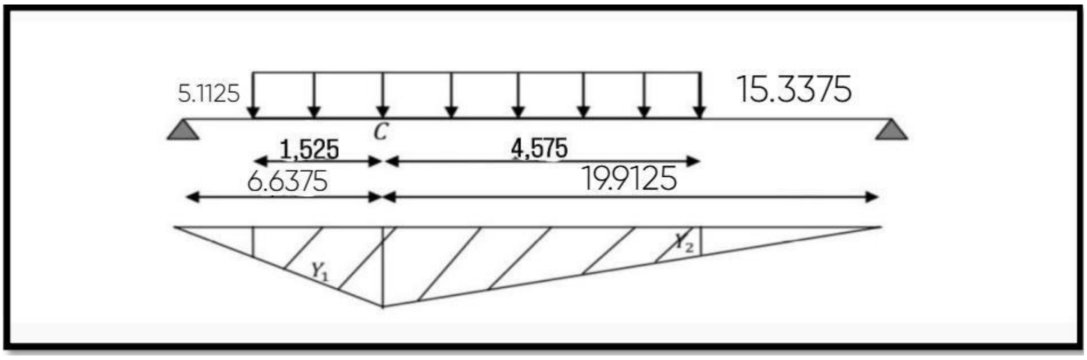


Fig. V.26 Influence line for Mc120 at x=0.25L

X= 6.6375

Y= 6.6375 $(1-\frac{6.6375}{26.55})$= 4.9781

Y1=Y2= 3.8343m

S= ((3.8343+4.9781)× $\frac{1.525}{2}$)+((3.8343+4.9781) ×$\frac{4.575}{2}$)= 26.8777m2

Therfore : M = q× S × δc =19.697×26.8777×1.0923=578.2746t.m

- **Exceptional Load D240:**

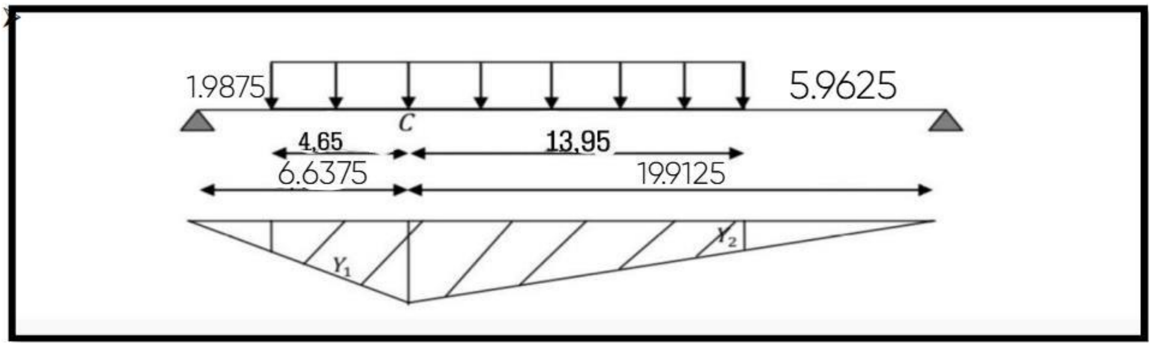


Fig. V.27 Influence line for D240 at x=0.25L

X= 6.6375

Y= 6.6375 $(1-\frac{6.6375}{26.55})$= 4.9781

Y1=Y2= 1.4906m

S=((1.4906+4.9781)×$\frac{4.65}{2}$)+((1.4906+4.9781)×$\frac{13.95}{2}$)= 59.8677m2

Therfore : M = q× S =12.903×59.8677=772.4729t.m

- ❖ **Summary table of bending moments due to live loads:**

Tab. V.19 Summary table of bending moments due to live loads

Designation		ForX=0.25L		ForX=0.5L	
		Mmax (t.m)	M0(M/7)	Mmax(t.m)	M0(M/7)
Live load A(L)	1 charged lane	267.6674	38.2382	356.892	50.984
	2 charged lanes	533.3348	76.1906857	713.784	101.979
Sidewalks	1 sidewalk	10.375	1.482	13.833	1.972
	2sidewalks	21.010	3.014	27.667	3.952
Bc system	1 charged lane	256.8247	36.6892429	322.046	46.006
	2 charged lanes	477.7951	68.2564429	599.132	85.590
Bt system	1 charged lane	164.9953	23.5707571	216.270	30.895
	2 charged lanes	332.6377	47.5196714	436.011	62.287
Mc120		578.2746	82.6106571	771.044	110.149
D240		772.4729	110.353271	1034.919	147.714

CHAPTER VI

TRANSVERSE DISTRIBUTION OF FORCE

I. Intoduction:

Diaphragms play a crucial role in distributing loads across the primary girders of a bridge. In cases where diaphragms are absent, this function is assumed by the deck slab itself. To accurately analyze the internal forces within a girder, the transverse distribution of live loads must be taken into consideration. This is done by applying a correction factor known as the Transverse Distribution Coefficient (TDC). Bridge girder decks are inherently three-dimensional structures, and numerous analytical methods have been developed for their assessment. These methods are typically categorized based on whether the cross-sectional behavior is assumed to be deformable or non-deformable.

- **Rigid non-deformable section :**

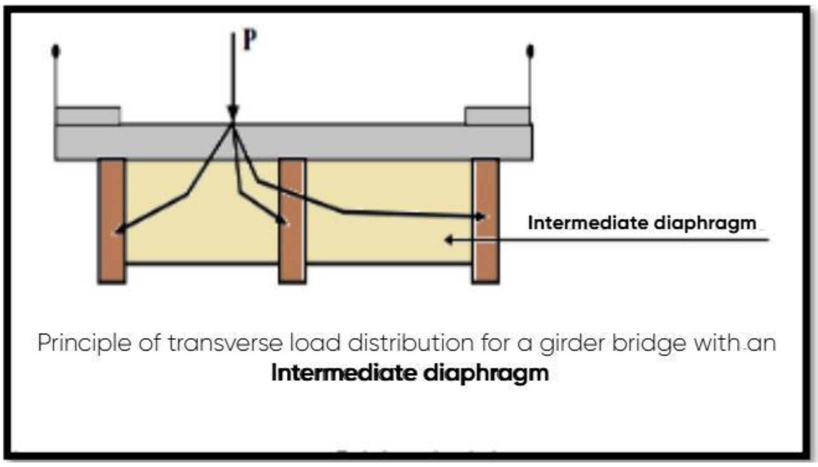


Fig. VI.1 Rigid non deformable section

- **Flexible deformable section:**

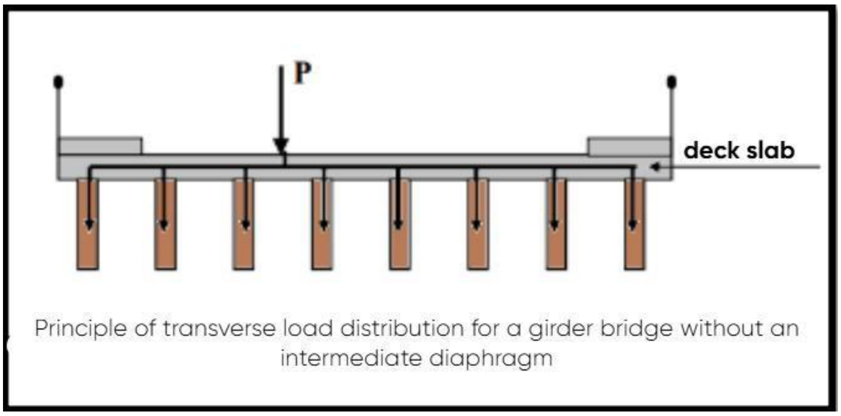


Fig. VI.1 Rigid deformable section

The choice of method consists in determining the bracing parameter ‘r’, which is defined by the following formula: $r=\frac{n}{2}\frac{a}{I}\sqrt[4]{\frac{Ip}{IE}}$

With:

n: number of girders

a: center-to-center spacing between girders

L: span of the girders

I_p: moment of inertia of a girder

I_e: moment of inertia of a cross beam (diaphragm or bracing)

If: ($r<0.3$)

- **J. Courbon's Method:** This method assumes that the deformations of the cross beams (diaphragms) are negligible compared to the deformations of the girders. In other words, the cross beams are considered to have infinite rigidity.

If: $r\geq0.30$

- **Guyon-Massonnet Method:** When the torsional rigidity of the bridge elements cannot be neglected, the bridge’s transverse section is considered deformable.

II. Application to Our Project:

n = 10 , a = 1.04 , l = 26.55

II.1 Our project includes diaphragms, which serve to distribute loads between the main girders. The diaphragms have the following dimensions:

L= 1,8m I= 0,98m e= 0,34m

Moment of inertia of a diphagram (IE) :

$IE = (d.h^3)/12$, $IE=(180\times34^3)/12=589560cm^4$

II.2 The prefabricated girder has variable cross-sections, so its **equivalent average moment of inertia** must be calculated:

$$I_P=I_0+(I_m-I_0)\frac{8}{3\Pi}$$

With:

I_0: Moment of inertia at the end section, including the slab.

I_m: Moment of inertia at the mid-span section, including the slab.

For an intermediate girder:

- $I0=22175060.93cm^4$
- $Im=16530721.24cm^4$
- $IP=22175060.86cm^4 = 0,221750m$

$$r=\frac{7}{2}\frac{1.50}{26.55}\sqrt[4]{\frac{22175060.93}{320982.666}} = 0.57 > 0.3$$

Since **r > 0.3**, the **Guyon-Massonnet method** is used.

III. GUYON-MASSONNET METHOD:

When the structure is loaded in accordance with the regulations, it is necessary to identify the critical sections—those subjected to the greatest stresses—both transversely and longitudinally.

The Guyon-Massonnet method is one of the simplest and most widely used approaches. It provides reliable results compared to other methods because it accounts for the non-negligible transverse stiffness of the bridge deck.

In this method, the applied load p(x) is assumed to follow a sinusoidal distribution, defined as:

$$\mathbf{P(x)=P1*Sin(\frac{\pi x}{L})}$$

p: is the constant value of the load.
L: is the span of the bridge.

Under the effect of a load with eccentricity *e*, applied along a line parallel to the bridge's axis, the bridge takes on a deformed shape as follows:

$$\mathbf{W\ (x, y) = W0.\ Sin(\frac{\pi x}{L})}$$

The latter takes on a cylindrical shape:

$$\mathbf{W0\ (x, y) = W0.\ Sin(\frac{\pi x}{L})}$$

It involves, for each load action, plotting the influence line of its transverse distribution coefficient for the various load eccentricities:

$e = \{\pm b;\ \pm\frac{3b}{4};\ \pm\frac{b}{2};\ \pm\frac{b}{2};\ \pm\frac{b}{4};\ \pm 0\}$ for the seven sections across the slab width.

$y = \{\pm b;\ \pm\frac{3b}{4};\ \pm\frac{b}{2};\ \pm\frac{b}{2};\ \pm\frac{b}{4};\ \pm 0\}$ It is sufficient to consider the symmetry (e.g; from 0 to b).

This allows for the accurate determination of the transverse distribution factors (Kα, εα, μα) and internal forces (bending moments, shear forces) at any point of the deck.

In the case of multi-girder bridges, the section under study is determined by the girder's position, which requires drawing influence lines for different load eccentricities. The section yielding the highest distribution coefficient values will be selected. All girders are identical and defined by the following characteristics:

- Their bending stiffness is given by **BP = E · IP**
- Their torsional stiffness by **CP = G · KP**.

Similarly, all cross beams (diaphragms) are identical and characterized by:

- Bending stiffness: **BE = E · IE**
- Torsional stiffness: **CE = G · KE** Where E is the Young's modulus.
- G: Shear modulus (modulus of torsion). With: **G=** $\frac{E}{2(1+\nu)}$

- ν: Poisson's ratio
- I_p: Flexural moment of inertia of the girders
- K_p: Torsional moment of inertia of the girders
- I_e: Flexural moment of inertia of the diaphragms (crossbeams)
- K_e: Torsional moment of inertia of the diaphragms (crossbeams)

Per unit length, these stiffnesses become:

Bending stiffness: $\begin{cases} \rho_P = \frac{B_p}{b_1} = \frac{E.I_p}{b_1} \\ \rho_E = \frac{B_E}{L_1} = \frac{E.I_E}{L_1} \end{cases}$

Torsional stiffness: $\begin{cases} \gamma_P = \frac{C_p}{b_1} = \frac{G.K_p}{b_1} \\ \gamma_E = \frac{C_E}{L_1} = \frac{G.K_E}{L_1} \end{cases}$

The behavior of the bridge is fully defined by two main parameters:

- Torsion parameter:: $\alpha = \frac{\gamma P + \gamma E}{\sqrt{\rho p . \rho E}}$
- Transverse bracing parameter: $\theta = b/L^4 \sqrt{\frac{\rho p}{\rho E}}$

In the case of multi-girder bridges, the section to be studied will be determined by the position of the girder. Influence lines will be plotted for the different load eccentricities, and the section that yields the highest coefficient values will be selected.

III.1 Determination of the calculation parameters:

The effective width is (2b). The bridge consists of 7 girders (n = 7) spaced at 1.5 m (center-to-center distance between girders), so the effective width of the bridge will be:

2b=n.b0=7×1,5=10.5 therfore : b=5.25

III.1.1 Cross-beam Flexibility Parameter (θ) (This parameter characterizes the transverse flexibility of the bridge deck):

$$\theta = b/L\sqrt[4]{(\rho p/\rho E)}$$

b=5.27 $\rho p = \frac{E.Ip}{b1}$

L=26.55m $\rho E = \frac{E.Ie}{L1}$

Calculation of Ip: The beams in our project have variable inertia.

- I_0 = Moment of inertia at the end section, including the slab.
- Im = Moment of inertia at the mid-span section, including the slab

- I0=22175060.93cm^4
- Im=16530721.24cm^4
- IP=22175060.86cm^4 = 0,221750m

Flexural rigidity of the girder:

$$\rho p = \frac{E.Ip}{b1} = \frac{E*0{,}221750}{1.5} = 0.14783E$$

Flexural rigidity of the diaphragms:

IE = $(b.h^3)/12 = (180\times 34^3)/12 = 589560cm^4$

L1= 26.55/18 = 1.5m with: 18 number of diaphragms

$$\rho E = \frac{E.Ie}{L1} = \frac{E*0.0058956}{1.5} = 0.00393E$$

$$\theta = b/L\sqrt[4]{\frac{\rho p}{\rho E}} = 5.27/26.55\sqrt[4]{\frac{0.14783E}{0.0039E}} = 0.492$$

III.1.2. Calculation of the torsion parameter α:

$$\alpha = \frac{\gamma P + \gamma E}{2\sqrt{\rho p.\rho E}}$$

$\gamma p = Cp / b_1$ → Torsional rigidity of the girders per unit width

$\gamma E = CD / L_1$ → Torsional rigidity of the diaphragms per unit width with:

Cp=G × [$\Sigma kiai.\, h$i³+$\frac{a.h^3}{6}$]

a: larger dimension

h: smaller dimension

Table VI.1 Coefficient K, as a function of b/a

$\frac{b}{a}$	1	1.2	1.5	1.75	2	2.25	2.5	3	4	4≤
K	0.141	0.166	0.196	0.213	0.229	0.240	0.249	0.263	0.281	0.333

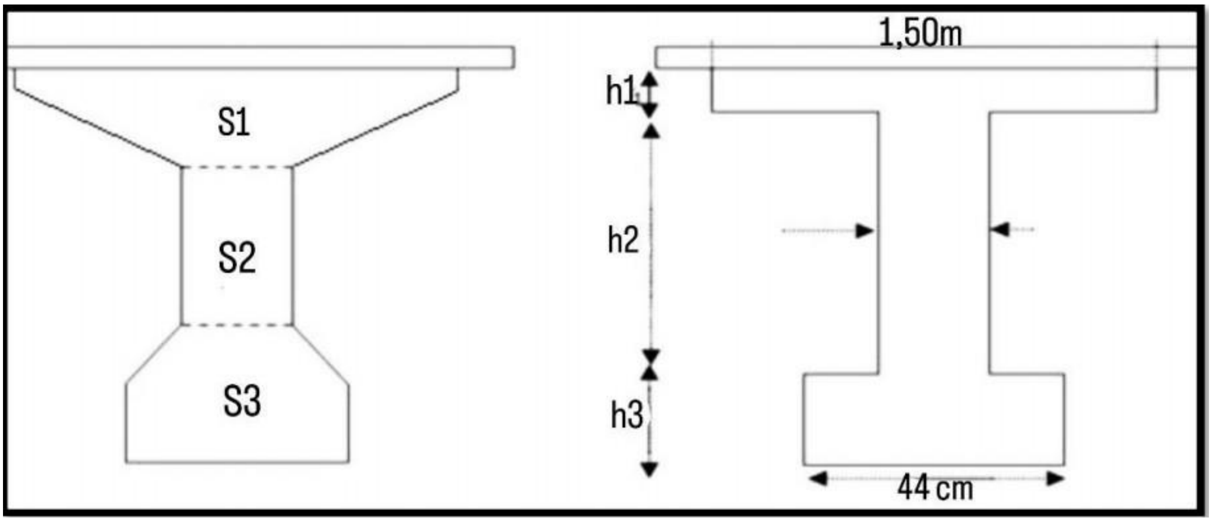


Fig. VI.1 Equivalent Beam

S1= 2186 cm^2= 100 × h1 ⟹ h1= 21.86 cm
S3 = 1672 cm^2=44 × h3 ⟹ h3 = 38 cm
Therfore: h2 = 130-(h1 +h3) =70.14 cm

Section 1:
$\frac{a}{b} = \frac{1}{0.2186}$= 4.575 ⟹ k= 0.333

Section 3:
$\frac{a}{b} = \frac{0.44}{0.38}$= 1.158 ⟹ k= 0.154

Section 2:

$\frac{a}{b} = \frac{0.7014}{0.18} = 3.897 \Longrightarrow k = 0.272$

$CP = \{(0.333 \times 1 \times 0.2186^3 + 0.154 \times 0.44 \times 0.38^3 + 0.272 \times 0.18^3 \times 0.7014 + \frac{1.8 \times 0.34^3}{6}) \times G\}$

$CP = 0.0201G$

$CE = \frac{1}{6} \times a \times b^3 \times G = 0.166 \times 1.8 \times 0.34^3 \times G = 0.01174G$

$\gamma p = Cp / b_1 = \frac{0.0201G}{1.5} = 0.0134G$

$\gamma E = CE / L1 = 0.01174G / 1.5 = 0.00782G$.

$G = \frac{E}{2(1+\nu)} = \frac{E}{2(1+0.2)} = \frac{E}{2.4}$

$\gamma p = 0.0134 \times \frac{E}{2.4} = 0.00558E$

$\gamma E = 0.00782 \frac{E}{2.4} = 0.00325E$

Therfore:

$\alpha = \frac{\gamma P + \gamma E}{2\sqrt{\rho p . \rho E}} = \frac{0.00558 + 0.00325}{2\sqrt{0.14783 . 0.00393}} = \frac{0.00883}{0.0214} = 0.36$

III.2. Transverse distribution of bending moments:

For any given α, the interpolation is not linear. It is provided by Massonnet.

$K = K0 + (K1 - K1) \sqrt{\alpha}$

For greater accuracy, Sattler proposed the following formulas:

$K = K0 + (K1 - K0)\, \alpha^{0,05}$ $0 \leq \theta \leq 0,1$

$K = K0 + (K1 - K0)\, \alpha(1 - e^{\theta o})$ $0,1 \leq \theta \leq 1$ with $\theta o = \frac{0,065 - \theta}{0,663}$

$K = K0 + (K1 - K0)\sqrt{\alpha}$ $\theta > 1$

K_0 and K_1 are given by the Guyon-Massonnet tables as functions of θ, e, and y (see the appendix).

$K_0 = K_0(\theta, e, y)$, $K_1 = K_1(\theta, e, y)$

θ : It varies from 0 to 1 in steps of 0.05

from 1 to 2 in steps of 0.10.

- $e = -b, \frac{-3b}{4}, \frac{-b}{2}, \frac{-b}{4}, 0, \frac{b}{4}, \frac{b}{2}, \frac{3b}{4}, b$
- $y = 0, \frac{b}{4}, \frac{b}{2}, \frac{3b}{4}, b$;For $y < 0$, the values are symmetrical

When the value of θ is not listed in the Guyon-Massonnet tables, the values of K_0 and K_1 must be interpolated (see appendix).

If: $\boldsymbol{\theta}0 \leq \boldsymbol{\theta} \leq \boldsymbol{\theta}1$ $k(\boldsymbol{\theta}) = k(\boldsymbol{\theta}0) + [\, (\frac{\theta-\theta 0}{\theta 1-\theta 0})\, (k(\boldsymbol{\theta}1) - k(\boldsymbol{\theta}0))]$

- **The values of K_0:**

- $\boldsymbol{\theta}$=0,45

Tab. VI.2 Values of K_0 for θ = 0.45

Théta=0.45	Alfa=0=> K0								
y / e	-b	-3b/4	-b/2	-b/4	0	b/4	b/2	3b/4	b
0	0.7355	0.8811	1.0194	1.1304	1.1783	1.1304	1.0194	0.8811	0.7355
b/4	0.073	0.3495	0.6242	0.8902	1.1305	1.3144	1.4148	1.4671	1.5059
b/2	-0.5152	-0.1402	0.238	0.6242	1.0194	1.4148	1.7857	2.1063	2.4061
3b/4	-1.064	-0.606	-0.1402	0.3495	0.8811	1.4671	2.1063	2.7708	3.434
b	-1.6003	-1.064	-0.5152	0.073	0.7355	1.5059	2.4061	3.434	4.5496

- $\boldsymbol{\theta}$=0,50

Tab. VI.3 Values of K_0 for θ = 0.50

Théta=0.50	Alfa=0=> K0								
y / e	-b	-3b/4	-b/2	-b/4	0	b/4	b/2	3b/4	b
0	0.6203	0.8288	1.0273	1.1877	1.2575	1.1877	1.0273	0.8288	0.6203
b/4	-0.0021	0.3111	0.6223	0.9226	1.1877	1.3721	1.4336	1.425	1.3968
b/2	-0.5198	-0.1466	0.2317	0.6223	1.0273	1.4336	1.8038	2.0981	2.3613
3b/4	-0.9828	-0.5703	-0.1466	0.3111	0.8288	1.425	2.0981	2.8125	3.514
b	-1.4286	-0.9828	-0.5198	-0.0021	0.6203	1.3968	2.3613	3.514	4.7981

- $\boldsymbol{\theta}$=0,49

Tab. VI.4 Values of K₀ for θ = 0.49

Théta=0.49	Alfa=0=> K0								
y / e	-b	-3b/4	-b/2	-b/4	0	b/4	b/2	3b/4	b
0	0.64334	0.83926	1.02572	1.17624	1.24166	1.17624	1.02572	0.83926	0.64334
b/4	0.01292	0.31878	0.62268	0.91612	1.17626	1.36056	1.42984	1.43342	1.41862
b/2	-0.5188	-0.14532	0.23296	0.62268	1.02572	1.42984	1.80018	2.09974	2.37026
3b/4	-0.9990	-0.57744	-0.1453	0.31878	0.83926	1.43342	2.09974	2.80416	3.498
b	-1.4629	-0.99904	-0.5188	0.01292	0.64334	1.41862	2.37026	3.498	4.7484

- $\boldsymbol{\theta}$=0,45

Tab. VI.5 Values of K1 for θ = 0.45

Théta=0.45	Alfa=1=> K1								
y / e	-b	-3b/4	-b/2	-b/4	0	b/4	b/2	3b/4	b
0	0.8933	0.9458	1.0032	1.0577	1.085	1.0577	1.0032	0.9458	0.8933
b/4	0.7355	0.8029	0.8804	0.9688	1.0577	1.1214	1.1318	1.1152	1.0938
b/2	0.6142	0.6881	0.7748	0.8804	1.0032	1.1318	1.2405	1.3013	1.34
3b/4	0.5202	0.5969	0.6881	0.8029	0.9458	1.1152	1.3013	1.4809	1.6291
b	0.4418	0.5202	0.6142	0.7355	0.8933	1.0938	1.34	1.6291	1.9476

- $\boldsymbol{\theta}$=0,50

Tab. VI.6 Values of K1 for θ = 0.50

Théta=0.5à	Alfa=1=> K1								
y / e	-b	-3b/4	-b/2	-b/4	0	b/4	b/2	3b/4	b
0	0.8609	0.9276	1.0028	1.0767	1.1146	1.0767	1.0028	0.9276	0.8609
b/4	0.6834	0.7617	0.8547	0.9642	1.0767	1.1557	1.1603	1.1293	1.0937
b/2	0.5516	0.6326	0.7308	0.8547	1.0028	1.1603	1.2911	13544	13376
3b/4	0.4538	0.534	0.6326	0.7617	0.9276	1.1293	1.3544	1.5704	1.7409
b	0.3751	0.4538	0.5516	0.6834	0.8609	1.0937	1.3876	1.7409	2.1362

- $\boldsymbol{\theta}$=0,49

Tab. VI.7 Values of K1 for θ = 0.49

Théta=0.461	Alfa=0=> K0								
y / e	-b	-3b/4	-b/2	-b/4	0	b/4	b/2	3b/4	b
0	0.86738	0.93124	1.00288	1.0729	1.10868	1.0729	1.00288	0.93124	0.86738
b/4	0.69382	0.76994	0.85984	0.96512	1.0729	1.14884	1.1546	1.12648	1.09372
b/2	0.56412	0.6437	0.7396	0.85984	1.00288	1.1546	1.28098	1.34378	1.33808
3b/4	0.46708	0.54658	0.6437	0.76994	0.93124	1.12648	1.34378	1.5525	1.71854
b	0.38844	0.46708	0.56412	0.69382	0.86738	1.09372	1.37808	1.71854	2.09848

- **Kα Values:**

Suttler's formula is applied:

If 0,1≤ $\boldsymbol{\theta}$ ≤ 1 Kα= K0+ (K1-K0)* α^{β} with : β=1- $e^{(0.065-\theta)/0.663}$

Tab. VI.8 Kα Values Table

Théta=0.49	Alfa=0.41=> K0								
Y / e	-b	-3b/4	-b/2	-b/4	0	b/4	b/2	3b/4	b
0	0.7819478	0.8961656	1.011589	1.11230	1.15938	1.11230	1.01158	0.89616	0.78194
b/4	0.4341754	0.5979011	0.769404	0.94643	1.11231	1.22957	1.25955	1.24352	1.21761
b/2	0.1511444	0.3428265	0.546405	0.76940	1.01158	1.25955	1.47896	1.63204	1.73167
3b/4	-0.091989	0.1179624	0.342826	0.59790	0.89616	1.24352	1.63204	2.0297	2.39709
b	-0.317538	-0.0919890	0.151144	0.43417	0.78194	1.21761	1.75642	2.39709	3.10896

Tab. VI.9 Kα Coefficient Values by Girder Position

	-b	-3b/4	-b/2	-b/4	0	b/4	b/2	3b/4	b
y	-5.27	-3.95	-2.63	-1.31	0	1.31	2.63	3.95	5.27
0	0.781947	0.896165	1.011589	1.112306	1.159388	1.112306	1.011589	0.896165	0.781947
1.5	0.393436	0.561185	0.737306	0.920953	1.097815	1.233889	1.291137	1.299447	1.291606
3	0.082993	0.279796	0.489341	0.721331	0.979235	1.255062	1.521873	1.743535	1.918195
4.5	-0.18596	0.0304826	0.2629589	0.5296821	0.8485749	1.2327276	1.68387050	2.182833	2.693705

Tab. VI.10 Kαm results

Bc system

K(Bc)	-3.48	-1.48	-0.98		3.52	2.52	1.02	y
1.0395405	0.937263544	1.099335092	1.124166658		0.933765853	1.019982535	1.122729024	0
1.0455963	0.623895454	0.897301915	0.965506495		1.296740668	1.286366884	1.203766513	1.5
1.032855	0.354407171	0.691454288	0.786300016		1.67132748	1.499639464	1.194001428	3
1.0054762	0.11325832	0.495331399	0.610013879		2.020292272	1.646275266	1.147686205	4.5

Bt system

K(Bt)	-4.28	-2.28	-1.28		3.72	1.72	0.72	y
1.0303727	0.867611212	1.038294663	1.113384408		0.916277398	1.12117921	1.12548928	0
0.992148	0.519248432	0.786000688	0.925003705		1.297999801	1.22884635	1.195789033	1.5
0.9371646	0.230595667	0.550854081	0.727238031		1.704912587	1.231556443	1.177830842	3
0.876846	-0.023629995	0.333681015	0.536984994		2.09589267	1.192982482	1.125164883	4.5

Sidwalks

K(S)	b		-3.69	4.22	b	y
0.838899811	0.781947822		0.918900666	0.872802932	0.781947822	0
0.894690767	0.393436116		0.595876278	1.297843947	1.291606726	1.5
1.025380029	0.082993379		0.321070457	1.779261275	1.918195005	3
1.217835185	-0.185967979		0.076273453	2.287329751	2.693705513	4.5

A(l)System

K(AL)	3.95	2.63	1.31	0		-1.31	-2.63	-3.95	y
1.0505576	0.8961657	1.011589476	1.112306183	1.159388673		1.112306183	1.011589476	0.896165675	0
1.0352366	1.2994478	1.291137527	1.233889809	1.09781561		0.920953426	0.737306402	0.56118587	1.5
0.9964185	1.7435355	1.521873757	1.255062234	0.979235835		0.721331832	0.48934149	0.27979643	3
0.9440787	2.1828331	1.683870503	1.232727653	0.848574903		0.529682106	0.262958972	0.030482666	4.5

Mc120

K(Mc120)	-0.08		4.22	3.22	0.92	y
1.028909495	1.156513406		0.872802932	0.959998535	1.126323108	0
1.218272488	1.087014866		1.297843947	1.294851969	1.19337917	1.5
1.384160762	0.963485972		1.779261275	1.620949821	1.172945978	3
1.535420882	0.829100534		2.287329751	1.906891676	1.118361567	4.5

D240

K(D240)		-1.6	0	1.6	y
1.12478385		1.090179028	1.159388673	1.090179028	0
1.080676228		0.880606731	1.09781561	1.246466959	1.5
0.985628961		0.670364257	0.979235835	1.313679917	3
0.875019004		0.471083841	0.848574903	1.33184237	4.5

- **Kαm Calculation Equations:**

Sidwalks: $\bar{K}=\frac{1}{4}[K\alpha(-b)+K\alpha(\frac{-3b}{4})+K\alpha(b)+K\alpha(\frac{3b}{4})]$

A(L) : $\bar{K}=\frac{1}{6}\left[\frac{k\alpha(\frac{-3b}{4})+k\alpha(\frac{3b}{4})}{2}+k\alpha\left(\frac{-b}{2}\right)+k\alpha\left(\frac{b}{2}\right)+k\alpha\left(\frac{-b}{4}\right)+k\alpha(\frac{b}{4})+ +k\alpha(0)\right]$

Bc: $\bar{K}=\frac{1}{6}[K1+K2+K3+K4+K4+K5+K6]$

Bt: $\bar{K}=\frac{1}{6}[K1+K2+K3+K4+K4+K5+K6]$

Mc120 $\bar{K}=\frac{1}{2}\left(\frac{K1+K2}{2}\times 1+\frac{K3+K4}{2}\times 1\right)$

D240 $\bar{K}=\frac{1}{3.2}\left[\left(\frac{K1+K2}{2}\times 1.6\right)+\left(\frac{K2+K3}{2}\times 1.6\right)\right]$

IV. Actual Values of Longitudinal Bending Moments Due to Live Loads:

TabVI.10 Actual Longitudinal Bending Moments Due to Live Loads

Bending Moment atX=0.5L			Girder 4		Girder 5		Girder 6		Girder 7	
Load case		M0	Kαm	Mreal	Kαm	Mreal	Kαm	Mreal	Kαm	Mreal
sidwalks (2)		3.952	0.838	3.314	0.894	3.535	1.0254	4.0526	1.2183	4.814
A(l)	1lane	50.984	1.050	53.566	1.035	52.784	0.9964	50.8011	0.94396	48.127
	2lanes	101.979	1.050	107.144	1.035	105.579	0.9964	101.613	0.94396	96.264
Bc	1lane	46.006	1.039	47.828	1.045	48.108	1.0329	46.5212	1.00551	46.259
	2lanes	85.59	1.039	88.981	1.045	89.500	1.0329	88.4089	1.00551	86.061
Bt	1lane	30.895	1.030	31.835	0.992	30.651	0.9370	28.9489	0.87654	27.080
	2lanes	62.287	1.030	64.182	0.992	61.795	0.9370	58.3635	0.87654	54.597
Mc120		110.149	1.028	113.338	1.218	134.246	1.3850	152.567	1.53680	169.277
D240		147.714	1.125	166.178	1.080	159.653	0.9856	145.589	0.87477	129.216

V. Load Combinations According to B.A.E.L:

- ***Moment Combinations at ULS (Ultimate Limit State) :***

Table VI.11: Load Combinations at ULS

ULS (ELU)	Girder 4	Girder 5	Girder 6	Girder 7
1.35G+1.6(AL+sidwalks)	486.902213	484.751168	479.233309	471.895049
1.35G+1.6(BC+sidwalks)	457.840176	459.025539	458.106443	455.570577
1.35G+1.6(BT+sidwalks)	418.162331	414.697432	410.033876	405.227296
1.35(G+MC120)	463.175121	491.400326	516.133787	538.692786
1.35(G+D240)	534.509454	525.700372	506.713106	484.60973

Moment Combinations at SLS (Serviceability Limit State) :

Table VI.12: Load Combinations at SLS

ULS (ELU)	Girder 4	Girder 5	Girder 6	Girder 7
G+1.2(AL+sidwalk)	362.304735	360.691451	356.553056	351.049362
G+1.2(BC+ sidwalk)	340.508207	341.397229	340.707907	338.806007
G+1.2(BT+ sidwalk)	310.749823	308.151149	304.653482	301.048547
(G+MC120)	343.092682	364.000242	382.321324	399.031693
(G+D240)	395.932929	389.407683	375.343041	358.97017

The most heavily loaded girder is Girder No. 7.

Mmax=1.35(G+Mc120) = 538.692786t.m

CHAPTER VII

DECK MODELING

I. ITRODUCTION:

SAP2000 is a powerful structural engineering software designed for the analysis and design of buildings and civil infrastructure. It offers a user-friendly graphical interface for modeling structural systems, supported by a robust library of elements tailored for accurate representation of real-world structural behavior. The platform enables comprehensive analysis of both static and dynamic loads, and includes advanced design and verification tools for reinforced concrete and steel frameworks. Additionally, SAP2000's built-in graphical post-processing tools streamline the interpretation of analysis results and assist in generating detailed design reports and calculation summaries.

The software supports the modeling process—such as geometry definition, ^numerical, or hybrid methods, leveraging an extensive set of integrated tools. Structural models can be built from subcomponents (e.g., frames, trusses, slabs, shear walls), each defined within its dedicated graphical environment and then assembled into a comprehensive analysis model. The software ensures automatic compatibility of connections between substructures. Additionally, finite elements can be generated based on predefined modeling templates (e.g., truss, frame, beam grid, slab, wall, shell), providing flexible and efficient system representation.

II. Modeling:

Since the structure is isostatic with two independent spans, only one span will be analyzed.
The girders are modeled as FRAME elements, supported on a simple support at one end and a double support at the other.
The slab is modeled as a SHELL element.

II.1 Modeling steps:

II.1.1 Choosing unites:

It is necessary to select the unit system before starting a SAP2000 session; we will adopt the ton–meter–Celsius (Ton–m–°C) system.

Fig. VII.1 Units tab

II.1.2 Structure creation:

Importing a model from the SAP2000 library: from the menu FILE > NEW MODEL, the following dialog box appears:

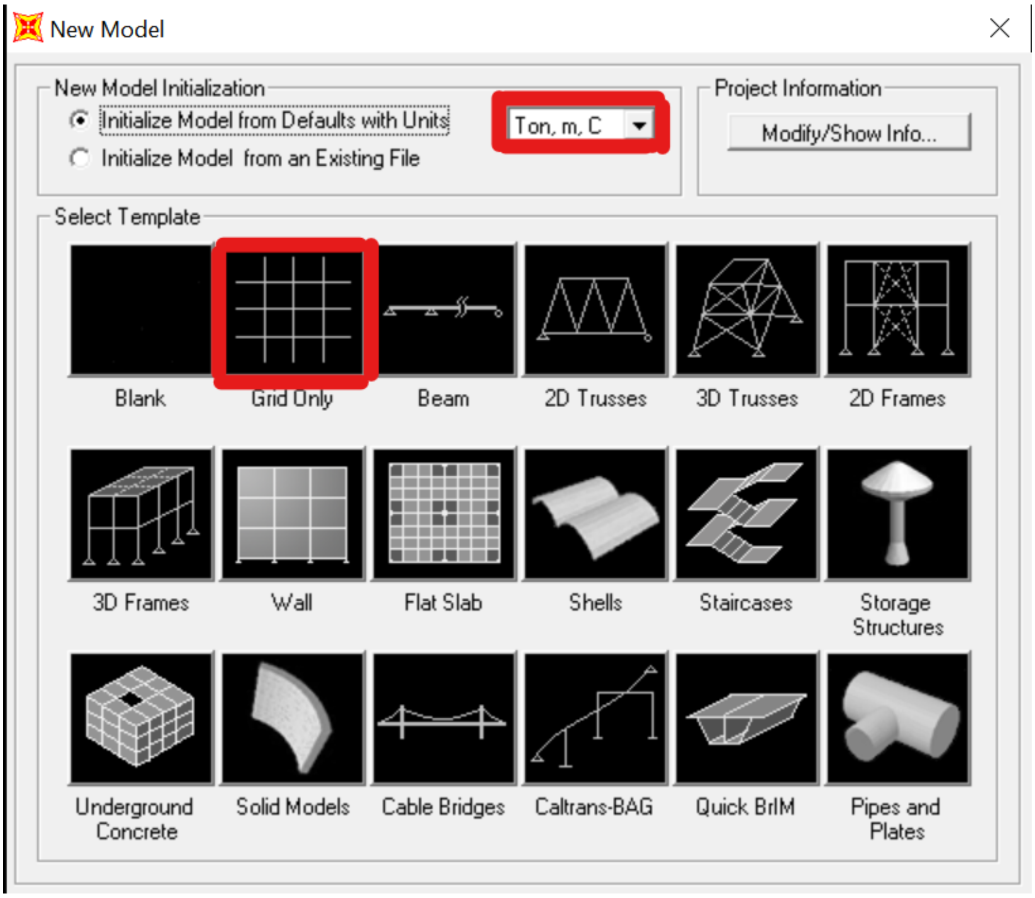
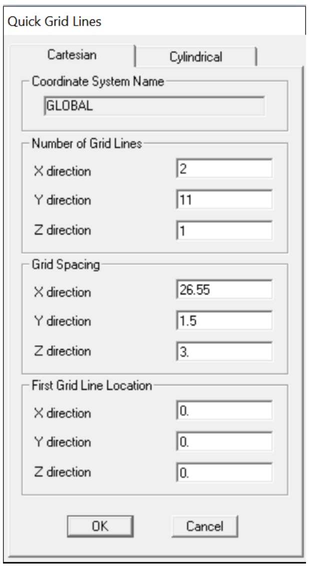


Fig. VII.2 Choosing the model's type according to the stucture's type

II.1.3 Span's dimension:



II.1.4 **Enter the structure coordinates:**

Right-click with the mouse → Edit Grid Data → Modify/Show System

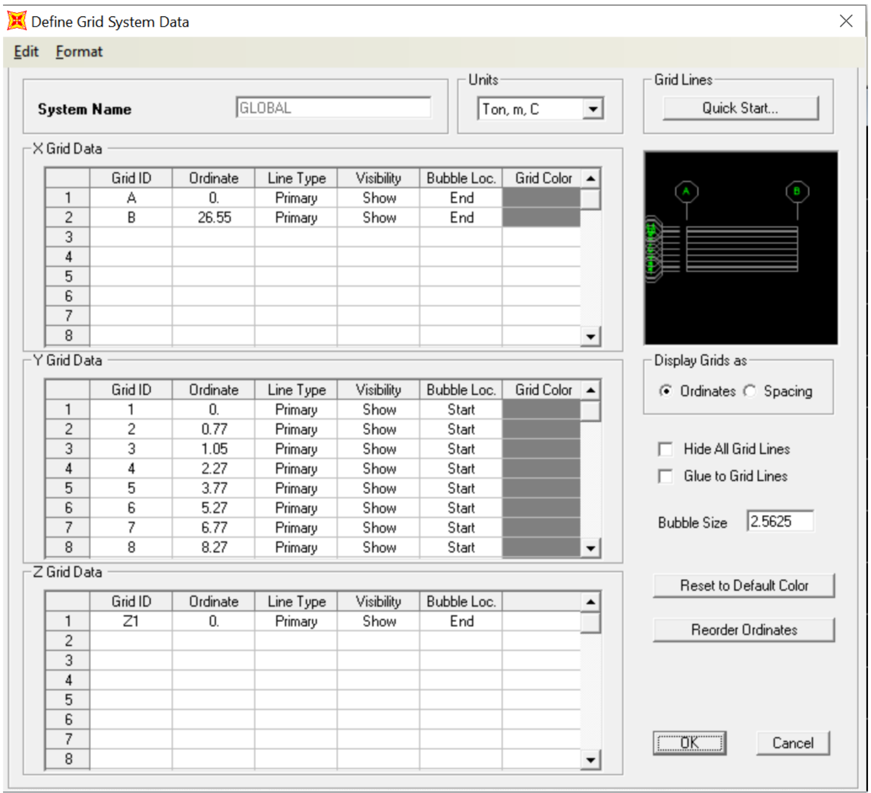


Fig. VII.3 Structurs coordinates

II.1.5 Selecting materials:

Define →Materiels →Add new material

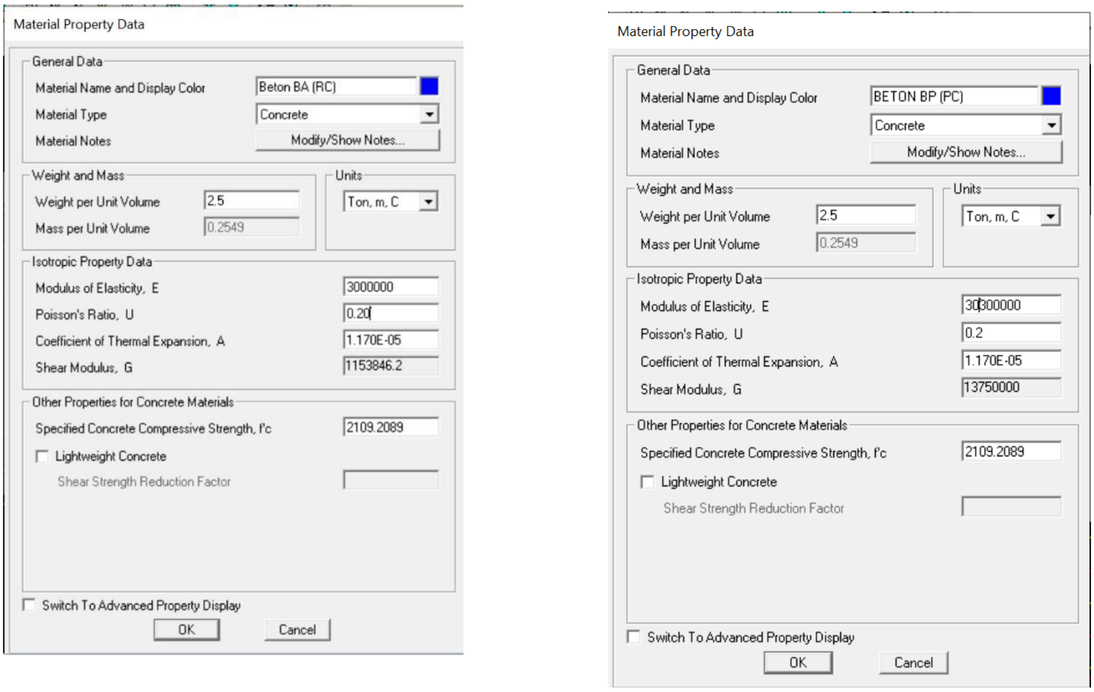


Fig. VII.4 Selecting Materials and its characteristics

II.1.6 selecting elements types:

Défine → Frame section → Add new property → Concrete → Rectangular section

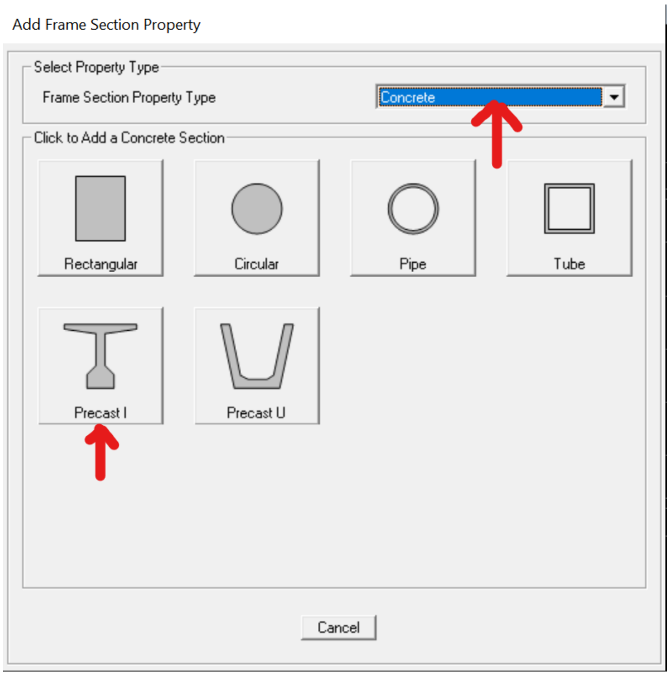


Fig. VII.5 Element type tab

Next, input the coordinates corresponding to the different girder's sections types.

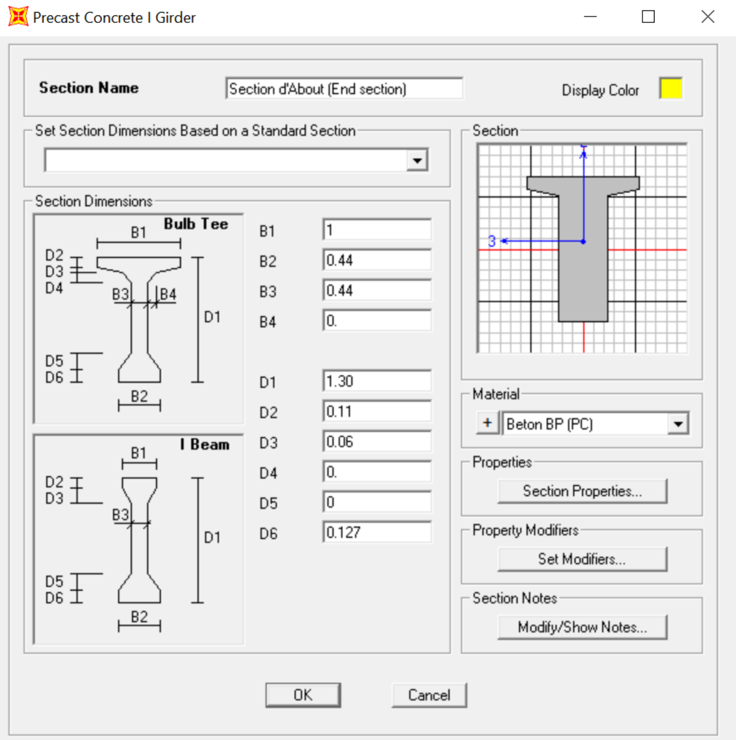


Fig. VII.6 End section

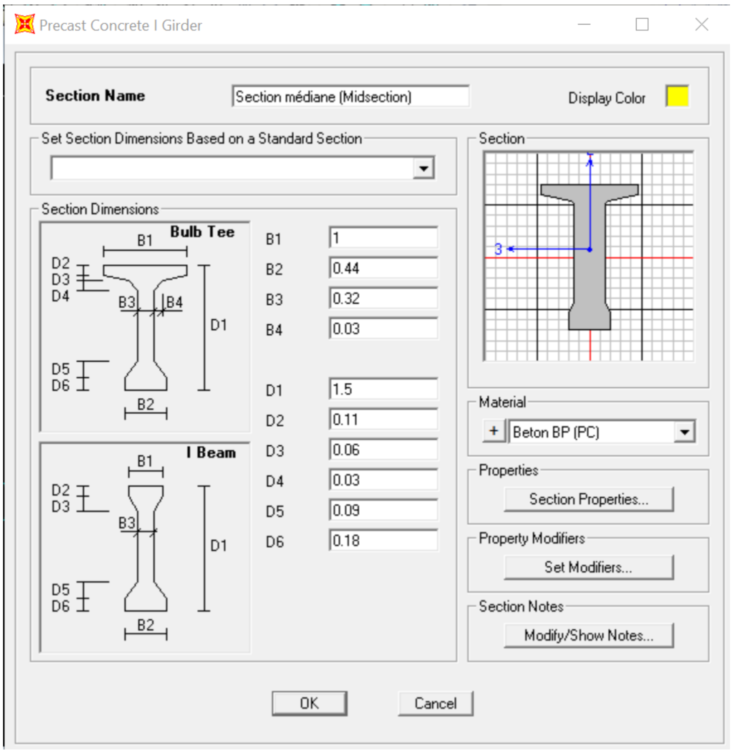


Fig. VII.6 Midsection

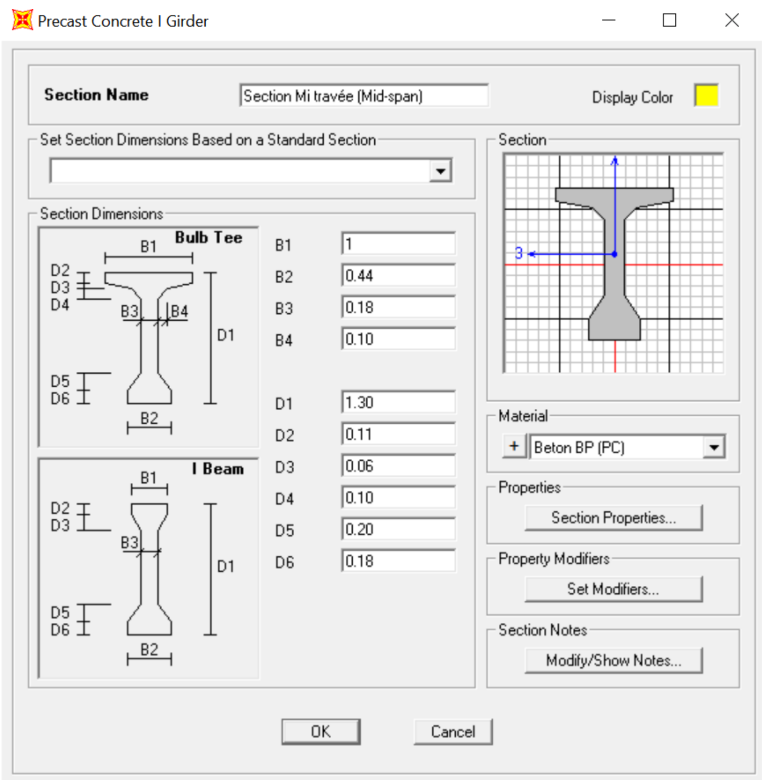


Fig. VII.7 Mid-Span section

This option allows the user to define the type of cross-section. In our case, we use three sections (end, midspan, and midsection), since the girder is prismatic. However, with a specific technique, it is possible to model a variable geometry girder.

Défine → Frame section → Add new property →other →nonprismatic

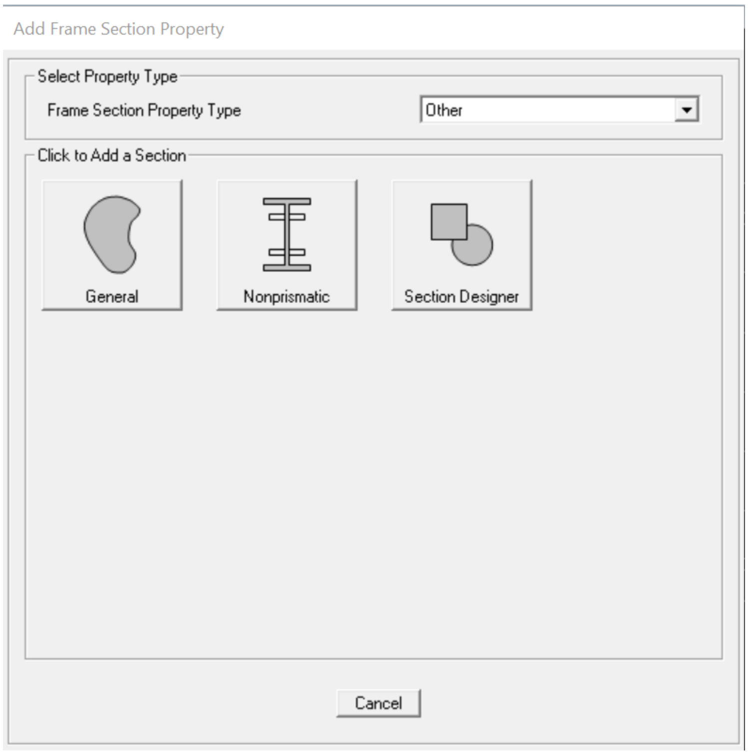


Fig. VII.8 Frame section property type

Then, the total geometry of the girder length is entered.

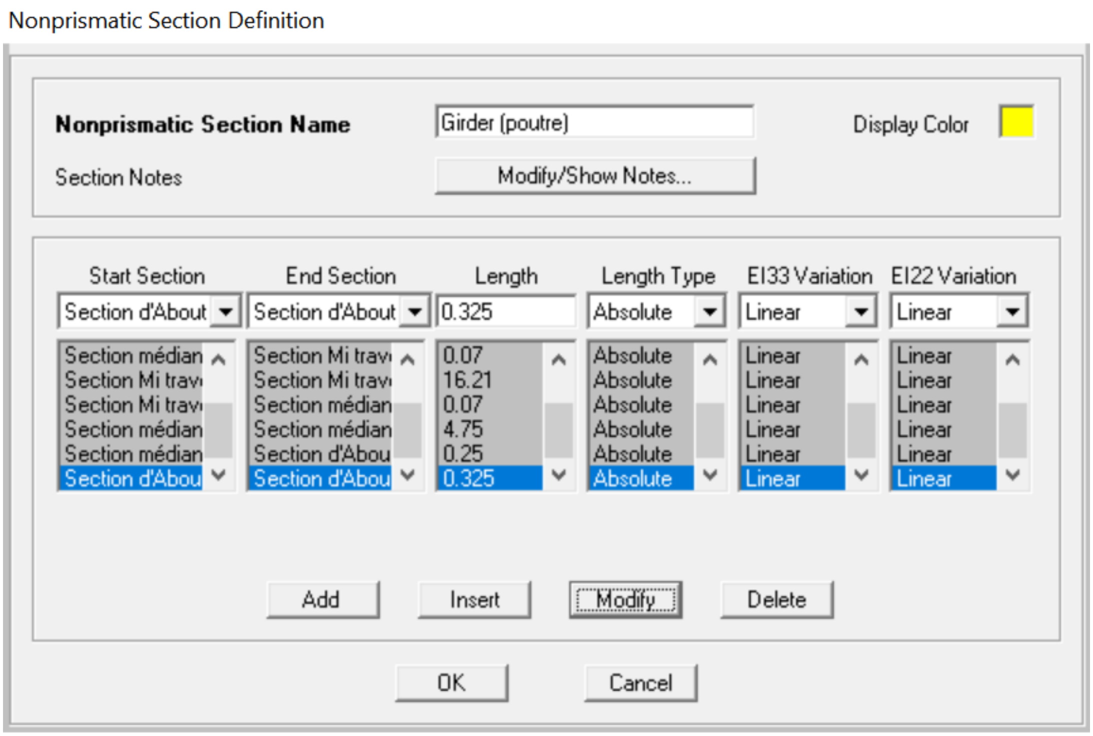


Fig. VII.9 Total geometry of the girder

II.1.7 Slab modeling:

Define →Area section

Define Area Section This option allows the user to define the type of slab. In our case, we use a thick plate with a thickness of 0.25 m.

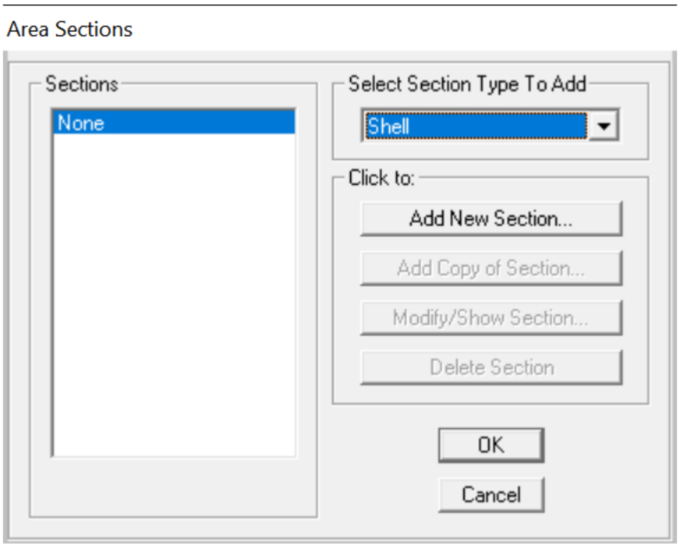


Fig. VII.10 Slab section selecting

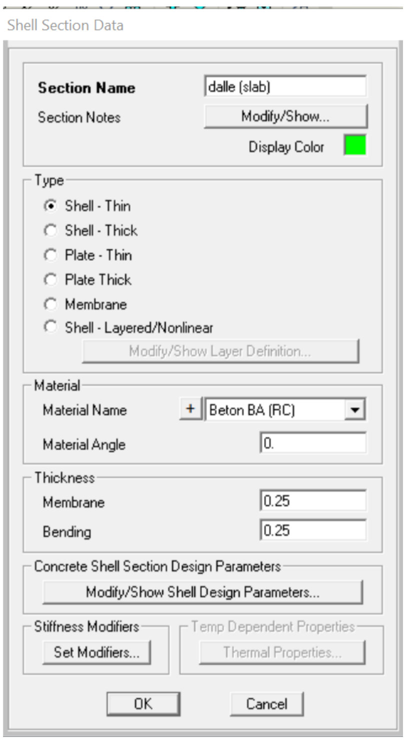


Fig. VII.11 Slab geometry defining

II.1.8 Draw the girders:

Draw→ Draw Frame → cable→ Tendon .

II.1.9 Draw the slab:

Draw →Draw rectangular area

II.1.10 Selecting support type:

Select the girders end points to place the supports.

Assign → Joint→ Restraints

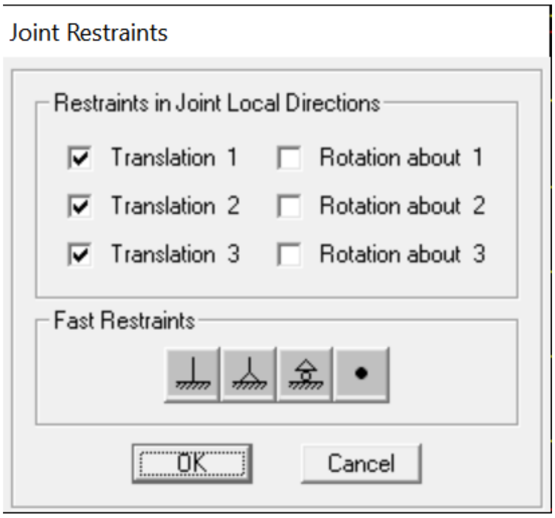


Fig. VII.12 Supports type tab

Then select the girders and:

Edit ⟹ Edit lines

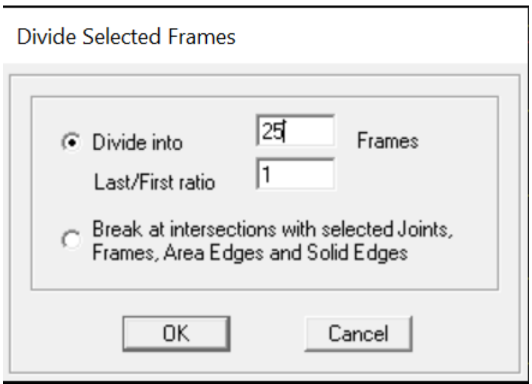


Fig. VII.13 Frame deviding

Select each slab:

Edit ⟹ Edit area

II.1.10Load Specification:

Define ⟹ load patterns

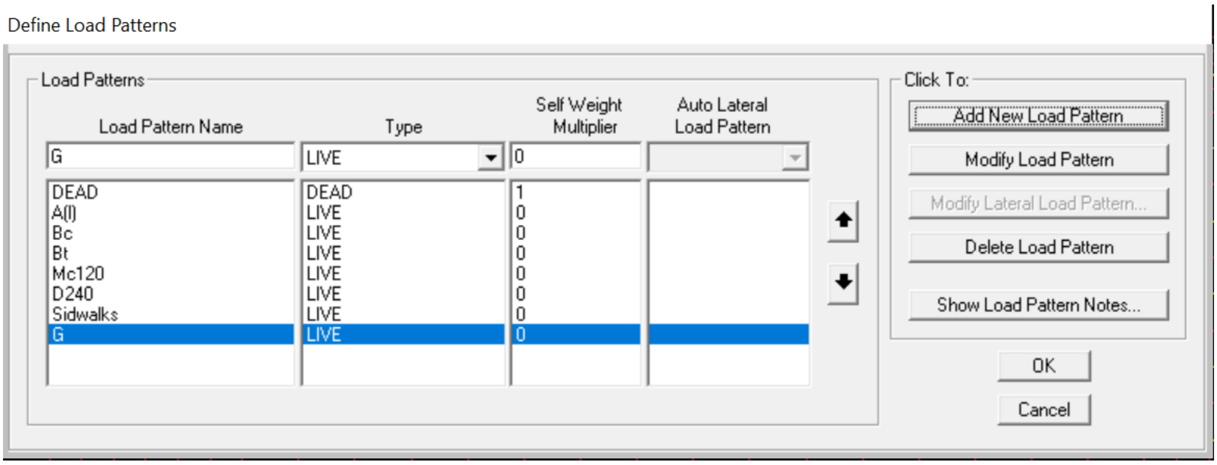


Fig. VII.14 load types defining

Then select each element and define the imposed loads:

Assign ⟹ Area load ⟹ uniform (shell)

For example: A(l)

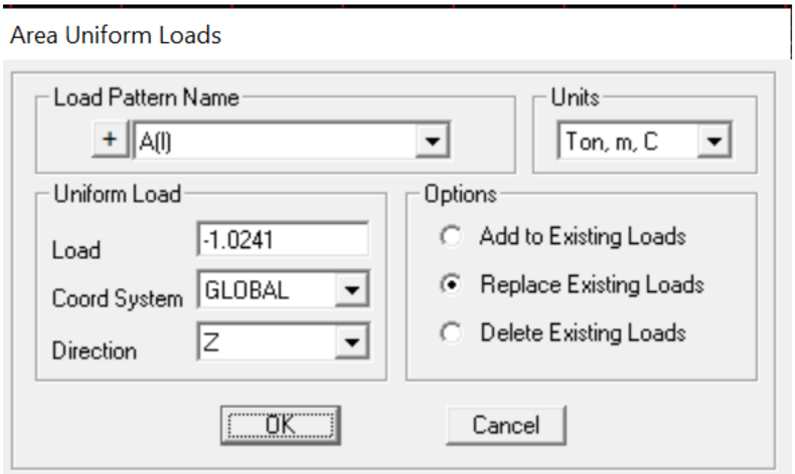


Fig. VII.15 A(l) loads defining

II.2 System Analysis:

Analyse ⟹ set analysis option ⟹ plan Grid

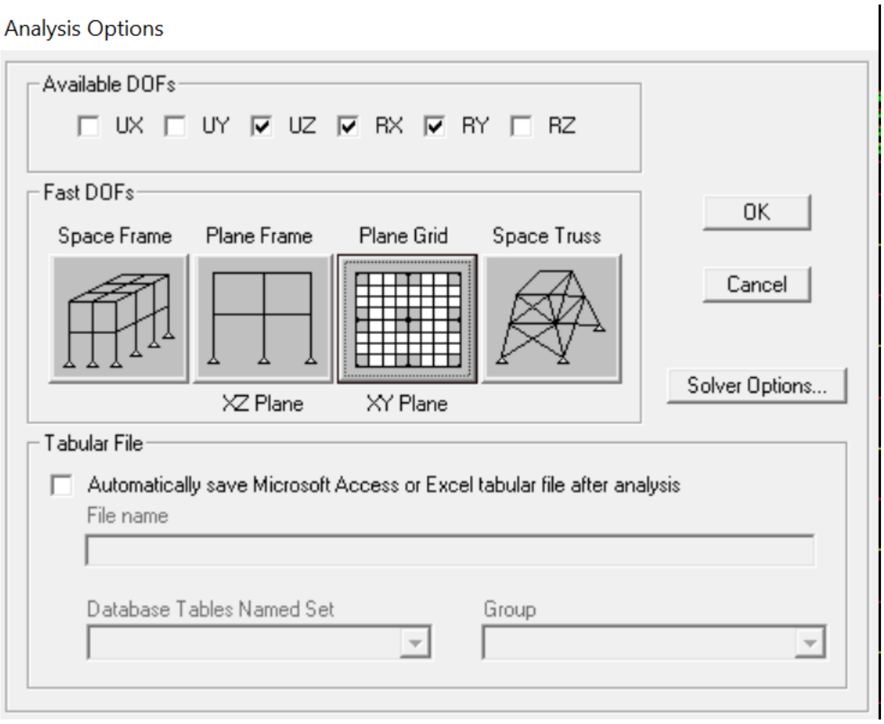


Fig. VII.16 Analyse options

Analyse ⟹ set load cases to run

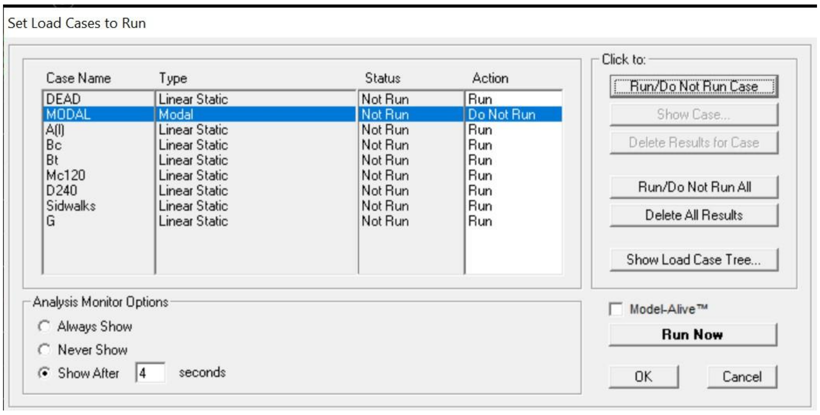


Fig. VII.17 load cases to run

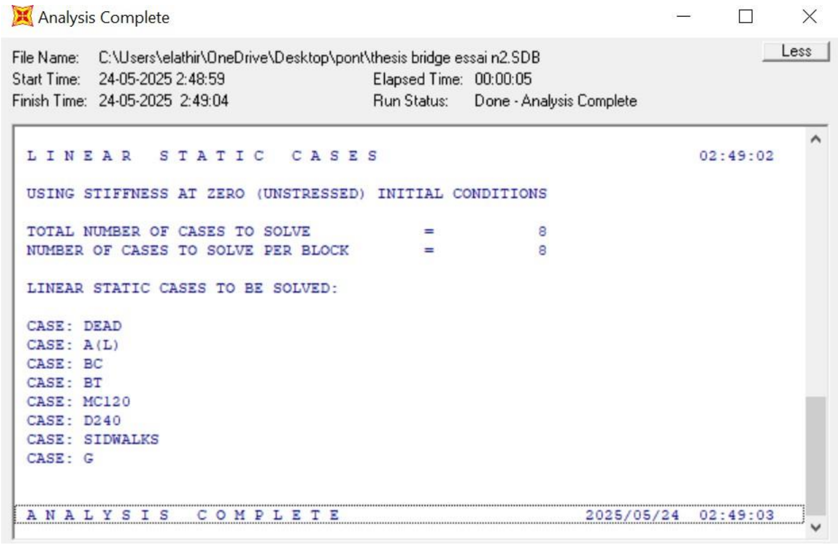


Fig. VII.18 Table of analysis

III. Comparison of Results:

- A(l)

Tab.VIII.1 A(l) G.MASSONET Vs SAP2000 results

N of girder	G.MASSONET	SAP2000	ΔM
Girder 4	107.244	101.343	5.901
Girder 5	105.579	99.381	6.198
Girder 6	101.613	94.154	7.459
Girder 7	96.264	87.239	9.024

- Sidewalks

Tab. VIII.2 Sidwalks G.MASSONET Vs SAP2000 results

N of girder	G.MASSONET	SAP2000	ΔM
Girder 4	3.314	2.992	0.322
Girder 5	3.535	3.00	0.535
Girder 6	4.0526	3.875	0.1776
Girder 7	4.818	4.00	0.818

Then compare D240, Mc120 ,Bc, Bt

CHAPTRE VIII :

PRESTRESS DESIGN

I. Principle of Prestressing:

Prestressing is a technique invented by Eugène Freyssinet in 1928, which involves tensioning the steel used as reinforcement in concrete, thereby applying a compressive force to the concrete when at rest.

A concrete structure is referred to as prestressed concrete when it is subjected to an artificially applied force system that induces permanent internal stresses. These stresses, when combined with those caused by external loads, result in total stresses that remain within the safe limits the concrete can indefinitely withstand.

Prestressing can be classified into two main categories:

Pre-tensioning (Prestressing by Pretension):

Tensioning of the steel tendons is performed before the concrete is poured.

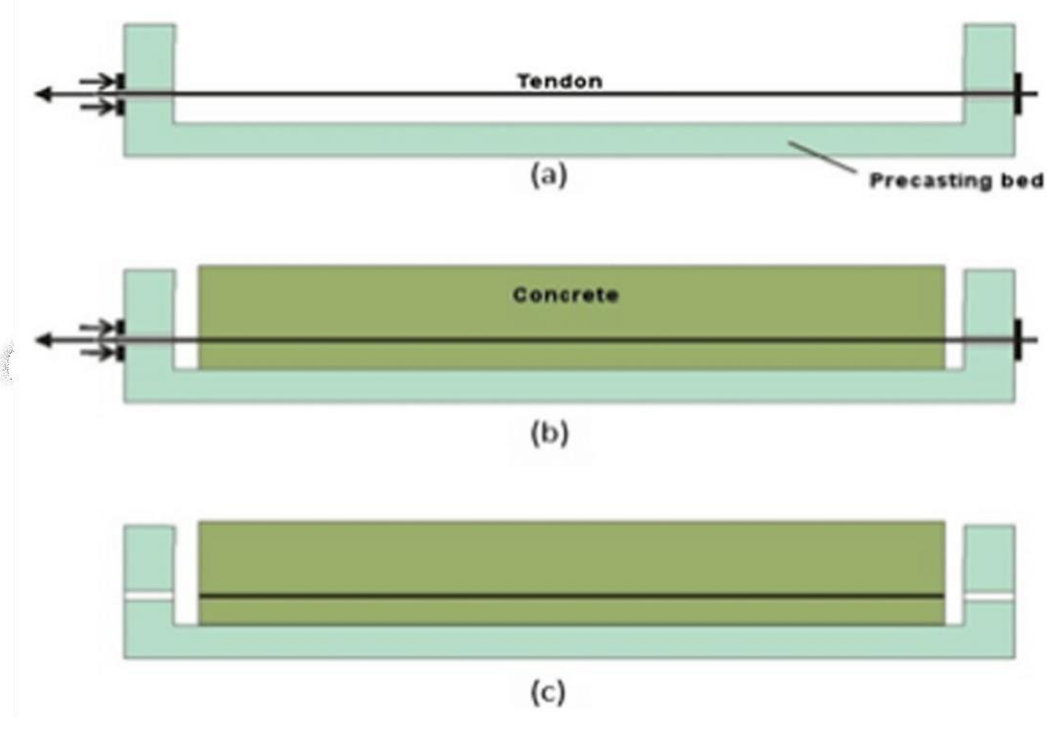


Fig. VIII.1 Longitudinal Section of a Prestressed Concrete Girder by Pre-tensioning

Post-tensioning:

Tensioning of the cables is carried out after the concrete has hardened.

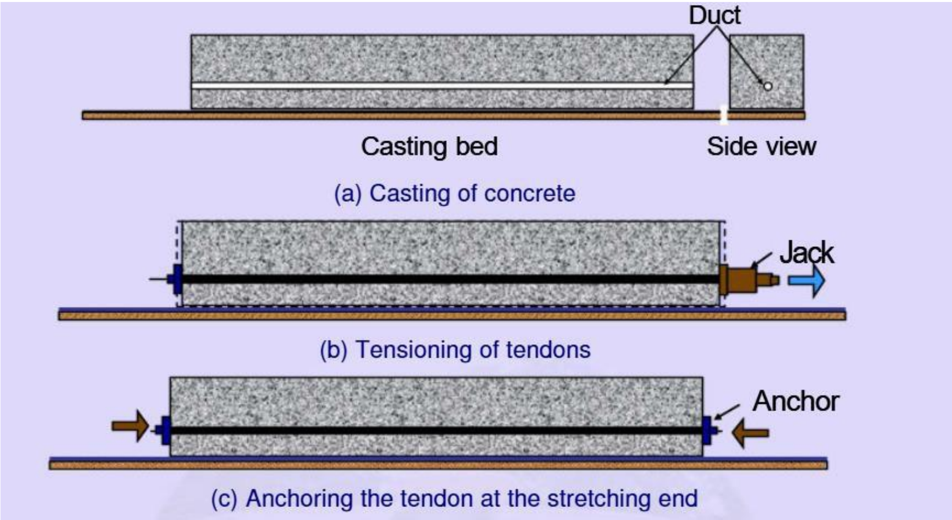


Fig. VIII.2 Longitudinal Section of a Prestressed Concrete Girder by Post-tensioning

In our case we use prestressing by post-tensioning

II. Post-Tensioning Prestressing:

The post-tensioning technique involves anchoring against already hardened concrete to tension the prestressing cable. The concrete element is first cast with ducts or sleeves reserved for the later installation of the prestressing system. Once the concrete reaches sufficient strength, the tendons (prestressing cables) are threaded through the ducts and then tensioned using hydraulic jacks.

Post-Tensioning Procedure:

- Installation of formwork.
- Placement of passive reinforcement (stirrups, ties, hoops, longitudinal bars, and support chairs for the ducts).
- Installation of the ducts and secure fixation onto the reinforcement cage to prevent any displacement during concrete pouring.
- Sealing of the ducts by applying adhesive tape to joints or accidental holes made during construction.

- Installation of bearing plates and adjacent confinement (frettage) near the ends of the ducts under the future anchorage locations.
- Concrete pouring.
- Allowing the concrete to harden until it reaches the minimum strength specified by the design office to permit tensioning.
- Once hardened, threading of the tendons (cables).
- Anchorage is secured using various wedge systems in a confined (fretted) concrete zone.
- Injection of cement grout into the ducts.

III. Prestressing Study:

The calculation is carried out on the most heavily loaded girder.

III.1 Input Data:

- Concrete:

- fc28 =25MPa (characteristic compressive strength at 28 days)
- ft28 =2.1MPa (tensile strength at 28 days)

- Prestressing Steel:

- The strands used in this project are 12T15 Super TBR with a nominal diameter of ϕ=15.7mm.
- Ultimate tensile strength $fprg$ =1770MPa
- Yield strength $fpeg$=1573MPa
- Modulus of elasticity Ep=190000MPa
- Cross-sectional area of one strand Ap=1800mm²
- Duct diameter: ϕg=82mm

III.2 Determining the Number of Cables:

The calculation will be performed for the most heavily loaded girder, subjected to a bending moment of 538.692786t.m

Geometrical characteristics of the mid-span section:

Tab. VIII.1 Geometrical properties of the mid-span section

	Girder alone	**Girder + slab**
S /Δ net	243635.765	751292.01
B net	3449.45	7011.9
V(cm)	59.3696778	47.8554803
V' (cm)	70.6303222	107.14452
IG (cm^2)	6806707.586	16530721.24
i^2 (cm^4)	1874.60964	2239.63165
ρ%	0.44704883	0.43679228

- **Initial Prestress (σ_0):**

σ0= min (0.8 fprg ;0.9fpeg)= min (0.8× 1770 ; 0.9 ×1573) = min (1416;1415.7)

σ0= 1415.7 =1416

- **Initial Prestressing Force:**

P0 = σ0× Ap = 1416 × 1800 × 10^{-6} = 2.548 MN.

- **Determining P:**

M girder = 93.183t.m = 0.93183MN.m

M min = M girder + M deck (slab +superstructure) = 229.754t.m =2.2975MN.m

M max = MG + Mc120 = 399.03t.m=3.9903MN.m (moments combinations tab)

ΔM = M max - M min = 1.6928MN.m

We have:

$$P_I = \frac{\Delta M}{\rho h} + \frac{Snette}{h}(v\bar{\sigma}_{ti} + \acute{v}\,\bar{\sigma}_{ts}) \qquad P_{II} = \frac{M_{max} + \rho S v \bar{\sigma} ti}{\rho v + \acute{v} - d\prime}$$

The equations that determine the minimum prestressing thus established, only the allowable tensile stresses **σ**ti and **σ**ts remain to be considered. These are conventional values and depend on the class of prestressing and the load combination in question. They are limited by (–ftj) within the concrete cover zone and –1.5 ftj outside this zone. Hence, σti = –2.1 MN and σts = –3.15 MN.

According to Prestressed Concrete at the Limit States by Robert Chaussin

• d_0: Concrete cover to prestressing steel

• d and d′: Minimum distances between the centroid of the prestressing tendons and the extreme tensile and compressive fibers (top and bottom fibers)

• e_0: Tendon eccentricity, defined as the distance between the centroid (center of gravity) of the concrete cross-section and the centroid of the prestressing tendons.

With:

d'=1.5ϕ =1.5×8.2=12.3cm

Where:

$P1=\frac{\Delta M}{\rho h}+\frac{Snet}{h}(V\sigma ti+V'\sigma ts)=\frac{1.6928}{0.436\times1.3}+\frac{0.7512}{1.3}\times((0.478\times-2.1)+(1.071\times-3.15))=-0.285\ MN$

$P2=\frac{Mmax\pm\rho Sv\bar{\sigma}ti}{\rho v+v-d'}=\frac{(2.2975+(0.436\times0.7512\times0.478\times-2.1))}{((0.436\times0.478)+1.071-0.123)}=2MN$

P1<P2 therfore:

Our section is overcritical; the tendon path intersects one of its boundaries, cutting through the concrete cover zone. Therefore, the economical prestressing force p_1 is no longer sufficient. As a result, the design is carried out on the overcritical section, assuming a full balancing of the permanent load.

• Determination of the required number of prestressing tendons:

The prestressing must counteract 100% of the permanent loads.

P min= sup (P1, P2) =2MN

Therfore: e0=v'+d=107.14452-12.3=94.844 cm=0. 94844 m

- **Nominal prestressing force:**

P0=min (0.8fprg× Ap ; 0.9fpeg ×Ap)

P0=min (0.8 ×1770 ×1800 ×10^{-6}; 0.9× 1573× 1800× 10^{-6}) = 2.54 MN

Number of cables, assuming prestress losses equal to 32%

n>$\frac{p}{0.68p0}$ = $\frac{2}{2.54\times0.68}$= 1.1579 (100%-32% =0.68)

We consider n=2 cables of 12T15

- **Verification of the upper limit of prestressing:**

Ensure that P_0 is not excessive in order to avoid overstressing the tendons

$nP0 \leq B_n \times \bar{\sigma}_{bc} - \frac{\Delta M}{\rho h}$

$\bar{\sigma}bc$: the permissible service stress in concrete

$\Longrightarrow$ $2\times2.54 \leq 0.70119 \times 20.77 - \frac{1.6928}{0.43679228\times1.55}$ $\Longrightarrow 5.08 \leq 12.8105$

- **Calculation of net and homogenized geometric properties:**

  ✓ **Net section:**

$B_n = B_b - nB_\emptyset$

$S_n = S_b - nB_\emptyset\times(\frac{0.123+0.18}{2})$

$I_n = I_b + (v'_b - v'_n)^2 \times Bn - [nI_\emptyset + B_\emptyset \sum_1^2 (v'_n - y_i)^2]$

Where: n = number of holes per section = 2

$\emptyset$: diameter of the duct=8.2cm

Yi=88.25cm

$B\emptyset = \frac{\pi\times\emptyset^2}{4}$, $I\emptyset = \frac{\pi\times\emptyset^4}{64}$

$\rho_n = \frac{In}{v_n\times v'_n\times B_n}$, $v'_n = \frac{Sn}{Bn}$, $v = h - v'_n$

- **Homogenized section:**

$B_h = B_n + k.n.Ap$

$$v'_h = \frac{v'_n \times Bn + k.Ap.\sum y_i}{Bh}$$

$$I_h = I_n + (v'_h - v'_n)^2 \times B_h + k.Ap\sum(y_i - v'_h)^2$$

Where:

k = 5: equivalence coefficient

n = 2: number of cables

Ap = 1800 mm²: cross-sectional area of one cable

- **Summary table (midspan section):**

Table VII.2 Geometric properties of the midspan section

	Girder	Girder+slab	
	net	net	Homogenized
B(m²)	0.3525	0.7275	0.7455
V'(m)	0.72	1.080	1.075
V(m)	0.58	0.47	0.475
I²(m⁴)	0.06785	0.1649	0.1655
ρ	0.46	0.44	0.4347

Table VII.3 Geometric properties of the end section section

	Girder	Girder+slab	
	net	net	Homogenized
B(m²)	0.6090	0.9840	1.002
V'(m)	0.71	0.97	0.968
V(m)	0.59	0.58	0.582
I²(m⁴)	0.085	0.221	0.2211
ρ	0.33	0.399	0.3916

- **Determination of the number of tendons at the end (without slab)** $\sigma_{sup} \geq \overline{\sigma_{st}}$ = -4.05MPa

The following equations must be verified:

$\sigma_{inf} \leq \overline{\sigma_{cs}} = 21MPa$

e0 = -v‘ +d‘ = -0. 6936+0.123 = -0.5706m (V' of the end section without slab)

III.3 Verification of normal stresses (at mid-span):

III.3.1 During construction (girder alone):

P0= n× P=2× 2.54=5.08 MN

- P1/B+(P1 ep + Mm) × v/I ≥ **σ**t
- P1/B-(P1 ep + Mm) × v’/I ≤**σ**c

P1=1.02×P0-0.8×Δpi× P0 =0.94 P0

P1=0.94×5.08=4.775 MN

P2=0.98×P0-1.2×Δpi× P0 =0.86 P0

P2=0.86×5.08=4.369 MN

- Verification:

- $\frac{4.775}{0.3525}$+(4.775× (-0.5706) +2.2975) ×$\frac{0.5936}{0.6806}$ = 12.828151MN >**σ**t=-4.05 MN (C.V)

- $\frac{4.775}{0.3525}$+(4.775× (-0.5706) +2.2975) ×$\frac{0.7063}{0.6806}$ = 13.102410MN < **σ**c=21MN (C.V)

III.3.2 In long-term service conditions:

- P2/B+(P2 ep + Mm) × v/I ≥ **σ**t
- P2/B-(P2 ep + Mm) × v’/I ≤**σ**c

P1=1.02×P0-0.8×Δps× P0 =0.82 P0

P1=0.82×5.08=4.166 MN

P2=0.98×P0-1.2×Δps× P0 =0.68 P0

P2=0.68×5.08=3.454 MN

- Verification:

- $\frac{3.454}{0.3525}$+(3.454 × (-0.5706) +2.2975) ×$\frac{0.5936}{0.6806}$ = 10.083474MN < **σ**c=21MN (C.V)

- $\frac{3.454}{0.3525}$+(3.454 × (-0.5706) +2.2975) ×$\frac{0.7063}{0.6806}$= 10.137563 >**σ**t=-4.05 MN (C.V)

III.4 Cables Positioning:

The two cables will be placed at the beam end in such a way that:

- Their distribution is approximately uniform.
- Their center of gravity coincides with that of the cross-section.
- The required clearances from the top and bottom fibers, as well as cable spacing, are respected.

A coordinate system (x, y) is selected, where:

- The x-axis passes through the bottom edge of the beam.
- The y-axis passes through the mid-span of the beam.

We set y = 0.5

$\sum M/_{fib\ inf} = 0 \Longrightarrow$ 2PV' = P1d1+P2(d1+y)

d1=$\frac{2v'-y}{2}$= $\frac{(2\times0.71)-0.5}{2}$= 46 cm

d2 =d1+y = 0.46 +0.5=96cm

This parabola follows a second-degree (quadratic) equation:

y=ax²+b

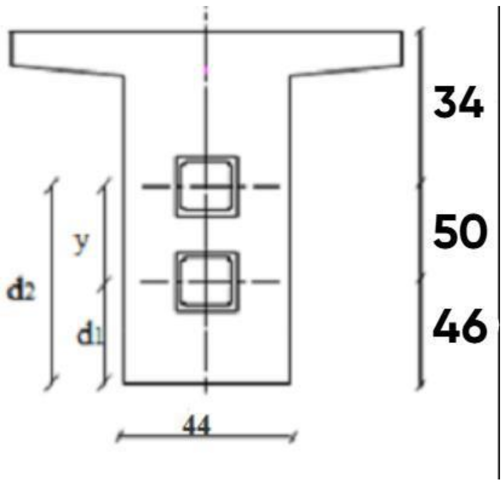


Fig. VIII.3 Cables positions

- **For cable 1:**

We have: for x=0 y1 =b1 =0.46m

For x=L/2 y1=0.123 = a(12.85)²+0.46

a1=-0.00204

Where : y = -0.00204x²+0.46

- **For cable 2:**

Y= a2x² +b2 with the same way we find: $Y= -0.00487x^2+0.96$

If we take the derivative of this equation with respect to x, we obtain:

$\frac{dy}{dx}$ =2ax = tgα l=26.55m

Summary table of yi and αi in function of x:

Table VII.4 Summary table of yi and αi as a function of x

cable	Position	0	1/8	1/4	31/8	1/2
1	yi**(m)**	0.46	0.438	0.370	0.257	0.100
	αi**(rd)**	0	0.01354	0.02707	0.04060	0.05411
2	yi**(m)**	0.96	0.906	0.745	0.477	0.101
	αi**(rd)**	0	0.03231	0.06456	0.09667	0.12859

III.5 Loss Calculation:

III.5.1 Instantaneous Losses:

a) Losses Due to Friction:

These are caused by the friction between the prestressing cable and the duct during tensioning.

The formula is given by:

$\sigma_{pfrot} = \sigma_{p0} \times - e^{-(f\alpha + \varphi L)}$ and $\Delta\sigma_f(x) = \sigma_{p0}(1 - e^{-(f\alpha + \varphi L)})$

With:

σ_{p0}: Initial prestressing stress

L: Distance from the considered section to the point of cable stressing (anchorage)

f: Friction coefficient of the cable in curves, f = 0.18

φ: Wobble coefficient (relative loss per meter), φ = 0.002

α: Total angular deviation of the cable in radians

We calculate friction losses at positions 0.5L and 0.25L using the profile of Cable 1.

Table VII.5 Losses Due to Friction

X(m)	0	L/4	L/2
σpfrot	1416	1390.522	1365.513
Δσpfrot	0	25.44	50.487

b) Losses Due to Anchorage Slip:

This tension loss results from the slip of the tendon relative to its anchorage, or from the deformation of the anchorage system itself.

$$d = \sqrt{\frac{g \times Ep \times L_{AB}}{\Delta\sigma_{AB}}}$$

With:

g: anchorage slip amount = 6 mm

Ep: modulus of elasticity of steel = 1.9×10^5 MPa

L: length of the tendon (cable)

$g \times Ep = 6 \times 10^{-3} \times 1.9 \times 10^5 = 1140$Mpa

Therefore: $d=\sqrt{\frac{6 \times 10^{-3} \times 1.9 \times 10^5 \times 6.6375}{25.44}} = 17.246$m

To calculate the losses at the different sections, we use "Thales" theorem applied to the stress diagram:

$$\Delta\sigma_{recul}\ 2\ \sigma_{p0}\ (f\times \frac{\alpha}{L} + \varphi) \times d$$

$$\Delta\sigma_{reel} = \Delta\sigma_{recul}\left(1 - \frac{x_i}{d}\right)$$

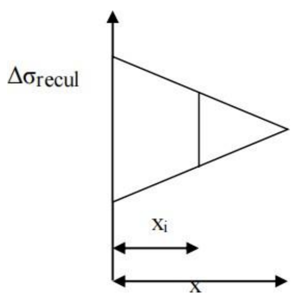


Table VIII.6 – Losses due to anchorage slip

X(m)	0	L/2	L/4	d
σrec	97.681	99.472	98.575	0
Δσreel	97.681	22.904	60.636	0

c) Losses due to elastic shortening of concrete:

$$\Delta\sigma_{élastique} = \frac{n-1}{2n} \times \frac{E_p}{E_{ij}} \sigma_b = \frac{2-1}{2\times2} \times \frac{1.9\times10^5}{11000\sqrt[3]{f_{c28}}} . \sigma_b = \mathbf{1.48}\, \sigma_b$$

σb: Concrete stress at the level of the cable at the time of stressing

$$\sigma_b : \frac{P}{B_n} + \frac{p.e_p^2}{I_n} + \frac{M_{poutre}.e_p}{I_n} \quad , \text{p= Ap}(\sigma_{p0} - \Delta\sigma_{frot}$$

ep: Eccentricity of the equivalent cable at the considered section; n: number of cables

Table VIII.7 Losses due to elastic shortening

X(m)	0	L/4	L/2
P(MN)	2.54	2.503	2.457
ep	-0.578	-0.591	-0.597
Mgirder	0	0.6817	0.9318
I(m^4)	0.085	0.06785	0.06785
B(m^2)	0.6090	0.3525	0.3525
σb (MPa)	14.154	14.057	11.693
Δσelastic	20.947	20.804	17.305

Table VIII.8 Summary table of instantaneous losses

X(m)	0	L/4	L/2
Δσpfrot	0	25.44	50.487
Δσreel	97.681	22.904	60.636
Δσelastic	20.947	20.804	17.305
Δσpi	118.628	69.148	128.428
$\frac{\Delta\sigma pi}{\Delta\sigma p0}$ %	8.377	4.883	9.070

III.5.2 Long-term losses:

a) Losses due to concrete shrinkage:

Shrinkage is a phenomenon of concrete shortening over time due to the evaporation of excess water contained in the concrete. The prestressing cables, being bonded to the concrete, undergo the same shortening.

The expression for this loss is taken as:

$\Delta\sigma r = Ea.\ \varepsilon r$

$\varepsilon r = 3 \times 10^{-4}$; and $Ea = 1.9 \times 10^{5}$ MPa

At the anchorage (abutment):

At 7 days: 15% of the shrinkage has dissipated.

At infinity (long-term): 100% of the shrinkage is dissipated.

Therefore: $\Delta\sigma_r = (1 - 0.15) \times 57 = 48.45$ MPa

b) Losses due to steel relaxation:

We use TBR (low-relaxation steel), since there's no cost difference between TBR and RN (normal-relaxation).

For TBR, we have $\rho_{1000} = 2.5\%$

$\Delta\sigma r\ (x) = \frac{6}{100} \times \rho 1000 \times (U - U0) \times \Delta\sigma pi\ (x)$

Where: σ_{pi} is the initial prestressing stress (after immediate losses)

$\sigma_{pi} = \sigma_{p0} - \Delta\sigma_{pi}$; $U = \frac{\sigma_{pi}}{f_{prg}}$; $U_0 = 0.43$] For TBR

Table VIII.9 Losses Due to Steel Relaxation

X(m)	0	L/4	L/2
σpi	1297.372	1346.852	1287.572
U	0.73	0.76	0.73
Δσr	53.382	34.22	57.79

c) Losses Due to Concrete Creep:

Concrete creep is the deformation that occurs under a constant sustained load over time. It results in a loss of stress in the prestressing tendons, given by the following formula: (According to BPEL):

$$\Delta\sigma_{fluage} = [\sigma_b^M + \sigma_b^f] \times \frac{E_p}{E_{ij}}$$

Where:

σ_b^M: the maximum compressive stress in the concrete at the centroid of the tendons

σ_b^f: the final compressive stress in the concrete

$$\sigma_b^M = \frac{P^M}{B_h} + \frac{P^M.e_p^2}{I_h} + \frac{M_G.e_p}{I_h} \; ; \; P^M = Ap(\sigma_{p0} - \Delta\sigma_i)$$

$$\sigma_b^f = \frac{P^F}{B_h} + \frac{P^F.e_p^2}{I_h} + \frac{M_G.e_p}{I_h} \; ; \; P^F = Ap(\sigma_{p0} - \Delta\sigma_i - \Delta\sigma_{iretr} - \Delta\sigma_{relax})$$

Tab. VIII.10 Losses Due to Concrete Creep

X(m) / losess	0	L/4	L/2
P^M	2.33	2.42	2.28
σ_b^M	5.480	5.91	5.50
P^f	2.15	2.275	2.126
σ_b^f	2.145	8.63	12.537
Δσfluage	45.0423	85.8905	106.5469

Tab. VIII.11 Summary Table of Long-Term Losses

X(m)	0	L/4	L/2
$\Delta\sigma_r$	48.45	48.45	48.45
Δσr	53.382	34.22	57.79
Δσfluage	45.0423	85.8905	106.5469
Δσdiffr	146.8743	168.5605	213.03

Tab. VIII.12 Summary Table of Losses

X(m)	0	L/4	L/2
Δσpi	118.628	69.148	128.428
Δσdiffr	146.8743	168.5605	213.03
Δσ =Δσpi +Δσdiffr	265.5023	237.7085	341.458

CHAPITRE IX

ABUTEMENT DESIGN

I. Introduction:

The abutment is one of the fundamental components of the overall bridge structure. It is also referred to as the end support and may be partially or fully embedded, located either at the crest or along the slope of an embankment. Massive backfilled abutments with an exposed front wall can also be constructed, typically positioned at the toe of the slope, especially when it is desired to minimize the deck length to the strict minimum. The associated wing walls can be either flared (splayed) walls or return walls.

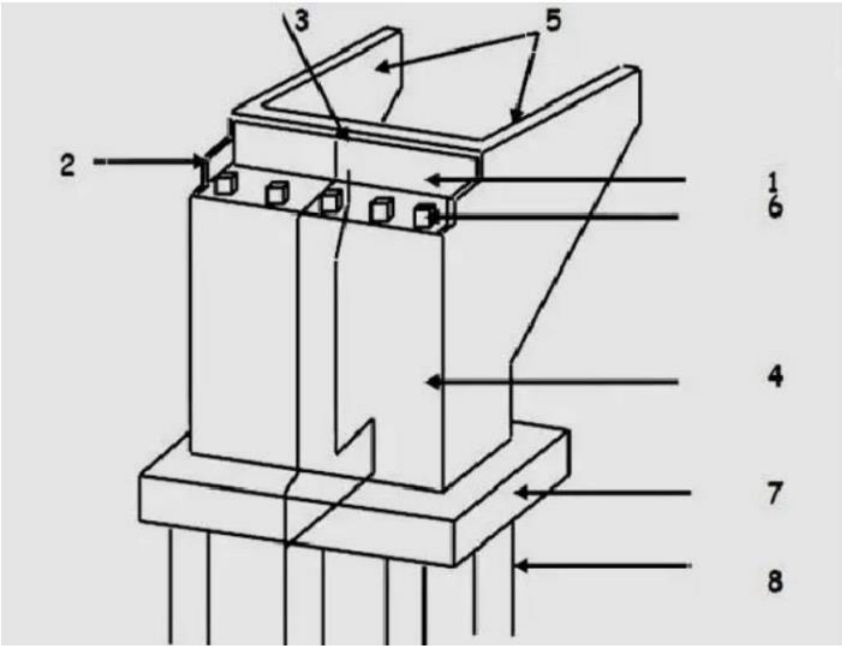


Fig. VIII.1. Structural elements of the abutment.

1) Bridge seat wall

2) Back Wall

3) Corbel

4) Abutment wall

5) Wing Wall

6) Bearing Pad / Elastomeric Bearing

7) footing

8) Pile Cap (Footing Cap)

II Verification of abutment stability:

The stability of the abutment shall be verified in both unloaded and service conditions, under both static and seismic loading scenarios.

II.1 Determination of the earth pressure coefficient Ka:

The earth pressure coefficient shall be determined under both static (non-seismic) and seismic conditions:

a) Under static conditions:

In static conditions, the active earth pressure coefficient is determined using Coulomb's theory.

$Ka = tg^2 \left(\frac{\pi}{4} - \frac{\varphi}{2}\right)$

b) Under seismic conditions (considering the effect of the earthquake):

Application of the « RPOA » standards.

- Seismic zone acceleration coefficient:According to the RPO (April 2007 version – Draft Seismic Regulation for Civil Engineering Structures), the zone acceleration coefficient A is defined based on the seismic zone and the importance level of the bridge, as shown in Table 1 below.

Tab. VIII.2 Seismic zone acceleration coefficient

Group de pont	Zone sismique			
	I	*IIa*	*IIb*	*III*
1	0.15	0.25	0.30	0.40
2	0.12	0.20	0.25	0.30
3	0.10	0.15	0.20	0.25

The dynamic earth pressure may be calculated using any scientifically validated method. If none is available, the Mononobe-Okabe method described below may be applied. Passive resistance and the weight of soil in front of the wall are generally neglected. Inertial forces of the wall or the soil resting on the footing are calculated using the seismic coefficients Kh and Kv as follows:

Kh = A

Kv = 0.3 × Kh A: zone acceleration coefficient.

Under seismic conditions, the total dynamic earth pressure which includes both the static and dynamic components of the active earth pressure is applied at mid-height of the wall (assuming a rectangular pressure distribution), and is given by:

$$\lambda_{ah} = \frac{\cos^2(\varphi+\alpha-\theta)}{\left[1+\sqrt{\frac{\sin(\varphi+\delta).\sin(\varphi-\beta-\theta)}{\cos(\delta-\alpha+\theta).\cos(\alpha+\beta)}}\right]^2 .\cos^2\alpha}.K.\frac{\cos(\delta-\alpha)}{\cos(\delta-\alpha+\theta)}$$

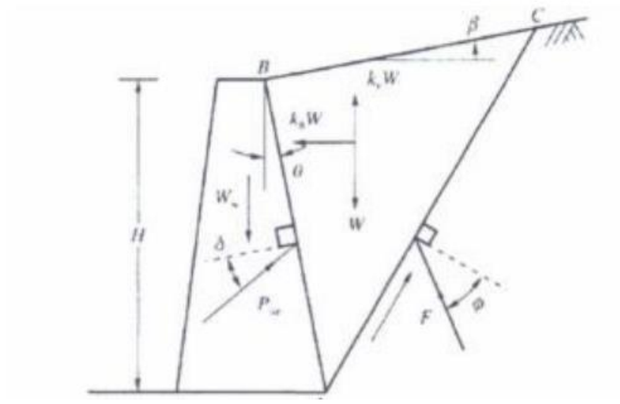


Fig. VIII.3 Mononobe-Okabe approach

With:

δ Wall-soil friction angle (friction between back of wall and soil)

λae Seismic active earth pressure coefficient

Θ: is the angle of the seismic inertial force $\theta=\tan^{-1}(\frac{kh}{1-kv})$

H is the height of the wall;

γ is the unit weight (bulk density) of the backfill soil;

ϕ = 30°, the internal friction angle of the cohesionless backfill;

β is the inclination angle of the backfill surface with respect to the horizontal.

α Inclination of the retaining wall from the vertical

K Seismic inertia correction factor

II.2 Load Calculation:

The earth pressure coefficients for the various cases are presented in the table below:

Tab. VIII.2 Earth pressure coefficient table

Static condition	K_h	K_v	K	θ	λ
Seismic action	0	0	1	0	0.33
H+V+	0.1	0,03	1.02	5.545	0.4
H+V-	0.1	-0,03	0.99	5.886	0.388

II.3 Calculation of the self-weights on the abutment:

II.3.1 Self-weight of the bridge seat wall:

✓ Height: H = height of the beam + height of the bearing device.

So: Height: H = 1.871 m

✓ Thickness (top cap): e = 0.32 m

✓ Length = width of the deck L = 10.42 m

$Ppw = \gamma c \times e \times H$

Ppw = 2,5 × 0,32 × 1,871 × 10,42

Weight Ppw = 15,596 t

II.3.2 For the transition slab:

✓ The transition slab has the same transverse dimension as the parapet wall.

The length is given by the following formula:

L= min(6m, max(3m,0.6H)).

H= height of the embankment= height of the front wall + parapet wall = 5.23 m.

Therefore: L=9.05m,. l=4.88m .

It is cast on a lean concrete (blinding layer) with a constant thickness of 30 cm, and supported on the corbel.

$P_{TS} = \gamma c .e.l .L$

$P_{TS} = 2{,}5 \times 0{,}3 \times 4.88 \times 9.05$

$P_{TS} = 33$ t

II.3.3 Weight of the corbel:

✓ Length = width of the deck – 2 × (thickness of the return wall).

Length: L = 10.42 – 2(0.63) = 09.16 m

$P_c = \gamma b \, S \, L$

S=S1 + S2

$S1 = 0{,}3 \times 0{,}3 = 0{,}09 \, m2$

$S2 = 0{,}3 \times 0{,}35/2 = 0{,}05 m2$

$S = 0{,}140 m2$

$P_c = 2{,}5 \times 0{,}140 \times 09{,}16 = 3{,}206$ t

II.3.4 Self-weight of the abutment wall:

Length: it is the same as that of the deck = 10.42 m

$P_{fw} = \gamma c \times e \times H \times L$

$P_{fw} = 2.5 \times 1.43 \times 3.45 \times 10.42$

$P_{fw} = 129$t

II.3.5 Self-weight of the return wall:

The return wall has a constant thickness of 0.63m to ensure proper reinforcement and good concreting.

It has a length of 5.68 m and a height of 5.321m, which is the same as that of the abutment.

$P_{RW} = \gamma C \times S \times L$

Where: S is the surface area of the return wall.

S = S1+S2+S3

$S = (2{,}66 \times 1) + (5{,}321 \times 3{,}02) + (\frac{2{,}66 \times 1.78}{2}) = 21{,}096 m^2$

The weight of the two return walls is:

$P_{RW} = 2{,}5 \times 0{,}63 \times 21{,}096 = 33{,}226$t

II.3.6 Self-weight of the footing:

Ps = γb×e×l×L

Ps = 2,5×1,5×5.60×11

Ps = 231 t

II.3.7 Weight of the soil:

a) On the transition slab:

P = γs H l L

P = 2×0,7×4.88×9.05

P = 61.829t

b) On the footing:

P = γs H l L

l = 1.9 m (width of the footing to the right of the abutment wall)

L = 9 m (length of the footing beneath the embankment)

H: height of the embankment above the footing

P = 2×3.45×3,12×9.15=196,981t

II.3.8 Surcharge loads:

P= γq.l.L

P= 1,2×4.88×9.05=48t

According to RPOA (Seismic Code for Civil Engineering Structures), we have:

➢ The horizontal seismic force:**Hs=kh P(t)**

➢ The vertical seismic force: **Vs=(1±kv) P(t)**

II.4 The calculation of moments on the abutment is carried out with respect to point 0:

Ms: represents the stabilizing moment due to vertical loads.

Mf: represents the overturning moment due to horizontal loads.

Mf=(horizontal forces×vertical lever arm)

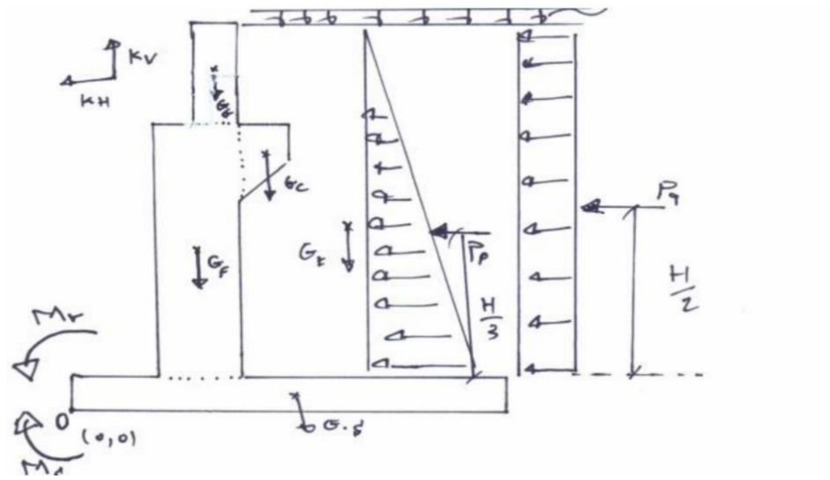


Fig. VIII.4 Moment analysis on the abutment about point 0

Table VIII.3 Values of stabilizing and overturning moments due to permanent loads.

Load Type		P(t)	Kh.P(t)	(1 ±Kv).P(t)	H(m)	V(m)	Mr(t.m)	Ms(t.m)
Bridge seat wall	C.S	15.596	0	15.596	2.32	5.885	0	36.18272
	H+V+	15.596	1.559	16.063	2.32	5.885	9.1790258	37.268201
	H+V-	15.596	1.559	15.128	2.32	5.885	9.1790258	35.097238
Abutement wall	C.S	129	0	129	1.765	3.225	0	227.685
	H+V+	129	12.9	132.87	1.765	3.225	41.6025	234.51555
	H+V-	129	12.9	125.13	1.765	3.225	41.6025	220.85445
Return wall	C.S	33.226	0	33.226	4.619	3.390	0	153.49747
	H+V+	33.226	3.322	34.222	4.619	3.390	11.26527	158.10239
	H+V-	33.226	3.322	32.229	4.619	3.390	11.265275	148.89255 0
Transition slab	C.S	33	0	33	4.92	5.922	0	162.36
	H+V+	33	3.3	33.99	4.92	5.922	19.54425	167.2308
	H+V-	33	3.3	32.01	4.92	5.922	19.54425	157.4892
Footing	C.S	231	0	231	2.8	0.75	0	646.8
	H+V+	231	23.1	237.93	2.8	0.75	17.325	666.204
	H+V-	231	23.1	224.07	2.8	0.75	17.325	627.396
Corbel	C.S	3.206	0	3.206	2.611	4.594	0	8.3715072
	H+V+	3.206	0.320	3.30218	2.61	4.594	1.4730608	8.6226524
	H+V-	3.206	0.320	3.10982	2.611	4.594	1.4730608	8.1203619
Weight of the soil on TS	C.S	61.829	0	61.829	4.92	6.172	0	304.19868
	H+V+	61.829	6.182	63.68387	4.92	6.172	38.163950	313.32464
	H+V-	61.829	6.182	59.97413	4.92	6.172	38.163950	295.07271
Weight of the soil on footing	C.S	196.98	0	196.98	4.65	4.160	0	915.957
	H+V+	196.98	19.69	202.8894	4.65	4.160	81.953529	943.43571
	H+V-	196.98	19.69	191.0706	4.65	4.160	81.953529	888.47829
Surcharge loade	C.S	48	0	48	4.08	7.321	0	195.84
	H+V +	48	4.8	49.44	4.08	7.321	35.1408	201.7152
	H+V-	48	4.8	46.56	4.08	7.321	35.1408	189.9648

II.4.1 Overturning moment values due to earth pressures:

Under seismic conditions, the total dynamic earth pressure including both the static and dynamic components of active earth pressure is given by :

$Pt = \frac{1}{2}\gamma ka H^2 L$

Ka: Earth pressure coefficient.

γ = 2 t/m³; Unit weight of the backfill.

Table VIII.4 Load values due to earth pressures

Load Type		P(t)	Kh.P(t)	(1 ± Kv).P(t)	H(m)	V(m)	Mr(t.m)	Ms(t.m)
BSW+AW	C.S	0.5×2×0.33×5.321²×10.42	97.3572228	/	/	3.27	318.35811	/
	H+V+	0.5×2×0.40×5.321²×10.42	118.008755	/	/	3.27	385.88862	/
	H+V-	0.5×2×0.38×5.321²×10.42	114.468492	/	/	3.27	374.31196	/
Footing	C.S	0.5×2×0.33×1.50²×11	8.1675	/	/	0.5	4.08375	/
	H+V+	0.5×2×0.40×1.50²×11	9.9	/	/	0.5	4.95	/
	H+V-	0.5×2×0.38×1.50²×11	9.603	/	/	0.5	4.8015	/

II.4.2 Overturning moment values due to backfill surcharge:

When the embankment supports a uniform load of intensity q, the corresponding total dynamic active earth pressure is taken as:

$$P_{ad}(q) = qK_{ad}H.L$$

Table VIII.5 Load values due to backfill surcharges

Load Type		P(t)	Kh.P(t)	(1 ± Kv).P(t)	H(m)	V(m)	Mr(t.m)	Ms(t.m)
BSW+AW	C.S	1.2×0.33×5.321×10.42	21.95614872	/	/	3.4105	74.88144521	/
	H+V+	1.2×0.4×5.321×10.42	26.6135136	/	/	3.4105	90.76538813	/
	H+V-	1.2×0.388×5.321×10.42	25.81510819	/	/	3.4105	88.04242649	/
Footing	C.S	1.2×0.33×1.50×11	6.534	/	/	0.75	4.9005	/
	H+V+	1.2×0.40×1.50×11	7.92	/	/	0.75	5.94	/
	H+V-	1.2×0.38×1.50×11	12.804	/	/	0.75	9.603	/

II.4.3 The sum of all load effects is summarized in the following table:

Summary table showing the overall moments and forces.

Table VIII.5 Summary table

Load Type		V=∑Kh.P(t)	H=∑(1±Kv).P(t)	∑Mr (t,m)	∑Ms (t,m)
Total	C.S	134.0148715	751.837	402.2238137	2650.89238 2
	H+V+	237.6259685	774.39211	743.1914078	2730.41915 3
	H+V-	237.8743004	729.28189	732.4062873	2571.36561 1

II.4.4 Overturning stability check:

In static conditions, the overturning safety factor (OSF) is Fe = 1.5

- ❖ $\frac{\Sigma Ms}{\Sigma Mr}$ > 1.5 Static condition

Under seismic conditions, the safety factor against overturning (S.F.O) is

Fv = 1.

- ❖ $\frac{\Sigma Ms}{\Sigma Mr}$ > 1 Seismic condition

Static condition:

C.S:

$\frac{\Sigma Ms}{\Sigma Mr}$ =6.590 > 1.5 C.V

Seismic condition:

H+V+:

$\frac{\Sigma Ms}{\Sigma Mr}$ = 3.673> 1 C.V

H+V-:

$\frac{\Sigma Ms}{\Sigma Mr}$ = 3.756> 1 C.V

In all three cases, it is clearly observed that both conditions (stability against overturning and sliding) are satisfied; therefore, the stability of the abutment is ensured.

III Reinforcement of the abutment elements:

The abutment elements will be reinforced in accordance with BAEL standards.

a) Concrete:

Unit weight of concrete: γb = 2.5 t/m³

Compressive strength: For 28-day-old concrete, we have: fc28 = 25 MPa for the concrete used in the superstructure, fc28 = 27 MPa for the concrete used in supports (abutments and piers), and foundations.

Tensile strength of concrete:

Ft28= 0,6+0,06fcj= 0,6+0,06 (27) = 2,22 MPa. (fc28=27MPa).

Allowable compressive stress (ULS):

$$\mu = \frac{M_u}{bd^2 f_{bu}}$$

fc28: characteristic compressive strength at 28 days.

γb = 1.5 for persistent or transient load cases.

γb = 1.15 for accidental load cases.

θ = 1 when the expected duration of the considered load combination exceeds 24 hours.

Service stress:

$\overline{\sigma_b}$ = 0.5 fc28 for the completed structure

$\overline{\sigma_b}$ = 0.5 fc28 during construction or in accidental situations

Poisson's ratio:

ν = 0 for cracked concrete (ULS)

ν = 0.2 for uncracked concrete (SLS)

b) Steel

The reinforcement used is standard high-bond steel (fe500).

The yield strength is: fy= 500 MPa

The modulus of elasticity is: Es= 210,000 MPa

Tensile stress limits:

ULS (Ultimate Limit State): $\sigma s = \frac{fe}{\gamma s}$

γs= 1.0 for accidental situations

γs= 1.5 for persistent or transient situations

SLS (Serviceability Limit State):

In cases of detrimental cracking:

$$\begin{cases} \text{En fissuration préjudiciable} : \overline{\sigma_{st}} = \min(\frac{2}{3} f_e ; 110\sqrt{\eta f_{tj}}). \\ \text{En fissuration très préjudiciable} : \overline{\sigma_{st}} = \min(\frac{1}{2} f_e ; 90\sqrt{\eta f_{tj}}). \end{cases}$$

η : coefficient de fissuration.

$\eta = 1$ pour les armatures rondes lisses.

$\eta = 1,\theta$ pour les armatures hautes adhérence.

II Design and reinforcement of the abutment:

II.1 The transition slab:

The transition slab rests on the corbel (double support) and on the ground, considered as an elastic support.

It is analyzed as a simply supported rectangular slab, although in reality, it is supported on elastic supports over its entire surface (direct contact with the soil).

II.1.1 Load Evaluation:

➢ Weight of the transition slab:

WTS = $\gamma c \times e \times 1 = 2.5 \times 0.3 \times 1 = 0.75$ t/m.l

➢ Weight of the backfill:

W1 = $\gamma s \times e \times 1 = 2 \times 0.5 \times 1 = 1$ t/m.l

➢ Self-weight of the pavement:

W2 = $\gamma bc \times e \times 1 = 2.2 \times 0.08 \times 1 = 0.176$ t/m.l

($\gamma bc = 2.2$ t/m^3: unit weight of bituminous concrete)

➢ Total dead load:

G = Wtot = WTS + W1 + W2 = 1.926 t/m.l

➢ Live loads:

Q = q × 1 = 1.2 × 1 = 1.2 t/m.l

Dead loads: G = 1.926 t/m.l

Live load: Q = 1.2 t/m.l

$MG=\frac{ql^2}{8}=\frac{1.926\times 4.88^2}{8}= 5.7331$ t.m/ml

$Mq=\frac{ql^2}{8}=\frac{1.2\times 4.88^2}{8}= 3.57216$t.m/ml

$VG=\frac{ql}{8}=\frac{1.926\times 4.88}{2}= 4.6994$ t.m/ml

$Vq=\frac{ql}{8}=\frac{1.2\times 4.88}{2}= 2.928$ t.m/ml

II.1.2 Loads combinations:

ULS:

MULS = 1,35 MG +1,6 MQ = 13,4550 t.m /ml.

VULS = 1,35 VG+1,6 VQ= 11,02899 t /ml

SLS:

MSLS = MG +1,2 MQ= 10,0196 t.m /ml.

VSLS = VG +1,2 VQ= 8,213 t /ml.

II.1.2.3 Reinforcement:

The reinforcement is designed for simple bending at the Ultimate Limit State (ULS), for a strip 1 meter wide.

a) **Vertical reinforcement:**
   A rectangular section is assumed.

- b=1
- d = 0,9h = 0,9*0,3 = 0,27m
- fc28 = 27 Mpa

$$\mu = \frac{Mu}{b.f_{bu}.d^2}$$

With, Mu =13,4550t .m/ml (under the effect of the maximum loadcombination)

$f_{bu} = \frac{0,85 \times f_{c28}}{\theta \times \gamma b}$; with Θ = 1 and γc = 1,5

Fbu=$\frac{0.85*27}{1*1.5}$= 15.3 Mpa

μ=$\frac{13.4550}{1*15.3*(0.27)^2}\times 10^{-2}$= 0.120

μ =0,120 < μ = 0.186 Thus, the calculation is carried out with respect to pivot point A

As=$\frac{M\mu}{Z\bar{\sigma}s}$ With: σs=$\frac{fe}{\gamma s}$=$\frac{500}{1.15}$=434.7826 MPa

$Z=d(1-0,4\alpha)$

$\alpha = 1,25(1-\sqrt{1-2\mu})$

$\alpha = 1,25(1-\sqrt{1-2\times(0,120)}) = 0,160$

$Z = 0,27(1-0,4\times 0,160) = 0,252m$

Hence, $As=\frac{M\mu}{Z\bar{\sigma}s}=\frac{13.4550}{0.252*434.782}\times 10^2= 12.2803cm^2$

b) Non-brittleness condition:

$As_{min} = 0,23\frac{bdf_{tj}}{f_e}$;

$Asmin=0.23\frac{100\times27\times2.22}{500}=$ 2.757 cm²

As =12.2803cm2 > Asmin = 2.757 cm2 ⟹ Condition verified

We use 8 HA14 bars with a total cross-sectional area of 12.31 cm².

c) Horizontal reinforcement:

At=As/3=4.0933cm²

That is, use 5 HA12 bars, with a total cross-sectional area of 4.52 cm²

II.2 Corbel:

The corbel serves as a support for the transition slab and is subjected to the reaction force from that slab.

- Reaction from the transition slab: R_1 = P.L/2 = 2,5*0,3*4.88*1/2 = 1,83 t/ml
- Reaction from the weight of soil: R_2 = P.L/2 = 2*0,5*4.88*1/2 = 2,44t/ml
- Reaction from the wearing course (deck finish):
  R_3 = P.L/2 = 2,2*0,08*4.88*1/2 = 0,429 t/ml
- Self-weight of the corbel: R4 = Wc =2,5*((0,3*0,3) +($\frac{0,35*0,3}{2}$))*1=0,3562t/ml
- Reaction from surcharge on the backfill: R5 =P.L/2 = 1,2*4.88*1/2 =2.928 t/ml

With: L = length of the transition slab bearing on the corbel (m)

II.2.1 Loads combinations:

Réactions : RULS= 1,35(RI + R2 + R3 + R4) + 1,6 (R5) = 11.5093 t/ml

RSLS = (RI + R2 + R3 + R4) + 1,2(R5) = 8.5688t/ml.

Moments : MULS= [1,35(RI + R2 + R3 + R4) + 1,6(R5)] x (L/4)= 0,8631 t.m/ml

MULS= [(RI + R2 + R3 + R4) + 1,2 (R5)] x (L/4)=0,64266 t.m/ml

II.2.2 Reinforcement:

a) Vertical reinforcement:

The corbel is considered as a cantilever embedded into the parapet wall.

The corbel is subjected to simple bending (pure flexure).

$$\mu = \frac{Mu}{b.f_{bu}.d^2}$$

With: Mu=0,8631 t.m/ml

- b=1
- d = 0,9h = 0,9*0,3 = 0,27m

$\mu=\frac{Mu}{b*fbu*(d)^2} = \frac{0,8631}{1*15.3*(0.27)^2}\times10^{-2}= 0.00773$

μ =0,00773 < μ = 0.186 Thus, the calculation is carried out with respect to pivot point A

As=$\frac{M\mu}{Z\bar{\sigma}s}$ With: $\sigma s=\frac{fe}{\gamma s}=\frac{500}{1.15}$=434.7826 MPa

$Z=d(1-0,4\alpha)$

$\alpha = 1,25(1-\sqrt{1-2\mu})$

$\alpha = 1,25(1-\sqrt{1-2\times(0,00773)}) = 0,0097$

$Z = 0,27(1-0,4\times0,0097) = 0,2689m$

Hence, As=$\frac{M\mu}{Z\bar{\sigma}s}=\frac{0,8631}{0,2689*434.782}\times10^2$= 0.738cm^2

b) Non-brittleness condition:

$$As_{min} = 0,23\frac{bdf_{tj}}{f_e} ;$$

Asmin=$0.23\frac{100\times27\times2.22}{500}$= 2.757 cm²

As =0.738cm² < Asmin = 2.757 cm² ⟹ C.N.V

We use 3HA12 bars with a total cross-sectional area of 3.39 cm².

c) Horizontal reinforcement:

At=As/3=0.919cm²

That is, use 2HA10 bars, with a total cross-sectional area of 1.57 cm²

According to SETRA documents, the corbel reinforcement is also provided using HA25 dowels spaced every 1 meter.

III. Bridge Seat Wall (Mur Garde-Grève):

III.1 Load Evaluation on the Bridge Seat Wall:

The bridge seat wall is primarily subjected to the following actions:

➤ Vertical forces:

- Self-weight.
- Reaction from a load directly applied to the wall.
- Reaction from the transition slab.

➤ Horizontal forces:

- Earth pressure.
- Pressure from a localized load behind the bridge seat wall.
- Braking forces from the axle load of a heavy truck (Bc).

Earth Pressure:

Is given by the following formula: $Pt = 1/2 \cdot \gamma \cdot Ka \cdot h^2$

The moment due to the earth pressure is given by: $Mt = 1/3.Pt.h = 1/6.Ka.\gamma.h^3$

Where:

Ka: Coefficient of active earth pressure

$Ka = tg^2\left(\frac{\pi}{4} - \frac{\varphi}{2}\right) = tg^2\left(\frac{180}{4} - \frac{30}{2}\right) = 0.33$

$\gamma = 2t/m$ Unit weight of soil

$h = 1.871m$ Height of the bridge seat wall

$\varphi = 30°$ Internal friction angle of the soil

Hence, **Mt = 0.648 t.m/m.l.**

Pressure due to local loads Bc:

$$\mathbf{M_q = \frac{12.Ka.bc.\delta bc}{(0,75+2h)} \times [(0,25+h)\ ln(1+4h)-h]}$$

With: **K = Ka.bc.δ.γ**

Load factor = 1.6 for ULS and 1.2 for SLS

bc = 1.1: load factor for the Bc loading system

δ = 1: dynamic amplification factor for a load on the backfill

Ka = 0.33

h = 1.871 m

$Mq = \frac{12.ka.bc.\delta bc}{(0.75+2h)} \times [(0.25+h)\ ln(1+4h)-h] = 2.58335$ t.m/m.l.

Moment due to braking force:

It is assumed that the effect of a single wheel is considered, and the load is distributed along directions inclined at 45° from the edge of the impact rectangle, hence:

Mf=$\frac{6.h}{0.25+2h}$ = 2,812t.m/ml

The total moment at the fixed support:

MULS= 1,35(Mpt) + 1,6(Mq + Mf) = 9,506 t.m/ml.

MSLS = Mpt +1,2 (Mq+ Mf) = 7,122 t.m/m

fc28 = 27 MPa ; b= 1m ; d = 0.27m.

III.2 Reinforcement:

a) Vertical reinforcement:

Mu=9,506 t.m/ml

Fbu=$\frac{0.85*27}{1*1.5}$= 15.3 Mpa

μ=$\frac{9,506}{1*15.3*(0.27)^2}\times 10^{-2}$= 0.0852

μ =0,0852 < μ = 0.186 Thus, the calculation is carried out with respect to pivot point A

As=$\frac{M\mu}{Z\bar{\sigma}s}$ With: σs=$\frac{fe}{\gamma s}$=$\frac{500}{1.15}$=434.7826 MPa

Z=d(1−0,4α)

α = 1,25(1-$\sqrt{1-2\mu}$)

α = 1,25(1−$\sqrt{1-2\times(0,0852)}$) = 0,111

Z = 0,27(1−0,4 × 0,111) = 0,258m

Hence, As=$\frac{M\mu}{Z\bar{\sigma}s}$=$\frac{9,506}{0,258*434.782}\times 10^2$= 8.4743cm²

b) Non-brittleness condition (Ductility condition):

$As_{min} = 0,23\frac{bdf_{tj}}{f_e}$;

Asmin=0.23$\frac{100\times27\times2.22}{500}$= 2.757 cm²

As =8.4743cm² > Asmin = 2.757 cm² ⟹ Condition verified

We use 6 HA14 bars with a total cross-sectional area of 9.24 cm².

At=As/3=2.824cm²

That is, use 3HA12 bars, with a total cross-sectional area of 3.39 cm²

c) Horizontal reinforcement:

According to SETRA guidelines, HA10 bars spaced at 15 cm in both directions will be provided as shown in the reinforcement drawing.

d) Ductility condition:

As> Asmin=$0.23\frac{b\times d\times ftj}{fe}$ Where (As: cross-sectional area of the tensioned reinforcement bars).

8.4743cm² >$0.23\frac{100\times27\times2.22}{500}$= 2.757 cm² ⟹ Condition verified

IV. Return Wall:

IV.1 Evaluation of Loads and Overloads:

The return wall serves to support the backfill of the bridge approach. It is subjected to the following loads:

The self-weight of the wall, including the superstructure.

Distributed horizontal earth pressures.

Concentrated loads applied 1 meter from the theoretical end of the wall, consisting of a vertical load of 4 tons and a horizontal load of 2 tons.

➢ Horizontal Forces:

- Earth pressure: Pt = 1/2×γs×ka×h²= 1/2×2×0 ,33×5,321²=9,3433t/m
- Backfill surcharge pressure: Psr = 1,2 × 5,321× 0,333 = 2,107 t/ml
- Concentrated surcharge pressure (2 t): Psc = 2 × 1,2 =2,4t/ml

Moments:

- Earth pressure: Mpt= 9,3433× 1/3×5,321= 16,571$t.m$ / ml
- Backfill surcharge pressure: Mpsr= 2,107 × 5,321/2 =5,605t.m /ml
- Concentrated surcharge: Mpsc= 2,4 × 5,321= 12,77 $t.m$/ ml

Combinations:

ULS (Ultimate Limit State): MULS = 1,35 Mpt + 1,6 ($Mpsr$ + $Mpsc$) = 51,770$t.m$ / ml

SLS (Serviceability Limit State): Mpt + 1,2 ($Mpsr$ + $Mpsc$) = 38,621$t.m$ /ml

➢ Vertical Forces:

- Self-weight of the wall: PG = 0.5×6.68 × 0,63 × 5,321× 2,5 = 11,1964 t / ml
- Concentrated surcharge (4 t): Psc = 4 × 1,2 = 4,800t/ml
- Self-weight of the superstructure: Psu = 0,3L= 0.3×6.68=2.004 t/ml

Moments:

- Self-weight of the wall: MG = 11,1964 × (6,68/3) = 24,930$t.m$ / ml
- Concentrated surcharge: Msc = 4,8 × 5,68= 27,264$t.m$/ml
- Superstructure self-weight: Msu = Psu×lth/2 = 6,693t. m/ml

Combinations:

ULS : $MULS$ = 1,35 (MG + Msu) + 1,6 Msc = 86,313$t.m$ /ml

SLS : $MSLS$ = (MG + Msu) + 1,2 Msc =64,339t.m / ml

IV.2 Reinforcement:

a) Horizontal reinforcement:

The reinforcement is designed for simple bending:

$MELU$ = 51,770$t.m$ /ml

$fc28$ = 27 MPa

$b0$ = 1ml

= 0,27

$Mu = \frac{51,770}{5,321} =$ 9.729t.m/ml

$Fbu = \frac{0.85*27}{1*1.5} = 15.3$ Mpa

$\mu = \frac{9.729}{1*15.3*(0.27)^2} \times 10^{-2} = 0.08722$

μ =0.08722< μ = 0.186 Thus, the calculation is carried out with respect to pivot point A

$As = \frac{M\mu}{Z\bar{\sigma}s}$ With: $\sigma s = \frac{fe}{\gamma s} = \frac{500}{1.15} = 434.7826$ MPa

$Z = d(1-0,4\alpha)$

$\alpha = 1,25(1-\sqrt{1-2\mu})$

$\alpha = 1,25(1-\sqrt{1-2\times(0.08722)}) = 0,1142$

$Z = 0,56(1-0,4\times 0,256) = 0,2576$m

Hence, $As = \frac{M\mu}{Z\bar{\sigma}s} = \frac{9.729}{0,2576*434.782} \times 10^2 = 8.6866cm^2$

b) Non-brittleness condition (Ductility condition):

$As_{min} = 0,23\frac{bdf_{tj}}{f_e}$;

$Asmin = 0.23\frac{100\times27\times2.22}{500} =$ 2.757 cm²

As =8.6866cm² > Asmin = 2.757 cm² ⟹ C.V

We use 5 HA16 bars with a total cross-sectional area of 10.05 cm².

c) Vertical reinforcement:

$MELU$ = 86,313$t.m$ /ml

$fc28$ = 27 MPa

$b0$ = 1md

= 0,27

$Mu = \frac{86,313}{5,68} = 15.19$ t.m/ml

$Fbu = \frac{0.85*27}{1*1.5} = 15.3$ Mpa

$\mu = \frac{15.19}{1*15.3*(0.27)^2} \times 10^{-2} = 0.136$

$\mu = 0.136 < \mu = 0.186$ Thus, the calculation is carried out with respect to pivot point A

$As = \frac{M\mu}{Z\bar{\sigma}s}$ With: $\sigma s = \frac{fe}{\gamma s} = \frac{500}{1.15} = 434.7826$ MPa

$Z = d(1-0,4\alpha)$

$\alpha = 1,25(1-\sqrt{1-2\mu})$

$\alpha = 1,25(1-\sqrt{1-2\times(0.136)}) = 0,183$

$Z = 0,27(1-0,4\times 0,183) = 0,250$m

Hence, $As = \frac{M\mu}{Z\bar{\sigma}s} = \frac{15.19}{0,250*434.782} \times 10^2 = 13.974 cm^2$

d) Non-brittleness condition (Ductility condition):

$As_{min} = 0,23\frac{bdf_{tj}}{f_e}$;

$Asmin = 0.23\frac{100\times27\times2.22}{500} = 2.757$ cm²

As =13.974cm² > Asmin = 2.757 cm² ⟹ C.V

We use 5HA20 bars with a total cross-sectional area of 15.710 cm².

VI. Front wall:

VI.1 Static condition:

a) Normal force:

$N = \sum(1 \mp Kv)P = 129t$

ULS = 1,35NG +1,6NQ

⟹ ULS = 1,35*129/10.42= 16.713t/ml

b) Moment:

Mnet =$\sum Mr - \sum M$s

= 227.685− 0 =227.685 t.m

ULS = 1,35*227.685 /10.42=29.498t.m

VI.2 Seismic Condition:

a) Normal force:

N = $\sum(1 \mp Kv)P$ = 258t

ELU = 1,35*258/10.42

=33,426t/ml

b) Moment:

Mnet =$\sum Mr - \sum M$s

= 455.0600− 83.205 =371.855 t.m

ULS = 1,35*371.855 /10.42=48.176t.m

VI.2 Reinforcement for Combined Bending and Axial Load:

a) Given data:

b = 1 ; h= 1,43; d= 0,9*1,43=1,29 ; d'=5cm

It is noted that the most unfavorable case is the seismic condition.

b) Seismic condition:

N ≤ 0 (E.T)

- Compressed

A=(0,337×h− 0,81d')b×h×fbu

A=((0,337× 1,43)− (0,81× 0,05)) ×1×1,43×15,3×10^6

=965,7609×10^4

B=Nu(d-d')- Mua

Nu= 33,426t/ml

Mua= Mu+Nu(d-$\frac{h}{2}$)

Mua= 48.176+33,426 (1,29 -$\frac{1.43}{2}$)

Mua=67.395t.m/ml

B= 33,426 (1,29− 0,05) – 67.395

= − 25,946

So, A > B

→ Partially compressed section

→ The design is subject to simple bending.

c) Vertical reinforcement:

$\mu = \frac{67.395}{1*15.3*(1,29)^2} \times 10^{-2} = 0.0264$

μ =0.0264< μ = 0.186 Thus, the calculation is carried out with respect to pivot point A

$As = \frac{M\mu}{Z\bar{\sigma}s}$ With: $\sigma s = \frac{fe}{\gamma s} = \frac{500}{1.15} = 434.7826$ MPa

$Z = d(1-0,4\alpha)$

$\alpha = 1,25(1-\sqrt{1-2\mu})$

$\alpha = 1,25(1-\sqrt{1-2\times(0.0264)}) = 0,0334$

$Z = 1,29(1-0,4\times 0,0334) = 1,272m$

Hence, $As = \frac{M\mu}{Z\bar{\sigma}s} = \frac{67.395}{1,272*434.782} \times 10^2 = 12.186 cm^2$

d) Non-brittleness condition (Ductility condition):

$As_{min} = 0,23\frac{bdf_{tj}}{f_e}$;

$Asmin = 0.23\frac{100\times129\times2.22}{500} = 13.173\ cm^2$

As =12.186cm² < Asmin = 13.173 cm² ⟹ C.N.V

Therfore: As = 13.173 cm²

We use 5HA20 bars with a total cross-sectional area of 15.710 cm².

e) Horizontal reinforcement:

At = As/3=13.173/3=4.391 cm².

We use 3HA14 bars with a total cross-sectional area of 4.620 cm².

GENERAL CONCLUSION

GENERAL CONCLUSION

The study presented in this thesis focused on the structural analysis of a prestressed girder bridge aiming ensure that the designed bridge would meet a broad range of requirements in both preformence and behavior under various loading conditions.

To achive the required results both analytical and specialized software methods were used the analytical approach provided a theoretical foundation and allowed for the verification of key structural parameters, while the use of the software "SAP2000" enabled more detailed modeling and simulation of the bridge's response.

The comparison between the analytical results and those obtained from the software revealed a high level of cinsistency, with only minor variations observed; which reinforces the reliability of the analysis.

This project allowed us to enhance and deepen our understanding of various key areas in engineering sciences, including material strength, continuum mechanics, and prestressing methods. It also helped us develop skills in structural analysis using specialized software to calculate internal forces and moments.

Additionally, we gained practical knowledge in designing reinforcement for the structural elements of a bridge.

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