PEOPLE'S DEMOCRATIC REPUBLIC OF ALGERIA

MINISTRY OF HIGHER EDUCATION AND SCIENTIFIC RESEARCH

UNIVERSITY OF ABOU BEKR BELKAID-TLEMCEN
FACULTY OF SCIENCE AND TECHNOLOGY
CIVIL ENGINEERING DEPARTEMENT

Dissertation submitted to the Department of civil engineering in pursue for a Master's degree in Transport and public works

STUDY OF THE DYNAMIC ANALYSIS OF THE RAIL-SLEEPER-BALLAST INTERACTION IN RAILWAY BRIDGES

Presented by:

Miss ALIYINZA CHARITY　　　　Mr. TAIBO BOSCO

Board of examiners composed of:

Dr. Nadia HAMIMED	President
Dr. Fethi HAMZAOUI	Examiner
Pr. Nadir BOUMECHRA	Supervisor

YEAR: 2020/2021

ABSTRACT

The research made in this dissertation, is about the study of the dynamic analysis of a railway bridge concerning the rail-sleeper-ballast interaction modeled by the help of the finite element method using SAP2000 software. A viaduct of decks in continuous spans of 40m+40m with a Fixed Pier and mobile abutments at the ends is studied following the Algerian and European code standards. The reconstituted welded steel beams, concrete and active and passive reinforced steel bars are the major materials used in the bridge design. The bridge elements and their corresponding material properties and geometric characteristics are defined and modelled by the SAP2000 software. Considering the train moving load with varying velocities, the analysis was done with variation of ballast thickness and ballast rigidities with reference to the finite ballast stiffness. According to the results from the analysis, the mid-span steel beam and the pier of the bridge are the most affected elements by the internal forces, increase in train velocities increases the vibrations of the railway track as well as the bridge elements.
Key words: Dynamic analysis, Finite element method, SAP2000

RESUME

La recherche faite dans cette dissertation de fin des études, porte sur l'étude de l'analyse dynamique d'un pont ferroviaire concernant l'interaction rail-traverse-ballast modélisée à l'aide de la méthode des éléments finis à l'aide du logiciel SAP2000. Un viaduc de ponts en travée continue de 40m+40m avec une jetée fixe et des culées mobiles à l'extrême est étudié selon la réglementation du code algérien et européen. Les poutres en acier soudé reconstitué, le béton et les barres d'acier renforcées actives et passives sont les principaux matériaux utilisés. Les éléments de pont et leurs propriétés matérielles correspondantes sont définis et modélisés par le logiciel SAP2000. Compte tenu de la charge en mouvement du train avec des vitesses variables, l'analyse a été effectuée avec une variation de l'épaisseur du ballast et des rigidités du ballast en fonction de la rigidité finie du ballast. Selon les résultats de l'analyse, la poutre d'acier de mi- portée et la pile du pont sont les éléments les plus touchés par les forces internes, l'augmentation de la vitesse des trains augmente les vibrations de la voie ferrée ainsi que les éléments du pont.
Mots-clés : l'analyse dynamique, la méthode des éléments finis, logiciel SAP2000

ملخص

يركز البحث الذي تم إجراؤه في أطروحة التخرج هذه على دراسة التحليل الديناميكي لجسر سكة حديد فيما يتعلق بالتفاعل بين السكك الحديدية والنائم والصابورة على غرار طريقة العناصر المحدودة باستخدام برنامج SAP2000.
تمت دراسة جسر من الجسور في مساحات متواصلة من 40 م + 40 م مع رصيف ثابت ودعامات متحركة في أقصى الحدود وفقًا للوائح الجزائرية والأوروبية. العوارض الفولاذية الملحومة المعاد بناؤها والخرسانة والقضبان الفولاذية هي المواد الرئيسية المستخدمة. يتم تحديد ونمذجة عناصر الجسر وخصائص المواد المقابلة لها بواسطة برنامج SAP2000. بالنظر إلى الحمل المتحرك للقطار بسرعات متفاوتة، تم إجراء التحليل مع اختلاف سمك الصابورة وصلابة الصابورة كدالة لصلابة الصابورة المحدودة. وفقًا لنتائج التحليل، فإن العارضة الفولاذية متوسطة المدى ورصيف الجسر هما أكثر العناصر تأثرًا بالقوى الداخلية، وزيادة سرعة القطارات تزيد من اهتزازات السكك الحديدية وكذلك عناصر الجسر.

ACKNOWLEDGEMENTS AND DECLARATIONS

First and foremost, we would like to thank the Almighty God for the good health and wisdom throughout our journey of this dissertation.

Secondly, we would like to express our sincere gratitude to our dear supervisor Pr. Nadir BOUMECHRA for his excellent guidance and supervision throughout this research project. Without his support and advise, this thesis would not have been possible.

We also want to express our appreciation to Dr Nadia HAMIMED and Dr Fethi HAMZAOUI the members of the thesis committee. Their professional knowledge and general interest in evaluating this thesis are valuable assets to our work. We would equally want to thank our university Abou-Bekr Belkaid Tlemcen and all the professors particularly in the civil engineering department for their academic support and advise throughout these academic years.

We would also like to thank all our colleagues in the civil engineering class especially in public works and transport for their friendship and guidance throughout the two years of master's studies and research.

Finally, we thank our parents, brothers, sisters and friends for their continuous emotional support and encouragement during our time of study especially during the roughest times we experienced

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

C_q – coefficient of contraction
E_b – ballast module of elasticity
E_s – dynamic module of elasticity of steel
γ - coefficient of security
Esp γ - elastic deformation
Esp s – deformation at ultimate limit state
K_b – ballast rigidity
Q – maximum tributary flow
Q_b – water flow rate
Φ - run off coefficient
ϕ - diameter
f_{yk} – minimum yield stress
% - percentage
$\boldsymbol{\Omega'_n}$- effective normal stress.
$\boldsymbol{\Omega_c}$–uniaxial compressive strength
$\int_f$ - shear strength at failure.

LIST OF ABBREVIATIONS

UIC – International Union of Railways
RPOA – Réglement Parasismique des Ouvrages D'art
ET – Electrical Traction
FEM – Finite Element Method.
TDR – Track Decay Rate
(DLR) – Dynamic Lateral Resistance
LWR – Long Welded Rails

CHAPTER 1: GENERAL INTRODUCTION

Background and problem definition

The railway track system is one of the most important transportation system networks used in most countries for the displacement of people and transportation for both agricultural and industrial products.

The enthusiasm to study the dynamic behaviour of the railway bridge increased significantly in recent years especially due to the development of high-speed railways in various countries. The main aim of developing these high-speed railway trains is to decrease the travel time among different destination. for example, in Algeria, the transportation system such as road transport and air transport have been the major systems of transport until recently a high-speed railway which will be able to handle train traffic speed up to 250km/h is being implemented connecting different districts such as Algiers, Oran, Blida, Tlemcen and many more.

figure 1. 1 High Speed Global transport network

Annually, large portions of budget are put in place to improve the railway track efficiency and decrease its maintenance costs of which it can be achieved only if a better understanding is obtained of the physical and mechanical characteristics of substructure and the ballast layer in particular. Less attention has been accorded to the substructure and the ballast in particular because its properties are more variable and difficult to define.

A lot of different complex phenomenon occur in these railway line structures and become more complicated when these lines are constructed on railway bridges and more so exposed to high-speed train traffic, which is the case considered in this thesis. The dynamic structural behaviour in the rail-sleeper-ballast combination is observed more crucially for high-speed railway bridges exposed to traffic speeds up to 200km/h. Particularly, a constant moving train load is considered as the only load on the railway composite bridge therefore the permanent and accidental charges were not considered in this thesis. It's very crucial to focus on the dynamic response of the rail-sleeper-ballast combination so as to mitigate the immediate phenomenal that might occur during the course of time in service state.

1.1 OBJECTIVE AND AIMS

The ultimate objective of this project is to understand how the rail-sleeper-ballast combination interacts with the railway bridge structure. The aims of this thesis can be stated as:

- To investigate the possible phenomenon that might occur once the rail-sleeper-ballast combination is not considered or taken into account.
- To understand how the variation of the ballast thickness and stiffness may affect the stability of the different elements of the railway composite bridge when a constant moving train load is applied.
- To analyse how train velocities can affect the railway track as well as the bridge.

1.2 THESIS OUTLINE

This dissertation is divided into four main chapters. A brief outline of these chapters is given below:

Following the introductory chapter, **Chapter 2** contains a literature review consisting of five parts: History of railway bridges, describing how since 1600's the train system was used in transportation of products such as coal until the 1800's when the first engine train 'Stockton & Darlington Railway' was Constructed. The compositions of the railway, the definition of the railway transport, its specific characteristics and its advantages as well as the disadvantages. The components of the railway such as the rails, rail pads and fastenings, sleepers with definition of the different types of sleepers used in railway construction and their functions. The ballast definition with its grade size gradation including particular experiments used to identify the grain size such as the grain size gradation test (ASTM D6913). The dynamic characteristics of the ballast, both physical and mechanical characteristics of the ballast were described including a few experiments like the petrographic analysis, direct shear tests, triaxial tests were carried out. The interaction of rail-sleeper ballast was also reviewed with referential recent research.

Chapter 3 consists of six parts: The first part describes the general description of the composite railway bridge, with particular dimensions such as the total length of the bridge of 80m of 2 decks section each of 40m, with a Fixed Pier (P01) and the 2 Mobile Abutments in the longitudinal direction. The next part explains the norms that were applied in this thesis and these include Algerian Earthquake Rules RPOA / 2008 version (for the coefficient of acceleration), Snow and Wind Regulations "R.N.V. 1999" (for the reference speed), Eurocodes NF EN 2003 (EIC 1 ,2,3 and 4). Construction materials such as the concrete Passive reinforced steel bars, Rail UIC60 with its dimensions recorded. Details of the construction with elements such as the column, mobile abutments, the evacuation of water from the platform was discussed. The foundation system was too described with dimensions of the column and abutments. The technological equipment such as the security barriers, electrical equipment, telecommunication and signalization equipment and the earthing system were too discussed.

Chapter 4 describes the finite element modelling of the railway composite bridge therefore, definition of the materials such as concrete and steel, defining the bridge elements such as beams, column, diaphragm, rails, slab, creating the continuity and the bearings and finally creating the moving constant train load.

Chapter 5 elaborates about the parametric analysis of the ballast railway bridge with its particular consequences on the different parts of the bridge such as the base of the intermediate pier, mid-span steel beam and steel beam at the support.

Our manuscript is finalized by a general conclusion where we present a global synthesis of the various results of the effect of ballast rigidity in the bridge/pier system behaviour and propose perspectives in relation to our work.

CHAPTER 2: **DYNAMIC INTERACTION OF VEHICLE-SLEEPER-BALLAST BRIDGE**

2.1 INTRODUCTION

The railway vehicles are an increasing means of transportation due to its reduced impact on environment and high level of comfort provided. A lot of case studies have been done considering divers scenarios of different models of bridge designing different dynamic moving load problem. To ensure effective operation of the railway track a lot of parameters have to be put into consideration. The physical, mechanical, mineralogical properties of ballast have to be studied keenly in that the type of ballast used is stiff enough, able to disperse the charges received from the train at a wider range and have the capacity to absorb a large quantity of charges induced by the trains. The ballast behavior of ballast settlements, its rate of degradation is to be studied as well. In addition, the interaction between the rail with the sleepers then sleepers with ballast have to be investigated during the study to ensure the stability of the railway track, durability of the rail comportments, as well as proper transmission and distribution of rail charges to the bridge. This combination of rails, sleepers and ballast distribute the load (lateral, longitudinal and vertical forces) over a certain area.

2.2 HISTORY OF RAILWAY BRIDGES.

Great Britain’s railway system is the world oldest. The system began with local wooden wagon ways which involved laying planks along cleared paths to make the transport of goods easier and faster. Historians note that in 1671 “railed roads” were used in Durham to make it easier to move coal. The first was the Tanfield Wagon way. The “roads” used straight and parallel timber rails on which carts with simple flanged iron wheels were drawn by horses, enabling several wagons to be moved simultaneously.
In 1807 the first public railway that carried passengers was the Swansea and Mumbles Railway at Oyster mouth. But while horse-drawn carts or carriages did move goods and people faster than other contemporary forms of transport, they were limited to the speed of the animals. Richard Trevithick designed and built the first steam locomotive to run on smooth rails in 1802. However, the Salamanca, a steam locomotive built in 1812 by John Blenkinsop and Matthew Murray, was the first commercially successful model. It ran on the Middleton Railway, which had rails that were four feet apart. The locomotive had cog wheels driven by two cylinders that were embedded into the top of the boiler.

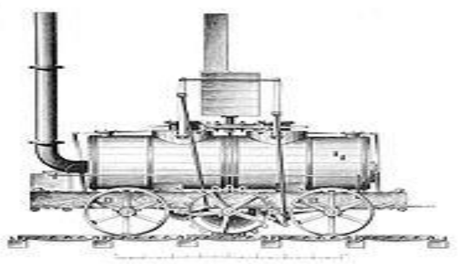


figure 2. 1: The Tanfield Wagonway (Britannica, 2019)

Stockton & Darlington Railway, in England is the first railway in the world to operate freight and passenger services with steam traction. In 1821 George Stephenson, who had built several steam engines to work in the Killingworth colliery, heard of Edward Pease's intention of building an 8-mile (12.9-km) line from Stockton on the coast to Darlington to exploit a rich vein of coal. Pease intended to use horse traction; Stephenson told Pease that a steam engine could pull 50 times the load that horses could draw on iron rails. Pease agreed to let Stephenson equip his line.

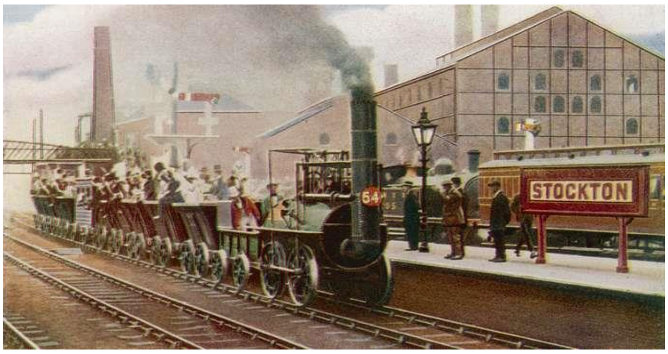


figure 2. 2: Stockton *and Darlington Railway*, centennial celebration of the Stockton and Darlington Railway,1925 (Britannica, 2019)

On September 27, 1825, the first engine ran from Darlington to Stockton, preceded by a man on horseback carrying a flag reading "Periculum privatum utilitas publica" meaning, ("The private danger is the public good"). When the horseman was out of the way, Stephenson opened the throttle and pulled his train of wagons carrying 450 persons at a speed of 15 miles (24 km/hour).

2.3 RAILWAY TRANSPORT

It is a means of transport, on vehicles which run on tracks (rails or railroads). The fundamental principle of rail transport is its efficiency.

figure 2. 3: Four north west river railway bridge (VEC civil engineering,2014)

2.3.1 characteristics of railway transport

* It is fast *it is reliable *it is convenient *it is economical

*it is safe and secure *it is fuel efficient *it is environmentally friendly

2.3.2 Advantages of railway transport

- It is dependable as it is least affected by weather conditions such as rain, fog among others compared to other mean of transport.
- It is economical, quicker and best suited for carrying heavy and bulky goods over long distances.
- It is the safest form of transport. The chances of accidents and breakdowns of railways are minimum as compared to other modes of transport.
- The carrying capacity of the railways is extremely large. Moreover, its capacity is elastic which can easily be increased by adding more wagons.

2.3.3 disadvantages of railway transport.

- The railway requires large investments of capital. The cost of construction, maintenance and overhead expenses are very high as compared to other modes of transport.
- Its inflexibility. Its routes and timings cannot be adjusted to individual requirements.
- Railway transport is unsuitable and uneconomical for short distances.
- Because of huge capital requirements and traffic, railways cannot be operated economically in rural areas.

2.4 COMPONENTS OF A RAILWAY

The track structure is composed of an ensemble of elements that facilitate the guidance of trains in a safe and economical manner as well as supporting the various kinds of charges induced by the train by the gradual increase in the load distribution surfaces which reduces the stress impact between the wheel-rail contact points. The different elements are defined further below.

figure 2. 4: Components of a railway (AGICO group, 2017)

2.4.1 rails

Rails are steel bars extending horizontally between supports which are used as a track for rail road, cars or other vehicles. They directly receive efforts exerted by the track that is to say;

- Vertical forces (weight of the track which is the maximum charge par each wheel)
- Transversal forces (centrifugal and centripetal forces)
- Longitudinal forces (acceleration of the track, the braking force, impact force of the rail joints, the dilation).

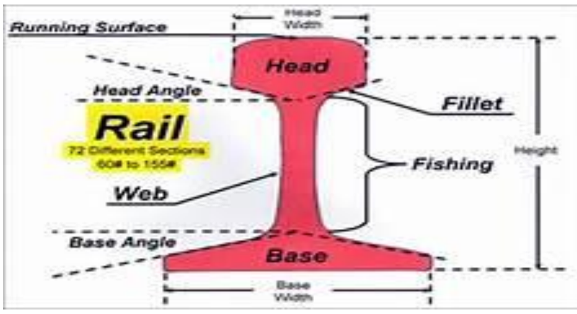


figure 2. 5: **Different parts of a rail (Slide share, 2015)**

They equally play other roles as listed below;

- Provide a smooth-running surface for the train wheels allowing a quiet running and limit the effect of the vibrations.
- The rails are derived from an I-profile, to provide bearing capacity in that direction.
- Transmit and distribute the charges to the sleepers
- Ensure the lateral guidance of the wheels, therefore the absorption of transverse forces and their transformation to the cross members.
- Resist breaking forces caused due to stoppage of trains.

Rails are divided into three types i.e., double headed rails, bull headed rails, flat footed rails

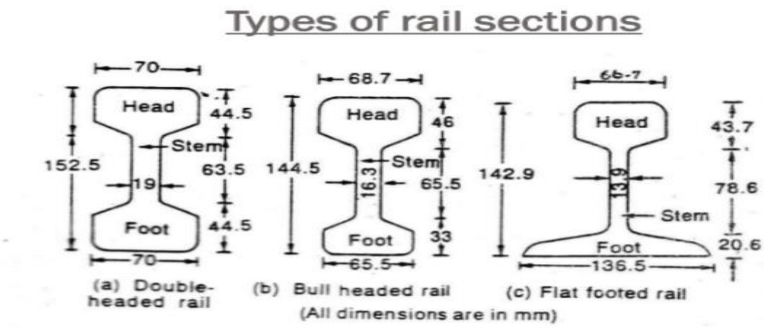


figure 2. 6: **Types of rail sections (Slideshare, 2014)**

The flat-footed rails are the most used today because of its various advantages as some stated below;

- They are economical since they don't need any chair given the fact that they can be spiked and fastened directly to the sleepers because of their spread-out foot that forms a base.
- They are much stiffer both vertically and laterally than the other types of rail sections.
- They are less liable to develop kinks and maintain a more regular top surface than bull headed rails.
- The loads from the wheels of trains are distributed over large number of sleepers and hence larger area which results in greater track stability, longer life of rails and sleepers, reduced maintenance costs, less rail failure and few interruptions to traffic.

2.4.2 Rail pads and fastenings

Rail pads are elastic polyurethane mats assembled between steel rails and rail sleepers to protect the sleeper top from wearing and impacting. The starting stiffness of rail pads is designed not to be high so that their relative deformation under spring fastening toe loads is greatly substantial. Rail-pads increase the track flexibility and they introduce part of the track dissipative properties. Soft rail-pads permit larger deflection of the rail and a better distribution of the load to the sleepers. Moreover, the softer the rail-pad is the better the isolation of sleepers and ballast from high-frequency vibrations. The rail-pad is composed by rubber material with non-linear behavior that is known to be influenced by the preload.

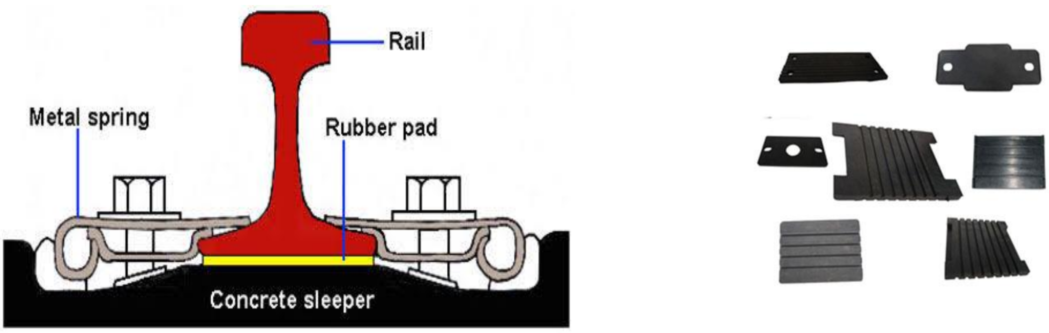
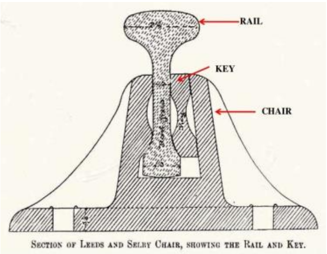


figure 2. 7 *:* Rail fastening and different kinds of rail pads (Malysian Rubber Board, 2002)

Rail pads can reduce fatigue cracking of concrete sleepers which is believed to be driven by impact and vibration from the passing train. In addition, the rail pads under the rail ensure;

- Load distribution over a large surface
- Elimination of load concentration and the resultant fatigue stresses
- Centering of loads on the supporting element
- Absorption of uneven contact surfaces between rail and sleepers
- Reduction of noise and vibrations
- Reduction of wear of steel rail and its support.

Rail fastenings are elements used to fix rails to sleepers to maintain the gauge and prevent the rails from moving longitudinally and laterally relative to sleepers. The fastenings used differ according to the material components of the sleepers, for examples for wooden sleepers we use clamps, spikes, bolts, for metallic sleepers we use bolts, chairs and keys as for concrete sleepers we use clamping plates, elastic blade, and clips.



Metallic spike fasteners

Clamping plate fasteners

Chair and key

figure 2. 8: Different fastening components (AGICO group, 2021)

2.4.3 Sleepers

Railway sleepers are components on which the rails are arranged with proper gauge, they generally rest on ballast, according to the nature of its purpose, sleepers need to be durable enough to resist the heavy traffic loads. Traditionally, railway sleepers were always made from wood, and it had continued for 50 years. With the intense steel production, steel sleepers were used for about 50 years. But nowadays, concrete sleepers are widely chosen with a demand for increased axle loads. The first concrete sleeper was introduced at the end of 19th century, and the first concrete sleeper experiment was made in Germany in 1906 between the line Nuremberg and Bamberg. In the period of World War II, the production of concrete sleeper was largely increased, and in 1939 the steel sleeper was stopped to be produced. The following below are the different types of sleepers that can be used in railway construction.

a- wooden sleepers

Wooden sleepers are only suitable for low-speed lines with the speed limit of 160 km/h. Acceptable species of wood for this type of sleepers are European oak, beech, pine and etc.

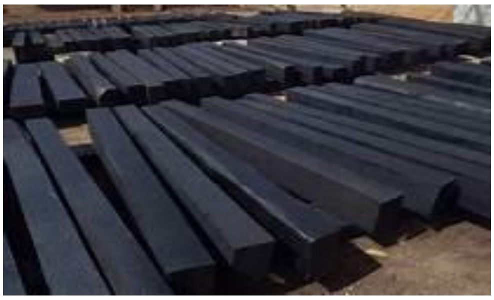


figure 2. 9: **Wooden sleepers (AGICO group, 2021)**

The advantages of wooden sleepers are:

- Easy to handle
- Good resilience
- Good electrical insulation
- Easily adapted to non-standard situations

The disadvantages are:

- They are subjected to decay, attack by white ants, warping, cracking and end splitting.
- Non reusable due to preservative chemicals.
- They have got minimum service life (12 to 15 yrs.) as compared to other types of sleepers.

To investigate the capacity of used wooden sleeper, a compliance test can be conducted, which consists of three experimental procedures, namely Rail Seat Vertical Load Test under Negative Moment (TRSN), Positive Moment (TRSP) and Centre Positive Bending Moment Test (CPM), see Siti (2008). For each test two samples are used. Test results can be seen in the figure below. Wooden sleeper is slenderer than concrete sleeper and the vertical central displacement for wooden sleeper is larger than concrete sleeper. This could be one of the reasons why wooden sleeper has been replaced by concrete sleeper.

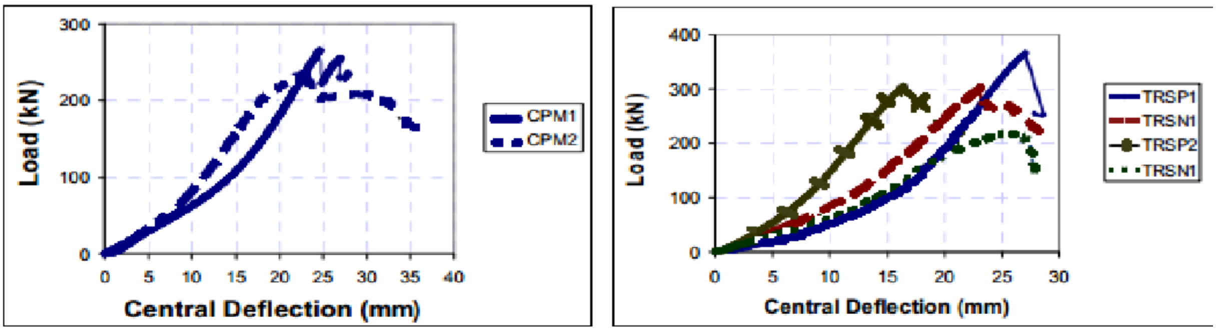


figure 2. 10: Load-deflection of CPM, TRSP and TRSN test,(Siti, 2008)

b- steel sleepers

Steel sleepers are lightweight, dimensionally more accurate than wooden or concrete sleepers. Classic steel sleeper is out of production today.

figure 2. 11: Steel sleepers (India Mart, 2020)

- ➢ The advantages of steel sleepers are:

- ❖ Easy to manufacture and install.
- ❖ They provide better lateral rigidity to the track.
- ❖ They can be easily handled as these are light in weight as compared to other types of sleepers. Hence, damages during handling and transporting are less.

- ➢ But the main disadvantages are:

- ❖ Low transverse resistance
- ❖ Difficult to maintain
- ❖ Sensitive to chemical attacks
- ❖ They get easily rusted and corroded.

c- concrete sleepers

Concrete sleepers do not rot like wooden sleeper, and they have longer life, much stability and need less maintenance compared to other types of sleepers. For these reasons, nowadays concrete sleepers are widely used all over the world.

figure 2. 12: **Concrete sleepers(AGICO group, 2019)**

- The advantages of concrete sleepers are:

- Longer life cycle time
- The concrete sleepers result in reduced rail bending stresses.
- Lower maintenance cost of fastening
- Smaller lateral displacement on account of large weight
- The concrete sleepers are suitable for track circuiting.
- The concrete sleepers are neither inflammable nor subjected to damage by corrosion or termites.

- But concrete sleeper still has some disadvantages:

- Vulnerable to impact
- difficult to handle because of large weight
- difficult to maintain longitudinal level due to higher moment of inertia and lower elasticity.
- The design and construction are both complicated.

In general, concrete sleepers can be divided into reinforced twin block concrete sleeper and prestressed monoblock concrete sleeper as presented below.

d- Twin block sleeper

Reinforced twin block sleeper is a type of sleeper where two concrete blocks are connected to each other through a steel rod or rigid steel beam. Increased lateral resistance and lower weight are the main advantages of this type of sleeper. It can be used under various loading conditions and its service life is about 50 years.

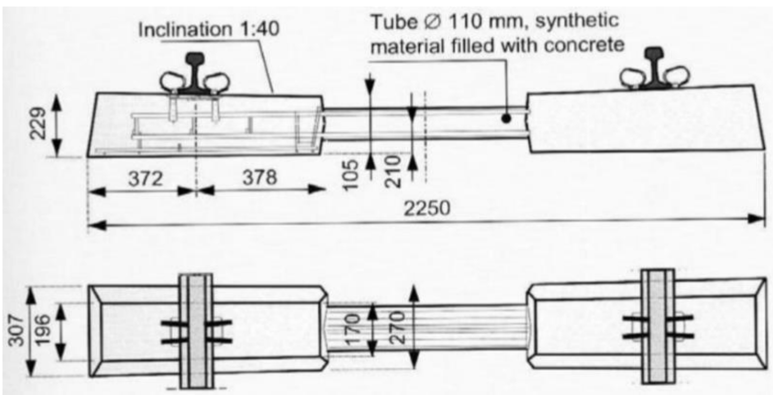


figure 2. 13: **Twin block sleeper(Shan Li, 2012)**

- The advantages of this type are:
  - Better lateral displacement resistance
  - More elastic behavior
  - Easy to handle due to low weight

- The disadvantages are:
  - Require elastic fastening
  - Declination of sleeper to the track center
  - Require special insulating accessories
  - Defect because of corrosion and fatigue of steel
  - Load distribution and flexibility less satisfactory

e- Monoblock sleeper

Prestressed monoblock sleeper which consists of one prestressing reinforced concrete beam was developed in UK. It can be used for high-speed railway and heavy loading. And the service life would be also about 50 years.

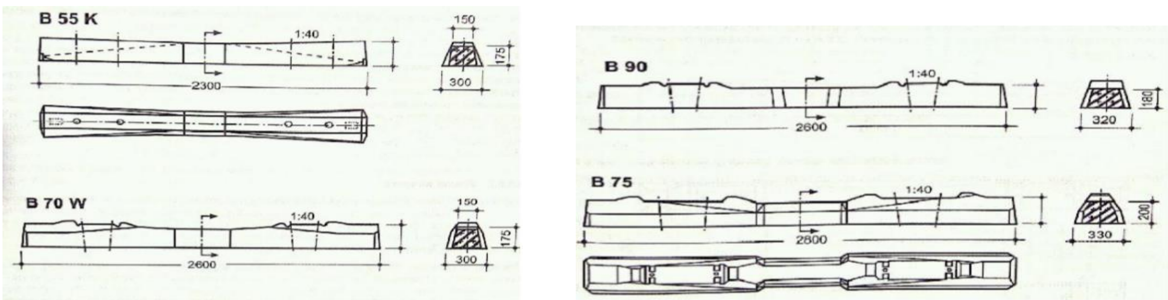


figure 2. 14: **Monoblock sleeper(Benhard, 2005)**

Different types of monoblock sleeper, see Bernhard (2005)

- Its advantages are:
  - Maintain track gauge in a good manner
  - Longer life time
  - Load distribution better than twin block
  - Good surface for maintenance inspection staff

- Its disadvantages are:
  - Require elastic fastening
  - Require special insulating accessories
  - No reinforcement against shear and torsional forces

- **Main functions of sleepers**
  - Transmission of forces from the rails down to the ballast bed.

- Maintenance of the characteristic dimensions between the two rails that is the gauge, level, and the alignment of the track.
- Offers sufficient mechanical resistance in the vertical and horizontal direction.
- Acts as an elastic medium between the rail and the ballast and absorbs the vibrations of the trains.

2.5 BALLAST

- **Definition of the ballast**

Ballast can be defined as a free-draining granular material used as a load bearing material in railway tracks. It is mainly composed of medium to coarse gravel-sized aggregates (30–50 mm), with a small percentage of negligible fine particles, (Esveld, 2001). The ballast is composed of a group of independent particles in close contact as shown in Fig 2.15 acting as a porous medium to facilitate the drainage of water and absorption of vibration produced by moving train to the sleepers which may cause ballast settlement and change of track geometries therefore influencing the track maintenance costs.

figure 2. 15: ***Ballast particles in close contact.***

The behaviour of the medium created by these particles depends on three essential factors; contact between its particles, the network of voids that settles and the water content in these voids. Due to its relatively low cost of construction and possibility of maintenance, Ballast is the most common railway structure used. (Chrismer, 1985; Jeffs et al, 1987; Esveld, 2001).

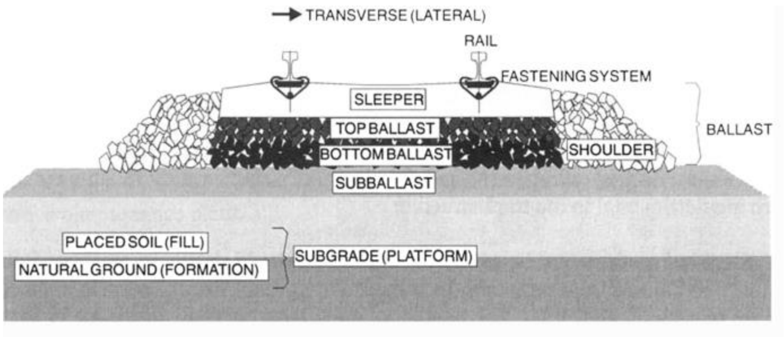


figure 2. 16: ***Cross section view of the ballast track. (Selig & Waters, 1994). (****Lim, Wee Loon (2004) Mechanics of railway ballast behaviour. PhD thesis, University of Nottingham.)*

The thickness of the ballast should be in a way that the subgrade is loaded as uniformly possible. The optimum thickness is usually 250–300 mm measured from the lower side of the sleeper (Esveld, 2001). Good-quality railway ballast should have angular particles, high specific gravity, high shear strength, high toughness and hardness, high resistance to weathering, rough surface and minimum hairline cracks (Chrismer, 1985; Jeffs et al, 1987; Indraratna et al., 1998, 2000, 2003b; Esveld, 2001).

The sources that contain high-quality ballast are so limited and therefore good quality ballast is rare to obtain. The usage of poor-quality ballast changes most ballast properties progressively because of breakage, deformation and fouling under dynamic loading conditions. Ballast fouling decreases permeability, and therefore causes hydraulic erosion, reduction in stability due to particle lubrication, subgrade attrition, and ballast deterioration due to the delay in dissipation of excess pore water pressures.

One of the main functions of ballast is to retain track position by resisting vertical, lateral and longitudinal forces applied to the sleepers. Ballast also provides resiliency and energy absorption for the track, which in turn reduces the stresses in the underlying materials to acceptable levels. Large voids are required in the ballast for storage of fouling materials and rapid drainage of water falling onto the track. Ballast also needs to have the ability to rearrange during maintenance level correction and alignment operations (Tutumluer *et al*., 2006).
The characteristics required for these functions are clearly contradictory in some aspects, and thus a particular type of ballast cannot accomplish all of them completely (Profillidis, 1995).

- **Grain size gradation of the ballast**

The grain size gradation of the ballast is the most common specification for new ballast and a common technique for the assessment of worn ballast. The grain size gradation of the aggregate mass allows a reliable correlation to strength, deformation, and drainage characteristics. Ballast specifications require a relatively narrow range of particle sizes, which maximizes inter-particle void volume. This large void volume facilitates drainage and provides a substantial storage of ballast-fouling material. When narrowly graded ballast is well compacted, it performs good and provides an adequate drainage and fouling material storage. More broadly graded ballast will provide increased strength and resistance to deformation due to the denser packing arrangement of the particles, but broadly graded ballast can be expected to have a lower void volume than narrowly graded ballast. Although a more broadly graded ballast can provide superior resistance to deformation.

- **Experiment to determine a good ballast gradation.**

Ballast gradation is an essential assessment to ballast performance and often most difficult to assess. Keeping in mind sample size requirements from the American Society of Testing Materials (ASTM), it is also important to follow sampling guidelines to ensure when a sample is obtained from the field.
The grain size gradation test (ASTM D6913) consists of washing and drying the ballast aggregates then placing a sample of these aggregates on top of a layer of sieves arranged in descending order with respect to the diameter of the sieving spacings (Figure 2.17). The particles on top of the sieve with a greater diameter sieving spacings falls through the stack as it is shaken mechanically, and larger particles are retained on the sieves of smaller diameters. The weight retained on each sieve is

measured and recorded then expressed as a percentage of the original weight. The percent passing each sieve size is plotted on a semi log chart of sieve diameter to percent passing, which is the grain size gradation plot (Figure 2.18).

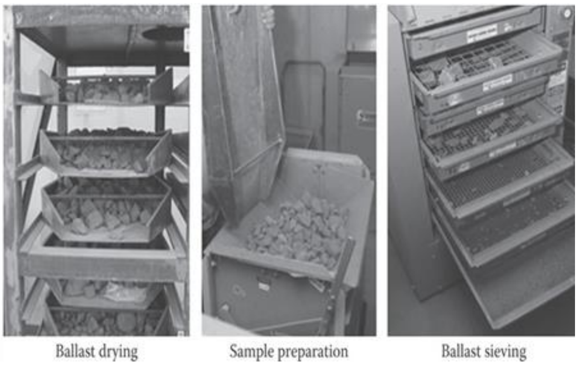


figure 2. 17: ***Ballast*** **gradation analysis. (Courtesy of Transportation Technology Center Inc., Pueblo, Colorado.)** (**Railway Geotechnics,** Dingqing Li, James Hyslip, Ted Sussmann, Steven Chrismer)

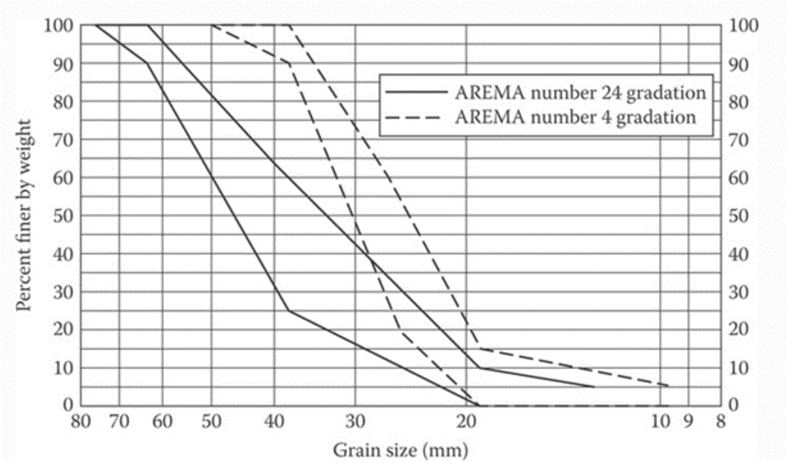


figure 2. 18***:*** **AREMA (24 and 4) ballast gradation particle size ranges.(** ***Railway Geotechnics Dingqing Li)***

The allowable grain size range of two common ballast gradations, according to the American Railway Engineering and Maintenance-of-Way Association (AREMA) manual of recommended practices and the percent passing certain sieve sizes for a few gradations are presented in Table below.

		AREMA ballast gradation percent-passing		
Sieve designation (ASTME 11-09)	**Sieve opening size (mm)**	4	4A	24
3’’	75.0	-	-	100
21/2”	63.0	-	100	90-100
2”	50.0	100	90-100	-
11/2”	37.5	90-100	60-90	25-60
1”	25.0	20-55	Oct-35	-
3/4"	19.0	0-15	0-10	0-10
1/2”	12.5	-	-	0-5
3/8”	9.5	0-5	0-3	-

table 2. 1: AREMA recommended ballast gradations 3, 4, and 24(*Railway Geotechnics* *Dingqing Li)*

Particle size is an important factor in ballast performance because the amount of void space between particles increases with particle size, and therefore the amount of void volume for storage of fouling fines increases.

2.6 DYNAMIC CHARACTERISTICS OF THE BALLAST

2.6.1 Physical characteristics of the ballast:

These physical properties of ballast are considered to be responsible for the effective performance of ballast in the field throughout the ultimate service period. We are going to tackle two different division of physical characteristics of ballast; properties of individual particles and properties of particles which are in close contact with each other but not influencing deformation.

- ✓ **properties of individual particles:**

These include properties like petrological properties, durability, shape and surface before the ballast can be considered suitable for use in the field.
Examination of the Petrological properties is regarded by the authors as one of the most important aspects of the evaluation of ballast materials, both potential ballast and that which has seen service in the track.

- ❖ **petrographic analysis**

It is a visual technique which is used to evaluate the source, composition, and the nature of the material constituting the sample. The analysis is performed on samples to provide an indication of performance and weathering and breakdown of materials.
The performance of ballast in the field depends on the characteristics of the parent rock, which can be determined by petrographic analysis. (Watters et al.,1987).
The analysis is mostly reliable on aggregate quality taste though results strongly depend on the competence of the petrographer. (Selig et Waters,1994).
The analysis of the samples is mostly a visual assessment of specimens under microscopic examination. It includes preparing thin sections as shown in the figure below. Thin sections represent slices of rock polished with thickness of 30µm, which are later examined under a polarising microscope to determine mineral modes, grain size, grain morphology and textural fabric.

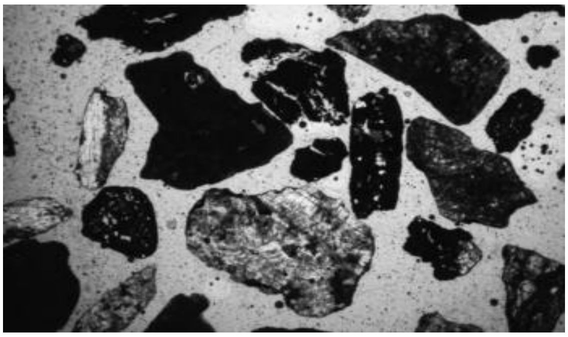


figure 2. 19: ***Examples* of granite/ syenite ballast of thin sections.(Railway Geotechnics,** Dingqing Li, James Hyslip, Ted Sussmann, Steven Chrismer)

The performance of rock ballast, subjected to the physical stresses of loading and the chemical and physical effects of weathering, depends to a great extent on its mineralogical, chemical, textural, and structural properties. Because these properties can readily be determined by petrographic analysis, the experienced petrographer should be capable of providing at least a qualitative assessment of the performance potential of ballast and a reasonable explanation of the failure or durability of ballast from track sections.

Many of the physical and chemical tests commonly applied to ballast materials (e.g., mill abrasion, Los Angeles abrasion, magnesium soundness, and absorption) essentially provide a quantitative measure of petrographic properties. It follows, therefore, that a complete petrographic analysis should always be the first step in the evaluation of ballast materials and should constitute the basis on which further testing is carried out. Furthermore, petrography should be regarded as a key factor in the selection of a ballast source (Watters et al., 1987).

The performance and durability of ballast particles is dependent on the material properties and loading conditions. Aggregate characterization is important for judging its performance as ballast, whether one is determining suitable quarry sources for new ballast or is assessing the performance of existing ballast in track.

The type of parent rock that the ballast is crushed from will affect the derived ballast particle size, shape, and angularity of individual particles, which consequently affects the frictional resistance to sliding between two particles in contact.

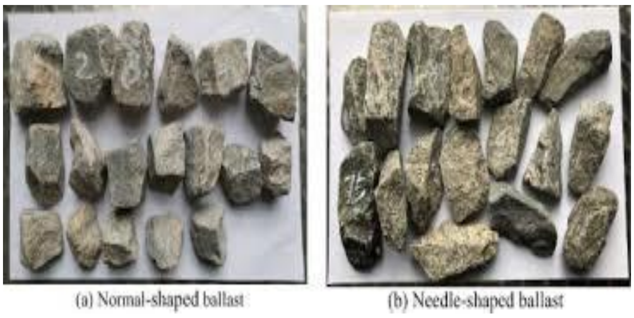


figure 2. 20: Ballast *of different angularity and shape*

Some rocks have cleavage planes and bedding layers that set up preferred failure planes that can control particle shape. An understanding of parent rock characteristics provides insight into ballast performance.

Natural variations within a quarry should be evaluated to determine if they could adversely impact ballast shape, size, or performance. Once a potentially suitable parent rock mass has been identified, a detailed evaluation of the crushed rock aggregate should be undertaken. (Dingqing et al. 2015)

- ✓ **properties of particles which are in close contact with each other but not influencing deformation:**

These properties are permeability, void ratio, bulk density and specific gravity. The permeability and void ratio are important factors that are ultimately responsible for long-term track drainage. Permeability decreases with increasing compaction and cementation, and increases with higher void ratio and coarser particles. The specific gravity and bulk density of the ballast are two properties that play pivotal roles in rail track safety. Bulk density is a function of the particle specific gravity and the void ratio. The use of denser particles and broader gradation increases the bulk density. When narrowly graded ballast is adequately compacted, it performs well and provides superb drainage and fouling material storage. More broadly graded ballast will provide increased strength and resistance to deformation due to the denser packing arrangement of the particles, but broadly graded ballast can be expected to have a lower void volume than narrowly graded ballast. Although a more broadly graded ballast can provide superior resistance to deformation and might provide the desired drainage and fouling material storage capacity. (Dingqing et al. 2015)
Ballast with higher specific gravity can improve the strength and stability of railway track, increase durability under cyclic loading, and minimise ballast settlement. (Jeffs and Marich, 1987; Indraratna et al., 1998).

2.6.2 Mechanical characteristics of the ballast:

Three important aspects of mechanical properties of ballast; shear strength, settlement and degradation, are discussed below

- **Shear strength**

Characterizing the aggregate material using the previous physical tests is often accompanied by assessment of the stress and strain strength behaviour of ballast to provide insight to performance of the ballast under these tensional and compressional forces. Assessment of ballast performance is a critical aspect of track structural design and performance evaluation. The shear strength of granular materials is assumed to vary linearly with the applied stress, and the Mohr–Coulomb theory is used to describe the conventional shear behaviour. Recent research by (Indraratna et al.,2000) and (Ramamurthy, 2001), among others, has shown that rocks at low normal stresses are tested; a non-linear shear strength response is obtained. (Indraratna et al.,1993) proposed a non-linear strength envelope obtained during the testing of granular media at low normal stress. This non-linear shear strength envelope is represented by the following equation:

$$\int f/\Omega c=m(\Omega' n/\Omega c)^{n}$$

Ω_c–uniaxial compressive strength of the parent rock determined from the point load test.
m and **n** are dimensionless constants.
$\int_f$-is the shear strength at failure.
Ω'_n-is the effective normal stress.
The non-linearity of the strength envelope is governed by the coefficient n. For small confining pressures (below 200 kPa) representative of rail tracks, n takes values in the range 0.65 – 0.75. A

large-scale cylindrical triaxial apparatus, which could accommodate specimens 300 mm in diameter and 600 mm high. (Indraratna et al.,1998) to verify the equation above.

a-Experiment on shear strength

Testing of ballast performance is often challenging due to difficulties in obtaining representative samples and preparing performing tests that accurately represent field conditions.

I -Direct shear test

The shear strength of ballast obtained from a direct shear test is characterized by the stresses at failure and represented as the friction angle ϕ.

figure 2. 21: Direct shear test experiment.(Railway Geotechnics, Dingqing Li, James Hyslip, Ted Sussmann, Steven Chrismer)

II-Triaxial test

The evaluation of the ballast strength with the triaxial test provides both the friction angle ϕ and young’s modulus (stiffness). The stiffness attained by the ballast after it stabilizes following a large number of repeated load cycles is an important parameter than is the initial stiffness.

Triaxial testing of ballast requires a large test apparatus as shown in figure 2.22. The diameter of the aggregate sample being tested should be at least 2.5–3 times greater than the largest individual particle size according to ASTM standards. The triaxial test can be used in monotonic loading to determine the ultimate shear strength of ballast. However, as the load on ballast does not monotonically increase until failure, a better use of the triaxial test is for repetitive load testing, where the samples are tested for particle breakage and permanent deformation after a large number of load cycles.

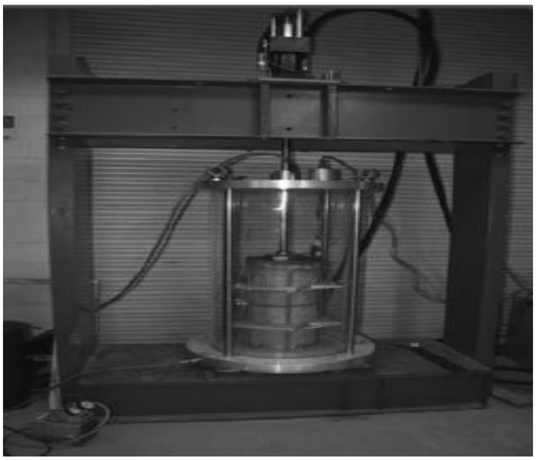


figure 2. 22: **Large** ***(0.3 m, 12 in. diameter) triaxial test apparatus. (Courtesy of University of Illinois Urbana-Champaign, Champaign, Illinois.)***

- **Ballast degradation**

The mechanical degradation of ballast grains produces finer particles that vary in size. The volume of voids of a ballast layer of a newly constructed track is around 45%. When the railway track settles under cyclic heavy train loads the ballast grains rearrange into a more packed position, hence reducing the volume of voids. During this stage, a primary ballast crushing takes place at the contact points of coarser grains. The corners and sharp edges are lost and collected in the voids between grains. This enables further grain rearrangement followed by additional ballast crushing upon traffic loading. The sliding and/or rolling action of grains on each other results in degradation of aggregates through attrition. The product of this degradation mode is pulverized ballast particle, which contribute to the fouling of ballast voids.

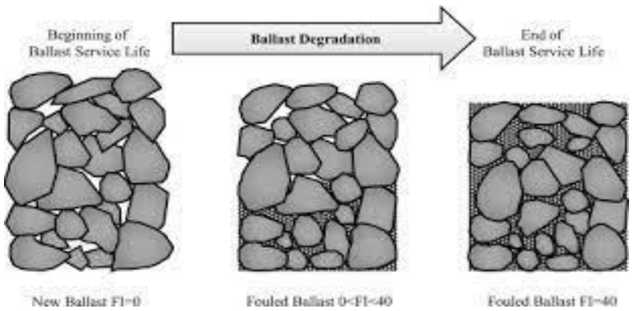


figure 2. 23: *Cycle of ballast degradation(Raymond and Diyaljee, 1979, Railroad ballast load ranking classification)*

The main causes of ballast degradation are excessive cyclic loading and vibration, temperature and moisture fluctuation, and impact load on ballast due to severe braking. The degradation of ballast particles can occur in three ways (Raymond and Diyaljee, 1979);

- the breakage of particles into approximately equal parts (this is responsible for the long-term stability and safety of the track)
- the breakage of angular projections, which influences the initial settlement
- the grinding-off of small-scale asperities (the presence of fines can cause fouling and reduce drainage).

The main factors that have effects on ballast breakage can be divided into the following categories. (Indraratna et al., 2003b):

- ballast properties related to the characteristics of the parent rock (e.g., hardness, specific gravity, toughness, weathering, mineralogical composition, internal bonding and grain size)
- particle properties associated with the blasting, crushing and transportation processes (e.g., soundness, particle shape, particle size and surface smoothness)
- factors related to the field/experimental variables (e.g., confining pressure, initial density or porosity, thickness of ballast layer, ballast gradation, presence of water or ballast moisture content, dynamic loading pattern, including train speed and frequency).

- **Settlement**

The settlement of ballast can be both elastic (such as the initial settlement due to the compaction of ballast) and plastic (due to breakage of ballast particles). As identified by (Selig and Waters, 1994), settlement of ballast may not be a problem if it occurs uniformly along the length of the track. In fact, differential track settlement is more important than the total track settlement. In the long term, however, the subgrade is more influential. The settlement of ballast is influenced by large trainloads, the number of load cycles, and high speed of trains. There is an initial stabilisation stage where settlement is rapid, followed by settlement over an extended period at a decreasing rate.

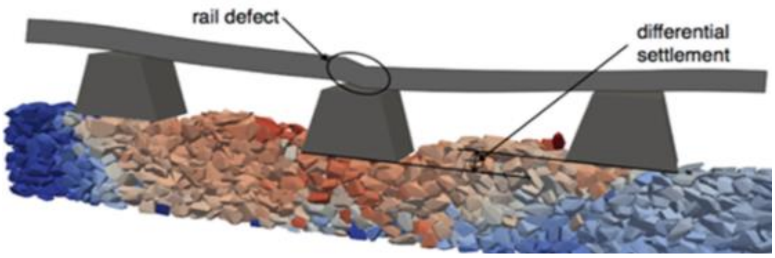


figure 2. 24: Differential *settlement*

(Jeffs and Marich, 1987) and (Indraratna et al., 2003b) among others, based on their experimental studies, reported that the relationship between the number of load applications and settlement of ballast is non-linear. The rail track settlement is usually related to the number of load cycles by a semi-logarithmic relationship such as;

$$S_N=a(1+k\log N)$$

where **SN** is the settlement of ballast at **N** load cycles, a is the settlement at the first cycle, and **k** is an empirical constant depending on the initial compaction, type of ballast, type of reinforcement and degree of saturation. Current data indicates a rapid initial settlement followed by gradual consolidation with increasing number of load cycles. Therefore, a more accurate relationship for settlement under cyclic load could be presented as a power function of the number of load cycles (Indraratna et al., 2000):

$$S_N=a.N^b$$

where **b** is an empirical coefficient determined from nonlinear regression analysis. It is important to note that the variation of the applied load affects only the coefficient **a**, while the coefficient **b** remains relatively unchanged.

✓ **Experiment to determine ballast settlement**

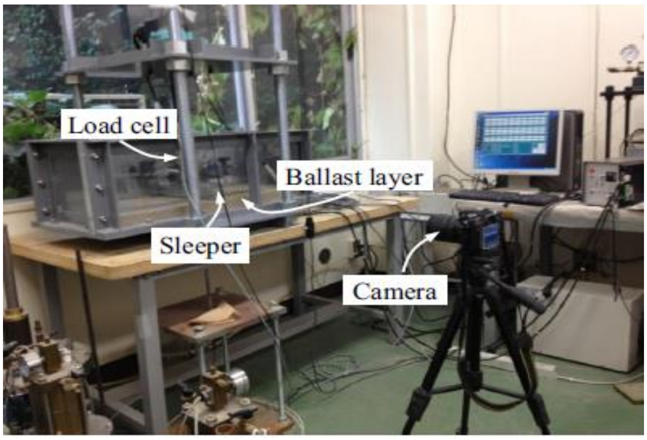


figure 2. 25: Set *up of the experiment*

A scaled-down (therefore a one-fifth of the field scale) model track-bed was constructed in a sandbox with the interior dimensions of 800mm length (i.e., Front side), 304mm width and 300mm height as shown in Fig 2.26. The front wall of the sand box is made of glass to facilitate capturing of the images during the loading process. A duralumin footing, 48 mm wide and 290 mm long, was used as the sleeper. In the model tests, the ratio of sleeper width to ballast thickness was designed to replicate field conditions. Therefore, the stress distribution zone of the model tests and field conditions were maintained equally. The axial displacement was measured using two external displacement transducers placed at the front and the back of the sleeper.

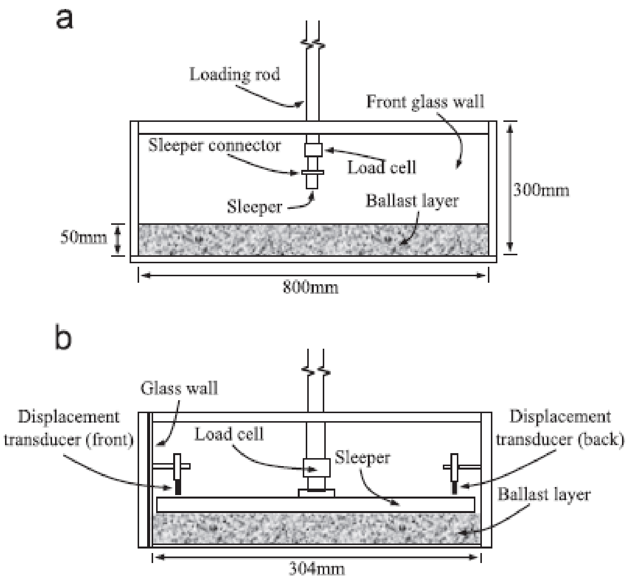


figure 2. 26: Sandbox *experiment (Soils andFoundations2016;56(4):652–663)*

✓ **effects of settlement.**

- Leads to reduction in the ballast strength thus reduction of its functionality.

- Reduces the void ratio which leads to reduction of permeability.
- It leads to deformation of the rails due to the detachment of the sleepers from the ballast.

figure 2. 27: **Track settlement zone due to insufficient compaction of ballast.(Railway Geotechnics,** Dingqing Li, James Hyslip, Ted Sussmann, Steven Chrismer)

- **Rigidity**

The increasing cost of railway track maintenance has made the selection of an appropriate aggregate for each ballast application a matter of considerable financial importance. The aggregate selection procedure must permit the engineer to identify the physical characteristics of a ballast so as to assess the differential aspects of the material with respect to other available materials with similar properties and to evaluate, in financial- terms, the expected costs and benefits from the use of each ballast.

To perform well in railway track, the aggregate for ballast must be tough enough to resist breakdown through fracturing under impact, and must be hard enough to resist attrition through wear at the ballast particle contacts. It must be dense enough so that it will have sufficient mass to resist vertical and longitudinal forces. The aggregate must be resistant to weathering so that weakening of the ballast does not occur from crystallization or acidity of impurities dissolved in rainwater or from daily or seasonal fluctuations in temperature or other weathering processes.

It must also be resistant to the chemical degradation resulting from the action of rainwater on foreign source fines. For example, trace elements such as sulphur in coal are highly likely to increase the acidity of any moisture trapped within the ballast. This acidity will cause solution weathering of the aggregate, particularly lime-stones. All ballast aggregate material may be expected to degrade to some extent with time. Because of this it is strongly recommended that aggregates be subjected to some form of petrographic examination to assess the long-term effect of the degradation and the ability of the ballast to remain free draining and elastic. Petrographic thin section examination that determines rock type, mineralogy, and structure, also establishes whether micro-fractures exist within the aggregate source and whether former micro-fractures have been weakly cemented with secondary minerals that might weather and soften quickly.

- ✓ **Origin of ballast aggregates**

Quarried stone ballast should be obtained from competent strata of reasonable thickness. The extent of the rock deposit should be sufficient for economic ballast production. A large variety of rock types are used as ballast. In general, the fine hard mineral-grained un-weathered aggregates make the best ballast. These include igneous rock types such as rhyolite, andesite, and basalt as shown below.

Basalt **Andesite** **Rhyolite**

figure 2. 28: Quarried *stones (geoscience news and information, geology.com)*

Second best are the coarser grained igneous rocks such as granite, diorite, and gabbro, along with the hard mineral-grained well-cemented sedimentary rock and hard mineral-grained metamorphic (or transformed) rock such as quartzite.

Granite **Diorite** **Gabbro**

figure 2. 29: Igneous *rocks (geoscience news and information, geology.com)*

Less satisfactory but often more commonly used because of their cheaper production cost and wider availability are the sedimentary rock types such as limestone, dolomite, sandstone, and siltstone. Rock types such as shale and slate shown below, result in flaky or elongated particles, should not be permitted because these shaped particles do not result in good interlocking, particularly when subjected to vibrations. Similarly, sedimentary and metamorphic rock types that contain visible quantities of secondary minerals, which weather quickly, should be rejected.

Shale **Slate**

figure 2. 30: sedimentary *rock (geoscience news and information, geology.com)*

A typical example would be small quantities of pyrites, which oxidize to produce a ferric compound and then sulphuric acid, which is highly corrosive to the metallic parts of the track structure.

- **damping effects of the ballast**

Damping is the phenomenon by which mechanical energy is dissipated (usually by conversion into internal thermal energy) in dynamic systems. Knowledge of dumping in a dynamic system is important in the utilisation, analysis, and testing of the system. Damping Force describes energy dissipation mechanism which induces a force that is a function of a dissipation constant and the velocity. Ballast is typically made of course stone, in such a way that sleepers can be embedded into it. The ballast layer supports the vertical and lateral forces and it provides a considerable flexibility and dissipation to the system.

Ballast damping is a characteristic of a system that decreases the ratio of response to force especially at frequencies near the modal frequencies of the system. Therefore, in order to see effects of damping on the system vibrations, the system has to be excited at frequencies near its modal frequencies or to be freely vibrated since train running speed varies from place to place and it causes different loading frequencies.

2.7 INTERACTION OF RAIL –SLEEPER-BALLAST

Two transmission directions of rail vibrations are mostly considered; along the rail and downwards to the sleepers. The downward force is a combination of a static load and a dynamic component superimposed on the static load. The static load is the dead weight of the train and superstructure, while the dynamic component which is known as the dynamic increment depends on the train speed and the track condition. (Ahlbeck et al., 1978) assumed that the load transmitting from a sleeper to the ballast approximately coincides with the cone distribution. That is to say, the stresses of the ballast are uniformly distributed over the cone region and zero outside the cone. The inclination of the cone is just the ballast stress pervasion angle corresponding to the Poisson's ratio. Thus, the effective acting region of the ballast under each sleeper can be determined, as shown in the figure 2.31 below.

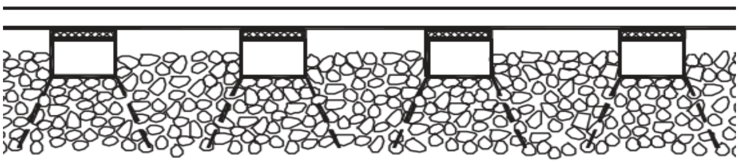


figure 2. 31: Load distribution region in continuous granular ballast(W.M.Zhai, K.Y.Wang, J.H.Lin, 2014)

The vibrations transmitted along the rail control the effective sound radiating length of the rail. The ratio of the transmission rate between the acceleration amplitudes of the sleepers and the on-support rail in the same section, reflect the transmission characteristics of rail vibrations in the downward direction which is associated with ground-borne vibrations and sleeper sound radiations.

The interaction between ballast particles and sleepers should be investigated in all three directions in order to have a reliable structure. The curved railway tracks are the critical locations where high lateral loading conditions are generated due to abrupt change in the route direction. For this reason, it is believed that a large impact force is induced by the train to the rail components, particularly at the beginning of the curve. Accordingly, the assessment of lateral interaction between sleeper and ballast layer becomes much imperative, especially under dynamic loading circumstances.

The dynamic lateral response of track and its components, however, has not yet been sufficiently appreciated by most of the studies which makes it hard to explain the interaction between the vibration transmissions in different directions, especially the dynamic lateral interaction between the track and the ballast layer. (Jones et al.) compared Track Decay Rates (TDRs) of different track structures according to their simulations and measured the frequency response functions, (Li and al.) calculated the dispersion relation and TDRs of the ballast were simplified as semi-analytical finite element method, but fasteners, sleepers, and ballast were simplified as semi-analytical finite element method.

2.7.1 recent research

- **(Selig & Waters, 1994)** conducted a parametric study of a three-dimensional, multi-layer model for determining the elastic response of the track structure. They found that as the sleeper spacing increased from 250mm to 910mm, the load applied to the sleeper beneath the wheel increased by a factor of about 4. They also found that for an increase of rail moment of inertia increased from 1610cm^4 to 6240cm^4, the load applied to the sleeper beneath the wheel decreased by 40%. The vertical downward force at the rail-wheel contact points tend to lift up the rail and sleeper at a distance away from the contact point, as shown in the Figure 2.32 below

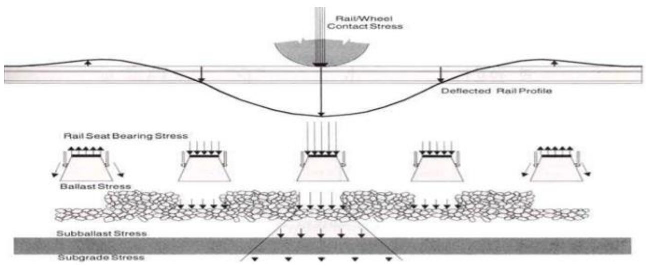


figure 2. 32: Typical wheel load distribution in track (Selig and Waters, 1994)

The uplift force depends on the wheel loads and self-weight of the superstructure. As the wheel advances, the lifted sleeper is forced downwards causing an impact load, which increases with increasing train speed. This movement causes a pumping action in the ballast, which increases the ballast settlement by exerting a higher force on the ballast and causing pumping up of fouling materials from the underlying materials in the presence of water. It is also noted that the impact load increases with the increase in track irregularity or differential settlement (i.e., impact load increases with the increase in the size of the gap underneath the sleeper). The increase of impact load would then lead to an increase in ballast settlement and lead to a larger gap underneath the sleeper. Thus, track geometry tends to degrade in an accelerating manner.

- **(Nguyen et al., 2003)** implement a continuum model of granular material to describe the no-tensional effects. A continuum model is used with an introduced discontinuous phenomenon that occur in granular materials. The hypotheses and the constitutive law for granular bodies in 3D are derived from a proposed modified elastic strain-energy function. Within the context of finite element method the proposed constitutive relations are numerically implemented. A simple example is presented to analyze the difference between the vertical stresses in two different cases, the nonlinear elastic case and the no-tension nonlinear elastic case. The example simulates a quasi-static test on ballast materials. The test shows that the vertical stresses in the no-tension nonlinear elastic case are higher than in non-linear elastic case. Therefore, the no-tension non-linear elastic model is the weaker one. Hence, it is likely that the no-tension effect in granular structures is of great importance.

❖ **(Morteza et al.,2016)** performed an experimental study on the interaction of pre-stressed concrete sleepers with ballast. A high-capacity accelerometer was attached to the pendulum loading test device hammer to measure the impact load. Three accelerometers and a nonlinear variable displacement transducer was used to record the sleeper lateral accelerations and displacements. The sleepers in a typical ballasted railway track are surrounded by ballast particles in three different contact areas namely base(B), crib(C), and shoulder(S).

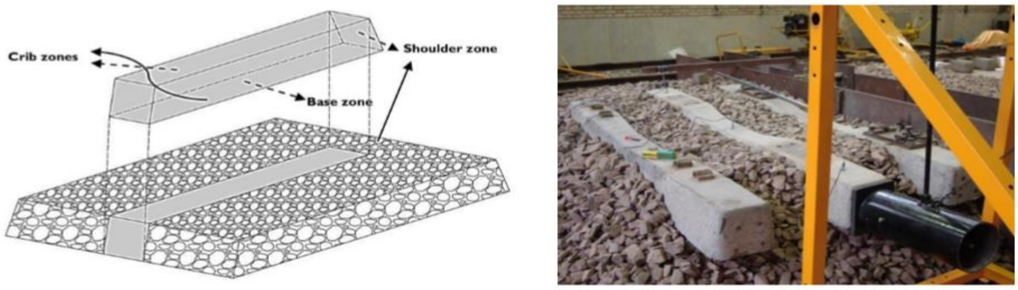


figure 2. 33: Base, crib and shoulder zones *and* Test setup apparatus (Morteza E. and Ahmad H., 2019)

To compute the system lateral impact load and inertia force, the accelerations of hammer and sleeper were multiplied by their masses, respectively. Subtraction of these forces from each other led to sleeper-ballast interaction force. The maximum of this force is called the sleeper dynamic lateral resistance (DLR) displays the portion of each ballast zone in total DLR of concrete sleeper in terms of lateral impact load. These contributions are averagely 48, 23, and 29 percent of the whole. In addition, the effect of crib zone on the sleeper DLR can be observed.

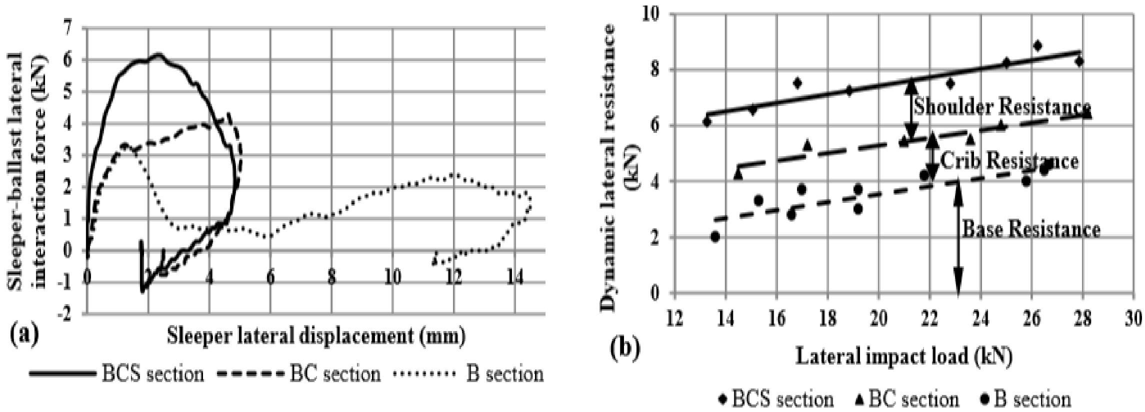


figure 2.34: Sleeper ballast lateral and Dynamic lateral (Morteza et al., 2016)

❖ **(Rauert et al., 2010)** study how the ballast affects the transfer of loads between two separated structures. Numerous two-track railway bridges often provide two separated bridge decks but with a continuous ballast layer. Experimental tests identifies the parameters which affect the load transfer. Furthermore, numerical analyses are done to investigate the influence of mechanical parameters. A stiffness parameter describing the load transfer between the structures is found. The tests show that the ballast introduces an additional bending stiffness to the structure. This is of great interest in terms of serviceability limit state design.

- **(Markine et al., 1998)** describe alternative track structures for high-speed trains. Most of the railway tracks used nowadays belongs to traditional ballasted track structures. High speed tracks cause increasing maintenance cost, which require high positioning accuracy of the rails. Using ballasted tracks for high-speed operations has shown problems. In particular, due to churning up of ballast particles at high speeds, serious damage of wheels and rails can occur. For the high-speed trains, the embedded rail structure is a serious alternative to the ballasted track in railway bridge design. The major advantage of ballast-less track is low maintenance effort and high availability. Furthermore, the mechanical properties of a track without ballast can be better determined and therefore the track behavior can be more accurately described and analyzed, using numerical methods.

2.8 CONCLUSION

In this chapter, our main focus was understanding the historical background of dynamic analysis and examine the interaction of ballast with other elements such as the rails and sleepers. we have come to realize how crucial the ballast is in the construction of a railway bridge especially for high-speed train just with the concrete study of its properties both physical and mechanical and therefore enables us to understand profoundly the dimensional aspects, conception and maintenance of the ballast through different phases of construction. Generally, the information obtained from this chapter will guide us through the next chapters such as modeling the ballast structure with the finite element method and the rail- sleepers- ballast interaction which is actually our main focus and point of interest. With a brilliant understanding of these interactions and the use of the finite element method, we can create a model close to the real structure in study and therefore a better conception and mitigation of problems that may surface during the service stage of the railway track.

CHAPTER 3: **PRESENTATION OF THE RAILWAY COMPOSITE BRIDGE**

3.1 INTRODUCTION

The railway viaduct project located in the town of Tlemcen designed following the euro code, Snow and Wind Regulations “R.N.V. 1999” and RPOA/2008 version norms to ensure proper conception and realization. It is composed of a deck sections with a continuous span, mixed metallic frames of type quadri-steel beams, a slab made of reinforced concrete resting on a fixed column as shown in figure 3.1. The viaduct is of 1 independent deck section of 80m long on which technological equipment are installed such as guard rails, earthing systems, electric systems, telecommunication and signalization equipment that serve a grand role in the project.

3.2 GENERAL DESCRIPTION OF THE RAILWAY COMPOSITE BRIDGE

The Viaduct of the new Oued-Tlélat / Tlemcen High Speed Line is a CURRENT RAILWAY VIADUCT whose typology is that of decks in continuous spans of 40 + 40m resulting in an expandable length of 80m which does not exceed the maximum length of 90m, so no rail expansion device is required. Consequently, the use of long welded rails (L.W.R.) generates the "Combined Track-Structure Response" to variable actions (vertical loads, acceleration, braking, thermal variations) which is studied among a global model of the viaduct including all its components (ballasted track, decks, piers, foundations and backfill behind abutments).

It is a viaduct with 2 decks of 40 + 40m for a total length of 80m, with a Fixed Pier (P01) and the 2 Mobile Abutments (AA / AB) in the longitudinal direction.

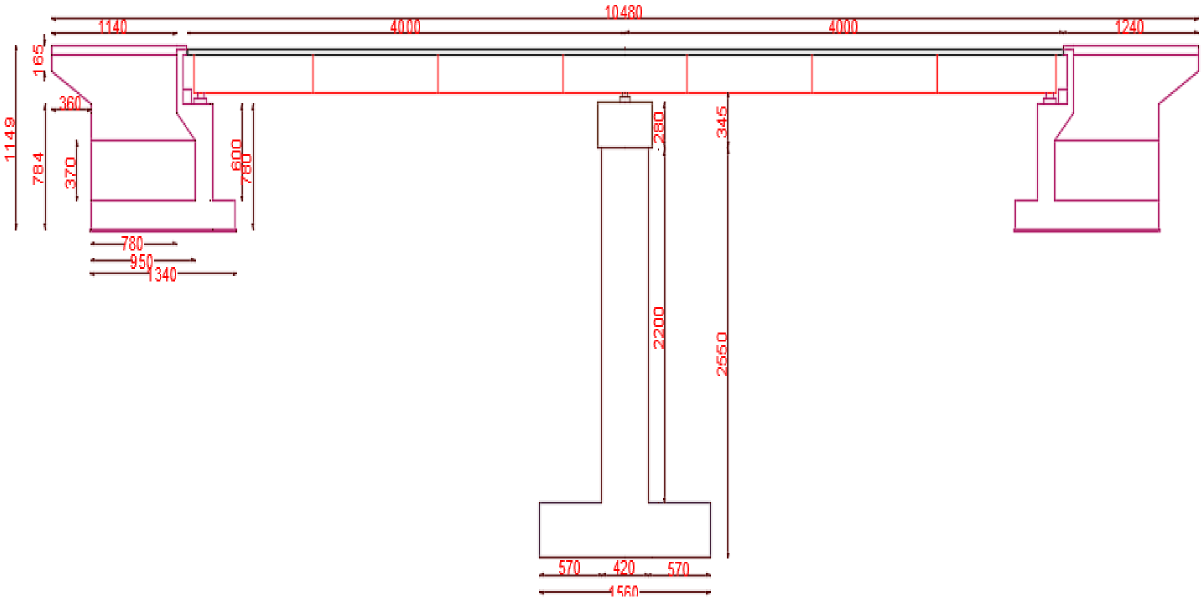


figure 3. 1: longitudinal *view of the railway bridge*

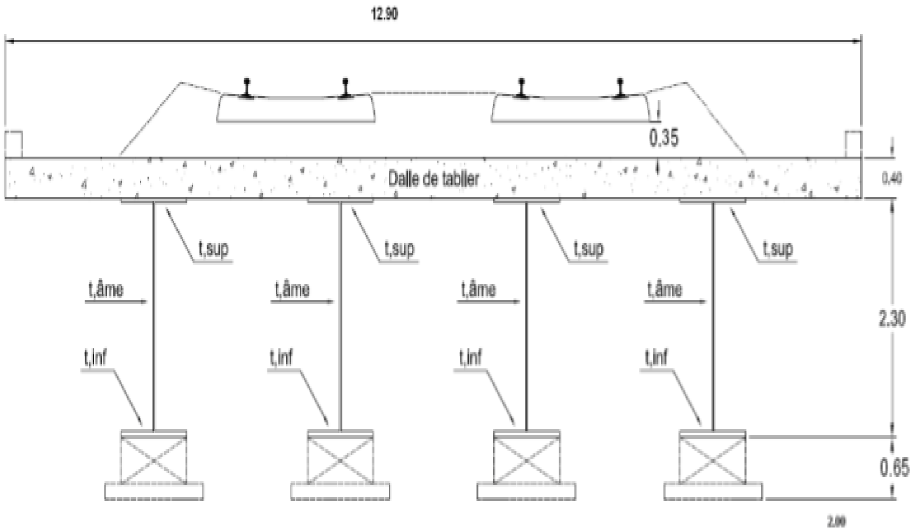


figure 3. 2: Transverse *view of the deck section.*

Regarding the long-welded rails (L.W.R.), the deck in continuous spans of 40m + 40m result in an expandable length of 2x40 = 80m, the viaduct deck presents the following support diagram:

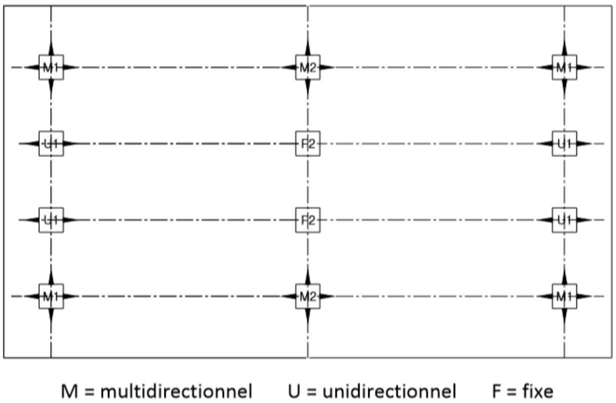


figure 3. 3: Arial *view of the deck section*

3.3 NORMS

The design of structures complies with the following Algerian Rules and Eurocodes with the French National Annexes.

- Algerian Earthquake Rules RPOA / 2008 version (for the coefficient of acceleration)
- Snow and Wind Regulations "R.N.V. 1999" (for the reference speed)
- Eurocodes NF EN 2003 (EIC 1 ,2,3 and 4)

3.4 CONSTRUCTION MATERIALS

3.4.1 Concrete

Concrete is a widely used construction material which is a mixture of cement, aggregates, water and admixture. The concrete is the very first choice for the civil engineering industry as it is a more conventional material readily available and cheap. It has good properties like excellent durability, mould-ability, in addition it can be produced with the industrial bi-products and with less energy input. Even though, in the trending world of the lightweight materials, concrete is the most construction material being used for the construction of foundations, tie beams, ground slabs and other underground and sea terrain structures.

figure 3. 4: Concrete formation(Civiconcepts, 2021)

The chemical reaction taking place within the concrete components are to provide the intended strength to the structure. The stiffening, solidification and hardening are the three main physical aspects of the setting and hardening process of concrete from its fresh state.

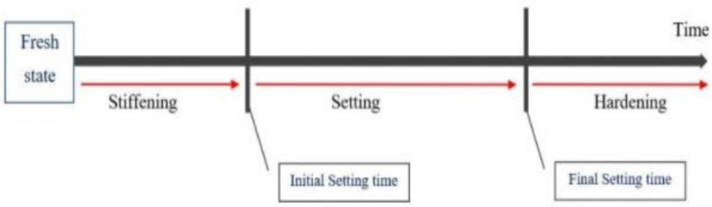


figure 3. 5: The physical processes of concrete formation(Diya, 2019)

The process of selecting the proper grade of concrete is important to arrive at an economical and durable material. In this manner the grade of the concrete is taking a big role here. Generally, the mix designs are prepared based on the requirement of strength, workability and durability for the project specifications. Different levels of strength and resistance were put in consideration in accordance to the elements of the project to be designed.

Below are concrete properties we used.

Weight per unit volume=25KN/m^3
Module of elasticity=32E6
Poison's ratio=0.15

3.4.2 Passive reinforced steel bars.

Passive steel reinforcing bars, also known as re-bars, should necessarily be strong in tension and at the same time, ductile enough to be shaped or bent. Steel rebar is most commonly used as a tensioning devise to reinforce concrete to help hold the concrete in a compressed state. Concrete is a material that is very strong in compression, but virtually without strength in tension. To compensate for this imbalance in a concrete slab behaviour, reinforcement bar is cast into it to carry the tensile loads. The surface of the reinforcement bar may be patterned to form a better bond with the concrete.

figure 3. 6: Steel re-bars (The constructer, 2009)

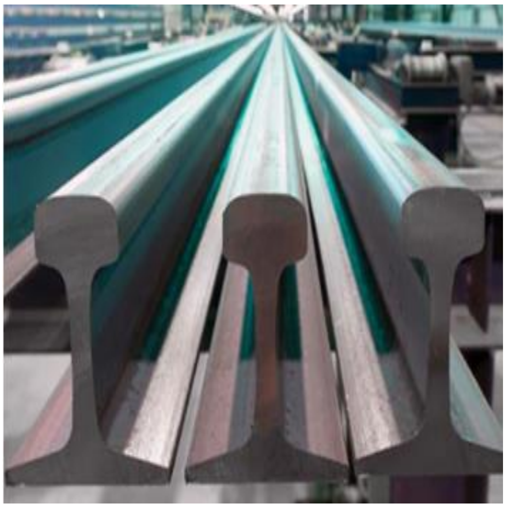
Below are the values we need that define the steel used. Steel of high adherence of FE400.

fyk = 400 N/mm²
γ = 1.15
eps_s = 0.0750 deformation at ultimate limit state.
eps_y = 0.00166 elastic deformation
Es = 250 000 N/mm²

3.4.3 Rail UIC60

The 60E1 (UIC60) rail model is manufactured according to the European standard EN 13674-1. It is used for railroad construction. This is a T-section type rail (flat bottom rails) with a mass of 60.21 kg per meter. For a standard track, the 60E1/UIC60 is used for a rail of medium and heavy load traffic.

The origin of the standard for the manufacture of this type of Steel Track comes from the International Union of Railways, founded in 1922, with the objective of promoting the global rail traffic and confront the transportation challenges and promote the sustainable development.



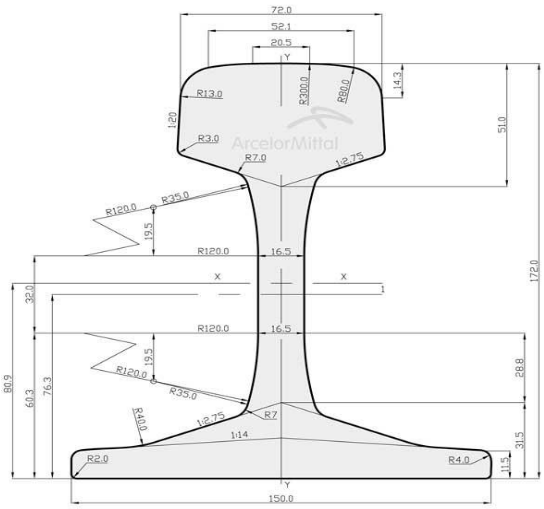


figure 3. 7: section steel track rail (ArcelorMittal,2012)

Dimensions of UIC60 Rail:

Type of rail	standard	Dimensions (mm)					Section(s)	Mass(m)
		H	B	C	D	E	Cm^2	Kg/m
		European standard EN 13674-1						
60E1(UIC60)	EN 13674-1	172	150	72	51	16,5	76,7	60,21

table 3. 1: table displaying rail dimension properties (ArcelorMittal,2012)

The details below define precisely the dimensional properties of the rail.

Cross sectional areas	$76.70cm^2$
Mass per metre	80.21kg/m
Moment of inertia x-x axis	$3038.3cm^4$
Section modulus of the head	$333.6cm^3$
Section modulus of the base	$375.5cm^3$
Moment of inertia in the y-y axis	$12.3cm^4$
Section modulus in the y-y axis	$68.3cm^3$
Indicative dimensions	A=20.456mm
	B=52.053mm

3.5 DETAILS OF THE CONSTRUCTION

3.5.1 General description of the viaduct

The purpose of this report is to describe the general characteristics of the studies used in the execution of the viaduct from PK 119 + 435 to PK 119 + 665. The viaduct represents the direct development of the characteristics at the study stage of execution.

The viaduct has a length of 80m designed according to the current viaducts, therefore a composite viaduct with regard to the deck, consisting of a continuous span of 40 + 40m with a mixed framework of solid core steel quadri-steel beam type and reinforced concrete slab resting on a column with a height of 28.30m and has a necessary function of Combined response of a Track-Structure.

The steel beams have a height of 2.30m and the average height of the reinforced concrete slab is 0.40m. They are provided with standard spacers every 10m on the axle support of a fixed column. This static scheme has a maximum expandable length of 80m (minor with the maximum length of 90m) and therefore we can use the long-welded rail for the track, without any expansion device.

The typical cross section of the viaduct, according to the typology defined, has a width of 12.90 (interaxis of the rails 4.20m) and the maintenance tracks have a width of 0.60m. The centre line of the electric traction poles is 3.25 from the centre line of the nearest rail. At the extremities in the cross section there are two borders each 0.20 in width.

According to the static continuous beam diagram, we have a stationary column and two abutments characterized by mobile supports. The column (fixed support) has a height of 28.30m from the head of the header/trimmer to the upper surface of the foundation footing.

3.5.2 COLUMN

COLUMN (P01)

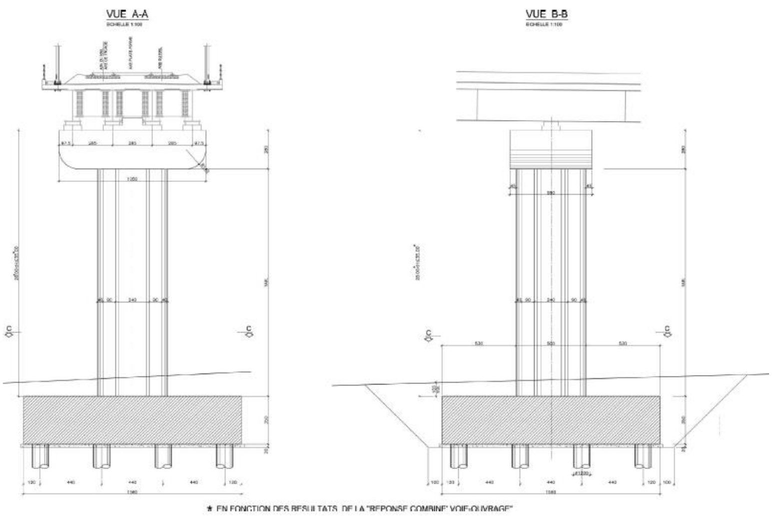


figure 3. 8: Fixed *support column*

3.5.3 ABUTMENTS

Abutment A is approximately 9m high plus 1.60m high for the foundation footing and the abutment B has a height of approximately 7.5m plus a height of 1.60m for the footing.

In the transition zone between the embankments and the abutments is planned for the realization of the "technical block" at the aim of limiting the differential settlements between the two types of structures (embankment / viaduct). The technical block is made with a gravel corner treated with 70kg / m^3 of cement, under the form layer. In the transition zone between embankment and viaduct, the transverse slope of the platform goes from 3% in backfill to 1.5% in viaduct.

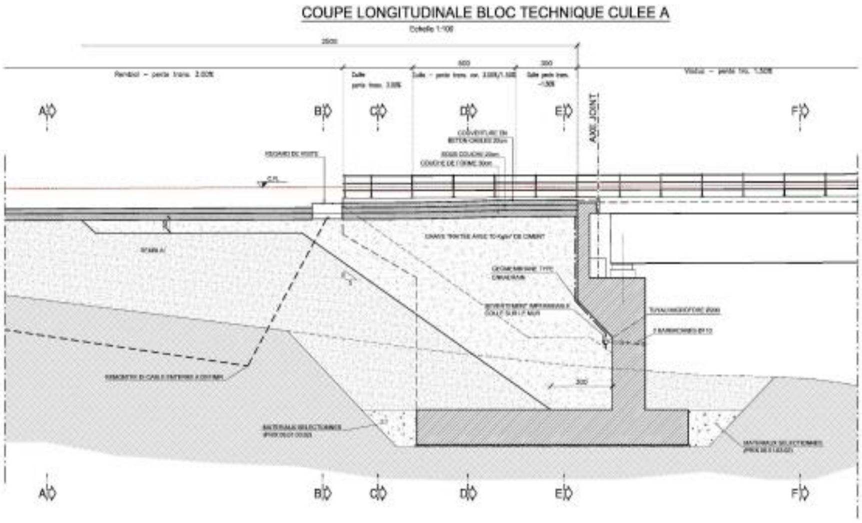


figure 3. 9: longitudinal section of the abutment.

3.5.4 EVACUATION OF WATER FROM THE PLATFORM

The waterproofing of the deck is a construction detail linked to the evacuation of water from the platform which in general leads to the durability of the structure. The waterproofing of the deck is carried out by means of a double waterproofing screed with thicknesses of 4mm and 3mm respectively, protected from ballast and atmospheric agents by a layer of bituminous material with a thickness of 4cm.

The water is evacuated from the platform using drainage pipes (or gargoyles) made with 200mm diameter galvanized steel pipes placed at 3.25m the axis of the rail closer, with a specific drainage grid and vertical protection with para-ballast function. The gargoyles are fixed to the deck and to the external beams by means of specific brackets.

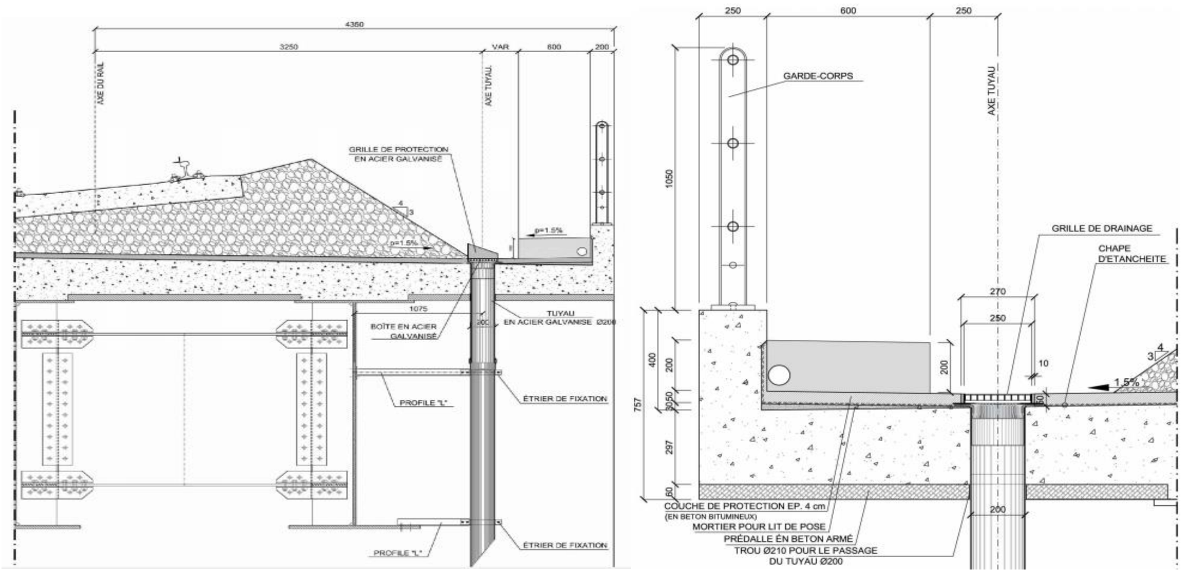


figure 3. 10: drainage *pipes (or gargoyles) for water evacuation water.*

$$Q\sqrt{}_b=C_qA\ (2gh)$$

Where:

Qb = water flow rate in the gargoyle (m3 / s);
A = wetted surface of the gargoyle (m2);
h = hydraulic head on the mouth (m);
Cq = 0.6 coefficient of contraction.

The calculations were carried out having imposed the planned geometry.

In particular:

- The pipe used has an external diameter of 200 Ø mm;
- The hypnotized maximum hydraulic load "h" (between the lower base of the grid and the top of the hydraulic surface) is equal to 9 cm, equivalent to the height of the grid (4cm) plus 5 cm, assumed as the maximum water height on the platform next to the ballast.

Using the formula of **Qb** there is a maximum flow rate of 23.12 l /s for the gargoyle isolated.

Once the maximum flow rate Qb for an isolated gargoyle was determined, the rainfall intensity was calculated of the project **Ic** (mm / h) corresponding to a return time equal to Tr = 10 years for each section rainfall along the route, with the following formula:

$$I_c=a.P_j10ans.tc^{(n-1)}$$

Where "**Pj10ans**", "**a**" and "**n**" are the parameters of the project and "**tc**" (in hours) is the concentration time assumed equal to 5 minutes.

The maximum flow rate flowing to each gargoyle is therefore:

$$Q=\frac{1}{3600}B_p.L_c.\phi.I_c$$

where:

Q = maximum tributary flow (l / s)
$\mathbf{B_p}$ = half width of the viaduct (12.90 / 2 = 6.5 m)
$\mathbf{L_c}$ = project centre distance (m)
ϕ = runoff coefficient equal to 1.00
$\mathbf{I_c}$= project rain intensity (mm / h)

By using such a relation, and imposing that the maximum flow rate is equal to $\mathbf{Q_b}$ (maximum flow rate of the isolated gargoyle), we obtain the maximum centre distance L at which to position the gargoyles:

$$L=\frac{Qb.3600}{Ic.B}$$

In continuation we report the results of computation obtained:

sections	Pk.in	Pk.fin	Pj_10ans	*A*	*n*	*tc*	*B_p*	*Ic*	*Qb*	*L*
	Km	*Km*	*mm/h*	*mm*	….	*min*	*M*	*mm/h*	*l/s*	*m*
BOUDJEBAA (Dar Esba)	**0+000**	**34+200**	**56,9**	**0,578**	**0,155**	**5,0**	**6,5**	**268,60**	**23,12**	**48,04**
SIDI BEL ABBES	**34+200**	**62+900**	**49,3**	**0,578**	**0,155**	**5,0**	**6,5**	**232,45**	**23,12**	**55,52**
LAMTAR*	**62+900**	**80+700**	**49,3**	**0,578**	**0,155**	**5,0**	**6,5**	**232,45**	**23,12**	**55,52**
OULED MIMOUN	**80+700**	**95+300**	**59,0**	**0,578**	**0,155**	**5,0**	**6,5**	**278,42**	**23,12**	**46,35**
BENBADIS	**95+300**	**114+400**	**59,4**	**0,578**	**0,155**	**5,0**	**6,5**	**280,23**	**23,12**	**46,05**
M'LILIA HENNAYA	**114+400**	**132+000**	**82,4**	**0,578**	**0,155**	**5,0**	**6,5**	**389,04**	**23,12**	**33,17**

table 3. 2: display *of the calculated values obtained*

Table 3.2 shows the values of L which represent, for each viaduct included in the respective sections, the maximum values of the longitudinal centre distance between two gargoyles. Depending on the results obtained, besides safety and according to the usual construction practices, we retain four suitable gargoyles (two for the left side and two for the right side) for each bay.

3.6 FOUNDATION

Regarding the foundations, depending on the geotechnical characteristics of the land, we have the two abutments with direct foundations; column(P01) will be built with foundation on piles with 16 piles of 1200mm in diameter (foundation footings 15.60 x 15.60 m and height 3.50m) and 25m pier lengths.

3.6.1 columns

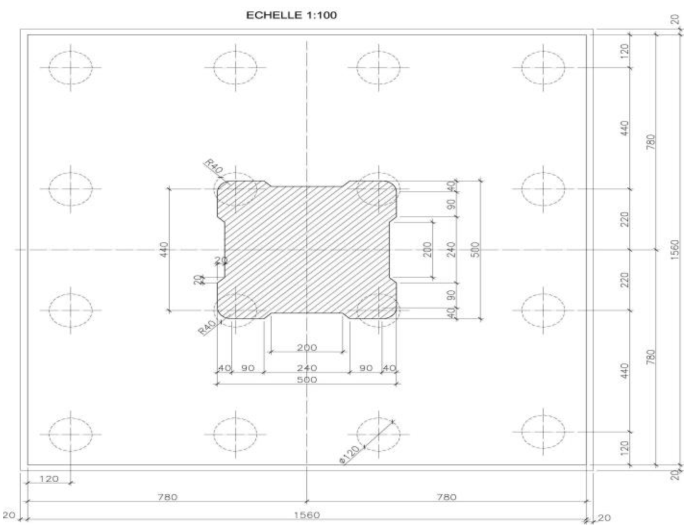


figure 3. 11: Foundation *of the column(P01)*

The foundations of the abutments present: the abutment A, has a footing 1.6 m high and dimensions in plan of 13.2m (length) x 13.5m (width), the abutment B, has a footing height of 1.6m and plan dimensions of 9.60m (length) x 13.5m (width).

3.6.2 Abutments

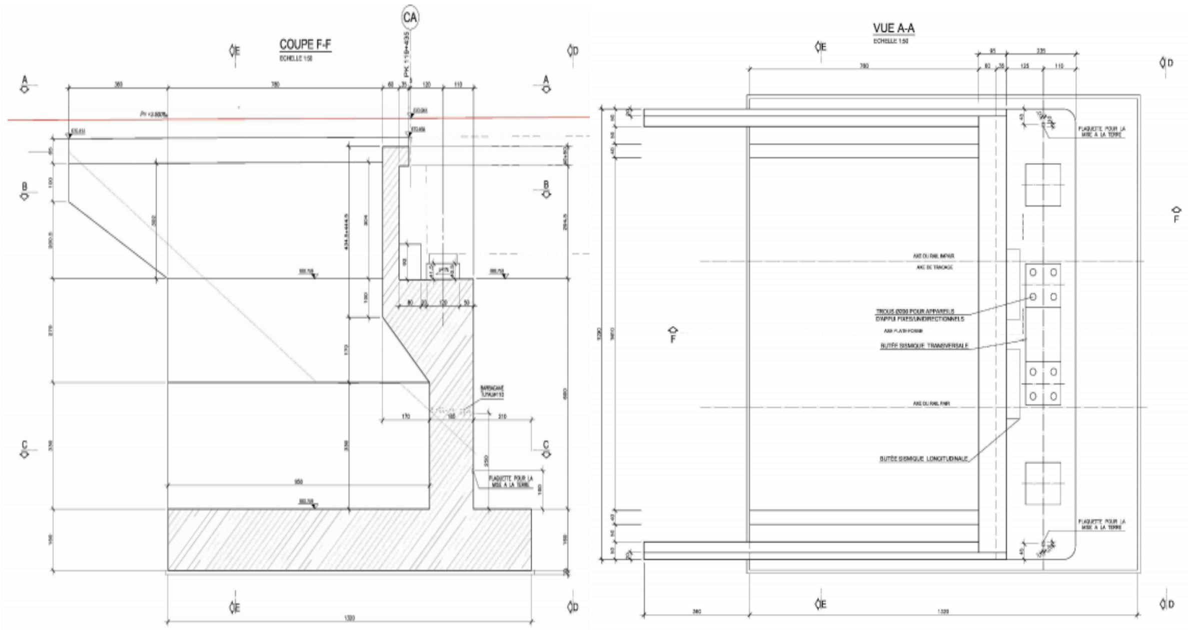


figure 3. 12: Abutment *sections*

The viaduct is part of the High-Speed Line and is located in the province of Tlemcen which, with regard to the earthquake, is a defined zone of low seismicity **(Zone 1 according to RPOA 2008)** with a corresponding acceleration coefficient of 0.15. The execution study was developed according to the indications of the Eurocodes and the Algerian norms such as R.P.O.A.

3.7 TECHNOLOGICAL EQUIPMENT.

3.7.1 security barriers (the guard rails)

The "guardrails" are necessary on both sides of the viaduct to guarantee the safety of the maintenance operations. The guardrails are made with vertical IPE100 profiles with a height of 1.05m (with plates welded to the base for anchoring to the edge of the viaduct) set up every 2.00m and three pipes (diameter 40mm) horizontally each 0.33m high. Guardrails will be made of prefabricated panels 6m. The connections between the prefabricated panels will be made by means of joints which guarantee the electrical continuity of the guardrail along the deck for earthing. In correspondence of the joints of expansion between the decks provide a junction detail of the guardrails which allows the relative movements of the decks. These guardrail joints between the decks will be dielectric to guarantee independent earthing of each guardrail.

Additional reinforcement is provided along the slab in concrete for solicitations generated by the electrical traction (ET) columns. The posts will be grounded with their connections to the ground cable consisting of a continuous electrical conductor implemented on both sides of the viaduct in the Cabling "channel".

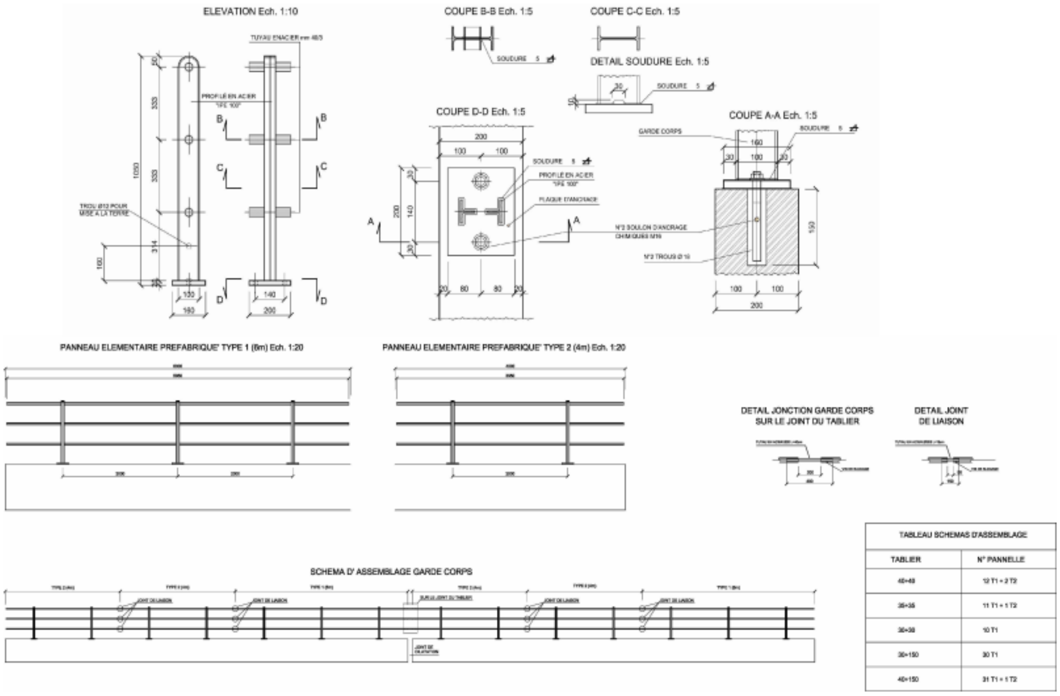


figure 3. 13: plan of the security ***barriers***

3.7.2 Electrical equipment.

Construction details needed to coordinate with Electric Traction (ET) focuses on:

- The possible anchoring of the ET posts.
- Earthing of the posts.

An anchoring system is provided (see the following figure) with two plates (the first welded to the post and the second installed under the reinforced concrete slab) connected by four lag bolts. A specific construction detail, shown in the following figure, has been studied to ensure the waterproofing of the slab also in correspondence of the ET posts.

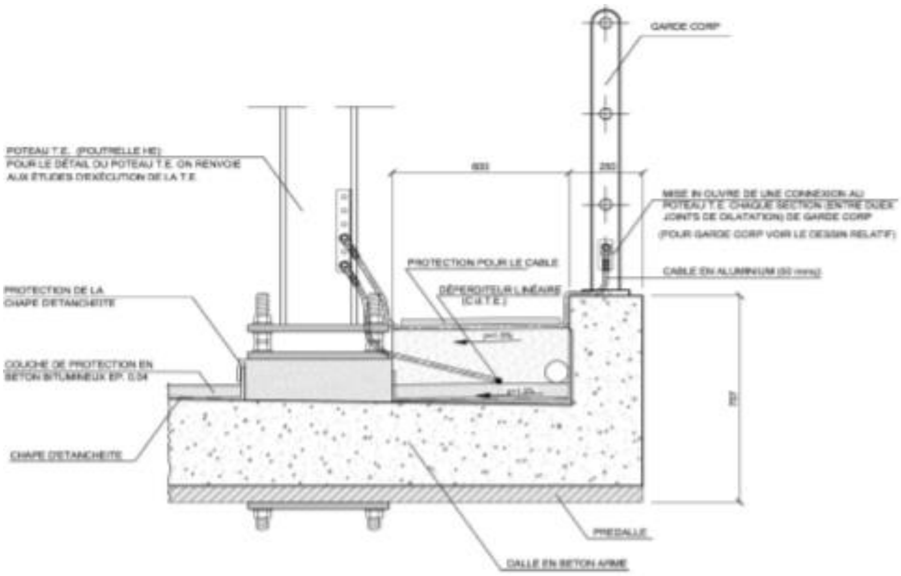


figure 3. 14: construction *detail of the anchoring system.*

3.7.3 Telecommunication and signalization equipment

Along the current viaduct there is no installation of telecommunications equipment and / or signalling, only the cabling of the optical fibre inside the specific "gutters". The wiring will be carried out inside pipes (protected with concrete pouring) to guarantee the cable slippage during viaduct movements due to thermal expansion and also travel under earthquake. The pipes will be cut in correspondence of the expansion joints where the cables will be protected using the solution in the figure 3.15.

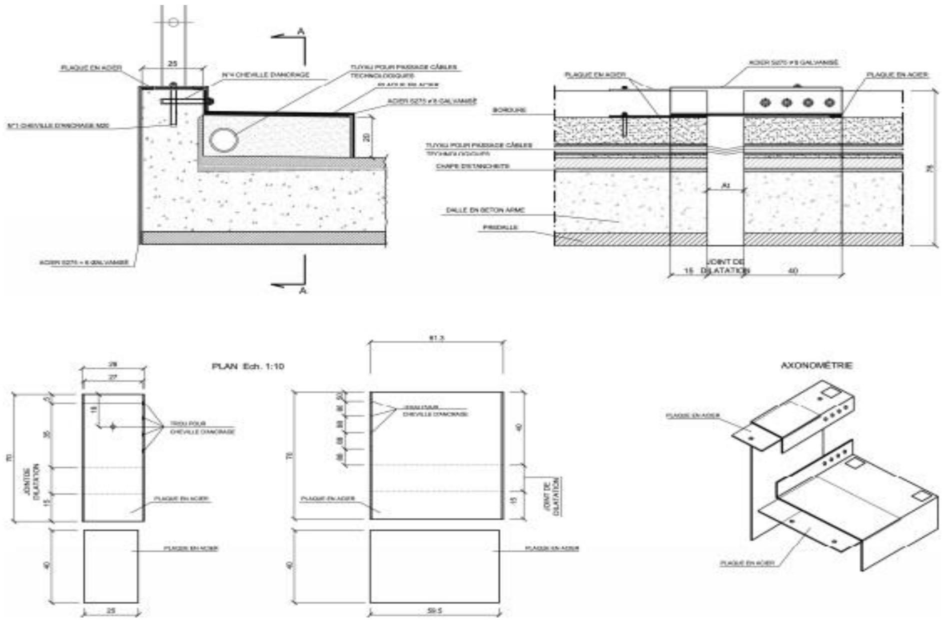


figure 3. 15: Telecommunications *and signalling equipment.*

The passage between the solution of the cable buried in backfill / cut and the solution in viaduct is carried out inside a manhole opening before the two abutments.

3.7.4 Earthing system.

In this paragraph we will provide a general description of the earthing system of the viaduct and in particular the necessary arrangements during the construction of the civil works and we refer for individuals to drawing and other coordinating drawings for all specifications. We have two separate earthing systems :

- The first is the ET earthing system to which all the steel parts which, in the event of an incident, can be tensioned by the catenary. These parts, such as guardrails, will be earthed with connection to the earth cable (linear loss) of the ET according to the predispositions in the figure below.
- The second system supported the grounding of the AB slab, steel beams the deck, the piers reinforcement cage with the pier and abutments.

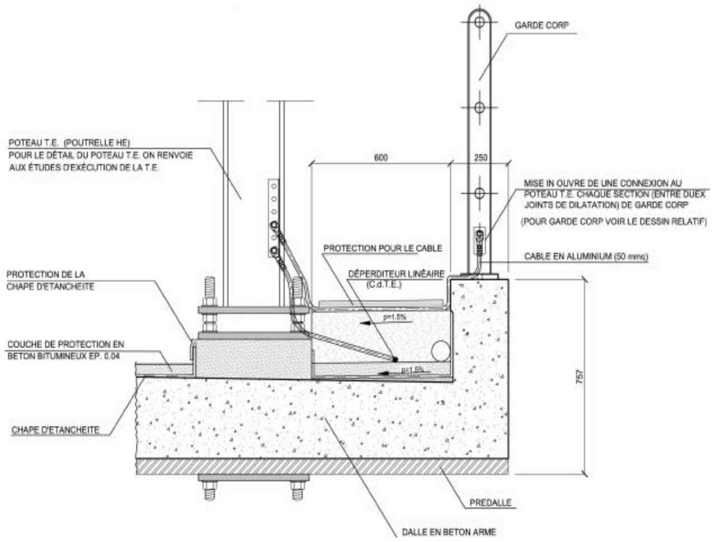


figure 3. 16: Earthing *system*

The earthing of the reinforcement of the deck slab is carried out with the predisposition of a dedicated welded reinforcement cage made up of three transverse loops at the end of the deck linked with 6 upper longitudinal bars and two lower bars of ϕ16mm. The loops will be welded to the studs of the beams to guarantee electrical continuity. With regard to abutments, reinforcement cages have been provided specific welding, dedicated to earthing. For each mobile stack, the connection is provided loops with five plates of which four on the headers, and a plate are linked to the lower loop for possible earthing, protected by a manhole. The header plates will each be linked to the beams by means of an aluminium cable with a minimum section of $50mm^2$ fixed to the bimetallic support. Regarding each abutment, we have the loops of the front wall and the side walls which will be welded together and connected to the two plates on the upper part of the front wall; the brochure for any earthing is provided at the bottom of the front wall. In case of pile foundations, also the reinforcement of each pile will be connected by welding reinforcement for the earthing of the column / abutment.

3.8 CONCLUSION

In this chapter, we outlined the general description of the project at hand, therefore, the viaduct and its relative supports such as the column, abutments and technical equipment used in respect with the different norms employed for this project.

The good quality of the construction is obtained by the appropriate use and satisfying study of the materials used and that's why we emphasized especially in regard to which materials are to be employed and their respective dosage as required. The description of the supports of the viaduct such as the column and the abutments give us a great understanding of the geography and the geotechnical nature of the environment where our project is implemented. A satisfying study of the foundations of the project was carried out to ensure great security and stability of the construction while in service state.

With a deep understanding of this chapter, we can now freely dive in the next chapter with sufficient knowledge concerning the project most especially the dimensions which we shall apply in the next chapter of calculations and assimilations.

CHAPTER 4: FINITE ELEMENT MODELLING OF THE RAILWAY COMPOSITE BRIDGE

4.1 INTRODUCTION

The software application used in our modelling of the studied railway composite bridge is SAP2000 which applies the principles of finite element method during the dynamic analysis execution of the railway bridge. With the help of this application, we are able to define and specify each element such as the columns, beams, slab, diaphragm, steel beam spacing, sleepers and many more. Material type such as concrete and steel are also defined with their respective properties. In addition, results of dynamic analysis are obtained from the same application which we interpret and use in accordance to our satisfaction.

4.2 DEFINING THE STEEL COMPOSITE BEAM

Clicking on the file icon displays a window that contains an icon of new model which consists of different models from which most structural element can be modelled.

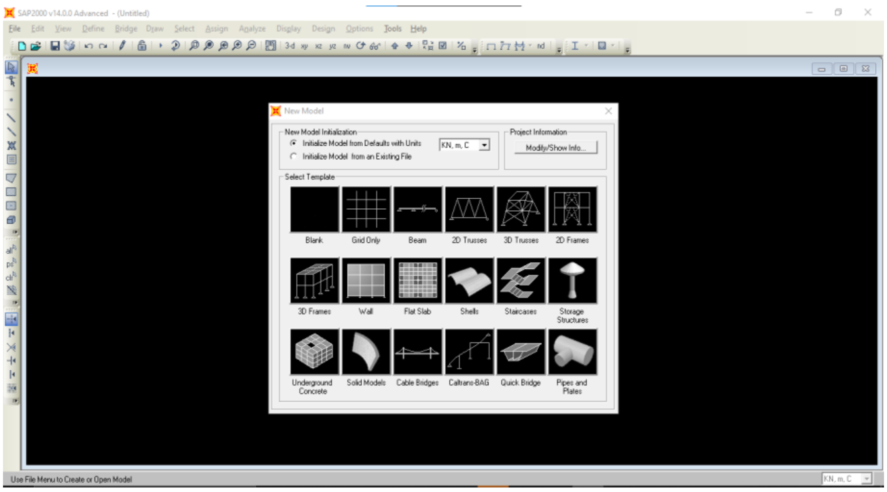


figure 4. 1: window *displaying designing options in SAP2000*

Considering our Project, we used the beam element to model one steel beam with two spans each of length 40m which later on we replicated to create three more similar beams.

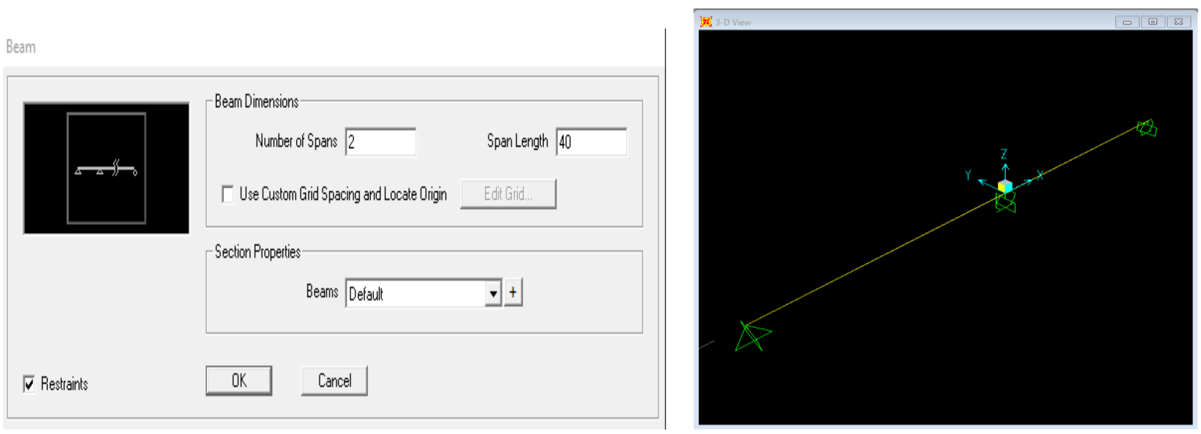


figure 4. 2: windows *displaying the creation of the beams (on the left) and how they appear after being created (on right)*

4.3 MATERIALS

The composite railway bridge consists of different types of materials used such as concrete **(4000Psi)** and steel **(A992Fy50)** which need to be defined from define-materials-define materials where you can as well modify the existing materials properties of each one of them.

4.3.1 Defining of concrete

To define the concrete, from the step "define materials" mentioned in 4.3, from the window given, two materials (4000Psi and A992Fy50) already exist in the programme, 4000Psi is selected and its material properties like the weight per unit volume, modulus of elasticity and Poisson's ratio are modified just as shown in the windows below.

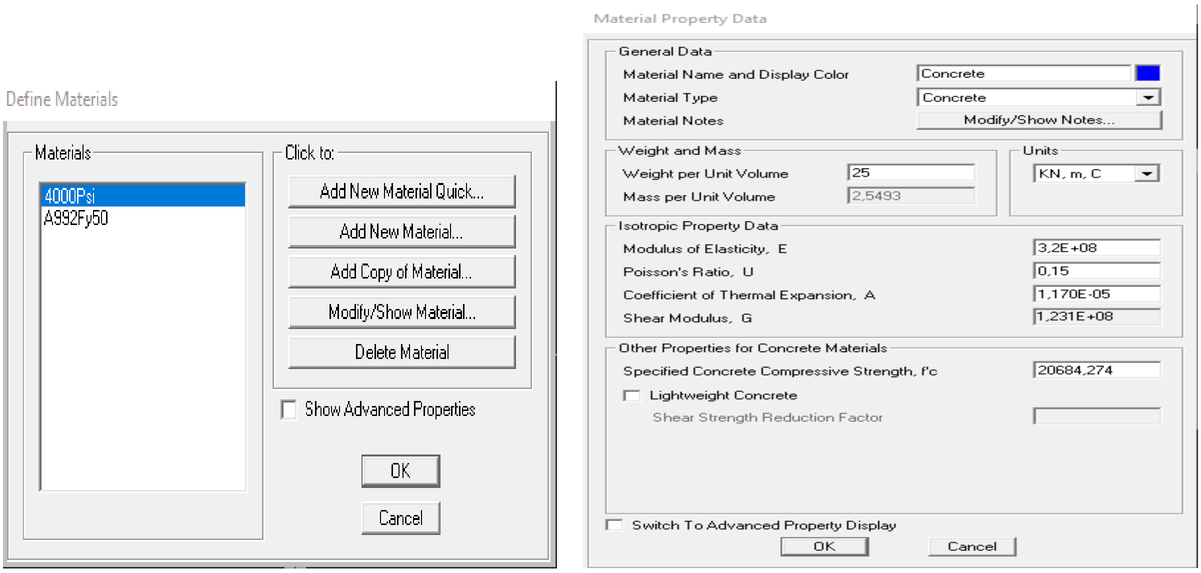


figure 4. 3: windows *displaying concrete property definition*

4.3.2 Defining of steel

To define steel properties, the material A992Fy50 is selected then its initial properties modified to match that of the steel we are using in our project.

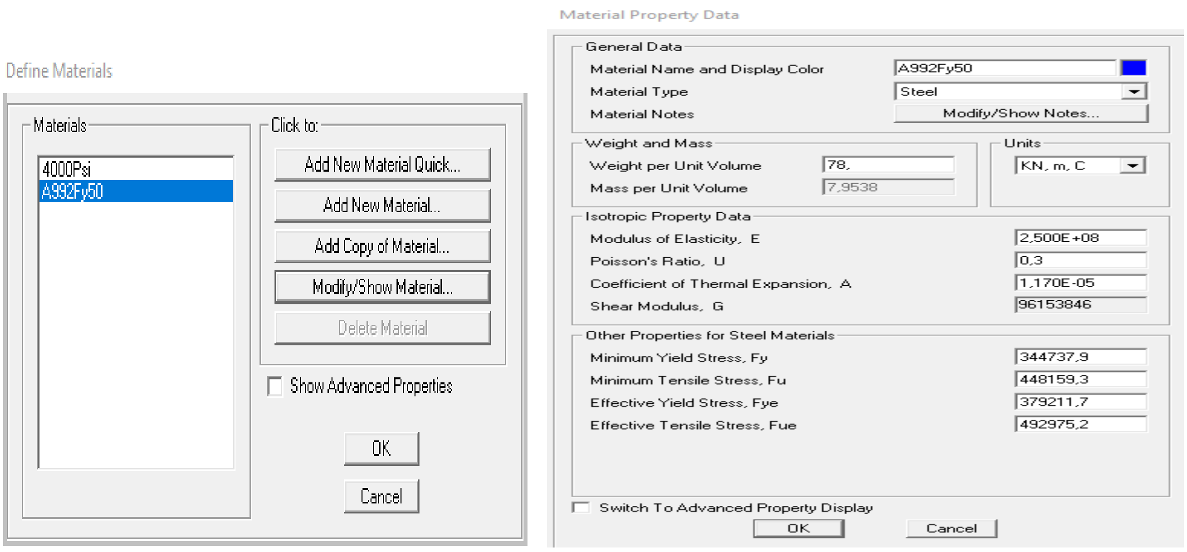


figure 4. 4: windows *displaying steel property definition*

4.4 DEFINING THE BRIDGE ELEMENTS

After defining the materials, the different frames such as the steel beams, columns, Diaphragm, steel beam spacing, sleepers and the rails are also defined from define- section properties-frame sections and a window is displayed with icons to add or modify the different frame sections.

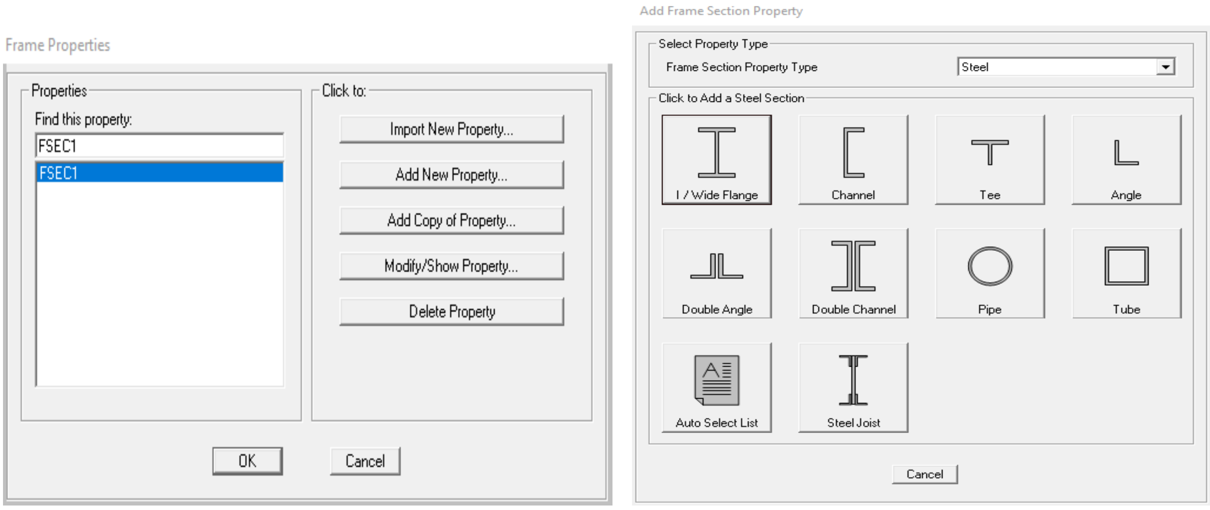


figure 4. 5: windows *from which different elements can be created depending on their material properties*

4.4.1 Step 1: Defining the steel beam section

The beam sections are made of steel materials, for our project the beams are of **I**-section so the I steel section is selected and the dimensions of the beams are entered in the window displayed.

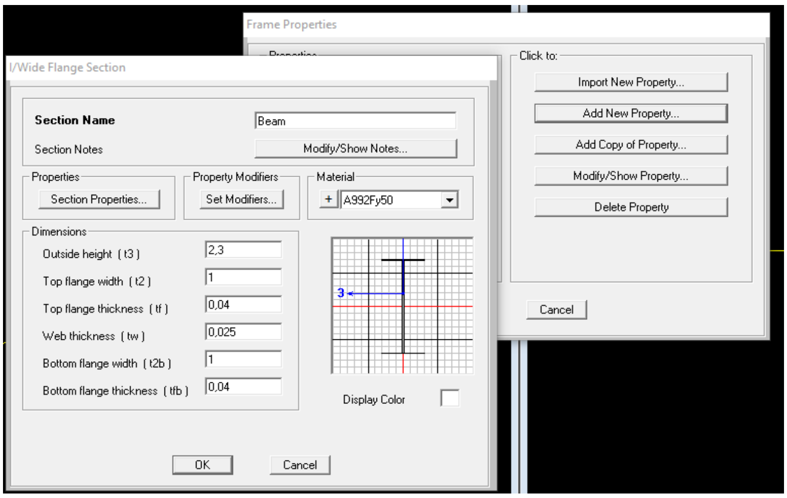


figure 4. 6: *window displaying the definition of the beam section.*

4.4.2 Step 2: Defining the column

The column is of concrete with a rectangular section,

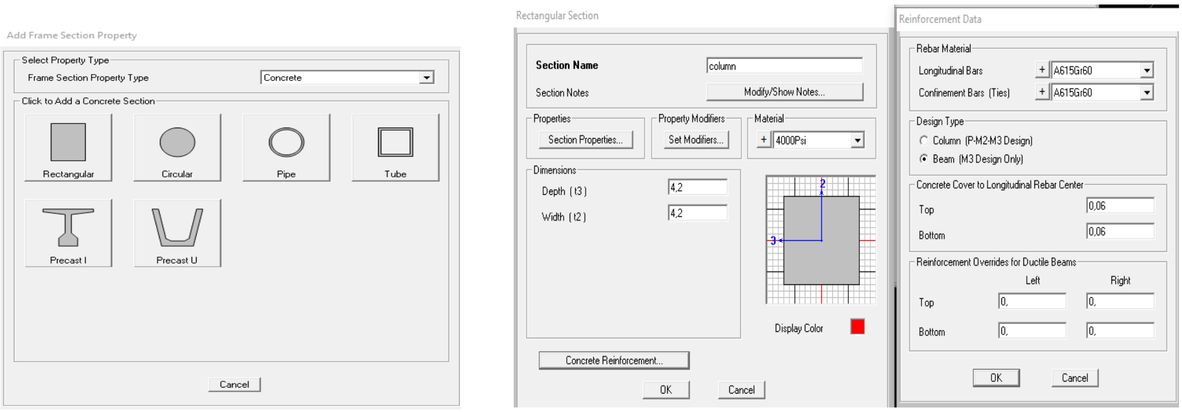


figure 4. 7: windows *from which the column material property(left) and section(right) is defined*

4.4.3 Step 3: Defining the Diaphragm

The diaphragm is equally modelled with concrete and is of a rectangular section.

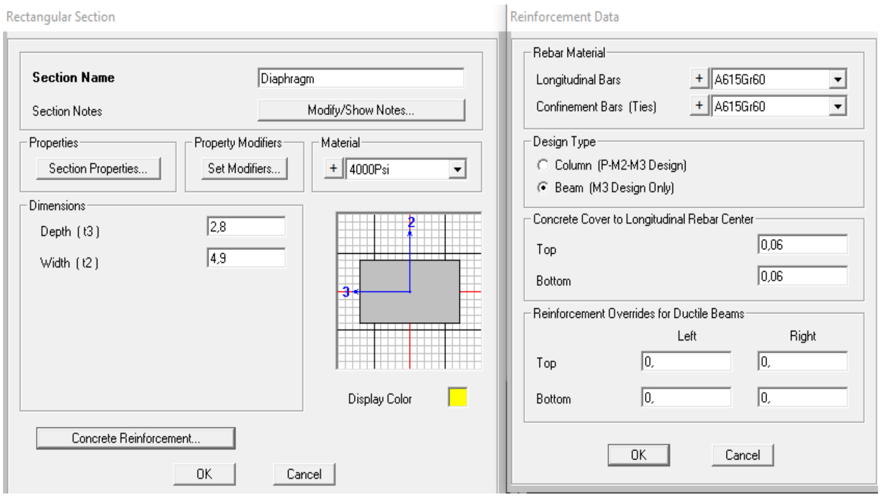


figure 4. 8: windows *displaying the definition of the diaphragm section.*

4.4.4 Step 4: Defining the steel beam spacing

The steel beam spacing material properties are those of the steel we defined in 3.3.2 and it is equally in I-section

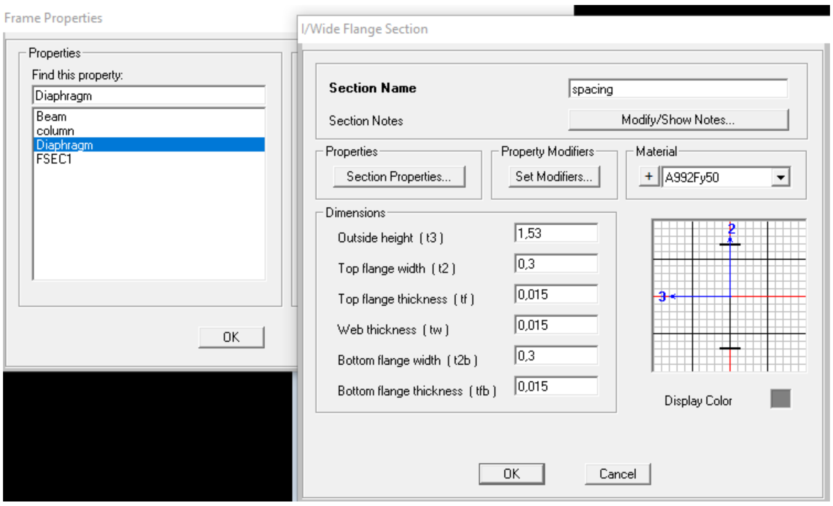


figure 4. 9: **definition *of dimensions of the steel beam spacing***

4.4.5 Step 5: Defining the slab section.

We create the section of the bridge slab, by going to define-section properties-area sections-add new section, shell thick is the slab type chosen. The slab section is in concrete with dimensions as shown as below.

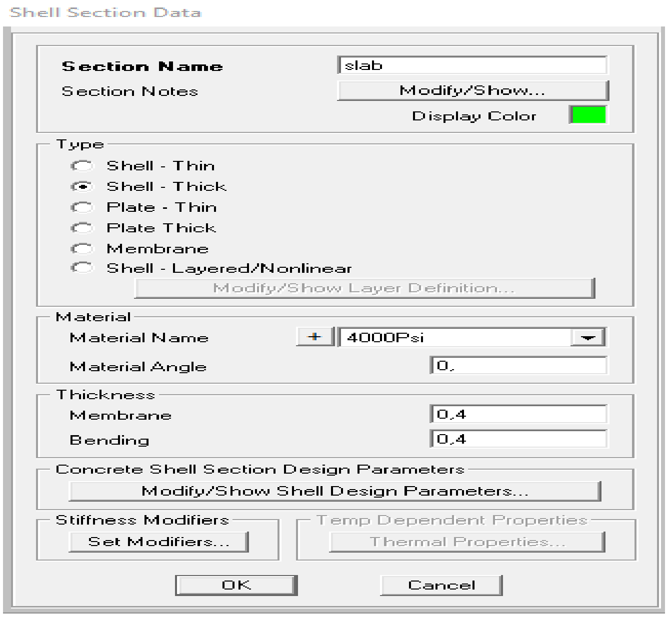


figure 4. 10: **definition *of the slab dimensions***

4.4.6 Step 6: Defining the sleepers.

The sleepers are made of concrete and are in rectangular sections, their dimensions are defined in fig 4.11.

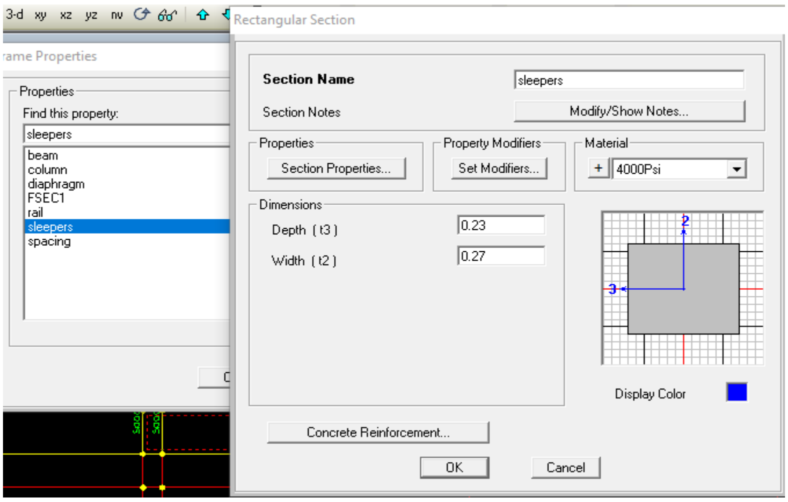


figure 4. 11: **definition *of sleepers***

4.5 DEFINING THE BALLAST

Using different values of elastic modulus of the ballast (E_b), different values of ballast stiffness are calculated as illustrated below. In addition, different values of ballast thickness are put into consideration in order to evaluate effectively how the variation in its stiffness and thickness affect the resistance and stability of the railway track.

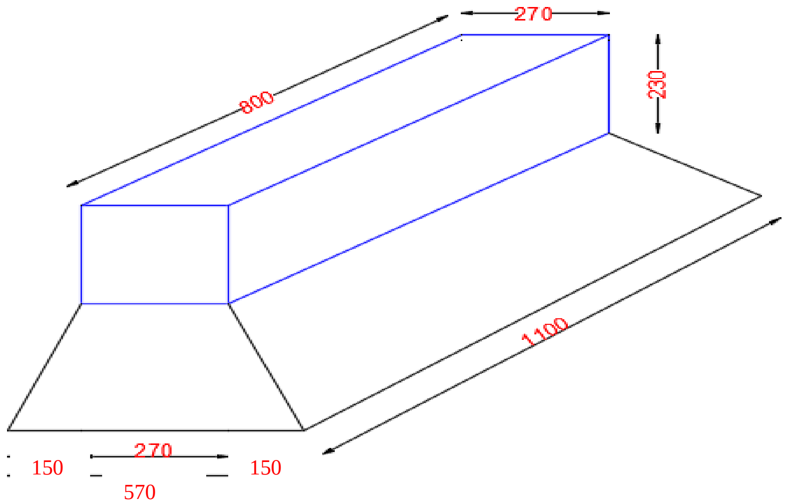


figure 4. 12: dimensions *of the sleeper and that of ballast*

Ballast stiffness is defined from define-section properties-link/support properties-add new property (ballast) from the window given we consider all directional properties fixed and from the section where we have 'modify/show for all', different values of ballast stiffness are considered for the U_1(x-axis) and the ones in the U_2 (y-axis) and U_3(z-axis) are considered infinitely rigid(1×10^{11}KN/m).

To calculate the ballast rigidity, the formula below was used with a varying elastic modulus **(E_b)** of (100MPa,200MPa and 400MPa) and **Amoy** representing the average surface area between the ballast and the sleeper.

$$kb = \frac{EbxAmoy}{L} \qquad (1)$$

With:

K_b=Rigidity of the ballast calculated
E_b=Elastic modulus of the ballast
Amoy=average area between the ballast and the sleeper
L=represents the respective height of the ballast

The average area was calculated from:

A-sleeper=**0,27x0,8=0,216m²**

$$Amoy = \frac{0{,}216+0{,}627}{2} = 0{,}4215m2$$

A-ballast=**0,57x1,1=0,627m²**

With varying difference in L (height of the ballast) for this project, we obtain different **K_b** values which then we used for the execution.

The table below shows the different ballast rigidities obtained when formula (1) is applied;

Eb (MPa)/L(cm)	100	200	400
	Different values of Kb (KN/m) obtained		
30	140500	281000	562000
40	127500	255000	510000

table 4. 1: Different *values of ballast rigidities calculated from their respectful elastic modulus.*

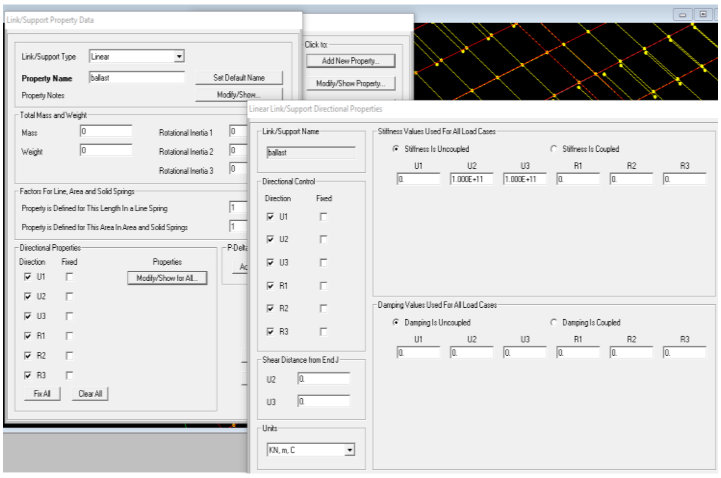


figure 4. 13: definition *of ballast stiffness*

4.6 DEFINING THE RAIL

The rail is designed and adjusted in dimension to resemble that of the standard UIC60 that is used in this project therefore from define-section properties-frame section-add new property-other which then displays a window that allows you to create your own section.

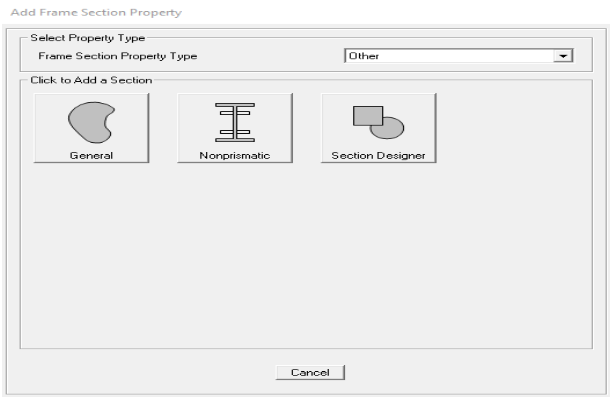


figure 4. 14: window *from which we can create our desired section of the rail*

From the window above, we select section designer option which displays a window as shown below from which the section name is entered and the section material property specified.

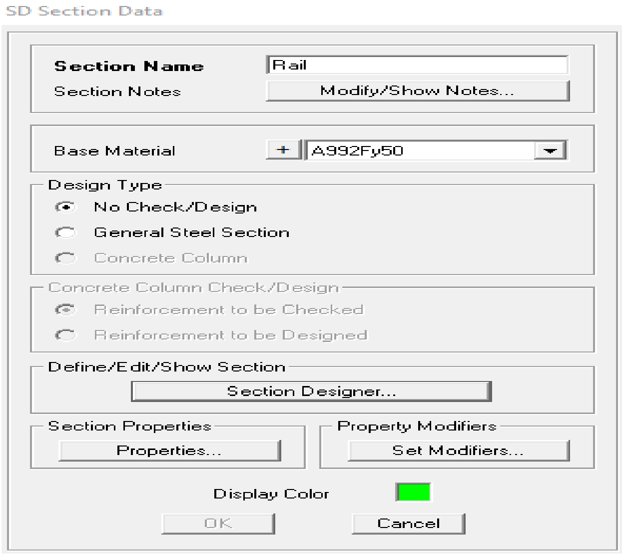


figure 4. 15: Defining *of the section rail and selecting the section property*

Using the draw structural shape icon in I form; we click on it and then transfer the pointer to the centre of our drawing space. A frame section in I form is given, right clicking on this section, a window is given to us which we use to modify the sections of our rail.

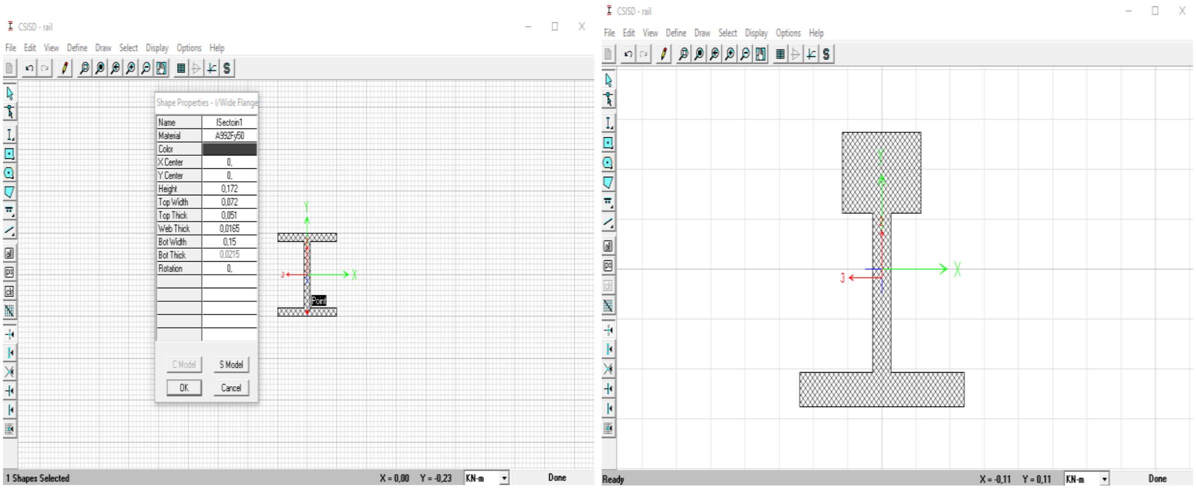


figure 4. 16: window *from which the rail section is drawn as desired*

We go on to replace the initial restraints from joints with nodes and then the frame section is selected and assigned as beam from assign-frame-frame sections-beam.

The section is selected and then replicated 3 times by clicking “ctrl +R” following the d_y direction with a spacing of 2.85m.

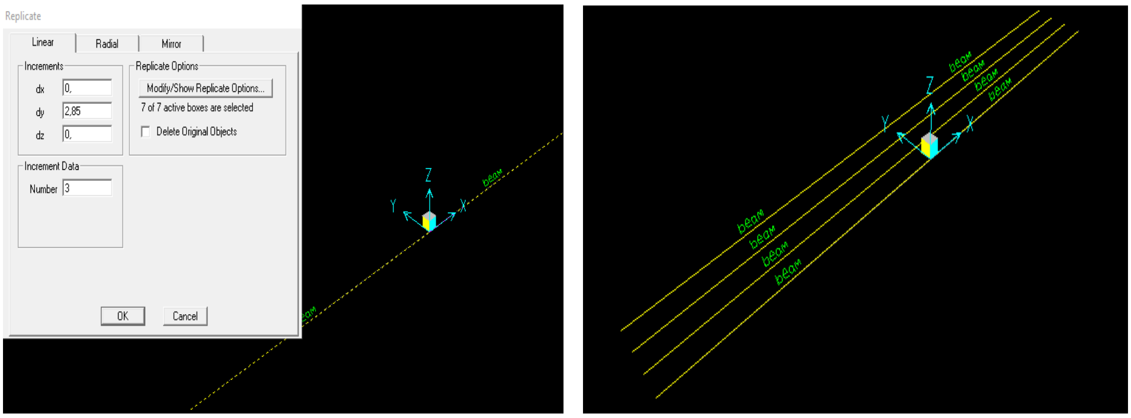


figure 4. 17: replication *of the beams*

To draw the spacing, we go to draw-draw frame/cable/tendon, a window is given from which we select the spacing section. The pointer is given which we use to draw the sections at the extreme perpendicular to the beams.

We select the sections of the spacing and then press "ctrl + R" to replicate them 8 times with a spacing distance of 10m.

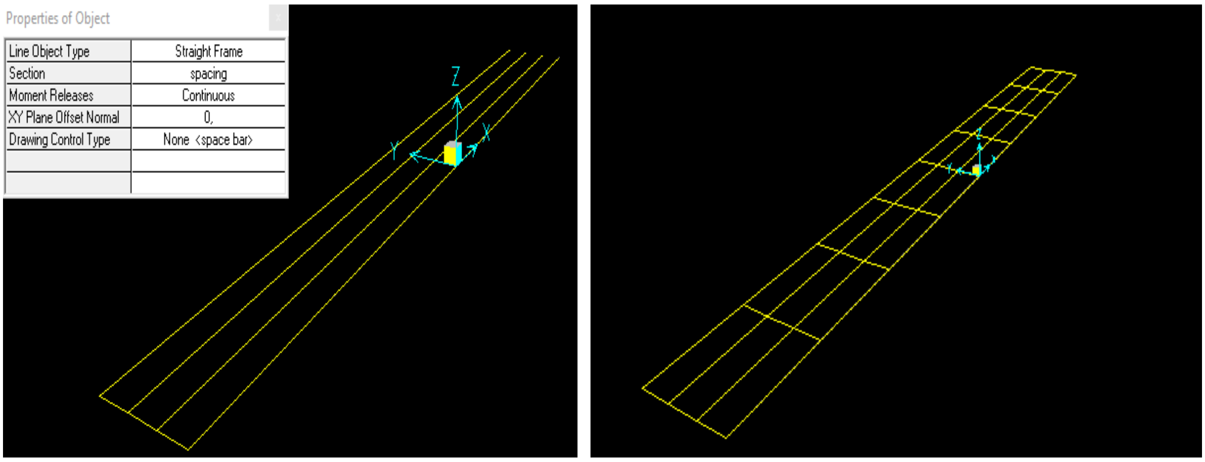


figure 4. 18: creation *of the steel spacing*

To draw the slab section, the nodes on the extreme beams are selected and replicated at a distance of 2.175m to create nodes parallel to the ones selected, then click on the icon of draw rectangular area element on the left side of the window, a "properties of object" window is given from which we select the slab area to be drawn section by section.

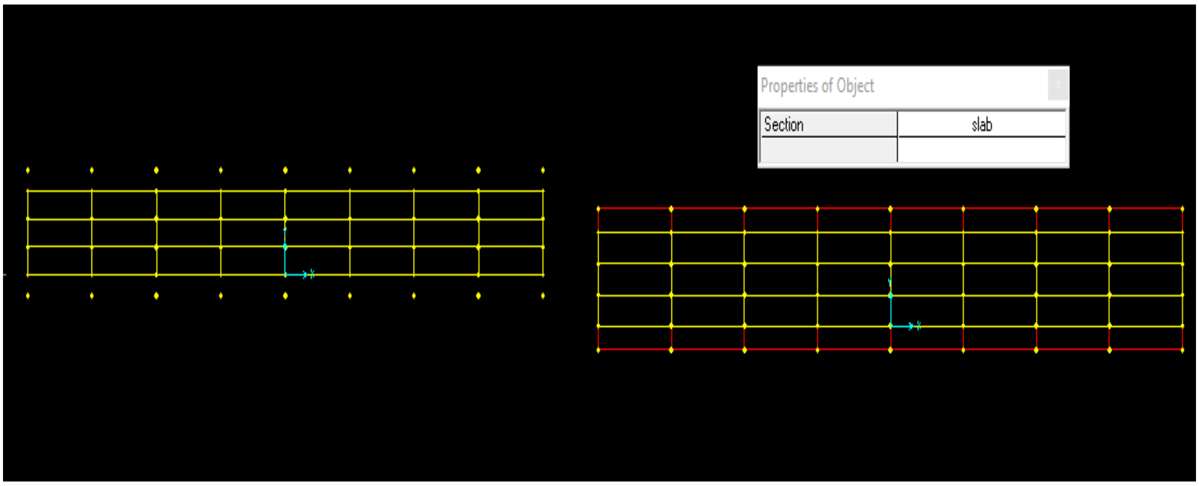


figure 4. 19: creation of the *slab section*

For more precise results, we further divide the area sections into finite elements by selecting the section and then divided into two from edit-edit area-divide area.

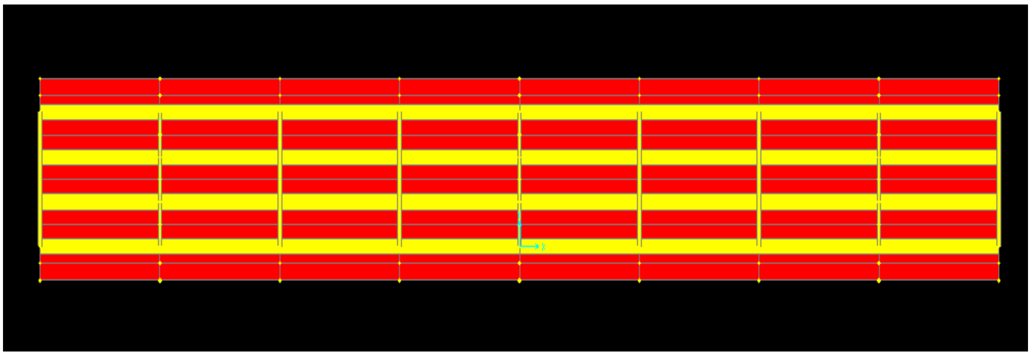


figure 4. 20: dividing *the area sections into smaller sections*

To align the spacing and the top fibre of the beam with the slab section, we select the beams and move them by -1.325m in the d_Z direction, and the steel beam spacing are moved by -0.94m.

Considering the height of the beam, spacing and the thickness of the slab,

$$beam = \frac{Hb}{2} + \frac{Hs}{2} = \frac{2.3}{2} + \frac{0.35}{2} = -1.325m$$

$$spacing = \frac{He}{2} + \frac{Hs}{2} = \frac{1.53}{2} + \frac{0.35}{2} = -0.94m$$

With:

H_b=height of the beam
H_e=height of the spacing
H_s=Thickness of the slab

Before alignment **after alignment**

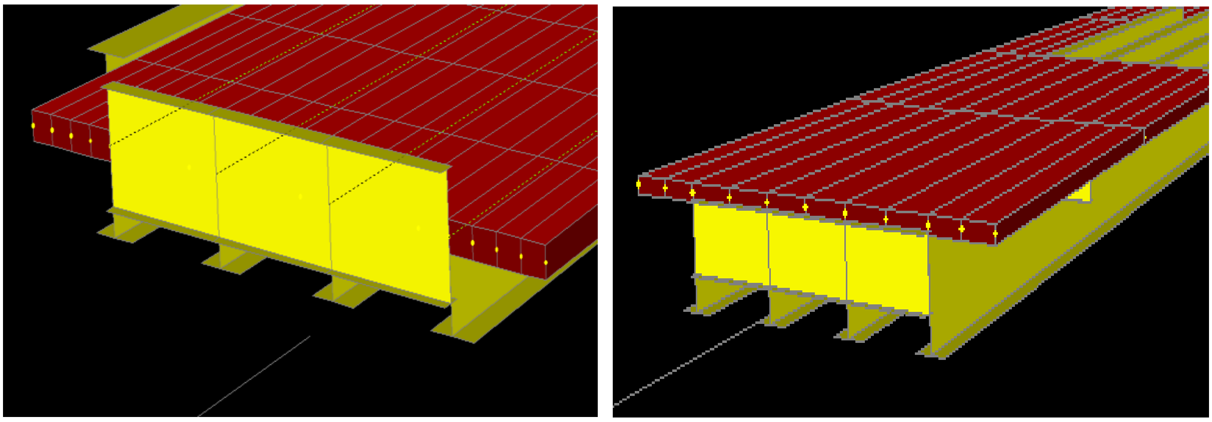


figure 4. 21: alignment *of the steel beams and steel beam spacing with the slab section*

4.7 CREATING THE CONTINUITY AND THE BEARINGS

To ensure the continuity between the superstructures of the bridge and the supports (the abutments and the columns), a link support is defined from define-section properties-link-support properties then “add new property” which was considered infinitely rigid in all directions therefore with stiffness of $K_b=1\times10^{11}$KN/m.

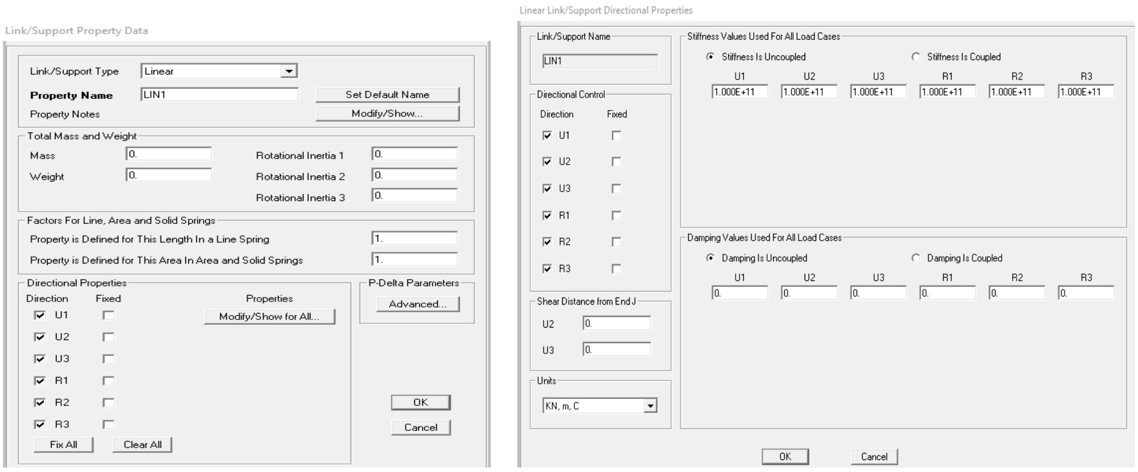


figure 4. 22: Defining the *stiffness of the link supports*

Furthermore, the bearings are linked to the link supports created, its rigidity is defined as infinitively rigid in the x direction ($U_1=1\times10^{11}$KN/m) and a certain degree of stiffness in the y (U_2) and z(U_3) direction of K_b=6300KN/m.

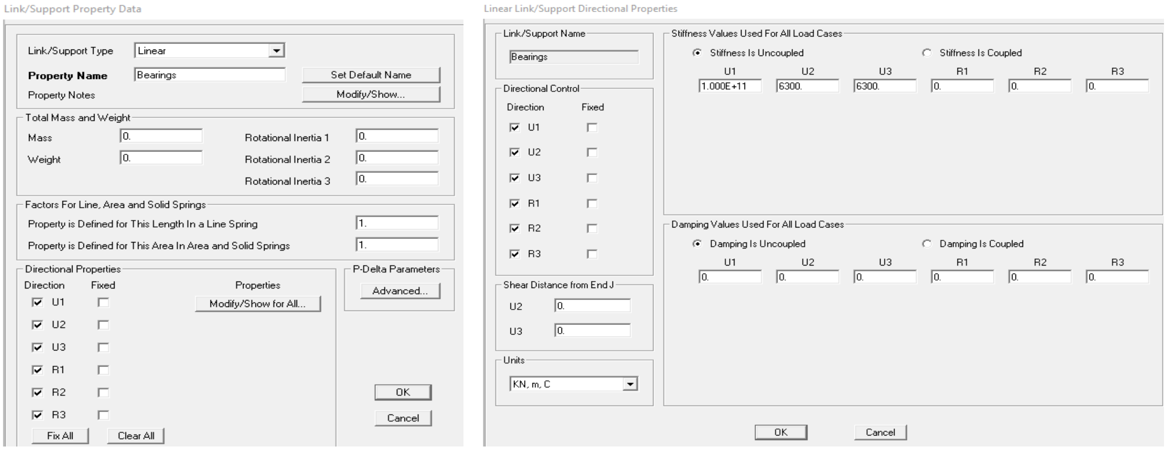


figure 4. 23: definition *of bearing stiffness*

Visualisation of the link supports and the bearings created at the left side of the bridge.

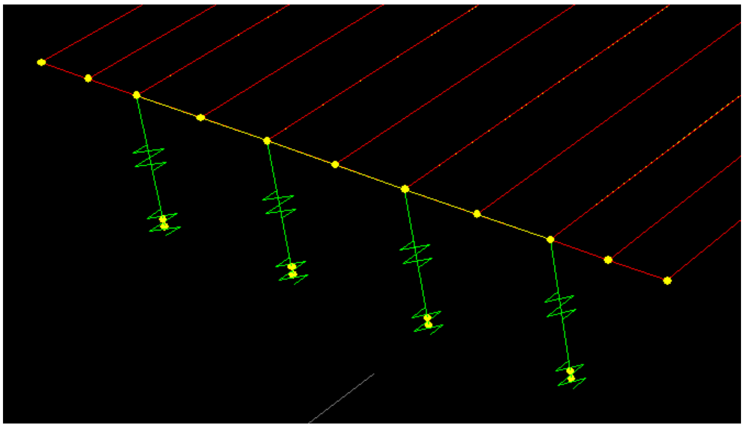


figure 4. 24: visualisation *of the link supports*

4.8 CREATING THE DIAPHRAGM

To create the diaphragm, the link supports (LN1) defined are used to ensure the continuity between the bearings and the diaphragm. A diaphragm of 10.5m long is drawn.

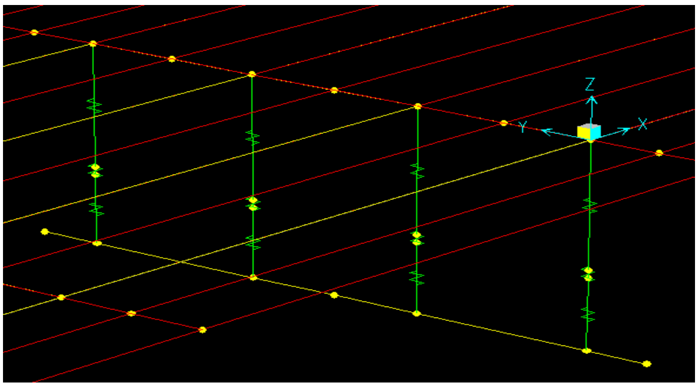


figure 4. 25: creation *of the diaphragm*

4.9 CREATING THE COLUMN

From the mid-section of the diaphragm, a point is created from which the column of height 28.3m is drawn perpendicularly to the diaphragm. At its base its assign with an embedded restraint.

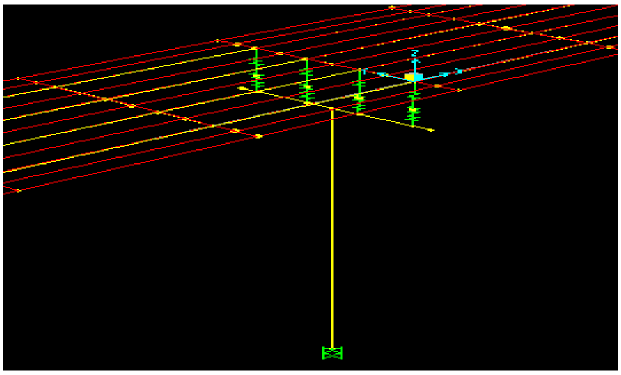


figure 4. 26: visualisation *of the column created*

4.10 CREATING THE RAILS AND THE SLEEPERS

In order to ensure the continuity of the rails, sleepers and the deck section, the deck section is divided further into smaller sections of 60cm which is the spacing between the sleepers following the longitudinal direction.

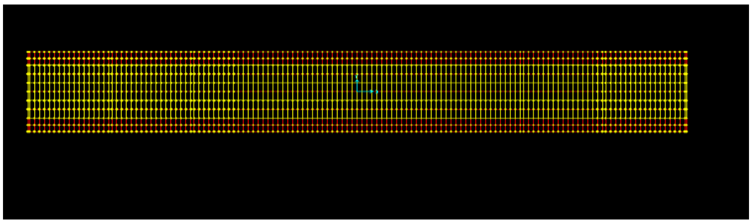


figure 4. 27: creation *of nodes of 60cm apart from each other on the area section*

To create the rails and the sleepers, ballast inclusive the nodes created on the deck section are used as reference to create the nodes that will facilitate the creation of the rails and the sleepers.

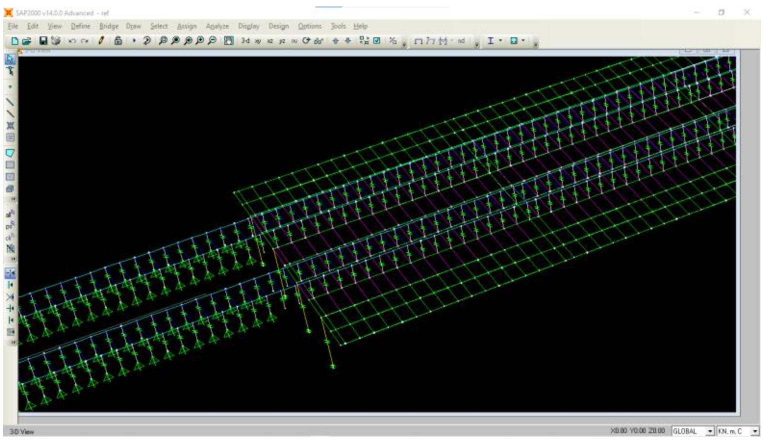


figure 4. 28: creation *of the sleepers, rails, and ballast*

From our illustration above, 20m of length of the railway was extended on both sides at the end of the bridge to ensure continuity of the railway line.

4.11 CREATING THE LANES

Two lanes are defined from Define-bridge loads- lanes-add new lanes defined from frames to create the railway road on which the train is to pass.

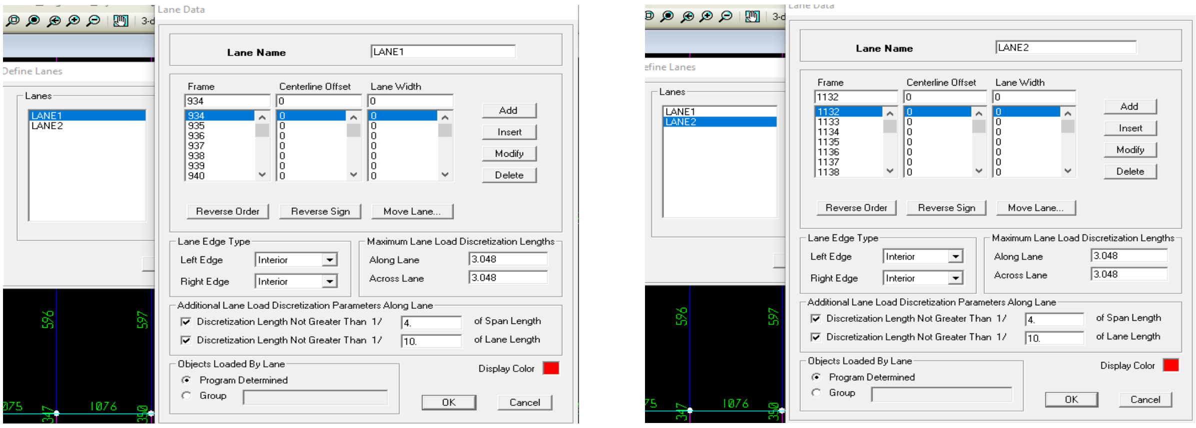
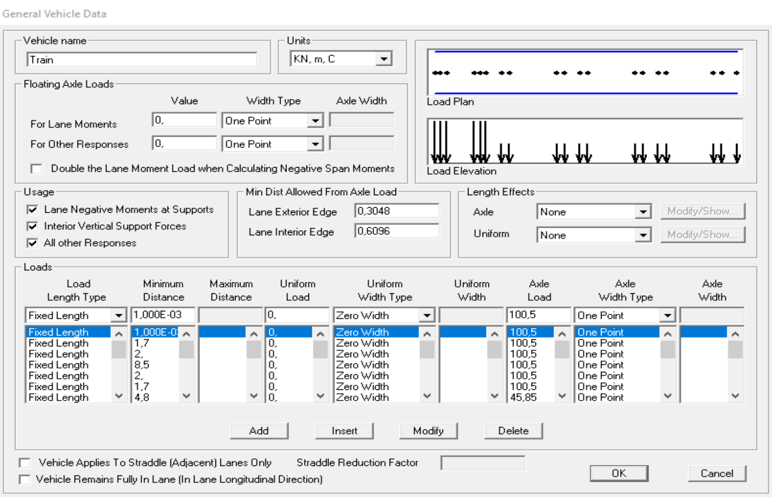


figure 4. 29: windows *visualising creation of lane 1(left) and lane 2(right)*

4.12 CREATING THE TRAIN

figure 4. 30: Maxres ***train***

The train is defined from define-bridge loads–vehicle-add general vehicle-add vehicle, from which a window is displayed and starting with the engine, different loads with their respective spacing are inserted then followed by the wagons of different loads and spacing too.



;

figure 4. 31: creation of *the train*

4.12.1 Defining the class of the Train:

The class of the train is defined from define- bridge loads-vehicle classes-add new class, a vehicle class data window is displayed from which the vehicle name “Train “is selected with a scale factor of 1.

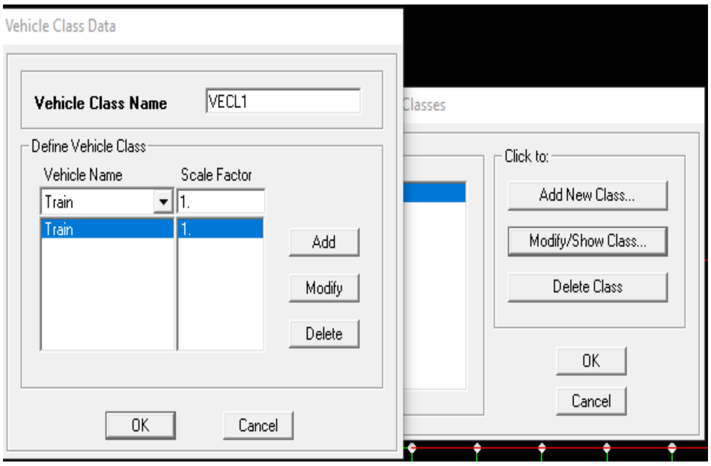


figure 4. 32: definition *of the train class*

4.12.2 Defining the velocity of the train:

The velocity is defined from define-load patterns-load pattern name (Train) and the load pattern type (bridge live) with a self-weight multiplier (0) then modify bridge live load pattern where the train lanes and velocity are assigned.

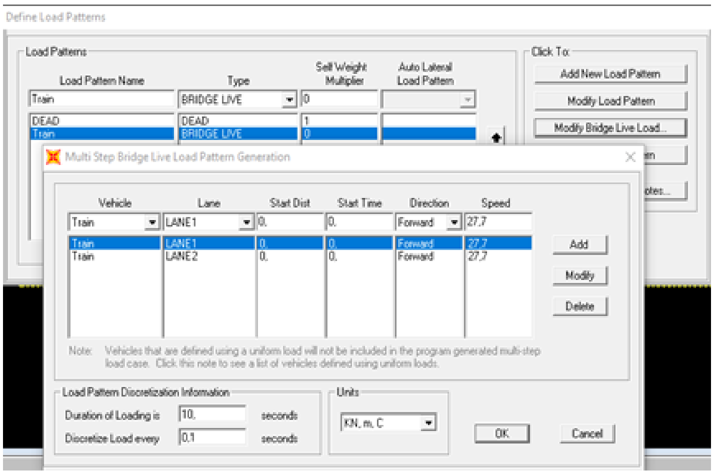


figure 4. 33: definition of *the train speed*

4.12.3 Defining the load case:

Load case is defined from define-load case-modify load from which load type «load pattern “is selected with function “UNIFTH” of scale factor of 1 and the load case type «Time history” is selected then added.

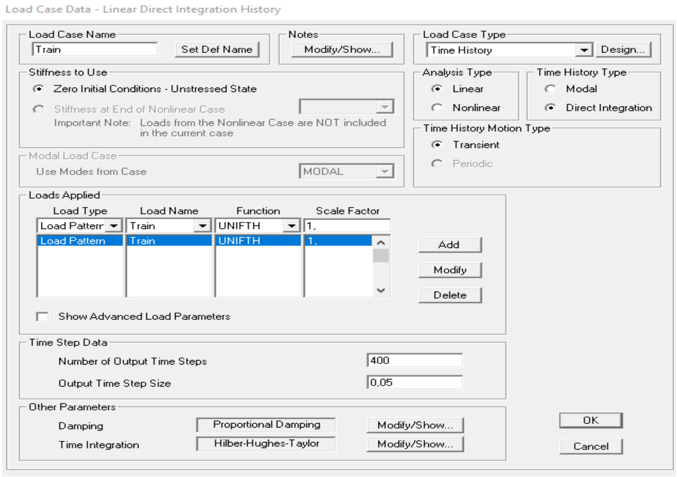


figure 4. 34: definition *of the train load case*

4.13 CONCLUSION

After modelling with the SAP2000 software, the program is run and the numerical results are obtained. The results are then interpreted or used in other projects depending on the individual. Generally, in this project, we kept all defined elements constant and only made changes to the ballast rigidity and the velocity of the train since these two play an important role in the dynamic analysis .

CHAPTER 5: PARAMETRIC ANALYSIS OF THE BALLAST RAILWAY BRIDGE EFFECT

5.1 INTRODUCTION

The railway bridge together with the moving train load were modelled and analyzed in the previous chapter. The analysis was specifically done for the train load without considering permanent and the accidental charges of the bridge. Our main aim is to study the dynamic effect induced by a moving train on the bridge. As we saw in the previous chapter, different cases were studied that is the ballast thickness of 30cm with correspondent ballast rigidities (140500 KN/m, 281000 KN/m,562000 KN/m) of the train moving at different speeds (100km/h and 200km/h) and ballast thickness of 40cm with correspondent ballast rigidities (127500 KN/m,255000 KN/m,510000 KN/m) with train speed of 100km/h and 200km/h. The numerical results obtained from the analysis are summarized in table 5.1.

For this analysis, we follow the variation of certain internal forces and displacement of elements in the studied composite bridge that we consider to be references. Therefore, the displacement at the first mid-span (node 534), the axial force, the shear force and the bending moment at the base of the intermediate pier (frame element 75), the bending moments of the steel edge beam at mid-span (frame element 11) and in support (frame element 12) are some of the internal stresses accounted for.

A summary of the numerical results as shown table 5.1 were obtained after the analysis with train speed (100km/h and 200km/h) with varying ballast rigidities and ballast height. These numerical values were recorded from the graphs obtained from the plot functions display window.

To explain the method of recording the numerical results, the following example shows the results obtained from the SAP2000 software. Considering the train speed of 100km/h, ballast thickness of 30cm and the ballast rigidity of 140500 KN/m, the following results were recorded.

- **Displacement of the node 534:**

	Disp (m)
Min	-5.760×10^{-3}
Max	9.150×10^{-4}

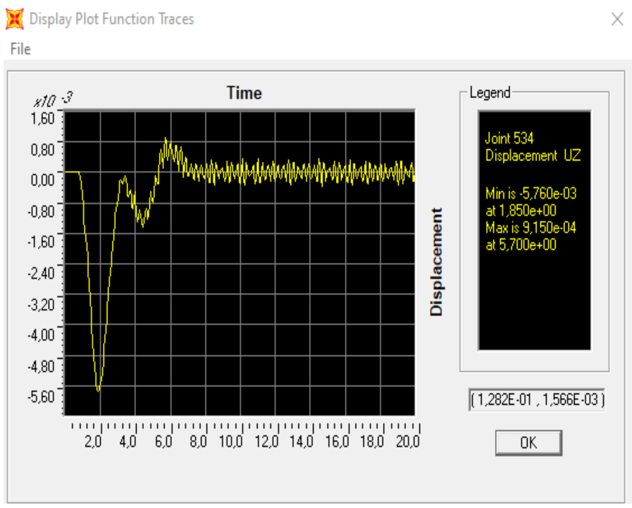


figure 5. 1: graph of *node displacement against time*

- **Axial force of the column 75:**

	N(KN)
Min	-1.480×10^{3}
Max	1.187×10^{2}

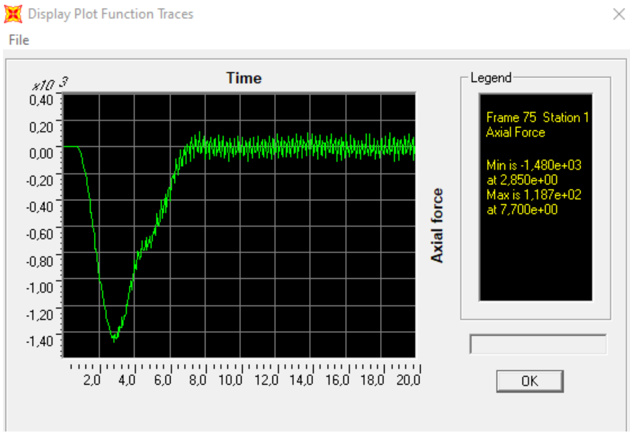


figure 5. 2: graph *of axial force against time*

- **Shear force of the column 75:**

	V_{22} (KN)
Min	-45.46
Max	33.37

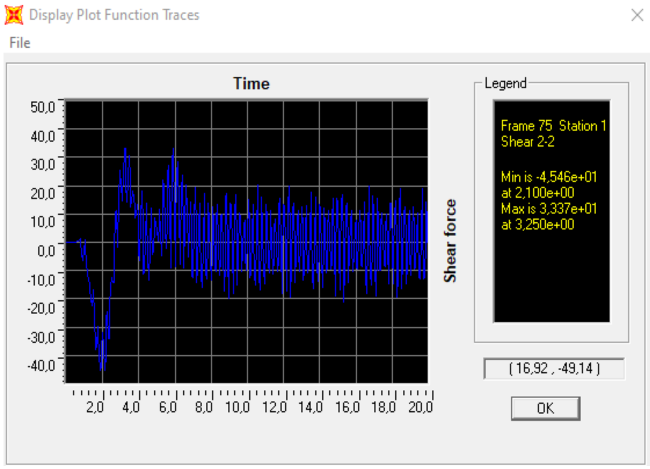


figure 5. 3: graph of shear force *against time*

- **Bending column moment of the 75:**

	M(KN.m)
Min	-1.218×10^{3}
Max	8.187×10^{2}

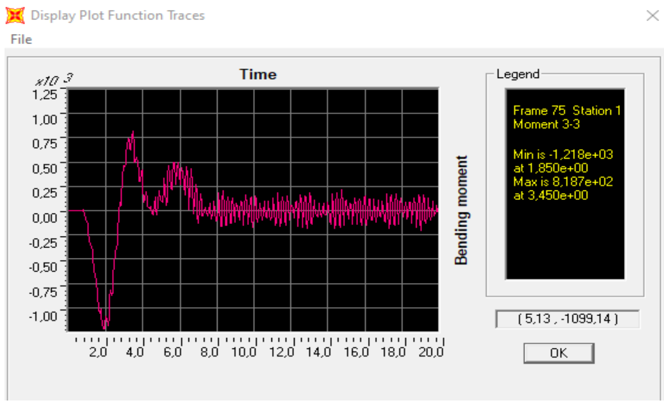


figure 5. 4: graph of bending *moment against time*

- **Bending moment of the central beam 11:**

	M(KN.m)
Min	-1.755×10^{2}
Max	1.310×10^{3}

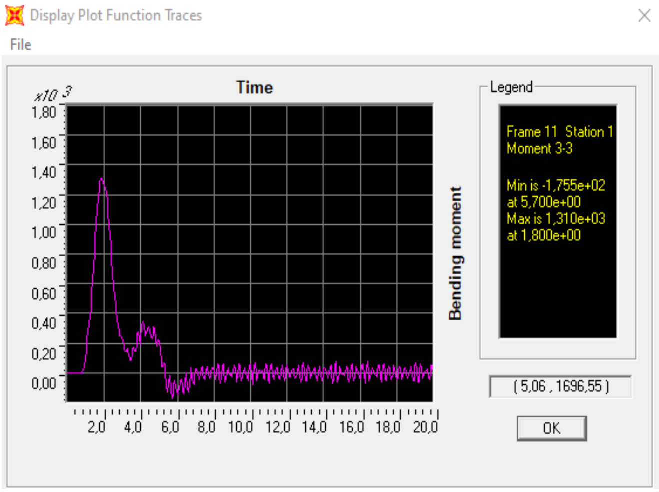


figure 5. 5: graph *of central beam bending moment against time*

- **Bending moment of the joint beam 12:**

	M (KN.m)
Min	$-3.I69\times10^{2}$
Max	7.115×10^{2}

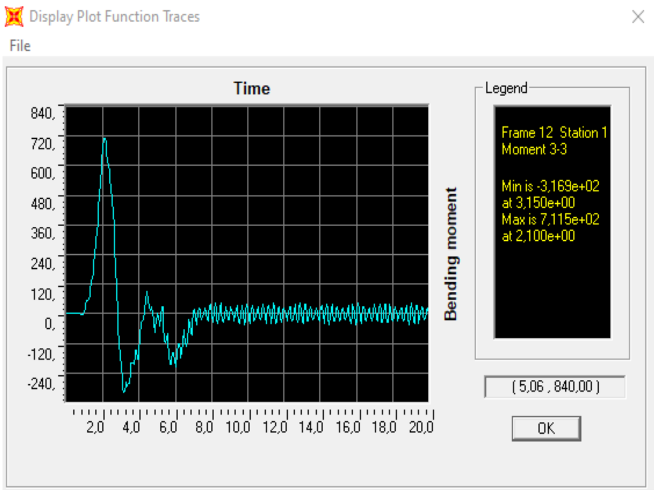


figure 5. 6: graph *of joint beam bending moment against time*

The percentage relative difference between the values recorded while using of infinite ballast rigidity and those of varying ballast rigidity were calculated from the formula as shown below;

$$\textbf{relative difference} = \frac{|\mathbf{R_k} - \mathbf{R_{k=\infty}}|}{|\mathbf{R_{k=\infty}}|}$$

With:

R_k= Value calculated while using the varying ballast rigidity

$R_{k=\infty}$= Value calculated while using the infinite ballast rigidity

Ballast thickness	Train speed (km/h)	$K_{Ballast}$ (kN/m)	Node 534 Displacement (mm)	Column 75 Axial force (kN)	Column 75 Shear force (kN)	Column 75 Basis bending moment (kN.m)	Beam 11 Bending moment (kN.m)	Beam 12 Bending moment (kN.m)
30cm	100 km/h	140500	-5.760/+0.915 (0.35% /0.44%)	-1480/+118.7 (0.06% /0.59%)	-45.46/+33.37 (12.5% /23.4%)	-1218/+818.7 (3.04% /5.19%)	-175.5/+1310 (0.97%/0.07%)	-316.9/+711.5 (0.31%/0.22%)
		281000	-5.752/+0.904 (0.21% /0.77%)	-1481/+118.3 (0% /0.25%)	-43.29/+32.55 (7.17%/20.4%)	-1204/+786.9 (1.86% /1.10%)	-172.6/+1310 (0.69%/0.07%)	-317.1/+711.8 (0.37%/0.18%)
		562000	-5.747/+0.902 (0.12% /0.98%)	-1481/+118.0 (0% /0%)	-41.88/+31.71 (3.68%/17.2%)	-1195/+784.8 (1.09% /0.83%)	-171.9/+1310 (1.09%/0.07%)	-316.8/+712.7 (0.28%/0.05%)
		1.10^{+11}	-5.740/+0.911	-1481/+118.0	-40.39/+27.04	-1182/+778.3	-173.8/+1309	-315.9/+713.1
	200 km/h	140500	-5.935/+0.870 (0.2% / 12.7%)	-1477/+171.6 (0.4% / 0.11%)	-45.50/+28.67 (0.3% / 2.6%)	-1340/+777.1 (6.2% / 2.3%)	-172.1/+1344 (20% / 0.2%)	-360.6/+684.3 (1.2% / 1.0%)
		281000	-5.930/+0.809 (0.13% /4.79%)	-1480/+170.5 (0.20% /0.52%)	-42.05/+27.17 (4.16% /2.68%)	-1303/+777.3 (3.24% /2.33%)	-158.4/+1343 (10.5% /0.14%)	-358.0/+687.6 (0.56% /0.56%)
		562000	-5.927/+0.784 (0.08% /1.55%)	-1482/+168.5 (0.06% /1.69%)	-40.13/+27.16 (0.59% /2.72%)	-1283/+780.7 (1.66% /1.90%)	-151.0/+1342 (5.37% /0.07%)	-357.0/+683.4 (0.22% /1.17%)
		1.10^{+11}	-5.922/+0.772	-1483/+171.4	-40.37/+27.92	-1262/+795.9	-143.3/+1341	-356.2/+691.5
40cm	100 km/h	127500	-5.749/+1.048 (0.07% /1.68%)	-1479/+117.1 (0.14% /3.62%)	-42.12/+34.90 (2.70% /8.45%)	-1133/+938.7 (2.71% /8.43%)	-220.2/+1304 (7.31% /0%)	-342.9/+713.1 (1.38% /0.48%)
		255000	-5.751/+1.055 (0.03% /1.03%)	-1480/+115.4 (0.07% /2.12%)	-41.39/+33.76 (0.92% /4.90%)	-1113/+908.2 (0.91% /4.91%)	-230.3/+1304 (12.2% /0%)	-345.0/+714.7 (0.77% /0.26%)
		510000	-5.752/+1.060 (0.02% /0.56%)	-1481/+114.1 (0% /0.97%)	-41.09/+33.06 (0.19% /2.73%)	-1105/+889.4 (0.18% /2.73%)	-204.2/+1305 (0.48% /0.07%)	-346.3/+715.6 (0.40% /0.14%)
		1.10^{+11}	-5.753/+1.066	-1481/+113.0	-41.01/+32.18	-1103/+865.7	-205.2/+1304	-347.7/+716.6
	200 km/h	127500	-5.925/+0.703 (0.49% /0.42%)	-1479/+167.0 (0.14%/1.53%)	-48.25/+31.29 (1.85% /5.78%)	-1298/+841.8 (1.88% /5.78%)	-138.0/+1342 (0.14% /0.52%)	-343.8/+680.6 (0.29% /1.41%)
		255000	-5.913/+0.7103 (0.28% /0.40%)	-1479/+167.0 (0.14% /1.53%)	-47.87/+30.75 (1.05% /3.95%)	-1288/+827.2 (1.09% /3.94%)	-139.1/+1339 (0.65% /0.3%)	-343.4/+685.3 (0.17% /0.73%)
		510000	-5.906/+0.710 (0.17% /0.56%)	-1480/+164.4 (0.07% /3.06%)	-47.65/+30.30 (0.59% /2.43%)	-1282/+815.1 (0.62% /2.42%)	-139.0/+1338 (0.57% /0.22%)	-343.1/+685.3 (0.08% /0.73%)
		1.10^{+11}	-5.896/+0.706	-1481/+169.6	-47.37/+29.58	-1274/+795.8	-138.2/+1335	-342.8/+690.4

table 5. 1: Summary *of numerical results.*

The table 5.1 summarizes the numerical results recorded from the SAP2000 Software after the analysis. With this information, we are able to reason and interpret critically the dynamic behavior of the railway bridge when the Train moving load is applied to specified different parts of the railway bridge such as Node 534 for displacement(mm), column 75 for the axial force (KN), shear force (KN) and the bending moment (KN.m), the beams 11 (central beam) and 12(nodal beam) for bending moment were considered to understand how they are affected by the dynamic vibrations and energy. The highlighted values in yellow indicate values that are significant (relative difference above 5%) and have a significant impact on the railway bridge behavior therefore visualized closely.

5.2 DISPLACEMENT AT NODE 534 (GIRDER MID-SPAN):

Generally, the absolute differences between the values of displacement at node 534 in both the minimum and maximum values and the values of the infinite rigidity ($1x10^{11}$) are negligible. Although, for ballast height 30cm and ballast rigidity 140500KNm with speed of 200km/h, it shows a crucial relative difference value of 12.7% which actually is negligible due to the fact that the displacement of 0,870mm is minimal to cause a regrettable impact on the railway bridge. When observed closely, the variation of speed, ballast rigidity and ballast thickness have no major impact on the displacement of node 534.

5.3 AXIAL FORCE OF COLUMN 75 (BASE OF THE INTERMEDIATE PIER):

The minimum and maximum absolute values of axial force for the two train speeds show no much difference even when varying the ballast rigidities and ballast thickness with respect to those of the infinite ballast rigidity. The relative difference calculated were negligible to cause any significant effect on the railway bridge. Therefore, the variation of the train speed, ballast rigidity and ballast thickness show no much effect on the variation of the axial force of the column 75 (Fig 5.1).
The graph below shows how the axial force for speed 100km/h and 200km/h with ballast thickness of 30cm and 40cm behaves with respect to the varying ballast rigidities. we realize that the maximum absolute values of the axial force increase slightly with increase in train speed but remains constant for the same speed.

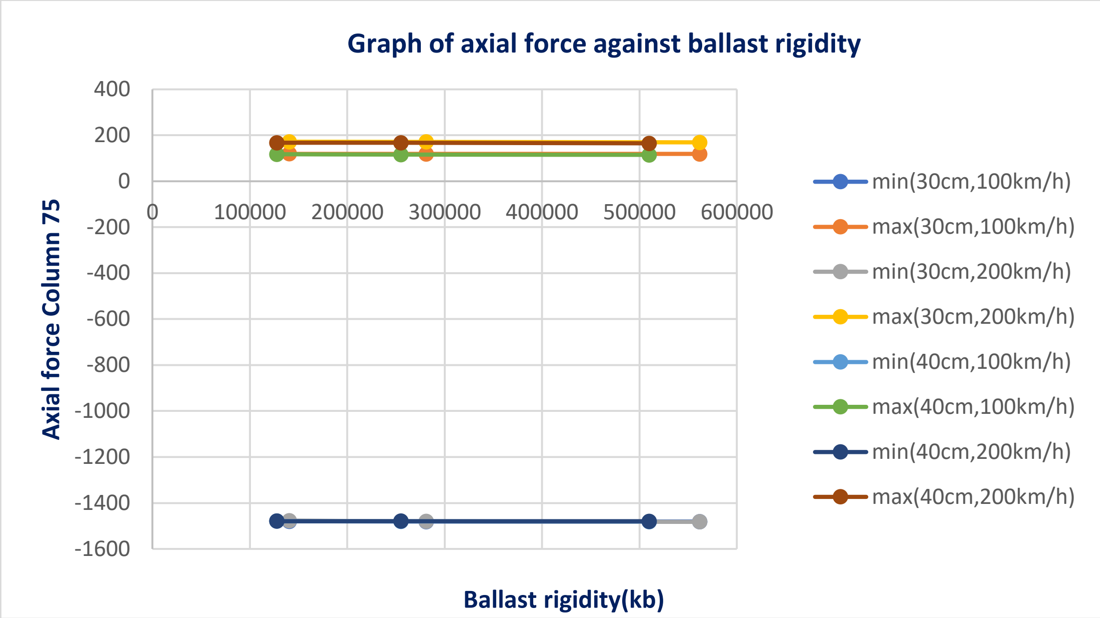


figure 5. 7: Variation *of the Axial force and the ballast rigidity*

5.4 SHEAR FORCE OF COLUMN 75 (BASE OF THE INTERMEDIATE PIER):

The shear force shows significant non-negligible values for both absolute and relative differences in regard to the values of infinite ballast rigidity. Increase in ballast rigidity leads to a decrease of shear force and the values in the table 5.1 show clearly that the change in speed and ballast thickness play an important role in the variation of shear force. Significant values for the absolute and relative differences (+23.4%) are more experienced in the first phase of the experiment with train speed 100km/h and ballast thickness of 30cm.Further details can be seen from the fig 5.8 showing how the increase in speed for a particular ballast thickness leads to a decrease in shear force though this effect is very much experienced in ballast thickness 30cm and train speed 100km/h.

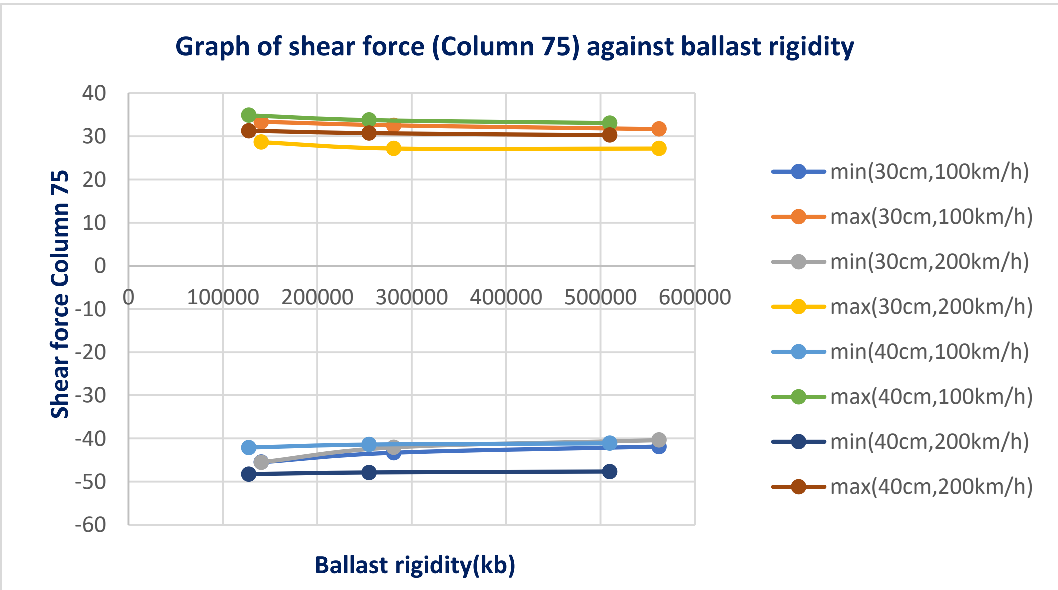


figure 5. 8: Variation of the *shear force and the ballast rigidity*

5.5 BENDING MOMENT OF COLUMN 75 (BASE OF THE INTERMEDIATE PIER):

For bending moment of column 75, the difference between the absolute values obtained is negligible except for ballast thickness of 30cm with ballast rigidity 140500KN/m of speed 200km/h and ballast thickness of 40cm with ballast rigidity 127500KN/m of speed 100km/h and 200km/h that show a significant difference of 8.43% in the value of the bending moment to that of the infinite rigidity, these values play a crucial role in the instability of this column 75. We notice that the values of bending moment increase with decrease in ballast rigidity and increase in speed as shown in table 5.9 Therefore, these factors have to be put into consideration while modelling.

The table 5.1 visualizes further how the bending moment varies with relation to ballast rigidity at variable train velocities (100km/h and 200km/h) and different ballast thicknesses. The difference between the curves min/max(40cm,100km/h) and min/max(40cm,200km/h) are far apart from each other compared to those of min/max(30cm,100km/h) and min/max(30cm,200km/h) respectively, this visualizes that increase in ballast thickness increases the rate at which the bending moment of the column increases.

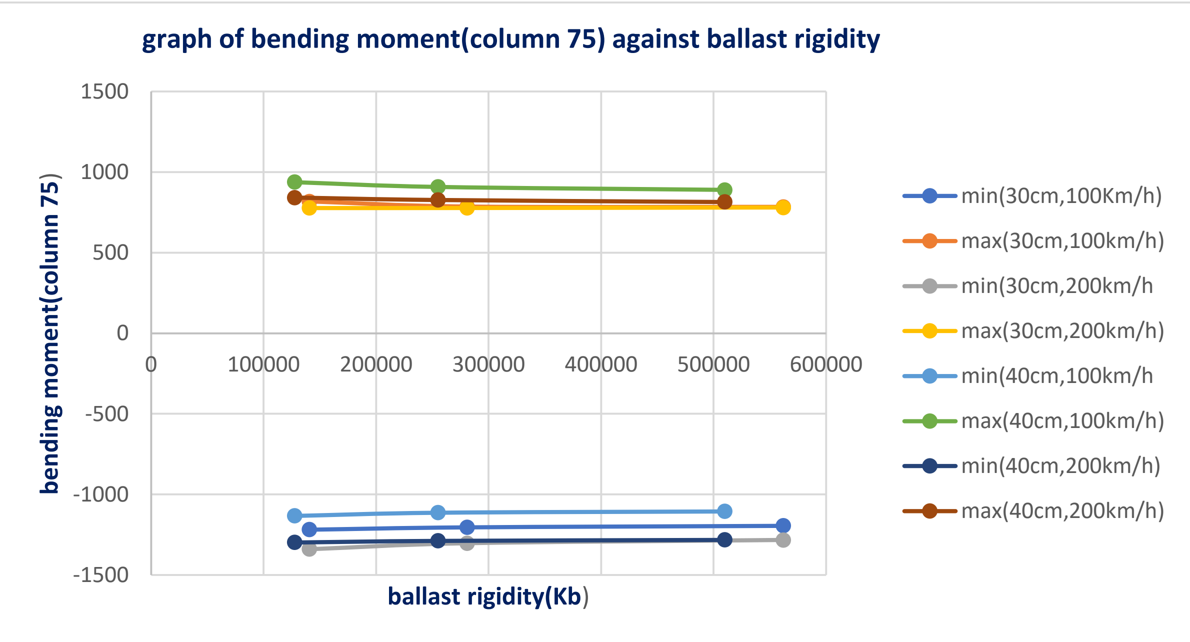


figure 5. 9: Variation of *the bending moment of column 75 and ballast rigidity (Kb)*

5.6 BENDING MOMENT OF BEAM 11 (MID-SPAN STEEL BEAM):

The difference between the absolute values of bending moment of the rigidities and that of the infinite ballast rigidity for ballast thickness of 30cm, speed of 100km/h and that of ballast thickness 40cm, speed of 200km/h are relatively negligible whereas those of ballast thickness 30cm, speed of 200km/h and ballast thickness 40cm, speed of 100km/h are non-negligible. The relative difference of the minimum values of the bending moment are significant while the relative difference of the maximum values is negligible. This implies that the inferior part of beam 11 (mid-span steel beam) is mostly affected by the bending moment induced by the moving train load.

We further visualize the variation of the bending moment on the central beam graphically as represented in fig 5.10, it is visualized that increase of the train speed for same ballast rigidities and thickness decreases the bending moment exerted on the central beam, this explains that the faster the train is the less time it takes to impose the charge on an element thus reduced charge effect.

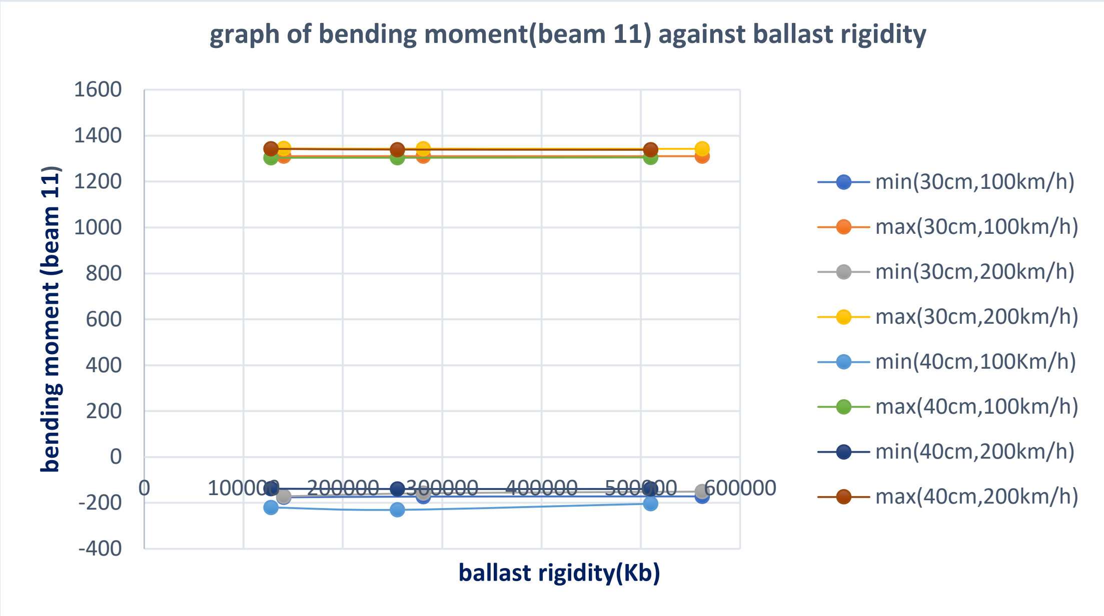


figure 5. 10: Variation *of bending moment of mid-span steel beam and ballast rigidity*

5.7 BENDING MOMENT OF BEAM 12 (STEEL BEAM AT SUPPORT):

For bending moment of beam 12, the difference in the values of the moment of the varying ballast rigidities with those of the infinite ballast rigidity is relatively negligible, from table 5.1, it is visualized that the bending moment of this beam increases with increase of the train velocity. It is equally visualized that increase of ballast thickness increases the effect of the bending moment on the beam, since increase in the ballast thickness increase the amount of charge imposed on the beam thus increasing the effect of the bending moment.

5.8 CONCLUSION

The main aim of this chapter is to help us identify how the displacement, axial force, shear force, and the bending moment at different bridge elements are affected by the dynamic vibrations and energy produced by the moving train load. With this information, we are able to cite out and focus on specifically which forces or moments are the most crucial and require maximum attention so as to mitigate and prevent the immediate problems that may occur. According to our results, the shear force and the bending moment in the base of pier and the bending moment of the steel beam are the elements mainly affected by the dynamic vibrations created by the moving train load of which their intensities increase to more than 20% compared to the values of the infinite rigid ballast. The other vectors such as the displacement, axial force at the pier and the bending moment of the steel beam at support which is the beam located hinge joint of the column show no much significant effects when the moving train load is applied and therefore their values are really minimal and negligible compared to the values obtained for the other elements.

Therefore, it is important to take into account seriously the rigidity of the ballast in the analysis and design of a railway bridge. During the design of the railway bridge, the ballast can be seen as a secondary element, but its effect is not negligible to some structural elements and to the variation of the internal forces. We observed that the train speed has also a perceptible influence to the bridge behavior in concordance to the rigidity of ballast. Eventually, a detailed study can be carried out to assess its effects in the future study project.

CHAPTER 6: **GENERAL CONCLUSION**

The application of the finite element method (FEM) has been presented for the analysis of the railway composite bridge. It's a computational technique used to obtain approximate numerical results to those obtained from the practical field experiments.

The FEM procedure permits the continuum to be discretized into a finite number of parts (or elements) and emphasizes that the characteristics of the continuous domain may be approximated by assembling the similar properties of the discretized elements per node.

A 3-D computational model was used to better represent the analysed railway composite bridge with a continuous 2-span slab and an intermediate reinforced concrete pier placed on a surface footing. The model created includes a linear-wheel-rail interaction, the ballast structure which later during the analysis its thickness is varied from time to time with variation of train speed and ballast rigidity. The effects of train speed are examined and analysed with varying ballast thickness and stiffness.

The dynamic behaviour of the railway composite bridge subjected to high-speed train traffic, varying thickness and stiffness of the ballast was studied and the conclusions below were outlined:

- Generally, the dynamic responses such as vibrations increase with increase in train speed therefore this explains why train speed is an important parameter that needs accountability so as to mitigate the phenomenon that may occur in case nothing is done about it.
- The dynamic response of the railway composite bridge reaches the greatest frequency peak when speed was increased from 100km/h to 200km/h and possibly we would have higher frequency peaks reached in case the speed was increased further. The magnification depends on the studied response, for our case we specify the displacement, shear force, axial force, bending moment and type of bridge and train models.
- According to the numerical results obtained in table 5.1, we realise that the displacement of the girder mid-span node was partially affected by the vibrations produced due to increase in train speed with the variation of ballast thickness and stiffness. Therefore, indicate that the displacement of most mid-span nodes is not much affected by the vibration frequencies from the moving train load.
- The axial force at the base of the intermediate pier was too partially affected by the vibration from the constant moving train load and therefore don't really show much threat to the stability of the pier. In addition, the bending moment at the base of the intermediate pier showed significant values especially with the lowest ballast rigidities which later reduced with increasing stiffness.
- The shear force at the base of the intermediate pier is the outstanding internal force amongst the other forces with remarkable significant values that is believed if neglected may cause in the course of time a phenomenal that is regrettable to the railway composite bridge.
- The bending moment greatly affected the mid-span steel beam due to increasing speed and variation of ballast stiffness and thickness. Since we considered the train as constant axle moving loads, the value of the bending moment in the direction of gravity is greater than that at the support. Therefore, precautions need to be taken

during the design stage of the railway bridge project to strengthen the mid-span steel beam and prepare them for greater and heavier charges with respective vibration frequencies.

There are still many unresolved problems before the dynamic behaviour can be fully understood although a lot of research topics have been studied. Below are some of the propositions for further study and research about the effects of dynamic vibration.

- Railway track irregularities can be one of the problems that might not have been considered in the course of time yet probably may affect the stability of the railway bridge structure.
- The vibrations caused by moving trains at high speeds can propagate in the slab and have a significant influence on the surrounding area which may affect the way vibrations are transmitted and therefore more studies could be done on them.
- The interaction of the column/pier with the substructure therefore the soil where the footing is implemented when the vibrations are transferred may also be a topic to consider for better understanding of the dynamic effects caused by the moving train loads.

REFERENCES

[1] W.M. Zhai, K.Y. Wang and J.H. Lin,2004, Modelling and experiment of railway ballast vibrations
[2] Britannica, The editor of Encylopaedia,2019, British Railways https://www.britannica.com/topic/british-railway
[3] Lim and Wee L., 2004, Mechanics of railway ballast behavior, PHD thesis, University of Nottingham
[4] Shai L., 2012, Railway Sleeper Modelling with Deterministic and Non-Deterministic Support Conditions, Master thesis, Royal Institute of Technology
[5] Chrismer,1985, Aggregate shape effects on ballast tamping and railroad track lateral stability.
[6] Esveld,2001, Modern Railway Track 2rd edition
[7] Jeffs et al.,1987, Ballast characteristics in the laboratory
[8] Indraratna et al.,1998-Shear behaviour of railway ballast based on large-scale triaxial tests
[9] Tutumluer et al., Aggregate shape effects on ballast tamping and railroad track lateral stability in proceeding of the 2006 AREMA annual conference Louis ville, Kentucky, pp.17-20
[10] Profillidis, 1995, Railway engineering 2nd edition
[11] Watters et al.,1987, Evaluation ballast materials using petrographic criteria
[12] Selig and waters (1994), Track geotechnology and substructure management
[13] Dingqing et al.,2015, Railway Geotechnics
[14] Jeffs T. and Marich S.,1987, Ballast characteristics in the laboratory. Proceedings of a Conference on Railway Engineering, Perth, pp. 141–147.
[15] Indraratna et al.,2000, Experimental and numerical modelling of shear behaviour of rock joints
[16] Ramamurthy 2001, Shear strength response of some geological materials in triaxial compression
[17] Indraratna et al.,1993, Large-scale triaxial testing of grey wacke rockfill
[18] Raymond and Diyaljee, 1979, Railroad ballast load ranking classification
[19] Indraratna et al.,2003b, Deformation and degradation mechanics of recycled ballast stabilised with geosynthetics
[20] Ahlbeck et al.,1978, The development of analytical models for railroad track dynamics
[21] Indraratna and salim,2005, Mechanics of ballasted rail tracks: A geotechnical perspective
[22] L. Björklund,2004, Dynamic Analysis of a Railway Bridge subjected to High-Speed Trains, master thesis, Royal Institute of Technology in Stockholm
[23] R. Ferrara et al., 2012, Numerical Modelling of Train Induced Vibrations, Procedia - Social and Behavioral Sciences, Elsevier, pp.155-165. https://hal.archives-ouvertes.fr/hal-00786945
[24] Xiaoyan L., Shenhua W., and Bin Z., 2016, Dynamic Analysis of the High-Speed Train and Slab Track Nonlinear Coupling System with the Cross Iteration Algorithm, pp. 17. http://dx.doi.org/10.1155/2016/8356160
[25] Nguyen, V., Duhamel, D. and Nedjar, B., 2003, A continuum model for granular materials taking into account the no-tension effect. Mechanics of Materials
[26] Rauert, T. et al., 2010. Prediction of the interaction effect caused by continuous ballast on filler beam railway bridges by experimentally supported numerical studies. Engineering Structures 32, 3981– 3988.

[27] Rikard R. and Daniel W., 2012, The influence of ballast on the vibrations of railway bridges,Master thesis, Royal Institute of Technology

[28] Chiara P., 2010, “Comportement du ballast sous l'action du bourrage et du trafic ferroviaire”, Doctorate thesis, “école polytechnique fédérale de Lausanne”

[29] Zijian Z., 2015, Finite element analysis of railway track under vehicle dynamic impact and longitudinal loads, Master thesis, University of Illinois at Urbana-Champaign

[30] Morteza E. and Ahmad H., 2019, Lateral interaction of pre-stressed concrete sleeper with ballast layer under various impact loads, conference on Railway infrastructure and Transportation, Phoenix Island,Jenju,South Korea

[31] Markine et al., 1998, A procedure for design and optimisation of a railway track structure