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ANALYSIS AND VERIFICATION OF THE BEHAVIOR OF INDUSTRIAL STEEL STRUCTURES WITH AND WITHOUT BRIDGE CRANES.

CASE STUDY: ZIN INDUSTRIES' FACTORY AT MVAN YAOUNDE

Thesis submitted in partial fulfillment of the requirements for the degree of Master of Engineering (MEng) in Civil Engineering

Curriculum: **Structural Engineering**

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DEDICATION

I dedicate this work to my late beloved grandfather

TSAFACK Constant

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This thesis is the result of combined direct and indirect contributions of numerous individuals whose names may not all be mentioned.

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LIST OF ABBREVIATIONS AND SYMBOLS

ABBREVIATIONS

EC	Eurocode
EN	European Norm
RC	Reinforced Concrete
SAP	Structural Analysis Program
SCI	Steel Construction institute
SLS	Serviceability Limit State
ULS	Ultimate Limit State

SYMBOLS

A	Cross-sectional area
A_b	Cross-sectional area of the bolt
A_c	Cross-sectional area of concrete
A_{eff}	Effective cross-section area
ah	Height below roof
A_{net}	Net cross-section area
A_p	Section of the plate
A_s	Cross-sectional area of reinforcement
A_{smin}	Minimum steel reinforcement
A_{smax}	Maximum steel reinforcement
aw	Extreme position of the wheel

$\boldsymbol{b}$	Width of the plate
$\boldsymbol{b_{eff}}$	Effective width
$\boldsymbol{c_e(z)}$	Exposure factor
$\boldsymbol{c_{min}}$	Minimum cover
$\boldsymbol{c_{nom}}$	Nominal value of the concrete cover
$\boldsymbol{c_0(z)}$	Orography factor
$\boldsymbol{c_{pe}}$	Pressure coefficient for the external pressure
$\boldsymbol{c_{pi}}$	Pressure coefficient for the internal pressure
$\boldsymbol{c_r(z)}$	Roughness factor
$\boldsymbol{C_t}$	Coefficient that depends on the moment resisting type of the structure
$\boldsymbol{d}$	Diameter
$\boldsymbol{d_j}$	Deflection at level *j*
$\boldsymbol{d_0}$	Diameter of the bolt holes
$\boldsymbol{d_r}$	Inter-storey drift
$\boldsymbol{E}$	Young modulus
e:	Minimum distance approach(distance between wheels)

e_i	Spacing of the connectors
e_1	Distance from the edge bolts to the end plate, in the direction of the load application
e_2	Distance from the edge bolts to the end plate, in the direction parallel to the load application
E_{cm}	Secant modulus of the concrete
E_s	Elastic modulus
e_v	Eccentricity
$F_{b,Rd}$	Bearing resistance of the bolts
f_{cd}	Design compressive strength of the concrete
f_{ck}	Characteristic compressive strength of the concrete
F_{pcd}	Tightening force of a bolt
$F_{t,Ed}$	Design traction force on the connection
$F_{t,Rd}$	Traction resistance of the bolts
f_u	Ultimate strength
f_{ub}	Ultimate strength of the bolts
$f_{u,p}$	Ultimate strength of the plate
F_v	Steel failure load of the fasteners

$F_{v,Ed}$	Design shear force on the connection
$F_{v,Rd}$	Shear resistance of the bolts
f_y	Elastic limit of steel
f_{yd}	Design yielding strength
F_w	Wind force
G	Shear modulus
$G_{k,j}$	Permanent loads
H	Lifting height
I	Moment of inertia
i	Radius of gyration
Itot	Moment of inertia of the running beam section plus rail
Lc	Bridge crane span
L_{EZ}	Buckling length
L_{cr}	Critical buckling length
$M_{b,Rd}$	Buckling resisting moment
$M_{c,Rd}$	Resisting moment
M_{Ed}	Design moments
$M_{pl,Rd}$	Plastic resisting moment
M_{Rd}	Resisting moment

My	Moment calculation along axis y-y
Mz	Moment calculation along axis z-z
N	Number of cycles done per hour
$N_{b,Rd}$	Compressive buckling resistance
$N_{c,Rd}$	Resistance to axial compression
N_{Ed}	Design axial force
$N_{pl,Rd}$	Axial force plastic resistance
N_{Rd}	Resisting compression force
p_1	Spacing of the bolts, in the direction of the load application
p_2	Spacing of the bolts, in the direction parallel to the load application
Q	Load
q_b	Basic velocity pressure
Qh,nom	Nominal hoist load
$Q_{k,j}$	Live loads
$q_p(z)$	Peak velocity pressure
Qr,max	Maximum load per wheel of the loaded crane
Qr,min	Minimum load per wheel of the unloaded crane

$S_{Qr,max}$	Maximum load per wheel of the loaded crane
$S_{Qr,min}$	Minimum load per wheel of the unloaded crane
Srail	Static moment of rail with respect to the neutral asix
t	daily operating time
tf	Thickness of flange
t_p	Thickness of the plate
tw	Thickness of web
T	Total working time
u	Perpendicular distance from the edge of the beam flange to the edge of the column
U	Fréquency of use
v_b	Basic wind velocity
V_{Ed}	Design shear force
Vl	Lifting speed
$v_m(z)$	Mean wind velocity pressure
$V_{pl,Rd}$	Plastic shear resistance
V_{Rd}	Resistance of the shear
$V_{Rd,min}$	Shear resistance of the end plate
Vt	Long travelling speed
Wc	Crane frame
Wcap	Hook load
Wcb	Crab weight

w_e	Wind pressure acting on the external surface
W_{eff}	Effective section modulus
W_{el}	Elastic section modulus
w_i	Wind pressure acting on the internal pressure
W_{pl}	Plastic section modulus
z_{min}	Minimum height
z_0	Roughness length
γ_M	Partial safety factors
γ_G:	Load amplification factor for permanent load
γ_Q	Load amplification factor for variable load
λ	Slenderness
χ	Reduction factor for relevant buckling mode
χ_{LT}	Reduction factor for lateral torsional buckling
$\psi_{0,j}$	Combination coefficients

ABSTRACT

The main objective of this thesis was to analyse and verify the behavior of an industrial steel structure with and without bridge cranes. This concerned the study and evaluation of the installation of a bridge crane with a capacity of ten (10) tons in an existing industrial hall intended to serve as a production and transportation of sheet metal in a factory located at Mvan in Yaounde. In order to model the steel building, the architectural plans of the building were obtained and Autocad 2020 was used to have detail views of the plans. This steel factory consists of eight (8) frames with each frame composed of braces and purlins. The preliminary design of the purlin, braces, columns and runway beams were made with the help of an excel program according to Eurocode 3-design of steel structures part1: general rules and rules of buildings EN 1993-1-1: 2005 and part 6: crane supporting structures EN 1993-6: 2007. For the bridge crane in particular a series of questions were asked to the enterprise to be able to obtain the necessary characteristics needed for the sizing of the latter. Once the loads have been determined and the lowering of the loads established, the two structures were modelled; one without and the second with a bridge crane. A complete study of the main frame taking into account all the parameters and load combinations were performed using SAP2000 v23.2.0 software in which the 3D models were created. This modeling and design allowed us to evaluate the forces and deformations applied to the structure under the different load combinations and verifications under constraints were conducted in order to perform a linear static analysis. Results showed that in the absence of the bridge crane, the steel structure presented lower values in terms of moment, shear, axial force and torsion. In the presence of the bridge crane, a high increase in the steel structure's moment, shear and axial force especially in the columns was observed. Equally, a cost estimate with prices from the world market was performed. Although the amount for construction of the steel structure with a bridge crane was higher, there is an increase in workspace safety, employees' efficiency of labour, cost-savings and gains due to the increased productivity and structural capacity while lifting heavier loads.

Keywords: industrial steel structure, bridge crane, moving loads, crane supporting structures.

RÉSUMÉ

L'objectif principal de ce travail était d'analyser et de vérifier le comportement d'un bâtiment industriel en acier avec et sans pont roulant. Il s'agissait ici de l'étude et l'évaluation de l'installation d'un pont roulant d'une capacité de dix (10) tonnes dans un hall industriel existant destinées à servir d'atelier de production et de transport de tôles dans une usine située à Mvan à Yaoundé. Afin de modéliser ce bâtiment industriel, les différents plans architecturaux, coupe et section du bâtiment ont été obtenus. Cette usine se compose de huit (8) portiques, dont chacun est constitué de contreventements et de pannes. Le prédimensionnement des pannes, contreventements et poteaux a été réalisé à l'aide d'un programme Excel selon l'Eurocode 3-calcul des structures en acier partie 1 : règles générales et règles des bâtiments EN 1993-1-1 : 2005 et partie 6 : Chemins de roulement EN 1993-6 : 2007. Pour le pont roulant en particulier une série de questions a été posées à l'entreprise pour obtenir les caractéristiques nécessaires pour le dimensionnement de ce dernier. Une fois les charges déterminées et la descente de charge établie, les deux structures ont été modélisées à savoir : une structure sans le pont roulant et la seconde structure avec un pont roulant. Une étude complète de la structure en prenant en compte toutes les charges a été réalisée à l'aide du logiciel SAP2000 v23.2.0 dans laquelle les deux modèle 3D ont été créé. Cette modélisation a permis d'évaluer les efforts et déformations appliqués à la structure sous les différentes combinaisons de charges et des vérifications sous contraintes afin d'effectuer une analyse linéaire statique. Les principaux composants de la structure ont ensuite été dimensionnés conformément aux réglementations EUROCODE 3 et CCM99. Les résultats ont montré qu'en l'absence du pont roulant, la structure en acier présentait des valeurs moindres en termes de moment, de cisaillement, et de force axiale. En présence du pont roulant, il a été observé une forte augmentation du moment, du cisaillement, et de l'effort axial, en particulier dans les poteaux. Aussi, une estimation des coûts avec les prix du marché mondial a été réalisée. Bien que le montant pour la construction de la structure en acier avec un pont roulant ait été plus élevé, il y a une augmentation de la sécurité de l'espace de travail, de l'efficacité du travail des employés, des économies de coûts et de gains dus à l'augmentation de la productivité tout en soulevant des charges plus lourd.

Mots clés : hall métallique, pont roulant, charge mobile, chemin de roulement et autres machines

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GENERAL INTRODUCTION

The extensive use of prefabricated factory-finished large-sized elements and the conversion of production into a mechanized process of assembly is what makes up an industrial building. Achieving greater span lengths, smaller deflections, lower costs and a better structural performance presents the day-to-day challenges of every structural system. The use of structural steel as a material for construction of industrial buildings is the most common with respect to its low mass, great resistance, ease of construction, although it presents some limits. The industry plays an important role in the economic growth and economic development of a country with the use of technological innovation. This is seen in the United States of America, China and Japan in the world and in Mauritius'island, South Africa and Rwanda in Africa. Cameroon being a developing country, gives room to the creation of more industries through its National Development Strategy, NDS30 which promotes the construction of larger industries with greater spaces by 2035.

The handling, loading, unloading and storage operations of manufactured products remain crucial and is among the most strenuous throughout the life of an industry. Indeed, handling is one of the physical activities that put a lot of strain on the body by carrying loads, walking with heavy loads, leaning posture, trunk extension with or without a load. These operations are repetitive and often carried out at a continuous rate which entails risks of Musculoskeletal Disorders (MSDs). There is therefore a need for a good structural system equipped with a bridge crane to cope for the limitations of the ancient approach. The main objective of this work is therefore clear, to design, analyse and compare the structural and functional performances of two identical steel structures of which one is equipped with a bridge crane and the other is not. This will bring forward the peculiar characteristics of steel structures with bridge cranes and the future perspectives for steel structures in Cameroon.

In order to achieve this objective, the study is divided into three parts. The first part consists of a literature review on the various forms of steel, a clever understanding of a bridge crane with an introduction to moving loads. The second part presents the methodology of the study, an elaboration on the collection of data, the method of analysis and design of a steel structure and a bridge crane using the norms EUROCODE 3 and CCM97. Lastly, the third part, which shows the results obtained from both structures and the comparison based on their general structural and functional performances and a cost estimation to conclude on a relatively convenient and best suited structural system.

CHAPTER 1: LITERATURE REVIEW

Introduction

Bridge cranes sometimes known as overhead bridge cranes are types of cranes that include two overhead runways built into the building's support structure. They typically span entire facilities and allow the users to lift, move, and position materials throughout the lifting area. This machine when suspended off the floor, may be selected over other types of overhead cranes to move loads around a warehouse or factory floor when floor space is in short supply as the machine can be introduced indoors. This chapter is meant for an in-depth explanation of the different concepts to facilitate the understanding of the subsequent work. As such the chapter starts by presenting the key concepts and principles necessary to understand the problem outlined in the thesis. Thus an overview about steel as a raw material, its production, characteristics, defects and uses. Secondly industrial steel buildings followed by bridge cranes where the characteristics, types, uses, actions are discussed with steel connection devices and design methods of both. Afterwards analysis methods of both industrial steel buildings and bridge cranes will be presented, and lastly the control and maintenance of bridge cranes.

1.1. Steel

Steel is a hard, strong bluish-grey alloy of iron with carbon and usually other elements, used as a structural material to improve its strength and fracture resistance compared to other forms of iron. Many other elements may be present or added. Stainless steels that are corrosion- and oxidation-resistant need typically an additional 11 % chromium. Because of its high tensile strength and low cost, steel is used in buildings, infrastructure, tools, cars, machines, electrical appliances, and weapons. Iron is the base metal of steel. Depending on the temperature, it can take two crystalline forms (allotropic forms): body-centred cubic and face-centred cubic. The interaction of the allotropes of iron with the alloying elements, primarily carbon, gives steel and cast iron their range of unique properties. In pure iron, the crystal structure has relatively little resistance to the iron atoms slipping past one another, and so pure iron is quite ductile, or soft and easily formed. In steel, small amounts of carbon, other elements, and inclusions within the iron act as hardening agents that prevent the movement of dislocations.

1.1.1. Raw materials of steel

The three main raw materials used to make pig iron (which is the raw material needed to make steel) for primary steel production in a blast furnace are the processed iron ore, scrap steel and coke.

1.1.1.1. Iron ore

Steel is made from iron ore, a compound of iron, oxygen and other minerals that occur in nature. It is usually rich in iron oxides and vary in colour from dark grey, bright yellow, or deep purple to rusty red. The iron is usually found in the form of limonite ($FeO(OH)\cdot n(H2O)$, 55 % Fe) , magnetite (Fe3O4, 72.4 % Fe), haematite (Fe2O3,69.9 % Fe), or siderite (FeCO3, 48.2 % Fe).goethite (FeO(OH),62.9 % Fe),

1.1.1.2. Scrap metal

This is a metal that is an important source of industrial metals and alloys, particularly in the production of steel. Scrap metal is the combination of waste metal, metallic material and any product that contains metal that is capable of being recycled from previous consumption or product manufacturing. Ironmaking furnaces require at least a 50% iron ore content for efficient operation. However, the cost of shipping iron ore means that it is often purified to some degree before being shipped - a process called beneficiation. This process includes crushing, screening, tumbling, floatation, and magnetic separation. The refined ore is enriched to over 60 % iron by these processes and is often formed into pellets before shipping. Scrap is the primary raw material used in smaller quantities in primary steel production in order to control the reduction reaction (from iron oxides to 'free' Fe ions).

1.1.1.3. Coal

Metallurgical coal, also known as coking coal, is used to produce coke-the primary source of carbon used in steelmaking. Coke making is effectively the carbonisation of coal at high temperatures. Production normally takes place in a coke battery located near an integrated steel mill. In this battery, coke ovens are stacked in rows. Coal is loaded into the ovens and then heated in the absence of oxygen up to temperatures around 1100 °C. Without oxygen, the coal does not burn but begins to melt. The high temperatures volatise unwanted impurities, such as hydrogen, oxygen, nitrogen, and sulphur. These given off gasses can either be collected and recovered as by-products or burned off as a source of heat.

After cooling, the coke solidifies as lumps of porous, crystalline carbon large enough to be used by blast furnaces. A blast furnace is fed with coke, iron ore and fluxes, and hot air is blown into the mixture. Air causes the coke to burn, raising temperatures to 1700 °C which oxidizes impurities. The process reduces the carbon content by 90 % and results in a molten iron known as hot metal. The hot metal is then drained from the blast furnace and sent to the basic oxygen furnace where scrap steel and limestone are added to make new steel. Other elements, such as molybdenum, chromium, or vanadium can be added to produce different grades of steel.

1.1.1.4. Alloying metals

An alloy is a mixture of chemical elements of which at least one is a metal. Unlike chemical compounds with metallic bases, an alloy will retain all the properties of a metal in the resulting material. By definition, steel is a combination of iron and carbon. Steel is alloyed with various elements to improve physical and chemical properties.

1.1.2. Steel making processes

Steel making is the process of producing steel from iron ore and/or scrap. In steelmaking, impurities such as nitrogen, silicon, phosphorus, sulphur and excess carbon (the most important impurity) are removed from the sourced iron, and alloying elements such as manganese, nickel, chromium, carbon and vanadium are added to produce different grades of steel. The raw materials for steelmaking are mined and then transformed into steel using different processes: the blast furnace/basic oxygen furnace route (indirect reduction) in combination with a converter, the electric arc furnace route, the crucible process, the bessemer process and the open hearth furnace. (Turkdogan, 1996).

1.1.2.1. Crucible process

Crucible process is a technique for producing fine or tool steel. The earliest known use of the technique occurred in India and central Asia in the early 1st millennium CE. The steel was produced by heating wrought iron with materials rich in carbon, such as charcoal in closed vessels. The process involves heating of either blister steel fragments or short lengths of wrought iron bars mixed with charcoal inside fire clay crucibles. The resulting molten steel is allowed to run through iron moulds. Such steel is called cast iron. Cast steel is extremely hard and perfectly homogenous. These are specifically used for making cutting tools and the finest cutlery items. The crucible steel process is one of the oldest documented methods of producing steel and, although it has been replaced with more efficient methods, it is still used to produce small quantities of high-quality materials for specialized applications.

Figure 1.1. Melting metal in crucible (https://www.dreamstime.com/stock-photo-steel-pouring-hot-plant)

1.1.2.2. Bessemer process

The bessemer process reduces molten pig iron in so-called bessemer converters. These are egg-shaped, silica, clay, or dolomite-lined containers with capacities of 5 to 30 tons of molten iron. An opening at the narrow upper portion of the bessemer converter allows iron to be introduced and the finished product to be poured out. Melting of metal typically is accomplished with coal and coke fires as shown in Figure 1.2. Air is forced upward through perforations at the wide bottom-end of the converter. As the air passes upward through the molten pig iron, impurities such as silicon, manganese, and carbon oxidize. Different property steels (Stability et al., 2008)are produced by introducing additional elements, such as spiegeleisen—an iron-carbon-manganese alloy—into the molten metal after the oxidation is completed. The converter is then emptied into ladles from which the steel is poured into molds. Since the whole process could be completed typically under one hour and for significantly larger quantities than with crucibles, bessemer steel production significantly helped meet steel demands during the onset of the industrial revolution.

Figure 1.2. Bessemer's converter (Matthias Ruth, 2004)

1.1.2.3. Open hearth furnace process

The speciality of open hearth furnaces is the extreme heat that can be obtained from them due to their regenerative process. The charge of pig iron, steel scrap, iron ore, and flux are together kept in a shallow container with a flame burning above it. The process is initiated inside reverberatory gas-fired regenerative furnaces for greater efficiency. Regenerators are placed below the furnace and positioned in two pairs. The pairs are heated alternately through the passage of hot gases given out from the furnace in their route to the chimney. This heat is retained by the regenerators and is reversed and given back to the furnace. This heat exchange procedure helps the furnace to maintain high temperatures even with less fuels.

Once the furnace is charged with pig iron, pure oxidizing ores like haematite are added to it from time to time, which helps oxidization and the removal of impurities like silicon, carbon, and manganese in the pig iron. Spiegel is also introduced when the carbon content becomes less than 0.1%, and ferro-manganese after the metal is tapped out into the ladle. The open hearth furnace is shown in Figure 1.3.

Figure 1.3. B-stall, 60-ton open-hearth furnace (viktormarsha.com,2022)

1.1.2.4. Basic oxygen steel process

The basic oxygen furnace (BOF) is a refractory-lined, tiltable converter into which a vertically movable, water-cooled lance is inserted to blow oxygen through nozzles at supersonic velocity onto the charge. The use of pure oxygen at high flow rates results in such fast oxidation of the elements contained in blast-furnace iron that only about twenty minutes are required to refine one charge. Converters vary in size and are operated for heats ranging from 30 to 360 tons. Basic oxygen furnaces use pig iron and small amounts of scrap as their primary material inputs and typically melt the furnace charge with coke oven, blast furnace, and natural gas (www.sciencedirect.com, 2022). Electricity and fuel oils are used to drive ancillary processes as shown in Figure 1.4. Basic oxygen furnaces have the ability to cut total cycle time to less than a tenth of the cycle time of open hearth furnaces, thus enabling significant savings of energy for a given metal mix, and substantial total cost savings even though material costs themselves may be higher.

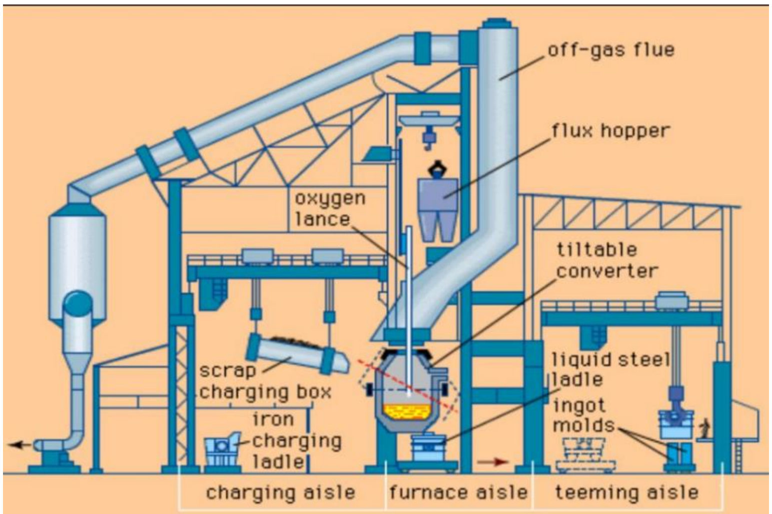

Figure 1.4. Basic oxygen furnace (https://cdn.britannica.com/oxygen-furnace-shop, 2022)

1.1.2.5. Electric arc furnace

An electric arc furnace is a steel cylinder lined with refractory and it has three electrodes (carbon rods) which are inserted through the furnace roof. The charge is mostly scrap steel and alloy materials. In this process electric arc or electric high frequency furnaces are used. In electric arc furnaces which are more common among the two processes, high voltage electric arc struck between carbon electrodes and the charge becomes the source of a very high temperature. The charge is collected directly from an open-hearth furnace, the intense arc heat keeps the charge in its molten state, and the impurities are removed in the form of slag. The high frequency furnace is based on the principle that when high frequency alternating current is applied to steel, eddy current starts flowing in them. If this induction is made very strong, it can heat up the steel and melt it. The electric arc furnace is shown in Figure 1.5.

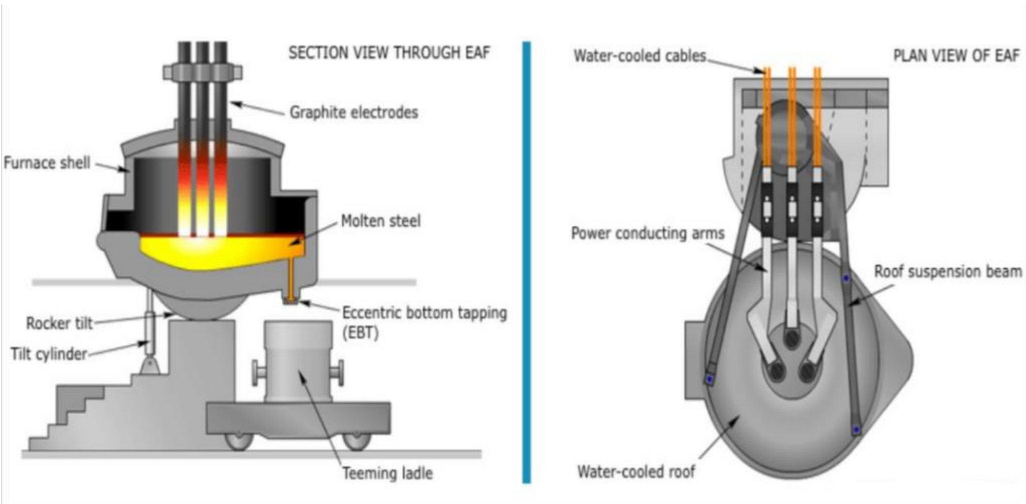

Figure 1.5. Electric arc furnace (Matthias Ruth, 2004)

1.1.3. Characteristics of steel

The characteristics of steel can be subdivided into mechanical, chemical, physical and technological characteristics.

1.1.3.1. Mechanical characteristics

Steel has a number of properties, including: hardness, toughness, tensile strength, yield strength, elongation, fatigue strength, corrosion, plasticity, malleability and creep. The properties that are most important in wear and abrasion-resistant steel are:

a. Hardness

Hardness is the material’s ability to withstand friction and abrasion. It is worth noting that, while it may mean the same as strength and toughness in colloquial language, this is very different from strength and toughness in the context of metal properties.

b. Toughness

Toughness is the ability to absorb energy without fracturing or rupturing. It is also defined as a material’s resistance to fracture when stressed and this quality depends on the maximum energy that can be absorbed before fracturing. It is important to distinguish this from hardness as a material that severely deforms without breaking, could be considered extremely tough, but not hard.

c. Yield strength

Yield strength is a measurement of the force required to start the deformation of the material (i.e., bending or warping).

1.1.3.2. Chemical characteristics

The chemical properties of steel include flammability, toxicity, acidity, reactivity and heat of combustion.

a. Flammability

Steel is a non-combustible material and consequently does not burn. Steel does not melt at temperatures typically encountered in a building fire and has a melting point of approximately 1500 °C.

b. Acidity

The temperature will rise when steel wool is in contact with the acids due to the oxidation reaction. The rise in temperature will be higher when steel wool is subjected to more acidic rainwater as higher hydrogen ion available will likely speed up the iron oxidation process and hence the rate of rusting.

1.1.3.3. Physical characteristics

The physical properties of steel include high strength, low weight, durability, ductility and resistance to corrosion.

- Steel offers great strength although it is light in weight. In fact, the ratio of strength to weight for steel is the lowest than any other building material available to us.
- Steel is ductile and can be molded easily to form any desired shape.
- The strength of steel refers to the deformation and fracture performance of metal materials under the action of external force, which generally includes tensile strength, bending strength and compressive strength.
- Steel has a unit weight of steel of 7850 kg/m3.
- Steel has a certain resistance against corrosion. This so-called passivity is due to a thin and invisible layer of oxides being formed on the surface.

1.1.3.4. Technological characteristics

The technological properties of steel include high strength, low weight, durability, ductility and resistance to corrosion.

a. Machinability

Steel can be cut into pieces permitting the removal of material with a satisfactory finish at lower cost. Good machinability is associated with high cutting speed, low power consumption and good surface finish.

b. Weldability

Steel can be welded to other metals under the fabrication conditions imposed in a specific suitably designed structure and to perform satisfactorily in the intended service.

c. Castability

Steel can be cast into different forms based on factors like solidification rate, gas porosity, segregation, shrinkage.

d. Formability

Formability is the ability of metals of forming into different shapes.

e. Malleability

Steel material undergoes too much change in shape under compressive stress without rupture. Materials like soft steel, wrought iron, copper and aluminium have good malleability. They can be hammered or rolled into the desired shape without rupture.

1.1.4. Types of steel

According to the American Iron & Steel Institute (AISI), steel can be categorized into four basic groups based on the chemical compositions namely carbon steel, alloy steel, stainless steel and tool steel (Aakanksha Gaur, 2019).

1.1.4.1. Carbon steel

Carbon steel is a steel with carbon content from about 0.05 up to 2.1 % by weight. The definition of carbon steel from the American Iron and Steel Institute (AISI) states that :

- no minimum content is specified or required for chromium, cobalt, molybdenum, nickel, titanium, tungsten, vanadium, zirconium, or any other element to be added to obtain a desired alloying effect;
- the specified minimum for copper does not exceed 0.40 %;
- the maximum content specified for any of the following elements does not exceed the percentages noted: manganese 1.65 % ; silicon 0.60 % ; copper 0.60 %. Generally, the greater the carbon content, the more the alloy can be hardened, increasing strength, wear and impact resistance but ductility affected (Martinez & Stern, 2002).

1.1.4.2. Alloy steel

Alloy steel is steel that is alloyed with a variety of elements in total amounts between 1.0 % and 50 % by weight to improve its mechanical properties. Alloy steels are broken down into two groups: low alloy steels and high alloy steels. Strictly speaking, every steel is an alloy, but not all steels are called "alloy steels". The simplest steels are iron (Fe) alloyed with carbon (C) (about 0.1 % to 1 %, depending on the type) and nothing else (excepting negligible traces via slight impurities); these are called carbon steels. However, the term "alloy steel" is the standard term referring to steels with other alloying elements added deliberately in addition to the carbon common alloyants including manganese, nickel, chromium, molybdenum, vanadium, silicon, and boron. Less common alloyants include aluminium, cobalt, copper, niobium, titanium, tungsten, lead, and zirconium. (Min, Kim, & Dornfeld, 2001).

1.1.4.3. Stainless steel

Stainless steel is a corrosion-resistant alloy of iron, chromium and, in some cases, nickel and other metals. It must contain at least 10.5 % chromium, the exact components and ratios will vary based on the grade requested and the intended use of the steel. (Aydoğdu & Aydinol, 2006).

1.1.4.4. Tool steel

Tool steels contain tungsten, molybdenum, cobalt and vanadium in varying quantities to increase heat resistance and durability, making them ideal for cutting and drilling equipments. (Imbert, Ryan, & McQueen, 1984).

1.1.5. Uses of steel

Steel possesses many properties which makes it very useful in different sectors. These sectors include the construction, transportation, mining, energy and machine industries.

1.1.5.1. Construction industry

Structural steel is tensile, ductile, flexible, and cost-effective. Metal fabricators across the world prefer using structural steel for construction. The plasticity and flexibility of structural steel make it suitable for the construction of residential and industrial buildings. A technique called light gauge steel construction is used to build residential buildings.

1.1.5.2. Transportation industry

- Steel provides strong, safe and sustainable transport solution to every mode of transport. It facilitates the mobility and transport of goods.
- For trains and rail cars: Rail transport requires steel in the trains, the rails and infrastructure. For short or medium haul journeys, rail reduces travel times and

carbondioxide emissions per passenger-kilometer compared to nearly all other forms of transport. Steel is essential and makes up 15 % of the mass of high speed trains. The main steel components of these trains are bogies (the structure underneath the trains including wheels, axles, bearings and motors).

1.1.5.3. Mining industry

Most of the mining infrastructure is created using structural steel and stainless steel. The chairs used in the mining units are mostly customized and made of steel. All underground mine workings must have a reliable support that meets the necessary technical requirements for strength, toughness, and flexibility.

1.1.5.4. Energy industry

The steel industry is extremely important, driving many economies and providing material for infrastructure all around the world. Steel helps extract primary energy from the environment and uses a fair amount of energy. Around 18 % of the world's industrial energy used goes into the mining, processing and manufacturing of iron and steel products.

1.1.5.5. Machine industry

If a product is not made of steel, the chances are that it will be made from a machine made of steel. Steel is essential in our modern world. Tools and machinery cover a wide range of equipment from small workshop tools to large factory-based robotic machinery and rolling mills. In 2017, tools and machinery represented approximately 15 % of global steel use.

1.1.6. Defects of steel

Defects and anomalies may occur in any heat treatment process associated with cold extrusion or forming, such as homogenization, preheating and annealing. Overheating and burning can reduce strength and toughness of steel. Another important reason of fracture may be caused by abnormal defects during surface hardening of parts. These defects are roll marks, quench cracking and pinchers.

1.1.6.1. Roll marks

Roll marks are also sometimes referred to as bruises, roll bruises, or whip marks. One type of roll mark also has aesthetic implications on the steel material. These usually don’t have any effect on the product's quality and are dismissed most of the time during the manufacturing process.

1.1.6.2. Quench cracking

The quenching cracks in steel originate from the stress produced by the increase of volume during the transformation of austenite to martensite. The martensite in the quenched state is very hard and has little ductility.

1.1.6.3. Pinchers

Pinchers usually appear as uneven ridges or grooves that may surface on the metal. It is more ubiquitous in steel products that have to be rolled, such as billets, pipes, or tubes.

1.1.7. Prevention of defects

Protective coatings, environmental control, and cathodic protection are useful ways to prevent corrosion in steel. However, these measures will be negligible without constant maintenance and monitoring.

1.1.7.1. Protective coatings

Protective coatings are a simple way to reduce corrosion, by limiting the exposure of the metal to a corrosive environment. Different types of protective coatings like zinc coating, powder coating act as a physical block between the metallic surface and the oxidising environment, thereby protecting the surface. Paint is a very common protective coating, but tar, pitch, bitumen and plastics are also used. Cathodic protection is possible to prevent corrosion by putting an opposite electrical current to the steel's surface. The most common example of cathodic protection is the coating of iron alloy steel with zinc, a process known as galvanizing.

1.1.7.2. Design modifications

The steps to prevent corrosion starts at the engineering level especially if the steel is in an environment where it is an easy target to corrosion. To lessen the corrosion, narrow gaps should be removed so that air or water does not enter and become dirty or stale.

1.1.7.3. Environmental measures

Corrosion is caused by a chemical reaction between metals and gases in the immediate environment. By taking measures to control the composition of the environment, these unwanted reactions can be lessened. This can be as simple as keeping down exposure to rain or sea water or more difficult measures such as controlling the amounts of sulphur, chlorine, or oxygen in the nearby environment.

1.2. Steel structure

An industrial steel structure is a metal structure produced with steel for the internal support and for exterior cladding, as opposed to steel framed buildings which generally use other materials for floors, walls, and external envelope.

1.2.1. Structural elements

Every structure has structural elements which support the building and carry out its main functions. The different parts of an industrial steel structure are shown in Figure 1.6.

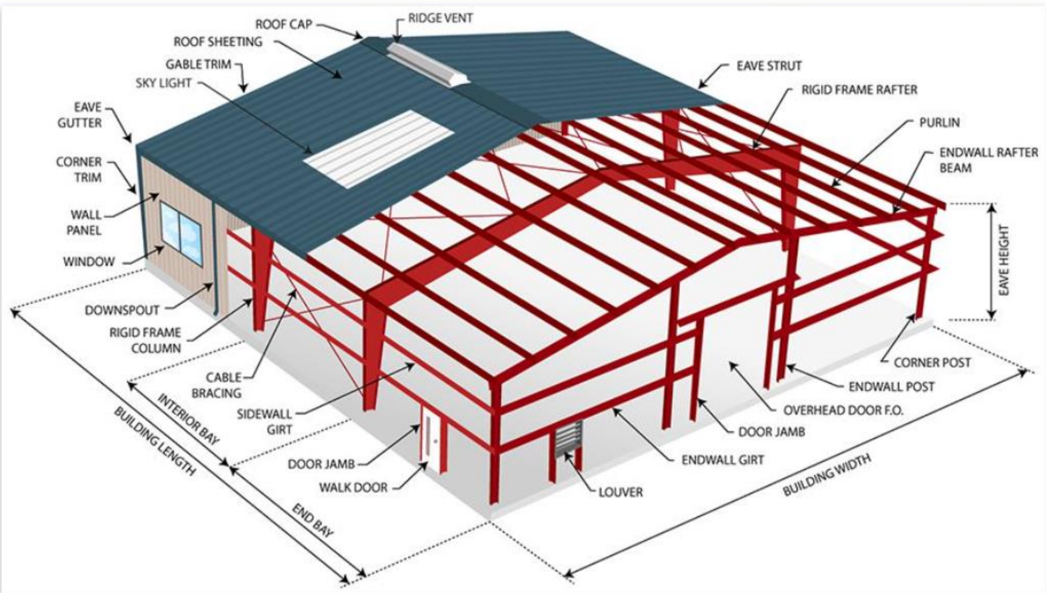

Figure 1.6. Parts of a steel building (https://essteelbuildings.com/parts-steel-)

The important elements of industrial steel structures are rafters, roof truss, purlins, wind bracing and columns as primary elements and eave gutter, eave strut, louver, doors as secondary elements.

- **Rafter**: a beam forming part of the internal framework of a roof.
- **Corner post**: the outside structural corner post of a steel building that supports the rafter within the end wall assembly.
- **Eave gutter**: the rain capture and diversion channel applied to the eve strut (fascia) of a steel building to transfer rain water to the downspouts.
- **Eave strut**: the lowest applied purlin of the roof framing that acts as the sub-fascia, and provides support for the rain gutters.
- **End wall post**: the structural support columns within the end wall assembly that support the end wall rafter beam.
- **Roof truss**: a triangulated system of straight interconnected structural elements. The individual elements are joined at the nodes by welding. The external forces applied to the system and the reactions at the supports are generally applied at the nodesEnd wall rafter beam – the rafter at the end of a steel building, normally part of the end wall assembly.
- **Louver**: a ventilation device that mounted into a wall assembly similarly to a window
- **Overhead door**: the general title for any large door that rolls, rotates or pivots to an upper position above the door opening.
- **Purlin**: the "framing" member applied to the rigid frame rafters that provides support for the roofing panels.

1.2.2. Characteristics of steel structure

The characteristics of an industrial steel structure include shock resistance, wind resistance, durability, heat preservation, sound insulation, fire resistance, comfort, quickness, and environmental protection.

1.2.2.1. Shock resistance

The roofs of low-rise houses are mostly sloped roofs, so most roof structures adopt triangular roof truss systems made up of cold-formed steel components. After sealing the structural plates and gypsum boards, the light steel components form a stable slab rib structure system. This structural system has stronger seismic resistance and resistance to horizontal loads and is suitable for areas with seismic intensity above eight degrees.

1.2.2.2. Wind resistance

The steel structure has lightweight, high strength, good overall rigidity, and strong deformability. The self-weight of the building is only one-fifth of the brick-concrete structure, which can resist seventy meters per second hurricane.

1.2.2.3. Durability

The steel structure is all composed of a cold-formed thin-walled steel component system, and the steel frame is made of super anti-corrosion high-strength cold-rolled galvanized sheet.

1.2.3. Typology

The types of industrial steel buildings include aircraft hangars , equipment storage, online fulfilment centres, industrial parks, industrial warehouse buildings, manufacturing facilities, meat packing plants, oil and gas buildings, processing plants recycling centers, sawmills and lumber yards, truck terminals and welding shops.(Poehlitz, 2017). The following lines present a breakdown of the top industrial uses for pre-engineered steel buildings.

1.2.3.1. Commercial office space

Many industrial businesses decide to opt for a pre-fabricated steel building that combines all needs in one. These steel structures can be built to accommodate office space while having sufficient room for equipment storage and having garage bays for industrial vehicles. The beauty of these pre-fabricated steel structures is that they are highly customizable to fit specific industry needs.

1.2.3.2. Equipment storage

One of the biggest benefits of pre-engineered steel buildings is the ability to store expensive equipment safely inside, protected from the elements and safe from thieves. These buildings serve as excellent tool rooms and machine shops, and are perfect for storing high-end industrial equipment.

1.2.3.3. Workspace

Pre-engineered steel buildings serve as the perfect workspace. Regardless of the specific industrial trade, these structures offer the ideal amount of space to get the job done. They can be set up due to the open concept steel construction.

Figure 1.7. Steel workspace (https://steelmillcranes.com/industrial-steel-structure/)

1.2.3.4. Garages for work vehicles

These pre-engineered steel buildings are also excellent garage structures. They can be designed to accommodate vehicles such as trucks, tractors parking them outdoors. It protects them from weathering, keeps them secure and serves as location to conduct usual maintenance.

1.2.4. Actions

The types of loads acting on a steel structure are: dead loads, variable loads, wind loads, snow loads, seismic loads, fire loads and moving loads (live load).

1.2.4.1. Dead loads

Dead loads, also known as permanent loads, are those that remain relatively constant over time and comprise, for example, the weight of a building's structural elements, such as beams, walls, roof and structural flooring components. Dead loads may also include permanent non-structural partitions, immovable fixtures and even built-in cupboards.

1.2.4.2. Live loads

A live load is a load that can change over time. The weight of the load is variable or shifts locations, such as when people are walking around in a building. Anything in a building that is not fixed to the structure can result in a live load since it can be moved around. Live loads are factored into the calculation of the gravity load of a structure. A live load can be expressed either as a uniformly distributed load (UDL) or as one acting on a concentrated area (point load).

1.2.4.3. Wind load

Wind load may not be a significant concern for small, massive, low-level buildings, but it gains importance with height, the use of lighter materials and the use of shapes that may affect the flow of air, typically roof forms. Where the dead weight of a structure is insufficient to resist wind loads, additional structure and fixings may be required. Wind load is required to be considered in structural design especially when the heath of the building exceeds two times the dimension transverse to the exposed wind surface.

1.2.4.4. Snow load

This is the load that can be imposed by the accumulation of snow and is more of a concern in geographic regions where snowfalls can be heavy and frequent. Significant quantities of snow can accumulate, adding a sizable load to a structure.

1.2.4.5. Seismic load

Earthquake load takes place due to the inertia force produced in the building because of seismic excitations. The magnitude of earthquake loading depends upon the weight or mass of the building, dynamic properties of the building and difference in stiffness of adjacent floors along with the intensity and duration of the earthquake.

1.2.4.6. Wheel/ Moving loads

A moving load is one that changes the point at which the load is applied over time. Examples include a vehicle that travels across a path and a train moving along a track.(EN, 2006). In case of buildings, the loads that affect the floor of the building, such as people and furniture, can all be called moving loads.

1.2.5. Design methods of steel structure

There are three different methods for the design of steel structure, i.e. simple design, continuous design and semi-continuous steel design. Joints in structures have been assumed to behave as either pinned or rigid to render design calculations manageable. In simple design the joints are idealised as perfect pins.(Dubina et al., 2013) Continuous design assumes that joints are rigid and that no relative rotation of connected members occurs whatever the applied moment. The vast majority of designs carried out today make one of these two assumptions, but a more realistic alternative is now possible, which is known as semi-continuous design.

1.2.5.1. Simple design method of steel structure

Simple design is the most traditional approach and is still commonly used. It is assumed that no moment is transferred from one connected member to another, except for the nominal moments which arise as a result of eccentricity at joints. The resistance of the structure to lateral loads and sway is usually ensured by the provision of bracing or, in some multi-storey buildings, by concrete cores. Assumptions regarding joint response must be ensured so that the detailing of the connections is such that no moments develop that can adversely affect the performance of the structure.

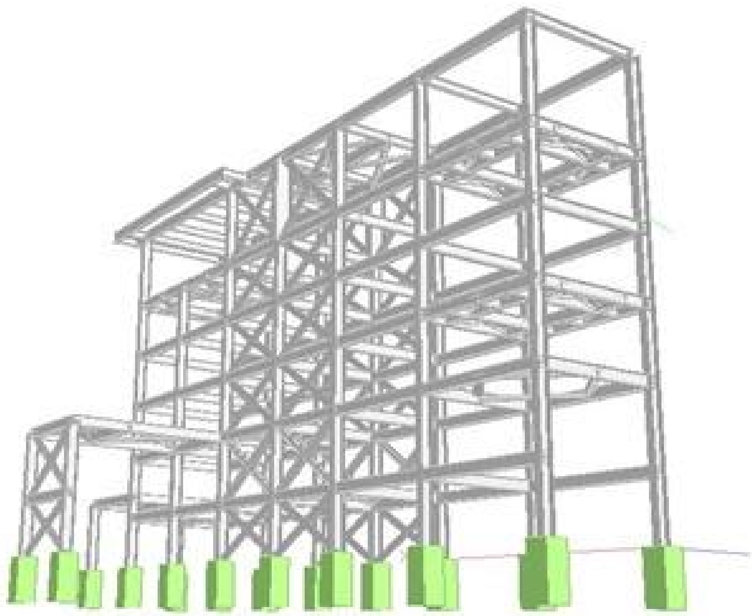

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Figure 1.8. Structure connected with Hinge joints (https://theconstructor.org/structural-engg/methods-of-steel-structure).

1.2.5.2. Continuous design method of steel structure

In continuous design, it is assumed that joints are rigid and transfer moment between members. The stability of the frame against sway is by frame action (i.e. by bending of beams and columns). Continuous design is more complex than simple design therefore software is commonly used to analyse the frame. Realistic combinations of pattern loading must be considered when designing continuous frames. The connections between members must have different characteristics depending on whether the design method for the frame is elastic or plastic. In elastic design, the joints must possess sufficient rotational stiffness to ensure that the distribution of forces and moments around the frame are not significantly different to those calculated. The joint must be able to carry the moments, forces and shear arising from the frame analysis. In plastic design, in determining the ultimate load capacity, the strength of the joint is of prime importance. The strength of the joint will determine whether plastic hinges occur in the joints or in the members, and will have a significant effect on the collapse mechanism. If hinges are designed to occur in the joints, the joint must be detailed with sufficient ductility to accommodate the resulting rotations.

1.2.5.3. Semi-continuous design method of steel structure

Semi-continuous design is more complex than either simple or continuous design as the real joint response is more realistically represented. Analytical routines to follow the true connection behaviour closely are highly involved and unsuitable for routine design, as they require the use of sophisticated computer programs. However, two simplified procedures do exist for both braced and unbraced frames; these are briefly referred to below. Braced frames are those where the resistance to lateral loads is provided by a bracing system; while in unbraced frames this resistance is generated by bending moments in the columns and beams. The simplified procedures are the wind moment method, for unbraced frames. In this procedure, the beam/column joints are assumed to be pinned when considering gravity loads. However, under wind loading they are assumed to be rigid, which means that lateral loads are carried by frame action.

1.2.6. Steel connection devices

Connections are structural elements used for joining different members of a structural steel frame work. Steel structure is an assembling of different member such as beams-columns which are connected to one another usually at member ends fasteners, so that it forms a single composite unit.

1.2.6.1. Components of a connection

Connections, or links, between steel beams are either bolted or welded. Both applications are economical and easy to perform(Jaspart & Weynand, 2016). However, bolts are commonly used in structures where safety is a primary concern as they tend to make stronger connections. They are represented as shown in Figure 1.9.

(a) (b) (c) (d)

Figure 1.9. Components of a connection: (a) Bolts (b) Welds (c) Connecting plates (d) Connecting angles (Vayas et al., 2019)

1.2.6.2. Classification of connections based on the connecting medium

The connections are an important part of steel structure and are designed more conventionally than any individual member. The classification in the connections (Michigan et al., 2008) provided in the steel structure is as follows: riveted connections, bolted connections, welded connections, and pinned connections.

a. Bolted connections

A bolted joint is one of the most common elements in construction and machine design. It consists of a male threaded fastener that captures and joins other parts, secured with a matching female screw thread. There are two main types of bolted joint designs: tension joints and shear joints. They are fastened together primarily by bolts. Bolts may be loaded in: Tension, shear, both tension and shear. There exist different types of bolts which are bearing type bolts, black bolts, turned bolts, and ribbed bolts.

Figure 1.10. Bolted connections (Michigan et al., 2008)

b. Welded connections

These are joints whose components are joined together by welds. Structural welding is a process by which the parts that are to be connected are heated and fused, with supplementary molten metal at the joint. A relatively small depth of material will become molten, and upon cooling, the structural steel and weld metal will act as one continuous part where they are joined. There exist the Groove, Fillet, Plug, Slot, Plug and slot weld.

Figure 1.11. Welded connections (Michigan et al., 2008)

1.2.6.3. Classification based on the type of internal forces.

There are three basic forces to which connections are subjected. These are axial force, shear force, and moment. Many connections are subject to two or more of these simultaneously. Connections are usually classified according to the major load type to be carried, such as shear connections, which carry primarily shear; moment connections, which carry primarily moment; and axial force connections, which carry primarily axial force (Brockenbrough & Merritt, 1994)

a. Shear connection

Shear connections (Figure 1.12) are the most common type of steel connections, and are referred to as “simple” or “semi -rigid” connections because no bending moment is considered. In structural engineering, a shear connection is a joint that allows for the transfer of shear forces between two members. So if a child member (for instance a beam) has some internal shear forces, these will be passed on as axial force into the column member. Common shear connections include plates, web angles and seat angles.

Figure 1.12. Shear connection (Brockenbrough & Merritt, 1994)

b. Moment connections

Moment connections (Figure 1.13) primarily carry moment loading, however are usually designed to resist shear and axial loading as well. For this reason, they are referred to as “rigid” or “fully restrained” connections and are used to create a frame(Brown et al., 2013). Common moment connections include directly welded members, flange plates, and end plate.

Figure 1.13. Moment connection (Brockenbrough & Merritt, 1994)

1.3. Bridge cranes

A bridge crane is a type of overhead crane or machine that allows you to lift, lower and move heavy materials from one location to another in a precise manner. Overhead travelling cranes, designed for handling loads up to 120 tonnes, are the ideal solution for heavy lifting and wide spans.

1.3.1. History

The first overhead cranes were steam powered and invented by the German company, Ludwig Stuckenholz, in the 1830’s and were mass produced in the 1840’s. But the first electric overhead crane did not appear until 1876, in the midst of the industrial revolution, in England. This crane was invented by Sampson Moore, a Liverpool engineer, for the Gun factory of the Royal Arsenal in London and was used to lift guns, ammunition and explosives. Soon afterwards, electric overhead crane use became widespread, especially in steel production. Further innovations to the overhead crane spurred widespread use throughout the world in many different industries. Inventions such as high capacity winch, electrical control systems, and then mass-produced electric hoists in 1910. The original hoist contained components mated together in what is now called the built-up style hoist. These built up hoists are used for heavy-duty applications such as steel coil handling and for users desiring long life and better durability.

1.3.2. Structural elements

A bridge crane is made up of the following parts as illustrated in figure 1.14. Each part will be defined to have a better understanding of the terms.

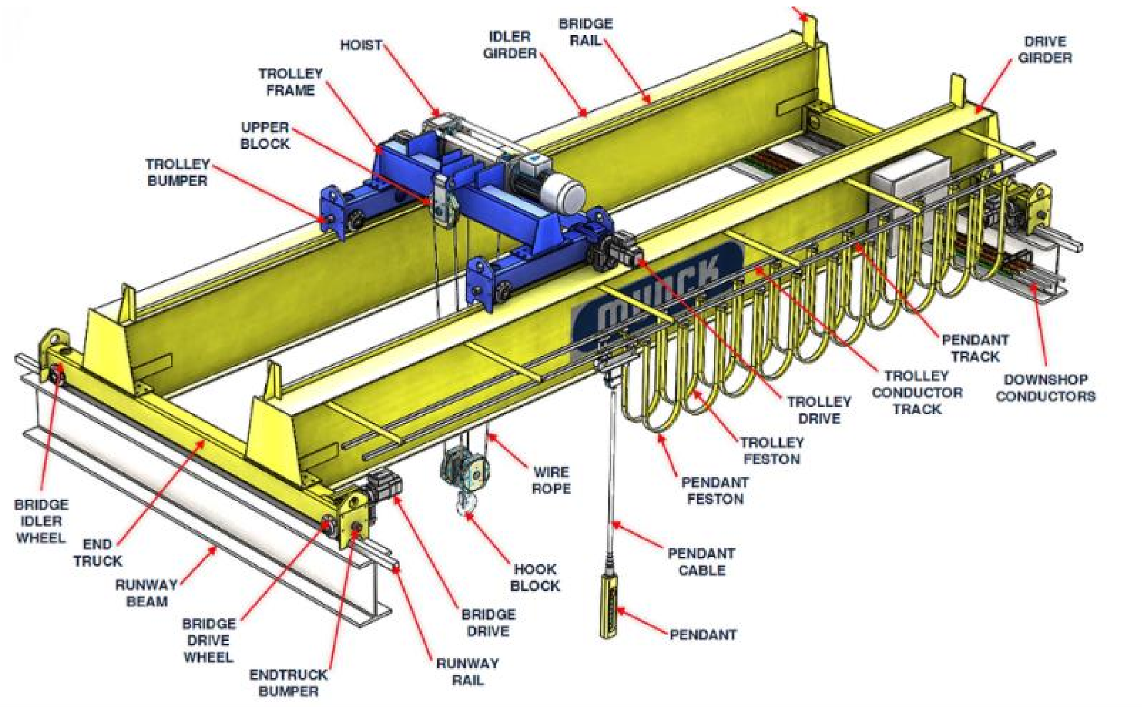


Figure 1.14. Parts of a bridge crane (https://www.munckcranes.com/crane-component-guide)

The parts which make up the structural elements of the bridge crane include the girders, trolley, hoist, end trucks and the bridge.

- **The Bridge:** The bridge is made up of either a single girder or a double girder (depending on the requirements and configuration) with a set of end trucks at both ends.

The bridge spans across the width of the bay and this travelling section is the main support structure of the overhead crane.

- **End Trucks:** Bridge crane end trucks are positioned on both sides of the span, where the bridge wheel assemblies are housed. This allows the whole crane to travel along the complete length of the bay. The wheel assemblies of the bridge end trucks travel along rails which are mounted to the runway beams or rails.
- **Girders:** The crane girders are connected to the end trucks and they are the structure by which the trolley is supported. Girders are the large horizontal beams and are considered to be the principal element of the bridge crane system.
- **Trolley Hoist:** This component is a combination of the hoist and the frame of the trolley. In dual hoist applications, two hoists can either be mounted to a single trolley frame or two trolley frames and can be manufactured each with independent hoists.
- **Trolley:** The bridge crane trolley rides across the span of the bridge along the girders and carries the hoist.
- **Hoist:** The hoist is designed to lift and lower the desired load and is fixed to the trolley frame using either a hook and/or custom lifting attachment to support the load. Two hoist models exist; an electric wire rope hoist or an electric chain hoist.

1.3.3. Types and features of bridge cranes

The bridge crane is made up of either a single girder or a double girder with a set of end trucks at both ends and includes two overhead runways built into the building's support structure. Bridge cranes have different configurations and can be comprised of one or two beams—more often referred to as a single girder or double girder bridge crane (Hernández, 1998). Girders can be made of rolled steel or can be fabricated by welding the beams into a steel box design for added strength and rigidity. The bridge is a load-bearing horizontal beam that runs the width of the crane bay and is the primary structural component that connects the runways and moves the hoist forward and backward using a trolley. There are two types of bridge cranes which are used in industries with many being highly specialized. They are the single girder bridge cranes and double girder bridge cranes,

1.3.3.1. Single girder bridge cranes

The crane consists of a single bridge girder supported on two end trucks. It has a trolley hoist mechanism that runs on the bottom flange of the bridge girder.

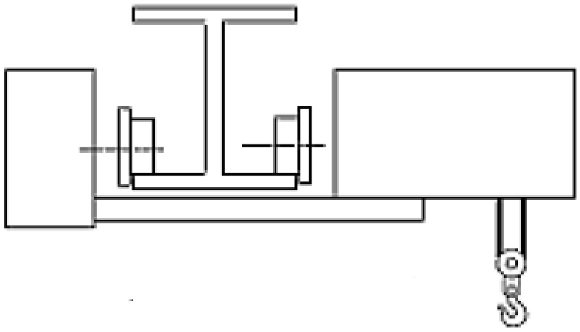


Figure 1.15. Section view of a single girder bridge crane (Treloar, 2011)

1.3.3.2. Double girder bridge cranes

The crane consists of two bridge girders supported on two end trucks (end carriages). The trolley runs on rails on the top of the bridge girders. Double girder bridge cranes are widely used in the industries because they can carry more loads with more span than any other type of crane. In this thesis, we will be concentrating mainly on double girder bridge crane.

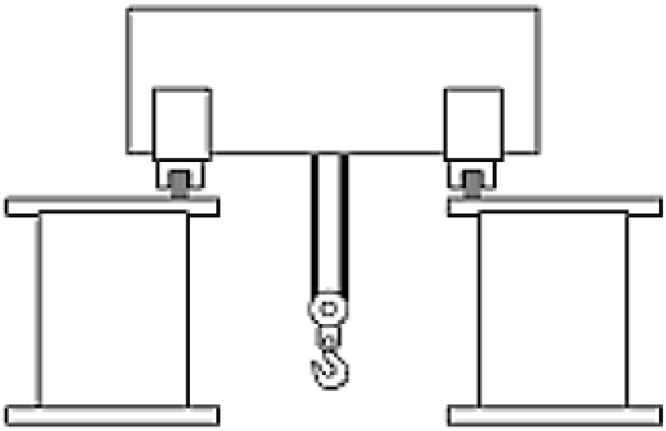


Figure 1.16. Section view of a double girder bridge crane (Treloar, 2011)

1.3.4. Bridge crane configurations

The trolley and hoist can be designed to be either top running or under running depending on the design of the building structure and the requirements needed to make the lift.

1.3.4.1. Top running bridge cranes

A top running bridge crane has a fixed rail system installed on the top of each runway beam allowing the end trucks to carry the bridge and hoist along the top of the runway system.

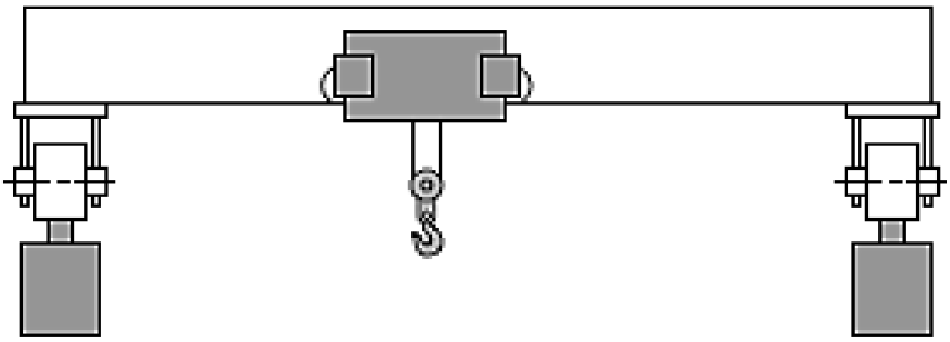


Figure 1.17. Single girder top running crane (Treloar, 2011)

Top running cranes can be configured in a single girder or double girder bridge design. These cranes are supported by the building structure, runway support columns, and are ideal for moving extremely heavy loads.

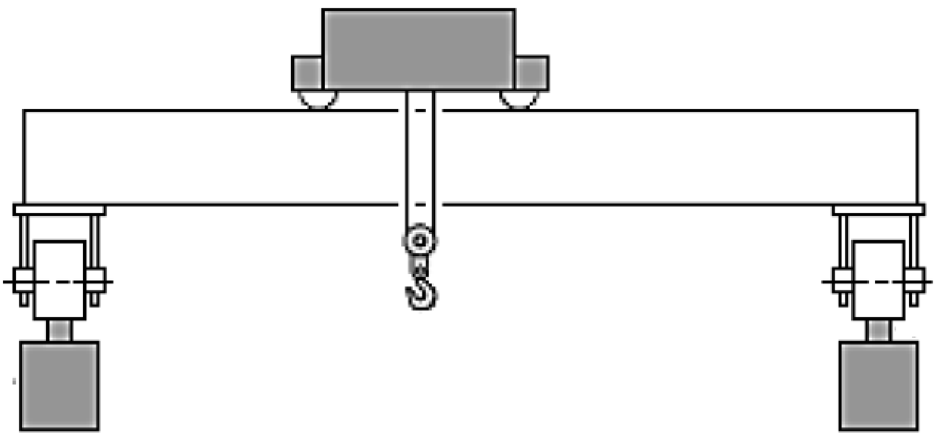


Figure 1.18. Double girder top running crane (Treloar, 2011)

1.3.4.2. Under running bridge cranes

An under running bridge crane, or commonly referred to as an "underhung" crane, uses wheels that are supported by the bottom flange of the runway beam to move the bridge up and down the runway. Under running bridge cranes are most commonly configured in a single girder design for lighter service and lower-capacity applications. They can also be built in a double girder design for higher capacities but it can become impractical and expensive to design.

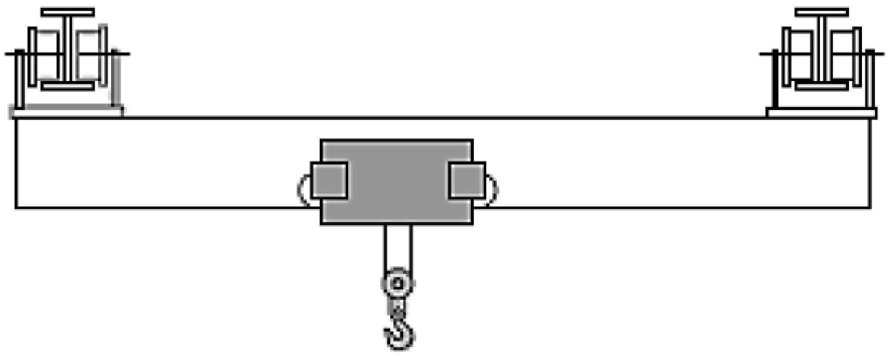


Figure 1.19. Single girder under running crane (Treloar, 2011)

1.4. Kinematics and mechanisms of bridge cranes

Kinematics describes the motion of points, bodies (objects), and systems of bodies (groups of objects) without considering the forces that cause them to move. Kinematics, as a field of study, is often referred to as the geometry of motion.

1.4.1. Kinematics of bridge cranes

The main axes of movement in a bridge crane are: Translation, direction, lifting and orientation. The first three movements are along three orthogonal axes and allow the hook or the gripping member to serve any point of the volume defined by the maximum deflection of the various movements. To obtain certain trajectories of the load, an additional degree of freedom is added to obtain the orientation.

1.4.1.1. Translation

The translation mechanism ensures the movement of the overhead crane on the runways. The axis of the rolling tracks (greatest distance) corresponds to an overall movement of the bridge. This movement is ensured by a motor controlling a transmission shaft connected to the rollers or by two or four synchronized motors each driving a roller.

1.4.1.2. Direction

The steering mechanism moves the hoist trolley(s) perpendicular to the direction of movement of the bridge (transverse axis).

1.4.1.3. Lifting

The lifting mechanism ensures the raising and lowering of the load in the vertical axis due to movement of the winch and therefore cables. It basically consists of a motor, a safety brake, a reduction gear, a drum for winding up the lifting cable and a chain nut for driving it.

1.4.1.4. Orientation

The orientation mechanism ensures the rotation of the load around a vertical axis. It can be integrated into the winch trolley, the gripping device (motorized rotation hook) or a lifting accessory.

1.4.2. Mechanisms of bridge cranes

The working mechanism of crane includes the lifting mechanism, operating mechanism and rotating mechanism.

1.4.2.1. Hoisting mechanism

This is a vertical lifting mechanism used to complete the material lifting. It is used for raising and lowering heavy loads to the desired height by using comparatively small efforts. It is mostly operated with a system of pulley and gears (either manually or mechanically) using power electric supply or fuel.

1.4.2.2. Operating mechanism

This is a mechanism that carries materials through the operation of a crane trolley. It can be divided into two types: the self-propelled type and the traction type according to the driving mode.

1.4.2.3. Rotation mechanism

This is a type of mechanism that swings objects around a vertical axis. The hanging beam adopts a cross structure which is reliable, safe and has a certain anti swing function.

1.4.3. Applications of bridge cranes

The basic function of bridge cranes is to lift heavy objects. Because of how bridge cranes are built, they can reliably lift objects that are very heavy in a way that is safer than many other lifting alternatives. They also often offer greater load capacity and lifting height. The bridge crane plays an important role in the following sectors.

1.4.3.1. Warehousing

The first industry that comes to mind when determining the need for a bridge crane is warehousing where supplies, equipment, and materials are constantly moved, positioned, and prepared. A major benefit of a bridge crane in warehousing is the movement of large items from storage to the shipping dock in a timely manner.

1.4.3.2. Assembly

Industries that assemble large equipment require a means for lifting and moving incomplete assemblies from one location to another for completion. Forklifts, Automated Guided Vehicles, and other forms of material handling are not adequate, which makes the use of a bridge crane a necessity. Assemblies can be moved easily as a natural part of the production process.

1.4.3.3. Transportation

The parameters that apply to warehousing are also true for transportation where heavy bulky products have to be loaded onto planes, trains and trucks. Bridge cranes can easily position large materials to be placed for transport. This can be especially true for overseas shipping where huge containers and materials have to be lowered into the holds of ships.

1.4.3.4. Equipment Repair

When the repair of a factory equipment is required, bridge cranes assist in facilitating the repair by gradually moving and positioning pieces for easy access. Bridge cranes can remove heavy equipment from their permanent location to a repair shop and then return them. Bridge cranes make it easier to lift machines up and other equipment to be placed at a repair station.

1.4.4. Design method of bridge cranes

An optimal design of the various components of a steel framework supposes the observation of the following elementary principles(BS EN1993-1-8, 2005):

- The standardization of elements (profile, steel grade) and assemblies (connecting parts, diameter, length and grade of bolts) as far as possible.
- The adoption of judicious solutions allowing to avoid as much as possible stiffenings or reinforcements at the level of the assemblies.
- The safe access for the achievement and inspection of welds.
- The development of an assembly procedure and the complete definition of the corresponding temporary stabilization system.
- By taking into account the impact of the rolling and manufacturing tolerances of the parts on their assembly by creating clearances at the level of the assemblies and by providing shims.
- By taking into account the transport jigs and the lifting means available on site in the definition of the sub-assemblies prefabricated at the factory.

1.4.5. Connections of bridge cranes to a building

There are several ways to integrate a bridge crane to a building as shown in figure 1.20.

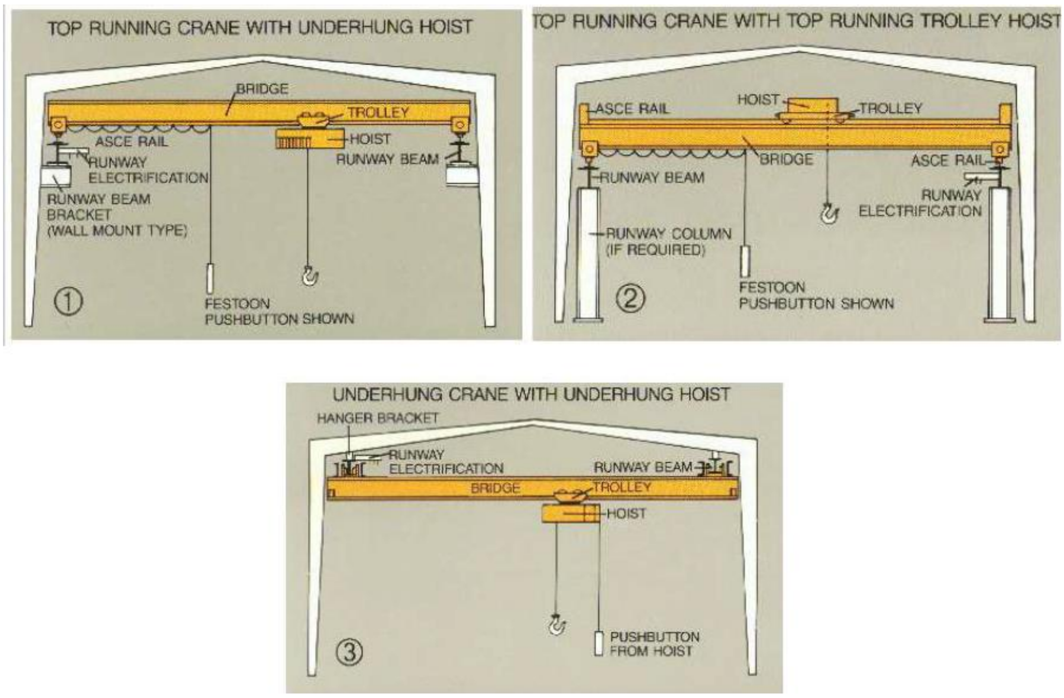


Figure 1.20. Crane with a hoist (https://www.munckcranes.com/crane-component-guide)

This can be done by:

- attaching the crane runways to the columns.

- installing separate columns (stanchions) that would support the bridge crane completely apart from the existing structure.
- attaching the runways to the roof.
- attaching the runways to the glubal beams.

1.5. Structural analysis

Structural analysis defines a sequence of procedures used to determine the effects of design actions on structures Several structural analyses can be performed on a structure or a structural element in order to design the latter. The steel structure and bridge crane are analysed differently with the first being discussed in section 1.5.1 and the second in section 1.5.2.

1.5.1. Steel structure

There are three different analysis methods which are global frame analysis, finite element method and design assisted by testing.

1.5.1.1. Global frame analysis

The global frame analysis determines the distribution of the internal forces and the corresponding deformations in a structure subjected to a specified loading. Global analysis of frames is conducted on a model based on many assumptions including those for the structural model, the geometric and material behaviour of the structure and of its members and the behaviour of the sections and of the joints. Once the analysis is completed, a number of design checks of the frame and its components (members and joints) must be performed. These checks depend on the type of analysis performed and the type of cross section verification (i.e., ultimate limit state criteria) used. The determination of the actual load-deformation response generally requires the use of a sophisticated analysis method. For practical purposes, assumptions for the frame and its component member and joint models are made that permit a safe bound for the ultimate load to be obtained. Hence models range from the simple linear elastic analysis or the rigid-plastic analysis to the most complex, elastoplastic analysis, which can provide a close representation of the real behaviour of the structure.

The methods for global analysis are: first-order elastic analysis, second-order elastic analysis, elastic-perfectly plastic analysis (Second-order theory), elastoplastic analysis (second-order theory), rigid-plastic analysis (first-order theory). According to EN1993-1-1 (CEN 2005a), the internal forces and displacements may be determined using either a global elastic analysis or a global plastic analysis. One important parameter in choosing the methods for global analysis is the cross section of the elements of the frame. EN1993-1-1 defines four classes of cross section as seen in Figure 1.21. The class into which a particular cross section falls depends upon the slenderness of each element (defined by a width-to thickness ratio) and the compressive stress distribution i.e., uniform or linear. The classes are defined in terms of performance requirements for resistance of bending moments as follows.

- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance;
- Class 2 cross-sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling;
- Class 3 cross-sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance;
- Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

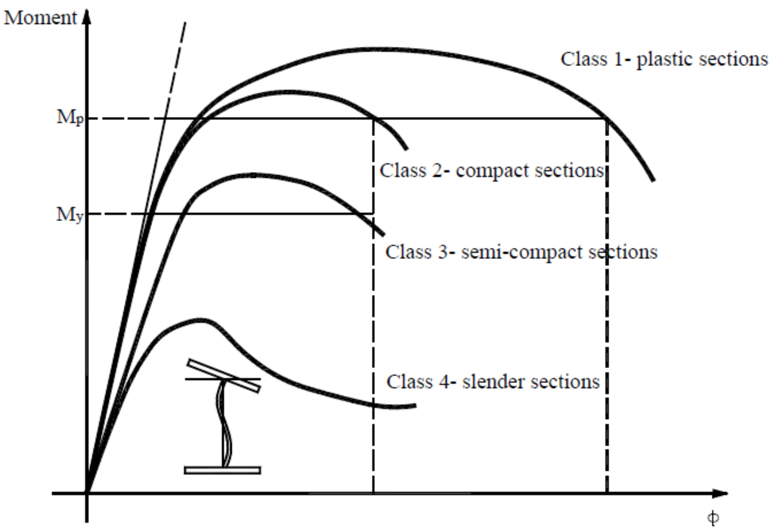


Figure 1.21. Cross section behavior classes (EN 1993 -1-3 :2006)

Thin-walled cold-formed steel members are usually made by sections of class 4 or, at most, of class 3. Compared with hot-rolled sections (of class1 or class 2). They are characterised by a reduced post-elastic strength and, as a consequence, by a reduced ductility (for example they do not have sufficient plastic rotation capacity to form plastic hinges. For this reason, only elastic global frame analyses are usually of interest. When a frame is analysed using an elastic method, the in-plane second-order effects can be allowed for by using: first-order analysis, “amplified sway moment method”; first-order analysis, “Iterative method”; or first-order analysis, with sway-mode buckling length.

1.5.1.2. Finite Element Analysis

The finite element method (FEM) is a popular method for numerically solving differential equations arising in engineering and mathematical modelling. The FEM is a general numerical method for solving partial differential equations in two or three space variables (i.e., some boundary value problems). To solve a problem, the FEM subdivides a large system into smaller, simpler parts that are called finite elements. This is achieved by a particular space discretization in the space dimensions, which is implemented by the construction of a mesh of the object: the numerical domain for the solution, which has a finite number of points. The finite element method formulation of a boundary value problem finally results in a system of algebraic equations. The method approximates the unknown function over the domain. The simple equations that model these finite elements are then assembled into a larger system of equations that models the entire problem. The FEM then approximates a solution by minimizing an associated error function via the calculus of variations.

1.5.1.3. Design assisted by testing

Under particular circumstances it may be favourable or necessary to carry out tests in order to obtain certain design parameters (Standard, 2002). Typical parameters determined from the tests are actions on the structure, resistance of the structure or structural component and material properties. Tests can be performed also to calibrate parameters in the theoretical model of resistance. The design value of the parameter is obtained from the test results as the estimated value of a certain fractile of the parameter in question. The procedures are explained for the determination of a single property and for the determination of a probabilistic model of resistance. The unknown quantities which are evaluated as a result of the tests may be actions on the structure.

The level of reliability of a structure designed by testing should be at least the same as for structures designed only by calculation models. The evaluation of test results should be based on statistical methods. Test results should in principle include probability distribution of unknown quantities, including the statistical uncertainties. This distribution is the base for obtaining the design values and partial factors. The classical statistical interpretation is possible if a large series of tests is performed, or a smaller series of tests is carried out in order to calibrate a model with one or more parameters. When only a small number of tests are performed, no classical statistical interpretation is possible. With the prior information about the distribution of the investigated quantities it is possible to interpret the test results as statistical using Bayesian procedures.

The design values for a material property, a model parameter or resistance should be derived from the tests either by (a) assessing the characteristic value and applying the appropriate partial and conversion factors, by (b) direct determination of the design value implicitly or explicitly accounting for reliability required and conversion of results. The derivation of the characteristic value should take into account the scatter of test data, statistical uncertainty associated with the number of tests and prior statistical knowledge. The partial factors should be taken from the appropriate Eurocode. The calculation model should take into account for differences between test arrangement and real behaviour.

1.5.2. Bridge crane

When designing bridge cranes, it is necessary to analyse the values of the maximum stresses occurring on the bridge crane. Precisely, in this thesis, the analysis of the stress conditions of the bridge crane in the steel factory in accordance with the standard of crane-supporting structures design has been done. The analysis of structural appearance in the main girder of the crane is carried out on the basis of the analytic solution of the software SAP 2000.

Before the analyses of the structural element and the structure load factors and combinations are being introduced. The loading adopted for the steel design is that specified in Eurocode 1: Actions on structures, and for bridge cranes situated in part 6 -EN 1991-6: Actions induced by cranes and machines BS National Annex's to Eurocode. Through the EN 1991-1-1 load combinations used in vehicle traffic areas. These fundamental combinations of actions are used to perform ULS safety verifications on shear and bending moment of cross-sections.

1.5.3. Structural analysis results

From a theoretical perspective, the primary goal of structural analysis is the computation of deformations, internal forces, stresses, support reactions, and stability. In practice, structural analysis reveals the structural performance of the engineering design and ensures the soundness of structural integrity in design without dependence on direct testing. In other words, these results are used to verify a structure's fitness for use.

1.5.4. Structural design

Structural design refers to a method or tool by which safe and economical specifications of a structure or a member of the structure sufficient to carry the load are found. That is, finding out cross-sectional dimensions, grade of material, amount of reinforcement, necessary to withstand the internal forces gotten from structural analysis.(Env, 2001)

1.5.5. Control and Maintenance of bridge cranes

The frequency and severity of accidents related to bridge cranes demonstrate to employers and workers the importance of working hand in gloves in order to make mechanical handling activities safer. Other than the safety of the structure itself, requirements for monitoring, operation, functional safety and maintenance of the bridge crane are key factors that must be considered as well.

1.5.5.1. Control of bridge cranes

Engineers and crane manufacturers have significantly improved overhead cranes—structurally and mechanically—over multiple decades. These improvements have made material handling easier, more productive, and ergonomic. Control systems are an essential part of a bridge cranes because they allow the crane operator to control the system from any location, in a safe and efficient manner. Although there are several control technology options available, AC power with inexpensive, effective ac-motors and Variable Frequency Drives (VFD) are perhaps the best option available today. VFD's have transformed the industry for supercharged AC crane controls, even exceeding the torque control and speed regulation abilities of DC shunt motor control. In fact, AC VFD provides enhanced performance and safety, and improves production by prolonging the life of the equipment, reducing maintenance costs, and allowing crane operators to do their job safely and effectively. Safety is built into modern VFD, including features that prevent operators from lifting loads over capacity, minimize load swing, prevent motor overheating, and limit unsafe programming modifications. They minimize the shock effect on the crane and the load by reducing high starting currents characteristic of AC induction motors.

1.5.5.2. Maintenance of bridge cranes

The greatest risk about a bridge crane structure is the potential hazard and danger that this can bring to the various users. Other than the safety of the structure itself, requirements for monitoring, operation, functional safety and maintenance of the bridge and connection with the surroundings are factors that must be considered as well. As there are many parts of the bridge crane, for different technical characteristics of the various components, in the actual work they will generally be maintained into periodic check in weeks. For this the following should be checked:

- The nut, cotter pins should be checked, the positioning plate on the brakes whether it is complete, lose or not, lever and spring is free of cracks, pin bolts on the wheel and the brake washer is loose cushion, or complete; if it is reliable when the brake is open.
- Checking the rope on the winding drum and the pulley is normal or not, if there is any phenomenon such as jumping from slot, channeling, knotting, and twisting.
- Checking if the installation of the safety switch and limit switch are accurate, flexible and reliable, especially the rise limit.
- Checking all lubrication points is in a good condition.
- Checking the track to confirm that no foreign body obstruct running of the bridge crane.
- If there is any abnormal noise on the transmission of the bridge crane.

Conclusion

The objective of this chapter was to present a review of the different key concepts and principles necessary to understand the problem outlined in the thesis. Thus industrial steel structures with a bridge crane, leaving from the raw materials to the prevention of defects passing by the characteristics, types and uses were explained. After that was discussed the principal constituents of an industrial steel building and bridge crane and how they are connected to each other. Afterwards were the types of loads been applied with an emphasis of moving loads in relation to bridge cranes. with the design methods. Lastly were the control and maintenance of a bridge crane. Given that an optimal design for a building passes through the choice of an adequate structural system, and our thesis being about the analysis of an industrial steel structure when constructed with and without a bridge crane, the next step will be the design of our different models. The next chapter will present the methodology used to reach our goal.

CHAPTER 2: METHODOLOGY

Introduction

The methodology is the part of the study that deals with the establishment of the research procedure adopted after the definition of the problem so as to achieve the set objectives. After the description of the case study and the listing of the norms used for the design, the procedures for the design of the structural elements, bridge crane and the connections in compliance with the Eurocode norms will be presented in order to perform a linear static analysis. The comparison criteria between the two steel structures will close this chapter.

2.1. General site recognition

The recognition of the site was done from documentary research whose essential objective is to know the physical parameters of the site of the case study, the location of the site, the climate, the relief, the population, the hydrology and the economic activities of the site.

2.2. Site visit

The site visit consists essentially of the physical description and observation of the case study. This led to a collection of data.

2.3. Data collection

The main data collected were the architectural plans. These plans include the floor plan views and elevation views which show the geometry and specific use of the level of the building.

2.4. Determination of actions and combination of actions

This is based on a number of standards. Depending on the geographical area, different standards are used for the design of geotechnical and structural structures. Thus, in the European countries the standards used are Eurocodes recommended by the European Committee for Standardization. In this study, the codes that will be used are listed as follows:

- EN 1990 - Eurocode 0: Basis of Structural Design
- EN 1991 - Eurocode 1: Actions on structures

- EN 1992 - Eurocode 2: Design of concrete structures
- EN 1993 - Eurocode 3: Design of steel structures

2.4.1. Actions

These standards define the different types of loads on the building. They belong to one of two broad categories: permanent loads, and variable loads.

2.4.1.1. Permanent loads

A permanent action, is an action that is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction until the action attains a certain limit value. In EN 1990, examples of the permanent actions are given based on this definition. These are the self-weight of structural elements and the self-weight of non-structural elements.

2.4.1.2. Variable loads

A variable action is one for which the variation in magnitude with time is neither negligible nor monotonic. These are the most difficult to quantify as their effect are variable in time and space. Examples of some live loads include furniture, equipment, and occupants of buildings.

a. Imposed loads

Depending on the occupancy of a building, EC1 prescribes imposed actions on the building arising from the occupancy of the building considered. With respect to these category, surface and concentrated actions are assigned as seen in Table 2.1.

Table 2.1. Categorization of roofs.

Categories of loaded area	Specific Use
H	Roofs not accessible except for normal maintenance and repair.
I	Roofs accessible with occupancy according to categories A to D
K	Roofs accessible for special services, such as helicopter landing areas

b. Lateral loads

Lateral loads are live loads that are applied parallel to the ground; that is, they are horizontal forces acting on a structure. These are observed during the acceleration and braking of the bridge crane. These forces are going to be calculated below.

c. Wind loads

An important issue is to bring out an analytical expression for the wind action on a given surface. The wind actions on a structure are functions of the building's dimensions and intrinsic properties of the wind. The former includes length, width and height of the buildings while the latter includes wind's speed, terrain orography and topography. The effect of wind on the structure as a whole is determined by the combined action of external and internal pressures acting upon it. In all cases, the calculated wind loads act normal to the surface to which they apply. The pressures created inside a building due to access of wind through openings could be suction (negative) or pressure (positive) of the same order of intensity while those outsides may also vary in magnitude with possible reversals. Thus, the design value shall be taken as the algebraic sum of the two in appropriate or concerned direction. The wind pressure on our structure is calculated using the equations provided in EN1991-1-4.

i. Basic wind velocity

The basic wind velocity, v_b, shall be calculated using Equation (2.1).

$$v_b = c_{dir} \times c_{season} \times v_{b,0} \tag{2.1}$$

Where:

c_{dir} and c_{season} are respectively the directional and season factors. EN 1991-1-4 recommends these values to be taken as 1.0;

$v_{b,0}$ is the fundamental value of the basic wind velocity, taken as $22.0m/s$.

ii. Peak and basic velocity pressure

The basic velocity pressure, q_b, shall be calculated using Equation (2.2).

$$q_b = \frac{1}{2} \times \rho_{air} \times v_b^2 \tag{2.2}$$

Where:

$\rho_{air} = 1.25 kg/m^3$ (air density).

The peak velocity pressure at height z, $q_p(z)$, which includes mean and short-term velocity fluctuations, was determined through.

$$q_p(z) = [1 + 7 \times l_v(z)] \times \frac{1}{2} \times \rho \times v_m^2(z) = c_e(z) \times q_b \qquad (2.3)$$

Where:

$v_m(z)$ is the mean wind velocity;

$$v_m(z) = c_r(z) \times c_0(z) \times v_b \qquad (2.4)$$

$c_0(z)$ is the orography factor, recommended to be taken as 1.0;

$c_r(z)$ is the roughness factor;

$$c_r(z) = k_r \times \ln\frac{z}{z_0} \quad \text{for} \quad z_{min} \leq z \leq z_{max} \qquad (2.5)$$

$$c_r(z) = c_r(z_{min}) \quad \text{for} \quad z \leq z_{min} \qquad (2.6)$$

z_0 is the roughness length;

z_{min} is the minimum height;

k_r is the terrain factor, depending on the roughness length z_0;

$$k_T = 0.19 \times (\frac{z_0}{z_{0,II}})^{0.07} \qquad (2.7)$$

$z_{0,II} = 0.05$ (terrain category II);

z_{max} is to be taken as $200\ m$;

l_v is the turbulence intensity;

$$l_v = \frac{k_l}{c_0(z) \times \ln(\frac{z}{z_0})} \quad \text{for} \quad z_{min} \leq z \leq z_{max} \qquad (2.8)$$

k_l is the turbulence factor and the recommended value is 1.0;

$$l_v = l_v(z_{min}) \quad \text{for} \quad z < z_{min} \qquad (2.9)$$

$c_e(z)$ is the exposure factor, function of height above terrain and the terrain category. The different values of $c_e(z)$ are shown in Figure 2.1 and the terrain categories in Table 2.2.

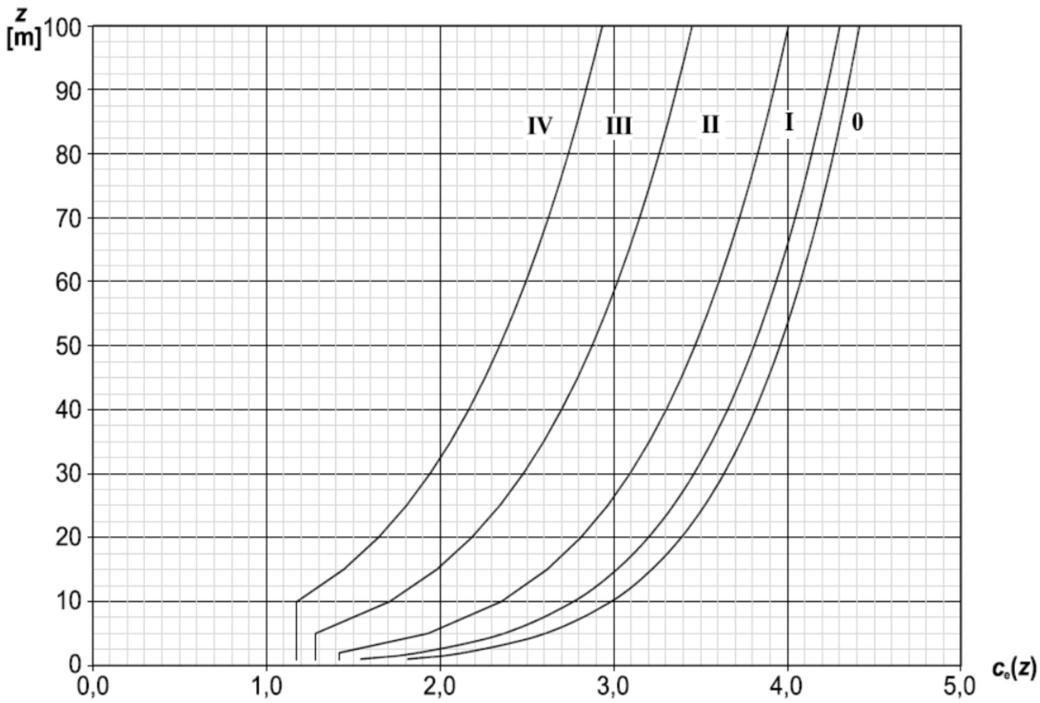


Figure 2.1. Values of the exposure factor Ce (z) for c_0=1.0 and k_1=1.0 (EN 1991-1-4, 2005)

Table 2.2. Terrain categories and terrain parameters (EN 1991-1-4, 2005)

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10

iii. Wind pressure on surfaces

A positive wind load stands for pressure while a negative wind load indicates suction on the surface. This definition applies for the external wind action as well as for the internal wind action. The pressure distribution is shown in Figure 2. 2.

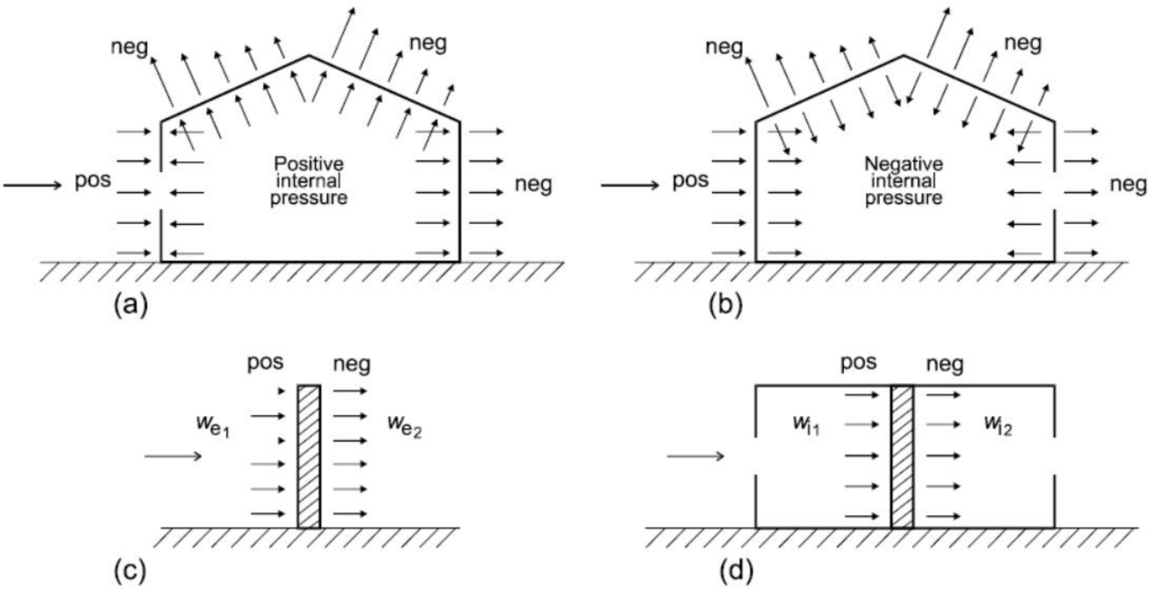


Figure 2.2. Pressure on surface (EN 1991-1-4, 2005)

The wind pressure acting on the external surfaces, w_e, should be obtained from Equation (2.10).

$$w_e = q_p(z_e) \times c_{pe} \tag{2.10}$$

Where:

$q_p(z_e)$ is the peak velocity pressure;

z_e is the reference height for the external pressure;

c_{pe} is the pressure coefficient for the external pressure.

The wind pressure acting on the internal surfaces of a structure, w_i, is given by Equation (2.11).

$$w_i = q_p(z_i) \times c_{pi} \tag{2.11}$$

Where:

$q_p(z_i)$ is the peak velocity pressure;

z_i is the reference height for the internal pressure;

c_{pi} is the pressure coefficient for the internal pressure.

iv. Wind force

The wind force, F_w, acting on a structure or a structural component, should be determined given Equation (2.12).

$$F_w = c_s c_d \times c_f \times q_p(z_e) \times A_{ref} \tag{2.12}$$

Where:

$c_s c_d$ is the structural factor, which value is recommended to be 1.0;

c_f is the force coefficient, taken as 1.6;

A_{ref} is the reference area of the structure or structural element;

2.4.2. Combination of Actions

A combination of actions consists of a set of load values applied to the structure simultaneously to verify its structural reliability for a given limit state (design limit states). The structure will be designed according to the corresponding limit states in such a way as to sustain all actions acting upon it during its intended life. This implies it will be designed having adequate structural stability (ultimate limit states) and remain fit for the use it is required (serviceability limit states).

2.4.2.1. Load combination at ultimate limit states

Ultimate limit state (ULS) corresponds to the loss of structural capacity of the whole structure or one of its fundamental elements (for example structural collapse). The loss of structural capacity includes loss of equilibrium of the whole structure or one of its fundamental parts, excessive displacements or deformations, reaching of the maximum strength capacity of parts of structures, joints, foundations, reaching of the maximum strength capacity of the entire structure, reaching of failure mechanisms in the soils and fatigue failure. It concerns the safety of people and/or the safety of the structure. The loss of structural capacity includes:

- Loss of equilibrium of the whole structure or one of its fundamental parts;
- Excessive displacements or deformations;
- Reaching of the maximum strength capacity of parts of structures, joints, foundations;
- Reaching of the maximum strength capacity of the entire structure;
- Reaching of failure mechanisms in the soils.

For ULS, the fundamental formulation is given in Equation (2.13).

$$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i\geq 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \tag{2.13}$$

2.4.2.2. Load combination at serviceability limit states

Serviceability limit state (SLS) is the inability of the structure to meet the specify service requirement. This includes mainly the functioning of the structure or structural members under normal use and comfort of people. This includes mainly:

- Functioning of the structure or structural members under normal use;
- Comfort of people
- The appearance of the construction works.

For SLS, the combinations include characteristic (rare), frequent and quasi-permanent combinations, which are represented respectively in Equations (2.14), (2.15) and (2.16).

$$\sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i\geq 1} \Psi_{0,i} Q_{k,i} \tag{2.14}$$

$$\sum_{j\geq 1} G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \sum_{i\geq 1} \Psi_{2,i} Q_{k,i} \tag{2.15}$$

$$\sum_{j\geq 1} G_{k,j} + P + \sum_{i\geq 1} \Psi_{2,i} Q_{k,i} \tag{2.16}$$

Where:

G_k are the permanent loads;

Q_k are the live loads;

$\Psi_{0,i}$ are the combination coefficients;

γ is the safety factor for permanent and variable loads;

2.5. Structural analysis and design

Structural analysis is a set of processes used to assess how design actions affect structures. The analysis type used for the assessment of the steel structure and the bridge crane is the linear static analysis. In this type of analysis, a linear relation exists between applied forces and displacements, which is applicable to structural problems where stresses remain in the linear elastic range of the used material. To facilitate structural analysis, numerical finite element analysis programs equipped with the various design and verification equations provided in the section will be used. The structure will be designed according to the corresponding limit states in such a way to sustain all actions acting upon it during its intended service life. This indicates that it will be designed with appropriate structural stability (ultimate limit states) and will continue to be fit for the purpose for which it is designed (serviceability). Before designing any element, it must be classified based on its ability to generate plastic hinges and rotational deformations.

The modelling and numerical analysis of our building will be performed using the software SAP2000 v23.2.0 which is a structural design software applying the finite elements method, and especially dedicated to the analysis of the stability and the resistance of structures.

Two nearly similar steel structures are modelled, one without a bridge crane and the second with a bridge crane are presented in section 3.3. Both structures are submitted to similar loading conditions (apart from the moving loads for the steel structure with the bridge crane) and adjusted to satisfy the structural and functional requirements. Modelling will consist of creating the appropriate material, section properties, loads and combinations. The steel elements shall be drawn according to plan and the supports conditions assigned to be fixed. The structure shall be loaded with respect to specific load patterns discussed in section 2.4.1. The load combinations will be defined prior to the analyses to satisfy the ULS and SLS conditions discussed in sections 2.5.2 and 2.5.3 respectively.

Considering the moving loads on the bridge crane, influence line diagrams are done to obtain stresses of maximum shear, bending moment and axial force to enable the design and verification of structural members.

2.5.1. Design of structural members

The sheet metal, purlin and gantry are going to be designed as follows:

2.5.1.1. Design of sheet metal

The sheet metal received all the actions. To design it we had to consider all the loads applied. The combination of load that will be used is the one which is the most unfavourable for the structure.

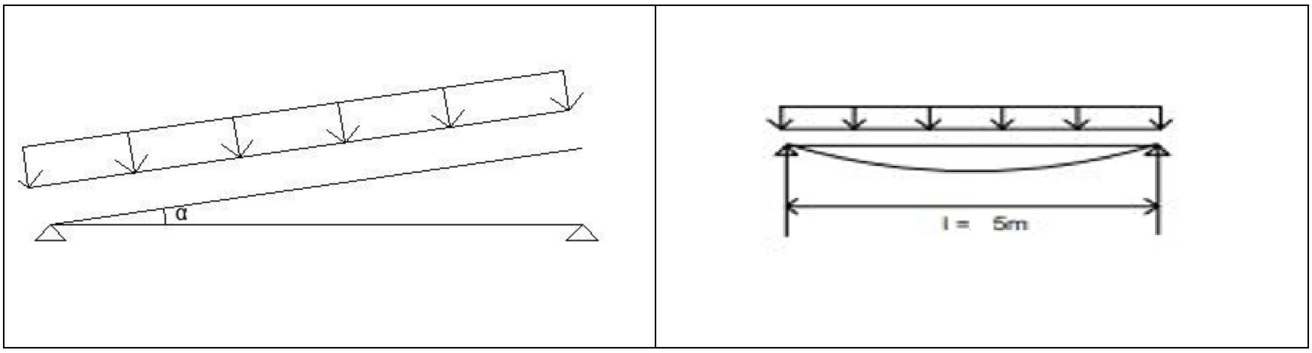


Figure 2.3. Static scheme of a sheet metal

The design moment is gotten from Equation 2.17.

$$Med = \frac{1}{8}Pl^2 \quad (2.17)$$

$$Med < Mrd = Wel*fs \quad (2.18)$$

$$\Rightarrow Wel > \frac{Med}{fs}$$

$$\Rightarrow \frac{bt^2}{6} > \frac{Med}{fs}$$

$$\Rightarrow t > \sqrt{\frac{6\,Med}{b*fs}} \quad (2.19)$$

Where $f_s = \frac{fy}{\gamma M0}$

Having t, the suitable metal sheet to be used can be found.

2.5.1.2. Design of purlin

After having the loads P acting on the purlin it is divided in two parts as shown in Figure 2.4.

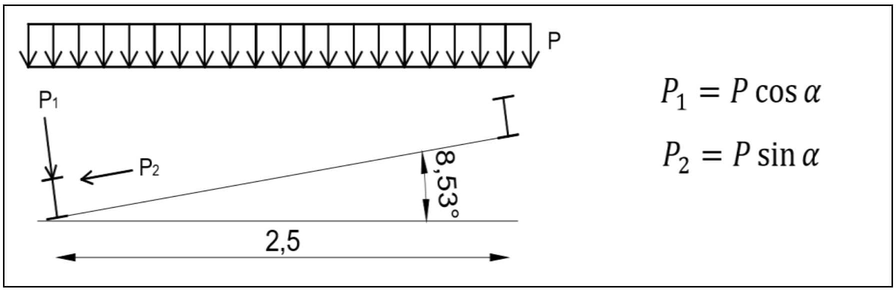


Figure 2.4. Wind load on the roof

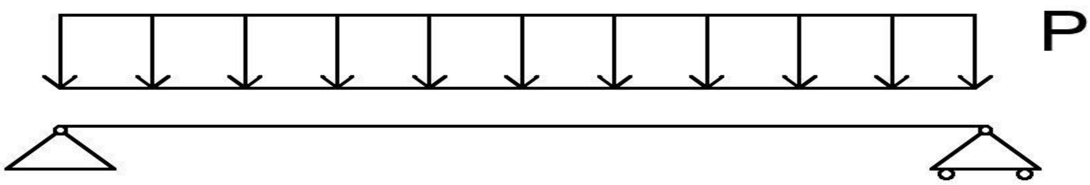


Figure 2.5. Static scheme use to design the purlin

The purlin is designed as a beam on two supports. The maximum bending moment are

My= $\frac{P1}{8}\, l^2$ and Mz = $\frac{P2}{8}\, l^2$

With the different bending moment, the plastic modulus of the good section is obtained using Equation 2.20.

$$Wpl = \frac{Mpl{,}rd*\gamma M0}{fy} \geq \frac{Med*\gamma M0}{fy} \tag{2.20}$$

Where:

M_{ed}= My or Mz

Fy is the steel grade

2.5.1.3 Design of gantry

The gantry, which constitutes the main frame of the buildings, are composed of rafters, which support the purlins, and columns which support the rafters.

a. Rafter

A rafter is a structural component that is used as part of a roof construction. Typically, it runs from the ridge or hip of the roof to the wall plate of the external wall. Rafters are generally laid in series, side by side, providing a base to support roof decks, roof coverings and so on. The loads applied on the rafter are the reactions of the two purlins. The static scheme retained for designing the rafter is shown in Figure 2.6.

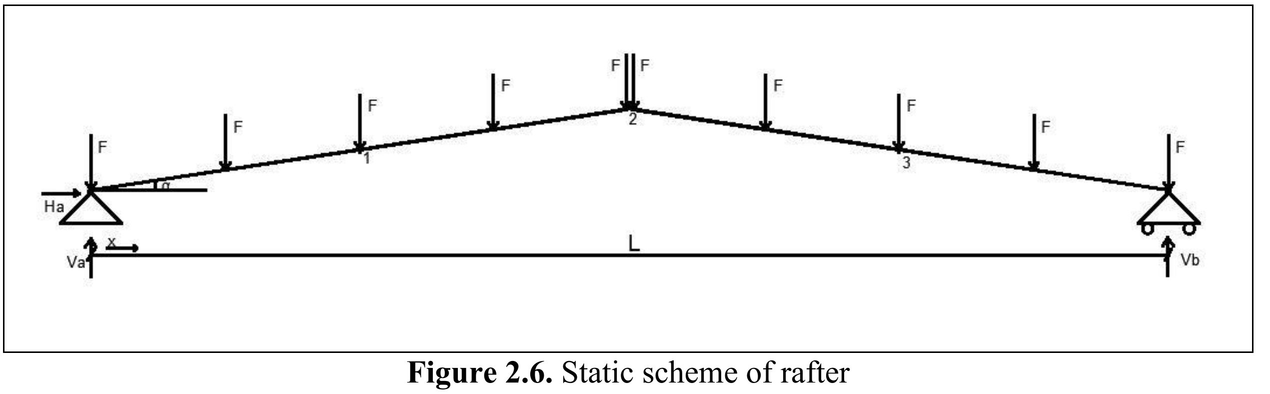


Figure 2.6. Static scheme of rafter

Then moment and reactions on the rafter are found using Equation

$$M_{max} = V_a * \frac{L}{2} - 2.61*FL \tag{2.21}$$

$$V_a = V_b = 5F \tag{2.22}$$

To find the section, the equation used is:

$$M_{pl,rd} > M_{ed} \tag{2.23}$$

$$\Rightarrow \frac{Mpl,rd*\gamma M0}{fy} \geq \frac{Med*\gamma M0}{fy}$$

$$\Rightarrow Wpl \geq \frac{Med*\gamma M0}{fy} \tag{2.24}$$

With the plastic modulus obtained the equivalent section can be deduced.

b. Column

The columns are stressed in compression with bending. This is why the buckling calculation must be made. It should be noted that the profile adopted for the column cannot be smaller than the one of the rafters.

2.5.1.4. Bracing of the roof

The force that is considered to design these members provide to the longitudinal pressure of wind on the building and the imperfection of the brace. The Figure 2.7 illustrates the static scheme adopted for the roof brace.

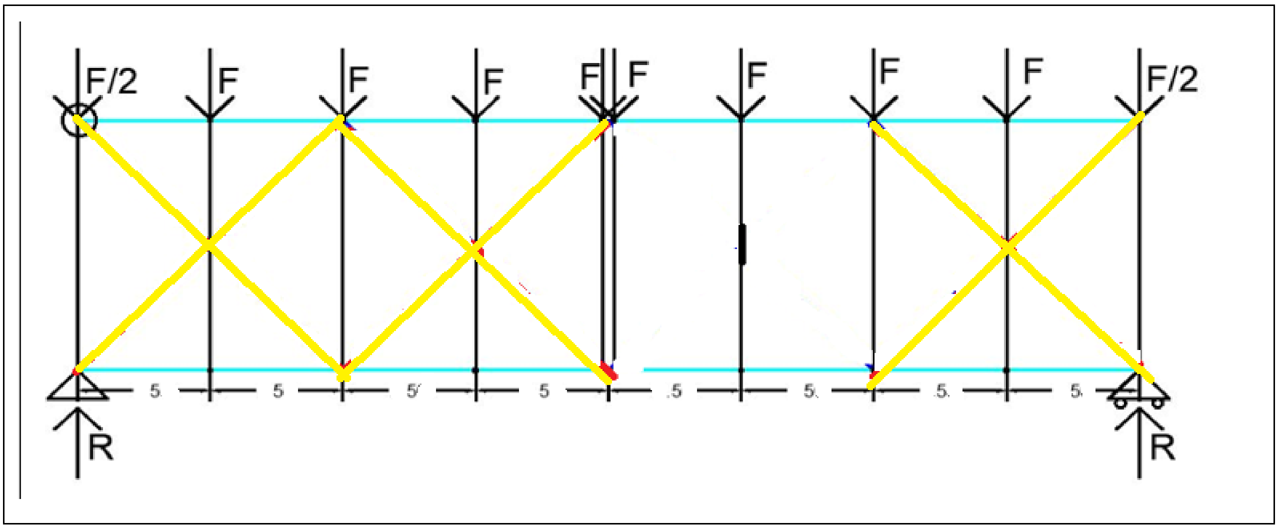


Figure 2.7. Distribution of wind load considered in the longitudinal direction

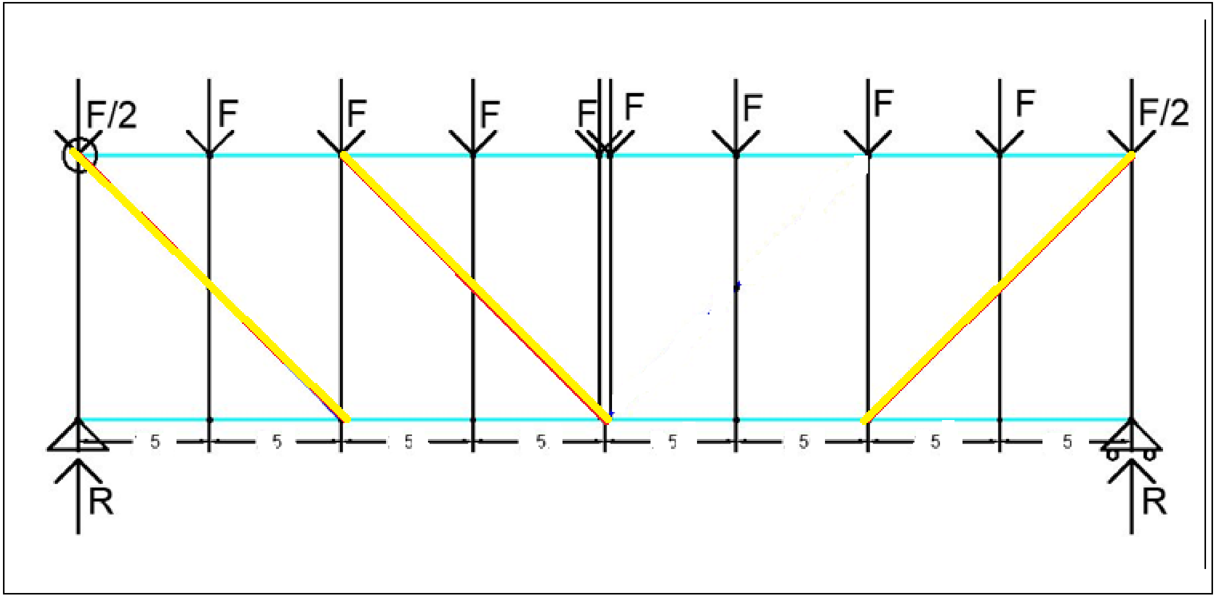


Figure 2.8. Static scheme of roof member with only traction member

The force on the brace is F = Fw + Fi

Where Fw is the force due to wind pressure while Fi comes from the imperfections of the brace.

To calculate the action of the wind the longitudinal direction and the windward wall is considered. The area covered is the multiplication of these two distances. Half of the wind action is absorbed by the foundation and the other half by the brace.

Having that, Fw = Q_w (windward side) *A

It is considered an equivalent geometric imperfection of the members to be constrained with an initial deflection.

$$e0 = \frac{\alpha_m L}{500} \text{ where } \alpha_m = \sqrt{0.5(1+\frac{1}{m})} \qquad (2.25)$$

$$qd = \Sigma Ned*8*\frac{e_0+\delta_q}{L^2} \qquad (2.26)$$

Where qd is the distributed equivalent load.

The deformation δ_q due to the wind (estimated as less than 1mm) is neglected and ΣNed is considered as the maximum compressive stress on the rafter.

Finally, F is gotten using Equation 2.27 and the axial tension solicitation S_1 is found using the equilibrium of forces.

$$Fi = qd * 2.5 \qquad (2.27)$$

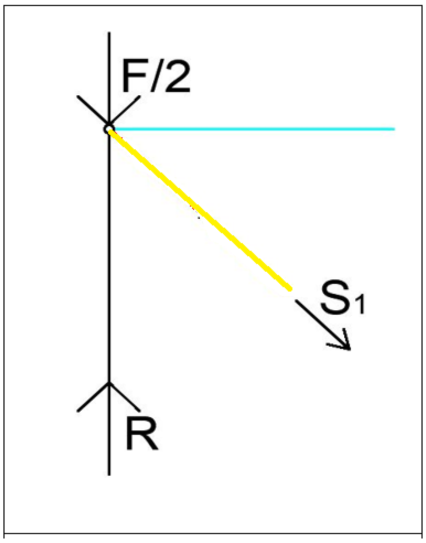


Figure 2.9. Detail of load distribution

Where R is the resultant of longitudinal forces

The section of the brace is gotten using Equations (2.28) and (2.29).

$$A = \frac{Ned*\gamma M0}{fy} \quad (2.28)$$

$$Anet = \frac{Ned*\gamma M2}{0.9*fy} \quad (2.29)$$

where Ned = S_1

2.5.2. Ultimate limit states design verification of members

Before any element is designed, it needs to be classified according to its capacity to develop plastic hinges and rotation deformations. The design verifications will be done with respect to EC3 1-1, 2005.

2.5.2.1. Classification of sections

The sections of the members to design are going to be classified as class 1, 2, 3, or 4 as depicted in Annex B. The design will consist of verifying the following in each of the members.

2.5.2.2. Members in bending

The columns will be designed as members in bending. The design procedure is given in this section. The ULS design verification procedure of the members in bending will take into considerations uniaxial bending, shear resistance and lateral torsional buckling.

a. Uniaxial bending

For uniaxial bending the required check will be given by:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1{,}0 \tag{2.30}$$

Where M_{Ed} and $M_{c,Rd}$ are respectively the design and resisting moments.

For class 1 or 2:

$$M_{c,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} \tag{2.31}$$

For class 3:

$$M_{c,Rd} = \frac{W_{el} f_y}{\gamma_{M0}} \tag{2.32}$$

For class 4:

$$M_{c,Rd} = \frac{W_{eff} f_y}{\gamma_{M1}} \tag{2.33}$$

Where:

f_y is the yielding strength;

W_{pl} is the plastic section modulus;

W_{el} is the elastic section modulus;

W_{eff} is the effective section modulus;

b. Shear resistance

The plastic shear resistance is given by:

$$V_{pl,Rd} = \frac{A_v \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} \tag{2.34}$$

Where A_v is the shear area that can be calculated as;

a) rolled I and H sections, load parallel to web	$A - 2bt_f + (t_w + 2r)t_f$
b) rolled channel sections, load parallel to web	$A - 2bt_f + (t_w + r)t_f$

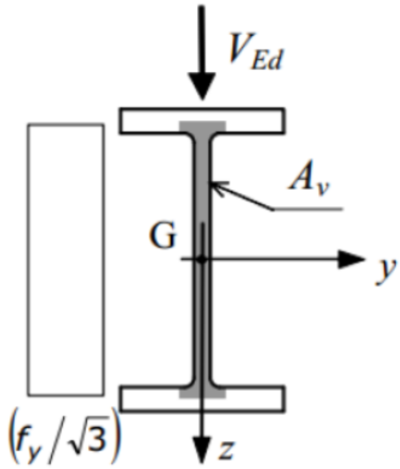


Figure 2.10. Beam shear resisting area

For webs with no stiffeners shear buckling will be verified as:

$$\frac{h_w}{t_w} > 72\frac{\varepsilon}{\eta} \tag{2.35}$$

Where:

h_w is the height of the web;

t_w is the thickness of the web;

$$\varepsilon = \sqrt{235}/f_y \tag{2.36}$$

c. Bending and shear interaction

In case of a bending moment - shear force interaction, Equation (2.31) for bending moment resistance may be modified when $V_{Ed} \geq 0.5V_{pl,Rd}$, with the introduction of a reduction factor ρ, to obtain Equation (2.37).

$$M_{pl,Rd} = \left(\frac{W_{pl} - \frac{\rho A_w^2}{4t_w}}{\gamma_{M0}}\right) f_y \tag{2.37}$$

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1\right)^2 \tag{2.38}$$

d. Lateral Torsional Buckling verification

For lateral torsional buckling it must be verified that:

$$\frac{M_{Ed}}{M_{b,Rd}} \leq 1{,}0 \tag{2.39}$$

$$M_{b,Rd} = \frac{\chi_{LT} W_y f_y}{\gamma_{M0}} \tag{2.40}$$

Where:

W_y is defined by the class (1, 2, 3 or 4);

χ_{LT} is the reduction factor for lateral-torsional buckling;

$$\chi_{LT} = \frac{1}{\Phi_{LT} + ({\Phi_{LT}}^2 - \beta{\bar{\lambda}_{LT}}^2)^{0,5}} \tag{2.41}$$

$$\Phi_{LT} = 0{,}5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta{\bar{\lambda}_{LT}}^2] \tag{2.42}$$

α_{LT} is the imperfection factor for buckling;

$$\bar{\lambda}_{LT} = \frac{1}{\sqrt{C_1}} 0{,}9\bar{\lambda}_z\sqrt{\beta_w} \tag{2.43}$$

$\bar{\lambda}_z$ is the non-dimensional slenderness;

$$\bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} \tag{2.44}$$

λ_z is the slenderness;

$$\lambda_z = \frac{L_{EZ}}{i_z} \tag{2.45}$$

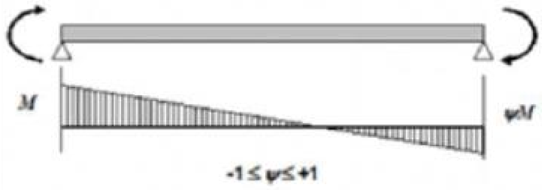
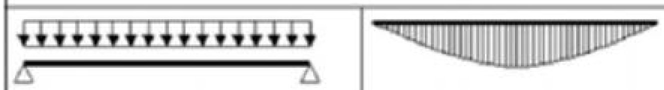
L_{EZ} is the buckling length and i_z is the radius of gyration with respect with to the Z-axis;

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} \tag{2.46}$$

E is the Young modulus;

C_1 is to be determined from Table 2.3.

Table 2.3. Values of C_1 (BS EN 1993 1-1, 2005)

End Moment Loading	ψ	$\frac{1}{\sqrt{C_1}}$	C_1
	+1.00 +0.75 +0.50 +0.25 0.00 -0.25 -0.50 -0.75 -1.00	1.00 0.92 0.86 0.80 0.75 0.71 0.67 0.63 0.60	1.00 1.17 1.36 1.56 1.77 2.00 2.24 2.49 2.76
Intermediate Transverse Loading			
		0.94	1.13
		0.62	2.60
		0.86	1.35
		0.77	1.69

2.5.2.3. Members in compression

This concerns column members. For members in compression, Equation (2.47) must be checked.

$$\frac{N_{ED}}{N_{C,Rd}} \leq 1 \qquad (2.47)$$

$$N_{c,Rd} = \frac{Af_y}{\gamma_{M0}} \qquad (2.48)$$

Buckling resistance will be verified using Equation (2.49).

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1 \qquad (2.49)$$

For class 1, 2 or 3:

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \qquad (2.50)$$

For class 4:

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \tag{2.51}$$

Where:

χ is the reduction factor for the relevant buckling mode;

$$\chi = \frac{1}{\Phi^2 + (\Phi^2 - \bar{\lambda})^{0.5}} \tag{2.52}$$

$$\Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right] \tag{2.53}$$

$\bar{\lambda}$ is the non-dimensional slenderness around the z or y axis.

For class 1,2 or 3:

$$\bar{\lambda} = \frac{L_{cr}}{i}\frac{1}{\lambda_1} \tag{2.54}$$

For class 4:

$$\bar{\lambda} = \frac{L_{cr}}{i\lambda_1}\sqrt{\frac{A_{eff}}{A}} \tag{2.55}$$

Where:

α is the imperfection factor;

Ncr is the elastic critical load (Euler's critical load) for the relevant buckling mode;

Lcr is the length of the corresponding buckling mode;

i is the radius of gyration of the cross section;

λ1 = π (*E*/ *fy*)0.5 = 93.9ε

ε = (235 /*fy*)0.5

The imperfection factor α and the associated buckling curve to be adopted in design of a given member depends on the geometry of the cross sections, on the steel grade, on the fabrication process and on the relevant buckling plane.

2.5.2.4. Members in tension

This includes members like braces. For members in axial tension, the design value of the tensile force N_{Ed} at each cross-section shall satisfy Equation (2.56).

$$N_{Ed} \leq N_{t,Rd} \tag{2.56}$$

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{2.57}$$

$$N_{u,Rd} = 0.9\frac{A_{net}f_u}{\gamma_{M2}} \tag{2.58}$$

Where:

$N_{t,Rd} = min\{N_{pl,Rd}; N_{u,Rd}\}$

A_{net} is the net cross-section area

γ_{M2} is the safety coefficient, whose value is to be taken as 1.25.

2.5.2.5. Design of Assembly

The connection between the structural members are done using bolted connections and they are going to be analysed at ULS based on EN1993-1-8 (2005). The connection is a bearing type bolted connection using non-preloaded bolts.

a. Beam-column connection

The beam to column connection presented in this work is an eave moment connection connecting a rafter with a column since the building is made up of a portal frame with eaves haunches (Vayas et al., 2019).

i. Shear resistance of the bolts

The shear resistance of the bolts is going to be verified according to equations (2.59) and (2.60).

$$F_{v,Rd} = 0.6 F_{ub} \frac{A_b}{\gamma_{M2}} \quad \text{for class 4.6, 5.6 and 8.8} \tag{2.59}$$

$$F_{v,Rd} = 0.5 F_{ub} \frac{A_b}{\gamma_{M2}} \quad \text{for class 4.8, 5.8, 6.8 and 10.9} \tag{2.60}$$

Where:

Ab is the cross-sectional area of the bolt at the shear plane determined by equation (2.61);

fub is the ultimate strength of the bolt.

$$A_b = \frac{\pi}{4} d^2 \qquad (2.61)$$

ii. Bearing resistance

The bearing resistance of the bolt on the plate is going to be verified according to Equation (2.62).

$$F_{b,Rd} = \frac{\text{k1abfudt}}{\gamma M2} \qquad (2.62)$$

d is the diameter of the bolts;

fu is the yielding strength of the plate;

α_b is the smallest of α_b; $\frac{\text{fub}}{fu}$ or 1;

α_b equals $\frac{e_1}{3d0}$ for end bolts and $\frac{p_1}{3d0} - \frac{1}{4}$ for inner bolts;

k_1 is the smallest of 2.8 $\frac{e2}{d0}$ − 1.7 or 2.5 for edge bolts;

k_1 is the smallest of 1.4 $\frac{p2}{d0}$ − 1.7 or 2.5 for inner bolts;

d_0 is the diameter of the bolts holes on the plate;

e1, e2, p1, p2 are the dispositions on the plate and are represented in Figure 2.11.

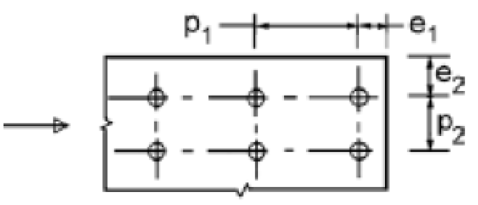


Figure 2.11. Spacing of the holes on the plate (EN 1993-1-8, 2005)

iii. Traction resistance

The resistance to traction of each bolt is given by equation (2.63).

$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{M2}} \qquad (2.63)$$

Where:

As is the tensile stress area of the bolt;

k_2 is equal to 0.9.

iv. Simultaneous traction and shear

Simultaneous traction and shear will be verified as shown in equation (2.64).

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4F_{t,Rd}} \leq 1.0 \qquad (2.64)$$

Where $F_{v,Ed}$ and $F_{t,Ed}$ are respectively the shear and traction forces.

v. Resistances of bolt rows in the tension zone

The effective design tension resistance for each row of bolts in the tension zone is limited by the least resistance of bending in the column flange, tension in the column web, bending in the end plate and tension in the rafter web. In bolted connections an equivalent T-stub in tension may be used to model the design resistances for the end plate and the column flange separately.

vi. Column flange in bending

. The geometry for a haunched connection in a portal frame in figure 2.12 would be similar although the beam would usually be at a slope and there will be more bolts rows which provides information to identify its basic joint components.

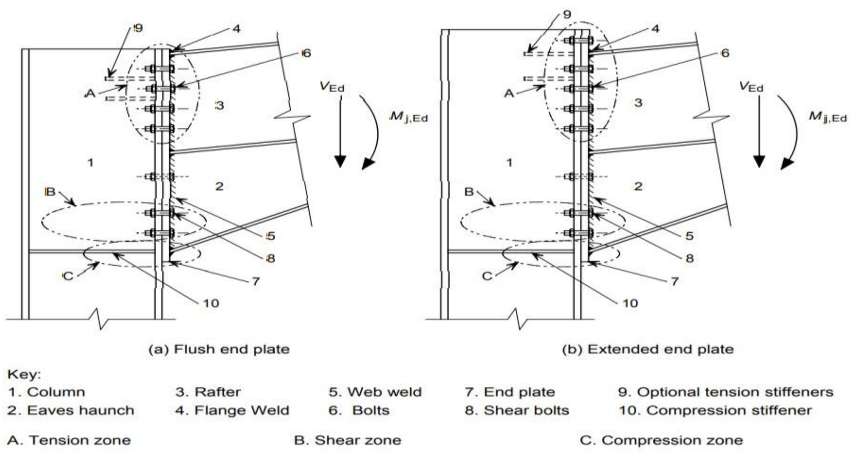
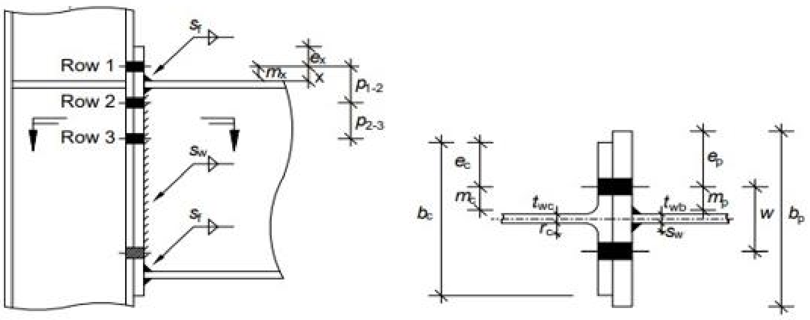


Figure 2.12. Portal frame eaves connections with bolted end plate (Brown et al., 2013)

The connection geometry for an end plate connection is shown in Figure 2.13.



For the end plate:

$$m_p = \frac{w}{2} - \frac{t_{wb}}{2} - 0.8s$$

$$e_p = \frac{b_p}{2} - \frac{w}{2}$$

For the column flange:

$$m_c = \frac{w}{2} - \frac{t_{wc}}{2} - 0.8r_c$$

$$e_c = \frac{b_c}{2} - \frac{w}{2}$$

For the end plate extension only:

$m_x = x - 0.8s_f$

Adjacent to a flange or stiffener:

m_2 is calculated in a similar way to m_x, above. m_2 is the distance to the face of the flange or stiffener, less 0.8 of the weld leg length.

where:

w is the horizontal distance between bolt centrelines (gauge)
b_p is the end plate width
b_c is the column flange width
t_{wb} is the beam web thickness
t_{wc} is the column web thickness
s is the weld leg length ($s = \sqrt{2}a$, where a is the weld throat) (subscripts f and w refer to the flange and the web welds respectively)
r_c is the fillet radius of the rolled section (for a welded column section use s, the weld leg length)

Figure 2.13. Connection geometry (Brown et al., 2013)

The resistances are calculated for three possible modes of failure and the least value is taken as shown in equation (2.65).

$$Ft_{fc,Rd} = \min\,(FT_{,1,Rd};\ FT_{,2,Rd};\ FT_{,3,Rd}) \tag{2.65}$$

• Mode 1

For the failure mode 1, the failure is due to the plate as shown in Figure 2.14.

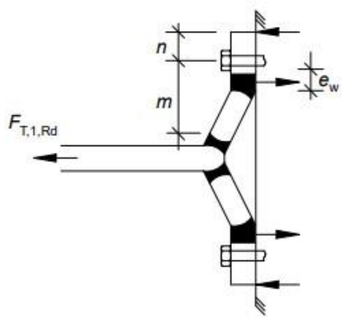


Figure 2.14. Complete flange yielding (SCI P398)

The design resistance of the T-stub flange is calculated as shown in equation (2.66).

$$F_{T,1,Rd} = \frac{(8n - 2ew)\ Mpl,1,Rd,}{2mn - ew(m+n)} \tag{2.66}$$

Where:

$M_{pl,1,Rd}$ is the plastic resistance moment of the equivalent T-stub for mode 1 as calculated in equation (2.67).

$$M_{pl,1,Rd} = \frac{0.25 \sum leff,1tp2fy}{\gamma M0} \tag{2.67}$$

e_w = dw/4

dw is the width across points of the bolt head.

$\sum leff,1$ is the effective length of the equivalent T-stub for mode 1;

$t_{w,b}$ is the web thickness of the beam;

n is the min {ec; ep; 1.25m}

e_c is the edge distance of the column flange;

e_p is the edge distance of the end plate.

• **Mode 2**

For the failure mode 2, the failure is due to the local yielding of the plate and bolts failure as show in Figure 2.15.

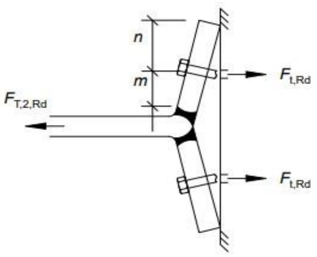


Figure 2.15. Bolt failure with flange yielding (SCI P398)

The design resistance of the T-stub flange is calculated as shown in equation (2.68).

$$F_{T,2,Rd} = \frac{2Mpl,2,Rd + n \sum Ft,Rd}{m+n} \tag{2.68}$$

$\sum F_{t,Rd}$ is the total tension resistance for the bolts in the T-stub (is equal to $2*Ft,Rd$ for a single row);

$M_{pl,2,Rd}$ is the plastic resistance moment of the equivalent T-stub for mode 2 as calculated in equation (2.69);

$$M_{pl,2,Rd} \quad \frac{0.25 \sum l_{eff,2} t_{pf}^{2} fy}{\gamma M0} \tag{2.69}$$

$\sum leff,2$ is the effective length of the equivalent T-stub for mode 2

• **Mode 3**

In mode 3, the failure is due to the bolts as shown in Figure 2.16

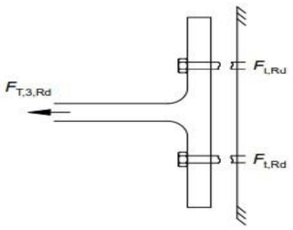


Figure 2.16. Bolt failure (Brown et al., 2013)

The design resistance of the T-stub flange is given by equation (2.70).

$$FT_{,3,Rd} = \sum Ft_{,Rd,u} \tag{2.70}$$

vii. Column web in transverse tension

The transverse tension resistance for a column web is given in equation (2.71).

$$F_{t,wc,Rd} = \frac{\omega b_{eff,t,wc} t_{wc} f_{y,wc}}{\gamma_{M0}} \tag{2.71}$$

Where:

ω is the reduction factor to allow for the interaction with shear in the column web panel as calculated in equation (2.72) .

$$\omega = \frac{1}{\sqrt{1+1.3(beff,c,wctwc/Avc)2}} \tag{2.72}$$

Avc is the shear area of the column. For rolled I and H sections it can be conservatively taken as hw, tw;

beff, t, wc is equal to leff..

viii. End-plate in bending

The design resistance and failure mode of an end-plate in bending, together with the associated bolts in tension, can be determined following the methodology used for column flange and using equation (2.73).

$$Ft{,}ep{,}_{Rd} = \min(FT{,}_{1,Rd;}\ FT{,}_{2,Rd;}\ FT{,}_{3,Rd}) \tag{2.73}$$

ix. Rafter web in tension

The resistance of the rafter web in tension can be calculated as shown in equation (2.74).

$$F_{t,wb,Rd} = \frac{b_{eff,\,wb}\, t_{wb} f_{y,\,wb}}{\gamma_{M0}} \tag{2.74}$$

where beff, t, wb is equal to leff.

x. Total resisting moment

The total resisting moment is obtained from the sum of the products of the traction resistance in each bolt row in the tension zone times their respective distances di from the center of resistance of the compression zone (neutral axis of the compression flange) as shown in equation (2.75).

$$M_{Rd} = \sum Ft,\ Rd,\ \mathrm{i} * di \tag{2.75}$$

The moment is designed considering the load at the centre of compression to be at the mid thickness of the compression flange of the beam.

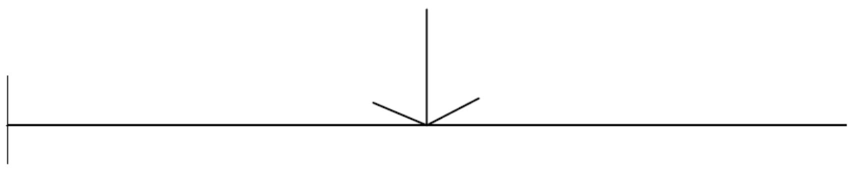


Figure 2.17. Load acting in the middle of the running beam

Where :

$$h_{r1}=h_b-\frac{tfb}{2}+x \tag{2.76}$$

$$h_{r2}=h_{r1}-100 \tag{2.77}$$

$$h_{r3}=h_{r2}-90 \tag{2.78}$$

b. Beam-beam connection

This connection is done between the two rafters of the portal frame as shown in Figure 2.18. The verification procedure will be done as for the beam to column connection. The shear resistance per bolt is computed using equation (2.59) or equation (2.60) depending on the bolt grade. Bearing resistance per bolt, traction resistance per bolt and total moment resistance will be verified using equation (2.62), (2.63) and (2.75) respectively.

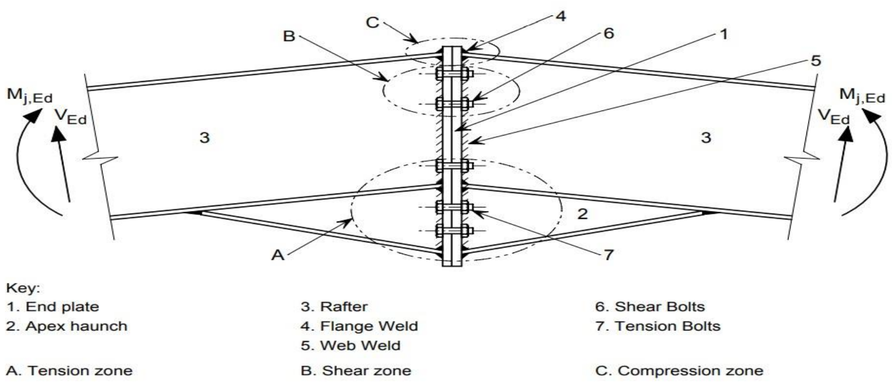


Figure 2.18. Portal frame apex connection with bolted extended end plate (Brown et al., 2013)

c. Brace connection

The presence of the brace is mainly to take horizontal loads and this loads transit through the brace connections. This connection will be a simple connection that is no moment. The shear resistance per bolt is computed using equation (2.59) or equation (2.60) depending of the bolt grade and the bearing resistance will be verified using equation (2.62).

d. Column base connection (foundation design)

The verification of the column base connection is done by verifying the concrete footing and the thickness of the plate.

i. Concrete footing

Firstly, the bearing capacity is verified using equation (2.79).

$$\sigma sol \leq \sigma adm \qquad (2.79)$$

Where σsol is the pressure exerted by the footing on the ground and it is calculated using equation (2.80)

$$\sigma_{sol} = \frac{\text{NSLS} + \gamma * \text{A} * \text{B} * \text{H}}{A * B} \qquad (2.80)$$

Where:

N_{SLS} is the SLS axial force solicitation at the level of the column base connection ;

A and B are the dimensions of the section of the concrete footing;

H is the height of the concrete footing;

γ is the unit weight of concrete.

Afterwards, the compressive resistance of the concrete block is verified with the help of equation (2.81).

$$\sigma < fck \qquad (2.81)$$

Where:

fck is the characteristic compressive cylinder strength of concrete ;

σ is the compression constraint exerted on the concrete bloc calculated using equation (2.82).

$$\sigma = \text{N/ab} \qquad (2.82)$$

With a and b being the length and width of the steel plate.

The bending resistance verification is given by:

$$\text{M}_{\text{Ed}} \leq \text{M}_{lim} \qquad (2.83)$$

Where:

$$\text{M}_{\text{lim}} = 0.167\text{f}_{\text{ck}}\text{bd}^2 \qquad (2.84)$$

$$\text{M}_{\text{Ed}} = 0.5\text{ql}^2 \qquad (2.85)$$

The longitudinal reinforcement of the footing is given by:

$$\frac{M_{Ed}}{0.87f_{yk}z} \leq A_s \quad (2.86)$$

$$A_{s,min} \leq A_s \quad (2.87)$$

$$A_{s,min} = \frac{0.15bd}{100} \quad (2.88)$$

ii. Plate

The design of the column base plate starts by determining the geometry of the plate. The section of the plate will be calculated using equation (2.89).

$$A_P = \frac{N_{Ed}}{f_{cd}} \quad (2.89)$$

Where fcd is the characteristic resistance of the concrete footing.

Admitting that part of the plate at the edge of the columns will be subjected to an uplift due to the reactions from the foundations, it will bend according to the tangent lines 1 and 2 as shown in Figure 2.19.

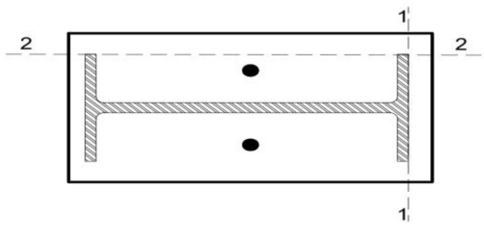


Figure 2.19. Tangent lines on the base plate which determine uplift (Becker et al., 2015)

The thickness of the plate is verified using equation (2.90).

$$t \geq u\sqrt{\frac{3\sigma}{f_y}} \quad (2.90)$$

Where u is the perpendicular distance from the edge of the beam flange to the edge of the column (lever arm).

Figure 2.20 shows the distribution of solicitations on the column base connection.

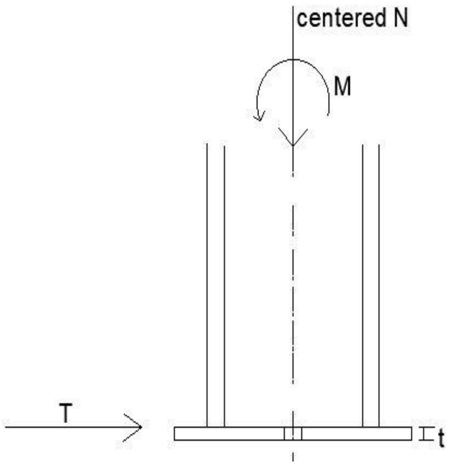


Figure 2.20. Distribution of solicitations on the column base connection

The areas As and A’s are presented in figure 2.21.

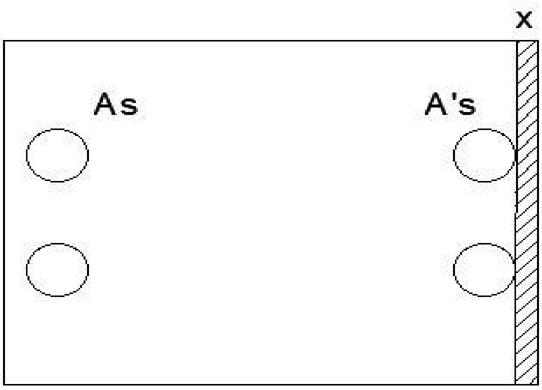


Figure 2.21. Region of As and A’s

From the displacement equilibrium, it is found that:

$$0.8bxfcd + A'sfyd - Asfyd = Ned \text{ then } x = \frac{Ned}{b0.8fcd} \tag{2.91}$$

From the rotational equilibrium about the geometric center of gravity of the section, its resistance moment is found

$$Mrd = 0.8xbfcd(\frac{h-0.8x}{2}) + fyd(As + A's)(\frac{h}{2} - c) \tag{2.92}$$

Afterwards the shear resistance, bearing resistance and traction resistance of the anchors will be verified using equation (2.46) or (2.47), (2.49) and (2.50) respectively.

2.5.2.6. Design of bridge crane

A bridge crane is composed of many structural parts and mechanisms necessary for the possible different movements that is vertical and horizontal movements (Metallique & Yezli,

2013) . These elements will be calculated according to Eurocode 3. The crane design study is structured as follows

- the bridge crane itself that is the running beam consisting of:
- bearing beam (standing);
- the rails.

a. Bridge crane itself

Bridge cranes have characteristics that were provided by the manufacturer. These are the

- Trolley type and size
- extreme position of carriage and hook
- wheelbase (distance between rollers).

To obtain the crane data sheet the following data was needed.

- Nominal load
- Span length
- Trolley speed, lifting speed and travelling speed.
- external load of the crane rollers on the running beam
- overall crane length
- type of rail necessary

To obtain these information, a series of questions were asked in order to ensure a proper design as seen in annex 1. Based on these pieces of information the maximum and minimum charge per wheel can be obtained as shown in Equation (2.97) and (2.98) in accordance with the Abus bridge crane catalogue as seen in annex A2. The design of the running beam was done according to CCM97 and the calculations done with the unfavourable combination.

i. Design working life

This is determined according to the number of duty cycles. 10 classes of duty cycles are defined in function of the number of cycles awaited.

The class is determined by the product of;

- The number of cycles of lifting that the machine will accomplish every day when in use.

- The average number of days in use per year.
- The number of years after which the bridge crane will be considered before been replaced.

The class was chosen corresponding to the number of cycles obtained.

ii. Classification of bridge cranes

The overhead cranes are divided into eight groups of machines (A1 to A8) according to their classes of use U0 to U9 (from the least used to the most used) and their classes of load spectrum Q1 to Q4 (from least loaded to most loaded). This classification was established by the European Federation of Handling (EFH)

- **Classification according to the state of loading**

Table 2.4 gives some examples of the classification of the most common overhead cranes commonly used by a combination of the state of loading and the frequency of use.

Table 2.4. Classification of overhead cranes according to the state of loading(Solutions, 2009)

Class	State of loading
Q1: Very light	The traveling crane only exceptionally lifts the nominal load and regularly low loads
Q2: Light	The traveling crane rarely lifts the nominal load and regularly lifts loads of the order of 1/3 of the nominal load
Q3: Average	The traveling crane does not lift the nominal load quite frequently and regularly lifts loads between 1/3 and 2/3 of the nominal load
Q4: Heavy	The traveling crane is regularly loaded close to the rated load

- **Classification according to frequency of use**

Table 2.5 shows the categorisation of cranes in classes U1 to U9 (from the least used crane to the most used) according to the frequency of use, i.e. the effective duration of operation during the operating period. In addition, the number of foreseeable lifting cycles during this duration, therefore to be considered for the fatigue check, is also shown.

Table 2.5. Classification of overhead cranes according to frequency of use(Solutions, 2009)

Class	Frequency of use	Predictable number of lifting cycles during service life
U0 U1 U2	Occasional non-regular use followed by a long period of rest	1.60 x10^4 3.20 x 10^4 6.30 x 10^4
U3 U4	Regular use in intermittent service	1.25x 10^5 2.50x10^5
U5 U6	Regular use in heavy duty	5.00x10^5 1.00x10^6
U7 U8 U9	Regular use in intensive service ensured at more than one station	2.00x10^6 4.00x10^6 >4.00x10^6

iii. Path of forces

The study of the actions due to a bridge crane on the runway requires the breakdown of the path of the forces from their point of application to the supports of the running beam and the foundations of the supporting structure. Thus, the static system must be determined successively for the three load application directions, namely:

- Vertically,
- Transversally,
- Longitudinally.

Since a running beam is a load-bearing structure that is dynamically and repeatedly stressed, the elastic-elastic calculation method is applied (elastic calculation of internal forces and elastic calculation of section resistance). The deformations are of course also calculated elastically.

- **Vertical loads**

The static system of a running beam is the isostatic beam. Since the bridge crane has a moving load that can in principle occupy any position on the running beam, the internal forces acting on a section are obtained through solid mechanics' methods. In common cases, the rolling beams are supported vertically and horizontally at each post. The path of the forces transmitted by this beam was done through the framework of the hall. The static system of the bridge crane can be represented as shown in Figure 2.22.

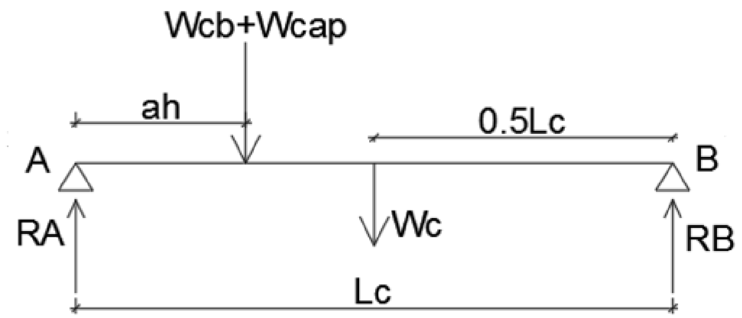


Figure 2.22. Static of the bridge crane

The reactions at supports A and B are gotten as shown in equation (2.93) and (2.94).

$$RA = \frac{Wc}{2} + \frac{Lc - ah}{Lc} * (Wcb + Wcap) \tag{2.93}$$

$$RB = \mathrm{Wc} + \mathrm{Wcb} + \mathrm{Wcap} - \mathrm{RA} \tag{2.94}$$

$$\mathrm{WvA} = \frac{\mathrm{RA}}{2} \tag{2.95}$$

$$\mathrm{WvB} = \frac{\mathrm{RB}}{2} \tag{2.96}$$

$$W'VA = 1.25WvA \tag{2.97}$$

$$W'VB = 1.25WvB \tag{2.98}$$

$$QmaxA = W'vA \times (2 - \frac{aw}{L}) \tag{2.99}$$

$$QmaxB = W'vB \times (2 - \frac{aw}{L}) \tag{2.100}$$

$$M1 = \frac{2W'VA}{L} \times (\frac{L}{2} - \frac{aw}{4})^2 \tag{2.101}$$

$$M2 = W'VA \times \frac{l}{4} \tag{2.102}$$

M3 max=max (M1, M2)

- **Horizontal forces**

The forces transverse to the track are caused by the braking of the truck, the oblique lifting of the load, the irregularities of the track and the crabbing of the bridge crane. They are transmitted to the beam by the rollers of the bridge crane through the running rail. A section of running beam and part of the two frames of the hall are represented schematically in figure 2.23(a), as well as the horizontal static system of the running beam in figure 2.23 (b). Based on this system, transverse reactions are determined analogously to vertical reactions.

The transverse support reaction acts on the supporting structure of the hall, in which it generates internal forces and deformations. The internal forces must be combined with those due to the other actions acting on the hall, according to the load cases considered. Figure 2.23(c) gives an example of a suitable static system for each of these cases.

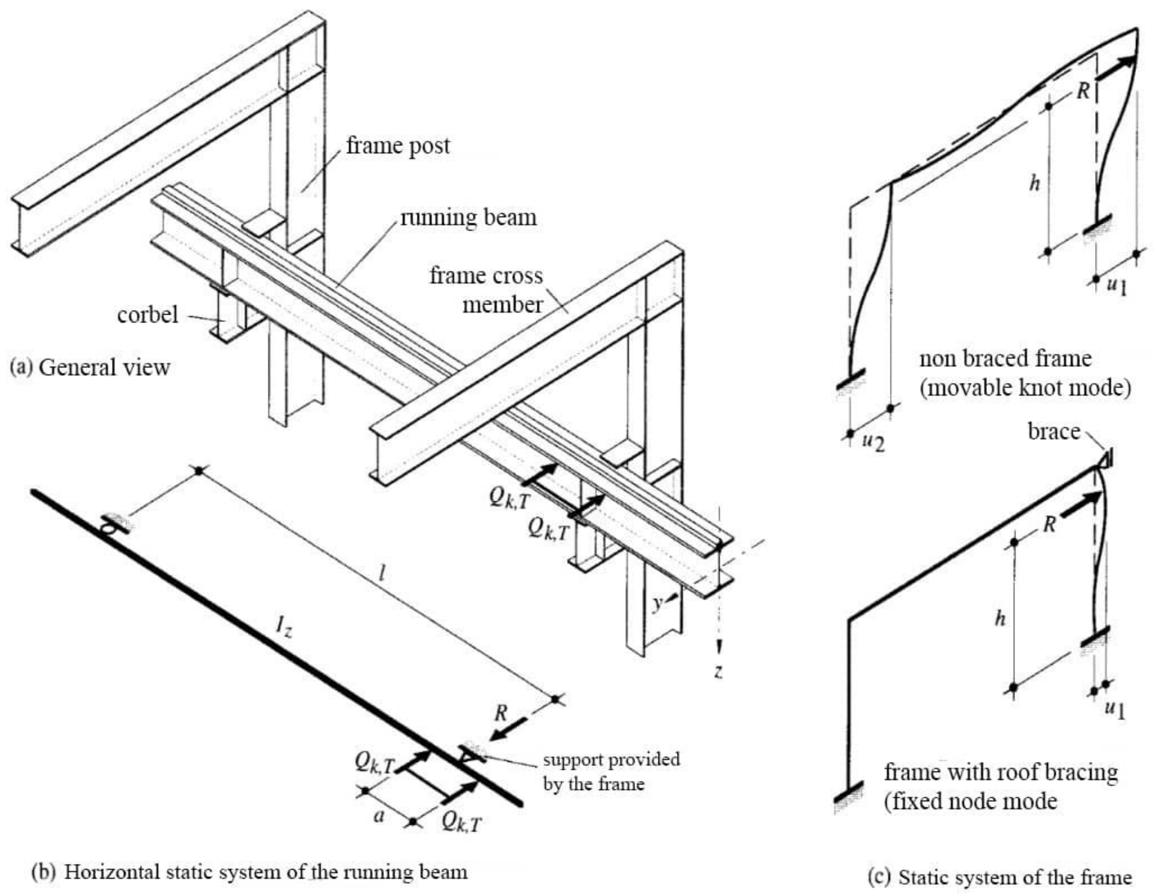


Figure 2.23. Transverse forces on the beam (a) General view (b) Static system of the running beam (c) static system of the frame (Hirt, 1949)

The horizontal forces were calculated as follows:

$$Wh2A = 0.05WvA \tag{2.103}$$

$$Wh2B = 0.05WvB \tag{2.104}$$

- **Longitudinal forces**

The longitudinal horizontal force due to the acceleration or braking of the traveling crane is transmitted directly by the running beam to the longitudinal fixed point.

The values of the partial factors γ_{Mi} for crane supporting structures shall be applied to the forces above for the ULS combinations. They are defined as follows:

Table 2.6. Dynamic amplification factors

γ_{M0}	γ_{M1}	γ_{M2}	γ_{M3}	γ_{M3ser}	γ_{M4}	γ_{M5}	γ_{M6}	γ_{M7}
1.00	1.00	1.25	1.25	1.10	1.00	1.00	1.00	1.10

iv. Verifications

The following verifications were considered:

- **Resistance of bottom flanges to wheel loads**

The design resistance $F_{f,Rd}$ of the bottom flange of a beam to a wheel load $F_{z,Ed}$ from an underslung crane or hoist block trolley wheel, was determined from:

$$F_{f,Rd} = \frac{\ell_{eff}\, t_f^{\,2} f_y / \gamma_{M0}}{4\, m}\left[1 - \left(\frac{\sigma_{f,Ed}}{f_y / \gamma_{M0}}\right)^2\right] \tag{2.105}$$

Where:

l_{eff} is the effective length of flange resisting the wheel load,

m is the lever arm from the wheel load to the root of the flange,

t_f is the flange thickness;

$f_{,Ed}$ is the stress at the midline of the flange due to the overall internal moment in the beam.

- **Resistance of the web to transverse loads**

The design resistance, to local penetration of a web of section in I, H or U is determined using the formula:

$$R_{y.rd} = \mathrm{S_y} \times \mathrm{t_w} \times \frac{\mathrm{f_{yw}}}{\gamma_{\mathrm{m1}}} \tag{2.106}$$

Where:

Sy is the rigid bearing length.

Ryrd is the force resisting crushing.

- **Resistance to compression**

According to CCM97, the design resistance R_{ard} to local penetration of a web of section in I, H or U is determined using the formula:

$$R_{a.rd} = \frac{0.5 \times {t_w}^2 \times (E \times f_{yw})^{0.5}\left[\left(\frac{t_f}{t_w}\right)^{0.5} + 3\left(\frac{t_w}{t_f}\right) \times \left(\frac{S_s}{d}\right)\right]}{\lambda_{m1}} \tag{2.107}$$

Where:

Ss is the length of the rigid support and the ratio $\frac{Ss}{d} < 0.2$

When the element is also subjected to bending moments, the following criterion should be considered:

$$f_{sd} \quad \le \quad R_{a,rd} \tag{2.108}$$

$$M_{sd} \quad \le \quad M_{c,rd} \tag{2.109}$$

$$\frac{f_{sd}}{R_{a,rd}} + \frac{M_{sd}}{M_{c,rd}} \le 1.5 \tag{2.110}$$

- **Web buckling resistance**

According to the CCM97 regulation, the design resistance to web buckling of an I, H or U profile is determined by studying the buckling of the web. beff is gotten by the formula:

$$b_{eff} = \sqrt{(h^2 + {S_s}^2)} \tag{2.111}$$

Also beff should be lesss than b.

- **Buckling of the compression flange in the plane of the web**

To prevent the possibility of buckling of the compression flange in the plane of the web, the ratio (d /t_w) of the web must satisfy the following criterion:

$$\frac{d}{t_w} \leq K \times \frac{E}{f_{yt}} \times \sqrt{\frac{A_w}{A_{fc}}} \tag{2.112}$$

Where:

Aw is the area of the web

Afc is the area of the compressed section

K is the coefficient taken equal to 0.3 for a class I section

- **Deflection**

The CCM97 requires the maximum admissible deflection

$$\mathbf{f} \leq \mathbf{f}_{\mathbf{adm}} = \frac{L}{750} \tag{2.113}$$

The deflection is obtained when the two loads are in a symmetric position. This method consists of superposing the loads caused by the two loads on the wheels.

$$\mathbf{a} = \frac{L}{2} - \frac{\mathbf{e}}{4} \tag{2.114}$$

$$f_1 = \frac{P \times a \times L^2}{24 \times E \times I_y} \times \left(3 - \frac{4 \times a^2}{L^2}\right) \tag{2.115}$$

This is illustrated in Figure 2.24.

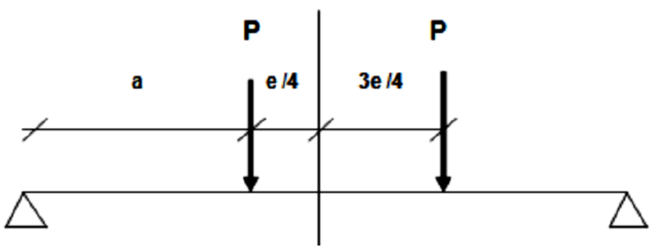


Figure 2.24. Two loads applied on the running beam

b. Runway beam (Running beam support)

The running beam is supported by the runway which is stressed by the following forces:

- The dead weight of the running beam and the rail;
- The vertical and horizontal actions of the overhead crane rollers;
- The self-weight of the runway beam;

i. Vertical Loads

The load combinations at the ULS and SLS are given in Equation (2.116) and (2.117) respectively.

ULS:

$$P = 1.35(G_T \times L) + 1.5\left[R_{vmax} \times \left(2 - \frac{e}{L}\right)\right] \quad (2.116)$$

SLS:

$$P = G_T \times L + R_{vmax} \times \left(2 - \frac{e}{L}\right) \quad (2.117)$$

ii. Horizontal load

$$P_H = 1,5 \times R_H \times \left(1 - \frac{e}{L}\right) \Psi^2 \quad (2.118)$$

iii. Runway beam design

The running beam support deflection is limited to $\frac{l}{500}$

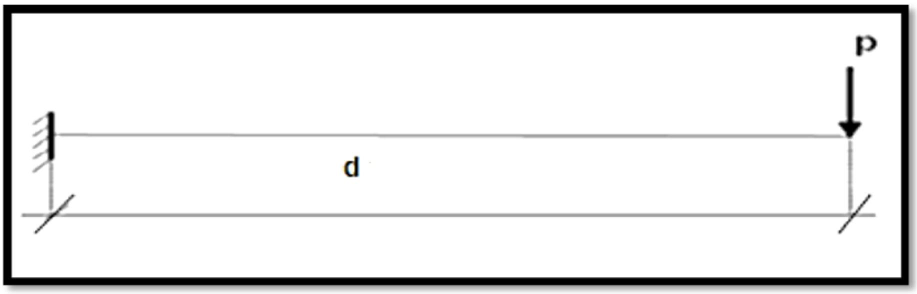


Figure 2.25. Static diagram of the bearing support

$$f = P_1 \times \frac{d^3}{3EI} \leq \frac{d}{500} \quad (2.119)$$

$$I_y > \frac{P_1 \times d^2 \times 500}{3 \times E} \quad (2.120)$$

From where the section with a greater moment of inertia is chosen. With the section chosen its verifications (class, buckling, shear) are the same as that described for the runway beam above.

c. Crane rails

The rails of the bridge crane should be designed as follows.

i. Rail material

The rail steel should both be made from special rail steels, with a specified minimum tensile strength of between 500 N/mm² and 1200 N/mm².

ii. Design working life

The grade of rail steel is selected to give the rail an appropriate design working life Lr. An account was taken for the need for rail replacement in selecting the rail fixings.

iii. Rail selection

The selection of crane rails should take into account the following: the rail material, the wheel load, the wheel material, the wheel diameter and the crane utilisation.

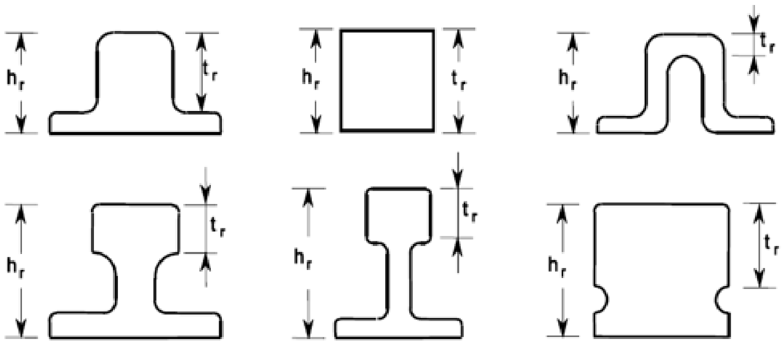


Figure 2.26. Minimum thickness below the wearing surface of a crane rail

The minimum width of the rail and the type of steel are defined by the crane manufacturer, in particular according to the support reactions of the rollers. The shape of the rail depends on its fixing system on the rolling beam and the size of the overhead crane. The type of rail chosen is the bolted rail

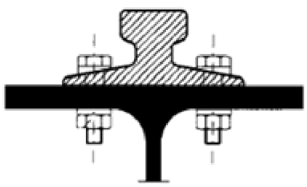


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Figure 2.27. Rail with a large bolted section

The eccentricity of application *e* of a wheel load *Q*r to a rail should be taken as a portion of the width of the rail head *b*r,

Where e =0.25br

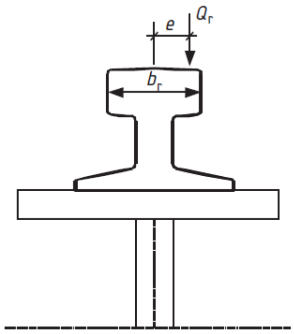


Figure 2.28 Point of application of wheel load

2.5.3. Serviceability limit states check for steel members

For the structural members of the steel structures, it shall be verified that the maximum deflection of each element is less than its maximum value according to Eurocodes.

For beam elements, the maximum vertical deflection should be less than $l/200$,

where l is the length of the beam in millimeters.

For columns, the maximum horizontal deflection should be less than $h/300$,

where h is the height of the column.

For the bridge crane and its support, the maximum deflection should be less than the maximum value according to Eurocodes as shown in Annex B.

2.6. Measuring and comparing the performance criteria

Haven modelled, analysed and designed the structure without a bridge crane and the structure with a bridge crane, the results shall be compared based on specific criteria which will help to judge the effect of bridge cranes in steel structures. The main judging criteria to consider will be divided into three parts:

- Structural (load carrying capacity)
- Functional (deflection) and,
- Economic (cost estimation)

The load carrying capacity is computed with respect to the load and resistance of a particular element(s) (most critical element(s)) The structural performances will be done after obtaining the results.

Regarding the evaluation of the cost, it shall be done based on the mass of the structure and erection cost. The mass of the structure is a sufficient robust instrument of measure (Pettang et al., 2016) as it provides 65 % certainty of the cost of the project. The construction process and man-power all contribute to 35 %. The mass of a structural project can therefore be used to compare two projects to estimate the costliest solution. The mass of the steel and the cost is estimated and compared between the two different structures. The price of the steel construction is obtained from an average of the world market price.

2.7. Reinforcement of steel structure due to effect of bridge cranes

The bridge crane has a dynamic effect on the whole structure and this brings forth many vibrations on the structure. As such the introduction of a bridge crane on an existing steel structure may bring about problems on the structural elements. These include the non-verification of the sections present during design, the removal of the roof during crane operation, and extra fatigue damage leading to the failure of the whole structure.

Conclusion

The aim of this chapter was to give detailed step by step procedures used all through the thesis work. Firstly, the chapter started by recognising the site through physical and geographical parameters. Secondly, the procedures for collecting data, assessing codes and standards used in this study, the different actions and load combinations used in the structural verification of the case study and the different analysis tools that were used for the work were detailed. Thirdly, the different analysis steps made using ideal modelling and structural analysis tools performed using the computer program Sap2000 v23.2.0 in order to statically verify the steel structures and analyse the behaviour of the bridge crane with the steel structure. Fourthly was presented the criteria that were used to compare both configurations of the designed steel structures with and without the bridge crane. The methodology being set, the proper design of the two steel structures and the analysis of the values obtained can now be discussed.

CHAPTER 3: PRESENTATION AND INTERPRETATION OF RESULTS

Introduction

This chapter, in this thesis, is engaged in the presentation of the design data obtained through the methodology previously described. The first phase of this section will be dedicated to the presentation of the case study. The next phase will concern the presentation of the influence line results which enables the proper design of the two steel structures. This goes from the choice and justification of the sections to the design of the connections between the elements. The chapter will conclude with a comparison of the results obtained for both configurations using the criteria defined in the methodology. The corresponding discussions proceed accordingly as the various results obtained are presented.

3.1. Presentation of the site

The bridge crane to be studied is found at Mvan, in the industrial zone of the Mfoundi district situated in the city of Yaounde. This part is reserved to a general presentation of Yaounde, presenting firstly its physical parameters as well as the human and socio-economic parameters, which can influence the design of the structure.

3.1.1. Physical parameters

Here, parameters like the geographical location, the climate, the relief and hydrology of the city will be presented.

3.1.1.1. Geographical Location

Yaoundé is the political capital of Cameroon and chief town of the Centre region. It is situated at latitude 3.87° (3° 50’) North and longitude 11.52° (11° 31’) East at an elevation of 760 m above sea level. Located 300km from the Atlantic Ocean and surrounded by seven hills, Yaoundé belongs to the Mfoundi division and covers a total surface area of 304 km², with an urbanized surface area of 183 km².

3.1.1.2. Climate

Yaoundé features a tropical wet and dry climate, with records of high temperatures of 36 °C, an average of 23.8 °C and a record low temperature of 14 °C. Primarily due to the altitude, the temperatures are not as high as would have been expected for a city located near the equator. The town of Yaoundé features a lengthy rainy season covering a nine-month span between March and November. However, there is a noticeable decrease in precipitation within the rainy season, observed during the months of July and August, giving the city the appearance of having two rainy and two dry seasons. The average precipitation is 1650 mm of rain per year and average humidity is 80 %.

3.1.1.3. Relief and hydrology

The land rises gently in escarpments from the south-western coastal plain before joining the Adamawa Plateau via depressions and granite massifs. The field is characterised by rolling, forested hills, the tallest of which have rocky tops.

The hydrographic network of Yaoundé is very dense and composed of permanent rivers such as the Mfoundi river which crosses the city from North to South, a few creeks and lakes. Yaoundé is part of the western sector of the Southern Cameroon Plateau. The area is characterized by gentle rolling chains of hills, and numerous valleys and wetlands; this varied physical landscape permits a combination of streams, hydromorphic soils and a great variety of plants and Fauna.

3.1.2. Human and socio-economic parameters

This section is concerned with the description of parameters like population, culture and economic activities.

3.1.2.1. Population

The city of Yaoundé extends over 304 km^2 including an urbanized area of 183 km^2 has an estimated population, in 2022, of 4,337,000 inhabitants and an average density of 14,266 inhabitants per km^2. The city of Yaoundé being a cosmopolitan city, there is a considerable portion of population coming from several other regions of the country. These population estimates and projections come from the latest revision of the UN World Statistical centre.

3.1.2.2. Economic activities

Most of the economy is centred around the administrative structure of the civil and diplomatic services. Due to these, Yaoundé has a higher standard of living and security.

However, Yaoundé is a tertiary city and there are a few industries: breweries, sawmills, carpentry, tobacco, paper mills, machinery sheet metal and building materials.

3.2. Physical description of the site

The project is located in the city of Yaounde, more precisely at Mvan as shown in Figure 3.1. Mvan is the industrial zone of the Mfoundi district situated in the city of Yaounde, located in the commune of Nkoteng and in the department of Haute-Sanaga. It is located at latitude 3° 49' 27.8" North and longitude 11° 31' 03.6" East.

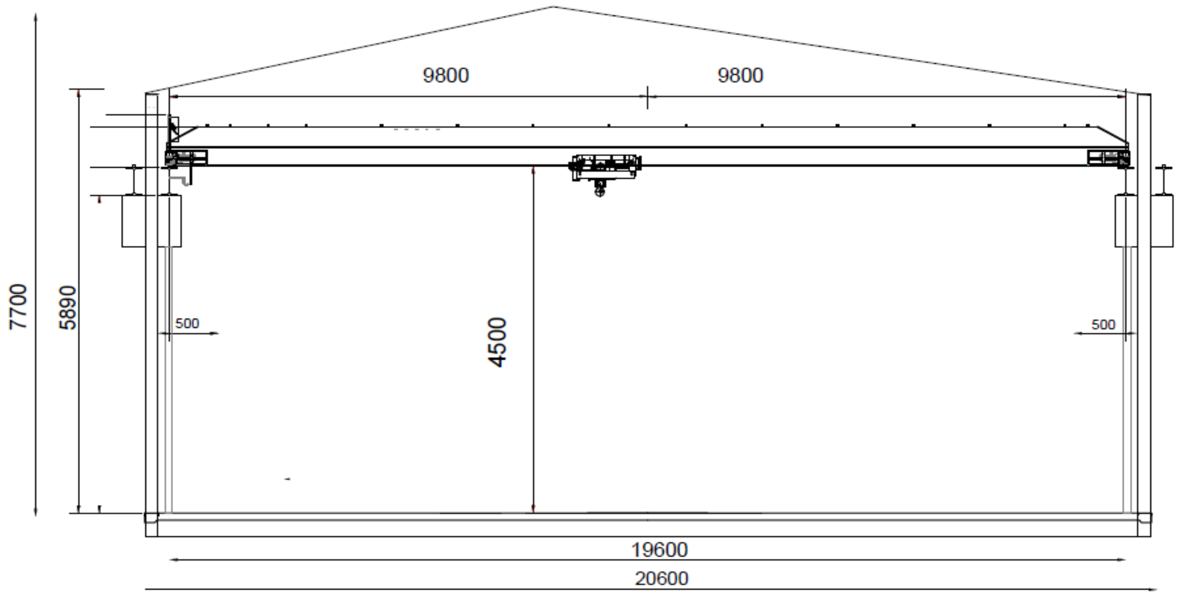
Figure 3.1. Location of the project site (Google Earth Pro, 2022)

3.3. Presentation of the project

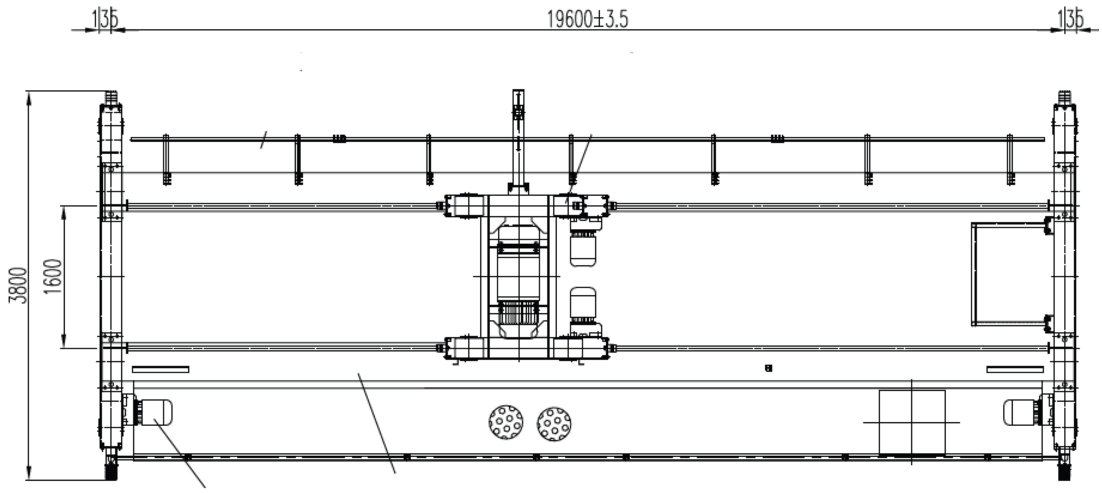
The steel factory is located in the locality of Mvan in the Center Region of Cameroon, at latitude 3° 49' 27.8" North and longitude 11° 31' 03.6" East. This consists of purlins, roof braces, and columns The bridge crane is a double girder which spans longitudinally and transversely along the building.

3.3.1. Building configuration of the structure with and without a bridge crane

The steel structure without a bridge crane is grounded on a 34 m long, and 20.6 m wide having a surface area of 700.4 m². Its total height is 7.7 m with the left side having columns of 5.89 m and the right side having columns of 4.73 m. Each bay is 5 m from each other. The structure is made up of a steel gantry linked together with purlins, roof braces, and columns. It is closed and covered with Aluminium 5/10 sheets. The foundation has isolated footing with tie beams.



(a) Front view



(b) Top view

Figure 3.2. Plan distribution of the factory with bridge crane: (a) Front view, (b) Top view

The steel structure with a bridge crane has the same components as the latter but with the addition of the bridge crane. The double girder bridge crane in the longitudinal direction has a span of 19.6 m with the rail installed above it and in the transversal direction it has a span of 34 m which travels along the whole building.

The numerical models of the two steel structures with the first without a bridge crane and the second with a bridge crane performed using the software SAP2000 are as shown in figure 3.3 and 3.4 respectively.

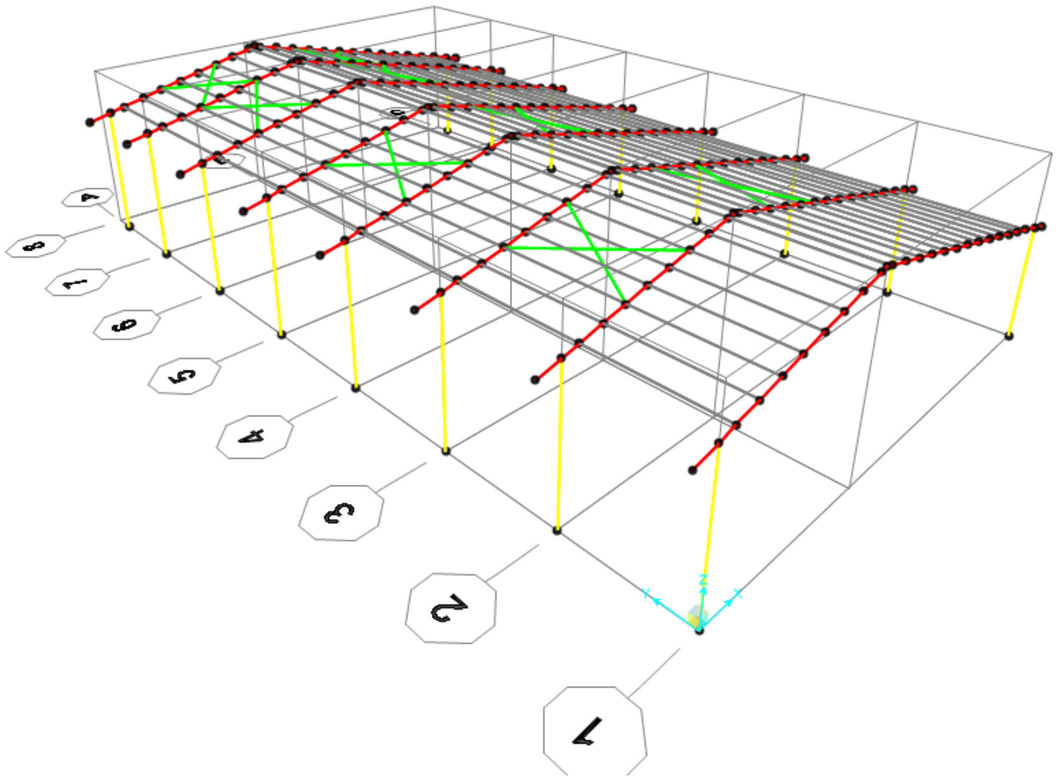


Figure 3.3. Numerical model of steel structure without a bridge crane

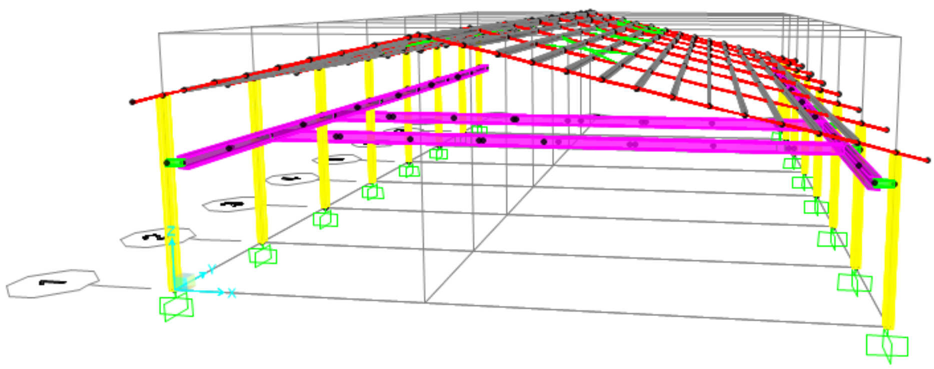


Figure 3.4. Numerical model of steel structure with a bridge crane

3.3.2. Materials characteristics

The structural steel used in the design of elements such as purlins, columns and bracings, will be designed using the grade S275. Its characteristics, useful to the design, are given in Table 3.1.

Table 3.1. Structural steel characteristics

Designation	Symbol	Value	Unit
Properties of structural steel			
Steel grade	**S275**		
Characteristic yielding strength	f_{yk}	275	N/mm^2
Elastic modulus	E_s	210000	N/mm^2

The bolts characteristics used to fasten the connections between structural members is as listed in Table 3.2

Table 3.2. Characteristics of bolts

Bolt characteristics			
Bolt class	**8.8**		
Nominal yielding strength	f_{yb}	640	N/mm^2
Nominal ultimate strength	f_{ub}	800	N/mm^2
Diameter of bolt	d	39	mm
Diameter of hole	do	42	mm
Shear area	As	976	mm^2

The concrete elements such as the concrete footing will be designed using a C25/30 concrete class. Its characteristics useful for the design are given in Table 3.3.

Table 3.3. Concrete characteristics

Designation	Symbol	Value	Unit
Properties of concrete			
Concrete strength class	**C25/30**		
Characteristic compressive strength	f_{ck}	25	N/mm^2
Design compressive strength	f_{cd}	14.17	N/mm^2
Secant modulus	E_{cm}	31000	N/mm^2

The reinforcement steel bars used in the concrete in the foundation to make it reinforced are graded B500C. The properties useful to the design are listed in Table 3.4.

Table 3.4. Properties of steel reinforcement in Reinforced concrete

Designation	Symbol	Value	Unit
Properties of reinforcement steel			
Steel grade	**B500C**		
Characteristic yielding strength	f_{yk}	500	N/mm^2
Design yielding strength	f_{yd}	434.78	N/mm^2
Elastic modulus	E_s	210000	N/mm^2

3.4. Actions on structure

This section presents in detail the actions considered in our structure, by displaying their nature and quantifying each according to the European norm.

3.4.1. Permanent loads

They are presented in two categories, which are structural and non-structural permanent loads. The former represents the weight of the elements, and will be introduced after the preliminary design phase, during the static design of the elements. Concerning the latter, the values determined are given in Table 3.5.

Table 3.5. Self-weight of structural elements.

Nature	Description	Value	Unit
G1	Self-weight of structural component	78.5	kN/m3

Table 3.6. Self-weight of non-structural elements.

Nature	Description	Value	Unit
G2	Aluminium roof 5/10 on steel elements	0.03	KN/m^2

Table 3.7. Non-structural permanent actions on the building

Designation	Symbol	Value	Unit
Non-structural actions on the roof			
Exterior sheet	-	0.15	kN/m^2
waterproofness	-	0.17	kN/m^2
Isolation	-	0.05	kN/m^2
Sheet support		0.05	kN/m^2
Vapour barrier		0.05	kN/m^2
Total	G2	0.5	kN/m^2

3.4.2. Variable loads

For the variable loads, the impose loads and the wind loads for the structure are going to be presented as follows.

3.4.2.1 Imposed loads

The main variable loads on the structure will be the maintenance of the roof. The roof is of category H as seen in table 2.1. The imposed loads q_k vary between 0 kN/m^2 and 1 kN/m^2. The value 0.5 kN/m^2 will be used.

3.4.2.2. Wind loads

The wind loads on the structure is determined for a terrain category II (area with regular cover of buildings). The parameters that are useful to the design are reported in Table 3.8.

Table 3.8. Computed wind coefficients and parameters

Designation	Symbol	Value	Unit
Basic wind velocity	v_b	22	m/s
Basic velocity pressure	q_b	0.3	kN/m^2
Peak velocity pressure	$q_p(z)$	0.8	kN/m^2
Windward pressure coefficient	c_{pw}	0.8	-
Leeward pressure coefficient	c_{pl}	-0.5	-
Windward force per unit length	$F_{w,w}$	2.69	kN/m
Leeward force per unit length	$F_{w,l}$	-1.67	kN/m

The computation of the wind load acting on the building is divided into two main parts, the load acting on the rectangular walls' contours of the structure and that acting on the roof. As mentioned in chapter 2, the norm used for wind load computation is BS-EN1991 E_2002. The following wind parameters are introduced considering the combination factors for the wind load to be $\varphi_0 = 0.6$ and $\varphi_2 = 0.5$.

a. Basic wind velocity

For the evaluation of the basic wind velocity, V_b, the following are used.

$C_{dir} = C_{season} = 1$ and $V_{b,0} = 22\ m/s$, hence is obtained $V_b = 22\ m/s$.

b. Peak and Basic velocity pressure

For the calculation of the peak and basic velocity pressure, the following are needed:

- Terrain category II, $z_0 = 0.01$ and $z_{min} = 1$
- Terrain factor, $K_r = 0.17$,
- Orography factor, $C_0 = 1$
- Turbulence intensity, $I_v = 0.21$.
- Exposure factor $C_e(z)$ taking into account turbulence $z \leq 10$ m, $C_e(z) = 2.6$

Hence, we obtain:

- Basic velocity pressure $(q_b) = 0.3025\ kN/m^2$
- Peak velocity pressure $(q_p) = 0.7865\ kN/m^2$

c. Wind pressure external on surfaces

Here, the loads acting on the columns and on the roof will be evaluated.

d. The load acting on the rectangular walls

Considering a conservative case, the pressure coefficients can be computed and the wind pressure can be obtained.

$C_{pe}(\text{windward}, w) = +0.90 + 0.30 = +1.20 \quad C_{pe}(\text{leeward}, l) = -0.3 - 0.2 = 0.5$

With, $C_s C_d = 1.0$ (framed buildings with structural walls less than 100 m high),it was obtained:

- On windward side, $W_e(w) = 0.3017\ kN/m^2$.
- On leeward side, $W_e(l) = -0.18\ kN/m^2$.

e. The load acting on the roof

For the loads acting on the roof, $W_e = -1.38\ kN/m^2$ was obtained.

3.4.3. Load combinations

For the verification of the structure, the load combinations are divided in two groups, the Ultimate limit states (ULS) and Serviceability limit state combinations (SLS). A load envelope is obtained for the ULS combinations to have the most unfavourable condition for an element.

$$ULS1: 1.3G + 1.5Qkw + 1.5Qkr \quad (3.1)$$

$$ULS2: 1.3G + 1.5Qkw \quad (3.2)$$

$$SLS1: G + Qkw + Qkr \quad (3.3)$$

$$SLS2; G + Qkw \quad (3.4)$$

For the crane loads, the load combinations are as follows:

$$\text{ULS: } 1.3G_k + 1.5Q_{kw} + 1.5Qkr + 1.5\gamma \text{Qmax} \quad (3.5)$$

$$\text{SLS: } G_k + Q_{kw} + Qkr + \gamma Q\text{max} \tag{3.6}$$

Where:

G is the permanent load

Qkw is the wind load

Qkr is the imposed load

γ is the amplification coefficient

Qmax is the maximum reaction per wheel

3.5. Static analysis of the structures

The static analysis of the structure is done under the static action meaning considering only the permanent and the imposed loads. It consists of obtaining the design solicitations on the members of the structure under ULS and the deflections under SLS.

3.5.1. Ultimate limit state design and verification of the structure without bridge crane

The results of the solicitations obtained in terms of stresses (moment, shear and axial force) were used to design the following structural elements.

The verifications are performed under the design load F_{Ed}.

$$F_{Ed} = 1.3G + 1.5Qkw + 1.5Qkr = 5.205\ kN.m^{-1}$$

3.5.1.1. Design verifications of the purlin

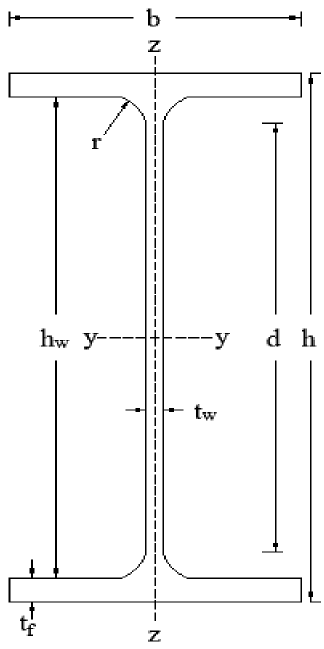
Here, the main interest will be to design the purlin element with maximum axial, shear forces and bending moment. The solicitations obtained are given in Table 3.9.

Table 3.9. Solicitations on the purlin.

Bending moment, M[kNm]	**Shear force, V[kN]**	**Axial force, N[kN]**
6.43	1.45	26.36

IPE 160 was used for the purlin's section. Its characteristics are given in Table 3.10.

Table 3.10. Properties of IPE 160.



G	15.8	Kg/m
h	160	mm
b	82	mm
d	127.2	mm
$\mathbf{h_w}$	145.2	mm
$\mathbf{t_w}$	5	mm
$\mathbf{t_f}$	7.4	mm
r	9	mm
A	20.1	cm^2
$\mathbf{A_{VZ}}$	9.66	cm^2
$\mathbf{I_Y}$	869	cm^4
$\mathbf{W_{PLY}}$	124	cm^3

Firstly, the section is classified and then the design verifications are presented in Table 3.11.

Table 3.11. Design verifications of the purlin.

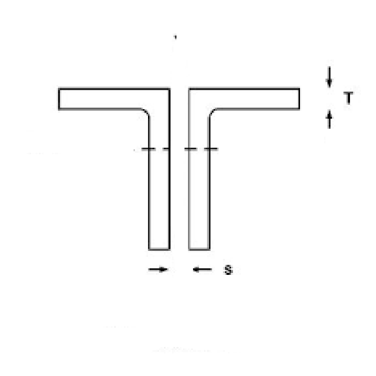
Designation	**Verification**	**Value**	**Observation**
Web in bending (Class verification)	$\frac{d}{tw} \leq 72\varepsilon$	25.44< 66.56	Class 1
Flange in compression (Class verification)	$\frac{c}{tf} \leq 9\varepsilon$	3.99 < 8.28	Class 1
Resistance in bending,	$M_{Ed} \leq M_{pl,Rd}$	6.43 KNm < 34.1 kNm	Verified
Resistance in shear	$V_{Ed} \leq V_{pl,Rd}$	1.45 kN < 153.37 kN	Verified
Moment and shear interaction ,	$V_{Ed} \leq 0.5V_{pl,Rd}$	1.45 kN < 76.69 kN	No bending moment reduction

Resistance in compression,	$N_{Ed} \leq N_{pl,Rd}$	26.36 kN < 552.75 kN	Verified
Bending and axial interaction	$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} \leq 1$	0.25 < 1	Verified
	$N_{Ed} \leq 0.25 N_{pl,Rd}$	26.36 kN > 138.19 kN	bending moment reduction
	$N_{Ed} \leq \frac{0.5\, h_w t_w f_y}{\gamma_{M0}}$	26.36 kN < 99.825 kN	No bending moment reduction

3.5.1.2. Design verifications of the gantry frame

To design the gantry frame, the elements in compression of the roof will be verified. To design the elements in compression of the roof, the element with the maximum compressive force will be verified. The section of the element under study is 2L30X30X4. Its characteristics are given in Table 3.12.

Table 3.12. Properties of 2L30X20X4.

	**t**	4	mm
	B=h	30	mm
	A	2.27	cm^2
	I	1.80	cm^4
	W	0.85	cm^3

The solicitations obtained from Sap 2000 are shown in table 3.13.

Table 3.13. Solicitations on the gantry.

Bending moment, M[kNm]	**Shear force, V[kN]**	**Axial force, N[kN]**
44.81	27.21	22.8

The design verifications of the element in compression are presented in table Table 3.14.

Table 3.14. Design verifications of the element in compression for the gantry frame.

Designation	Verification	Value	Observation
Web in bending	$\frac{d}{tw} \leq 72\varepsilon$	7.5 < 66.56	Class 1
Resistance to compression,	$N_{Ed} \leq N_{b,Rd}$	22.8 kN < 203 kN	Verified
Resistance in bending,	$M_{Ed} \leq M_{pl,Rd}$	44.81 KNm > 0.47 kNm	Not Verified

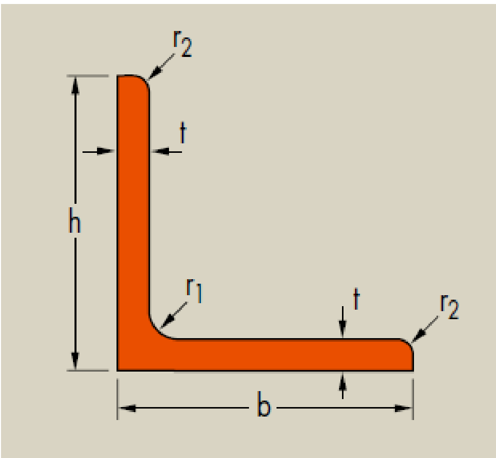
To cater for this non-verification, the section is changed to 2L150X150X15 for the bending moment to be verified. Thus the new value of design moment resistance is obtained.

$$M_{Ed} = 44.81 \text{ KNm} \leq M_{pl,Rd=}45.94 \text{ KNm}$$

3.5.1.3. Design verifications of the bracing system

In this section, the bracing system of the roof will be designed. For the bracing system of the roof, the elements with the maximum tension force will be designed. The axial force solicitations obtained is $N_{Ed} = 21.61$ KN. The section of the element under study is L30x30x4. Its characteristics are given in Table 3.15.

Table 3.15. Properties of L30X20X4.



t	4	mm
B=h	30	mm
A	2.27	cm^2
I	1.80	cm^4
W	0.85	cm^3

The design verifications of the roof bracing system's element.are presented in Table 3.16.

Table 3.16. Design verifications of the roof's bracing system.

Designation	Verification	Value	Observation
Web in bending	$\frac{d}{tw} \leq 72\varepsilon$	7.5 < 66.56	Class 1
Resistance in tension	$N_{Ed} \leq N_{pl,Rd}$	21.61 KN < 62.43 KN	Verified

3.5.1.4. Design verifications of the column

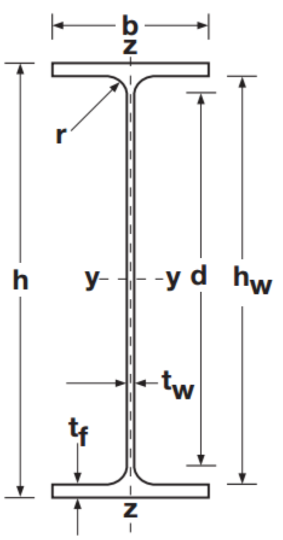
Here, the main interest will be to design the column element with maximum axial force, shear and bending moment. The solicitations obtained are given in Table 3.17.

Table 3.17. Solicitations on the column.

Bending moment, M[kNm]	Shear force, V[kN]	Axial force, N[kN]
587.21	108.60	60.55

HE200A is used with the cross-section and dimensions as shown in Table 3.18.

Table 3.18. Properties of HE200A for the column



G	42.3	kg/m
h	190	mm
b	200	mm
t_w	6.5	mm
t_f	10	mm
r	18	mm
A	53.8	cm²
h_w	170	mm
d	134	mm
I_y	3692	cm^4
I_z	1336	cm^4
i_y	8.28	cm
i_z	4.98	cm
$W_{pl,y}$	429.5	cm^3
$W_{pl,z}$	203.8	cm^3

The design verifications of the column are presented in table Table 3.19.

Table 3.19. Column design verifications

Designation	Verification	Value	Observation
Web in bending	$\frac{d}{tw} \leq 72\varepsilon$	21.87 < 66.56	Class 1
Flange in compression	$\frac{c}{tf} \leq 9\varepsilon$	5.52 < 8.28	Class 1
Resistance in bending,	$\mathrm{M_{Ed}} \leq \mathrm{M_{pl,Rd}}$	587.21 kNm > 204.77 kNm	Not Verified
Resistance in shear	$\mathrm{V_{Ed}} \leq \mathrm{V_{pl,Rd}}$	108.60 kN < 399.79 kN	Verified
Moment and shear interaction	$\mathrm{V_{Ed}} \leq 0.5\mathrm{V_{pl,Rd}}$	108.60 kN < 199.89 kN	No bending moment reduction
Resistance in compression,	$\mathrm{N_{Ed}} \leq \mathrm{N_{pl,Rd}}$	60.55 kN < 2112 kN	Verified
Bending and axial interaction	$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} \leq 1$	2.90> 1	bending-axial interaction
	$N_{Ed} \leq 0.25 N_{pl,Rd}$	60.55 kN < 528 kN	Axial forces doesn't affect moments
	$N_{Ed} \leq \frac{0.5\ \mathrm{h_w t_w f_y}}{\gamma_{\mathrm{M0}}}$	60.55kN < 212.44 kN	
Resistance to Buckling	$\chi \leq 1$	0.80 < 1	verified
Stability of the column	$\frac{\mathrm{N_{Ed}}\gamma_{\mathrm{M1}}}{\chi_{\min} A \mathrm{f_y}} + \frac{\mathrm{M_{eq,Ed}}\gamma_{\mathrm{M1}}}{\mathrm{W}_{pl}\mathrm{f_y}\left(1 - \frac{N_{Ed}}{N_{cr}}\right)} \leq 1$	0.70< 1	verified

For the resistance in bending to verify, the column section is increased to HE340B such that:

$$\mathrm{M_{Ed}} = 530.05 KNm \leq \mathrm{M_{pl,Rd}} = 662.2\ \mathrm{KNm}$$

$$\mathrm{N_{Ed}} = 60.55 KN \leq \mathrm{N_{pl,Rd}} = 4699.75\ \mathrm{KN}$$

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} = 0.83 \leq 1 \text{ which is OK}$$

$\frac{N_{Ed}\gamma_{M1}}{\chi_{min}Af_y} + \frac{M_{eq,Ed}\gamma_{M1}}{W_{pl}f_y\left(1-\frac{N_{Ed}}{N_{cr}}\right)} = 0.70 \leq 1$ which is ok

3.5.1.5. Design verifications of foundation

The footing of the column with the maximum axial force shall be designed. For the foundation, the section of footing used is a 100 cm x 150 cm x 50 cm the effective depth, d=50 cm. The longitudinal reinforcement for the top and bottom mesh is 12 Φ 12 for each mesh. The steel plate used has as dimension 430 x 260 x 15 mm.

The solicitations obtained are given in Table 3.20.

Table 3.20. Solicitations of the foundation.

Bending moment, M[kNm]	Shear force, V[kN]	Axial force, N[kN]
6.81	11.1	75

The design verifications of the foundation.are presented in Table 3.21.

Table 3.21. Foundation design verifications

Designation	Verification	Value	Observation
Bearing capacity,	$\sigma_{sol} \leq \sigma_{adm}$	0.01 MPa < 0.15 MPa	Verified
Compressive resistance,	$\sigma \leq f_{ck}$	0.04 MPa < 25 MPa	Verified
Resistance in bending	$M_{Ed} \leq M_{lim}$	6.81 kNm < 862.5 kNm	Verified
Longitudinal reinforcement of the footing	$\frac{M_{Ed}}{0.87 f_{yk} z} \leq A_s$	165.5 mm^2 < 1300 mm^2	Verified
	$A_{s,min} \leq A_s$	540 mm^2 < 1300 mm^2	Verified
Thickness of the plate,	$t_p \geq \mu \sqrt{\frac{3\sigma}{f_y}}$	15 mm > 9.3 mm	Verified

3.5.1.6. Design verifications of connection

The design of the connection is done here by choosing one of the connections of the structure. The selected connection is the one forming a junction between the column and the gantry beam. 4 bolts of class 8.8 and diameter 14mm were used for the gantry beam-column connection Table 3.22 shows the solicitations obtained for the beam to column connection.

Table 3.22. Bolt's solicitations

Shear force, V[kN]	**Axial force, N[kN]**
60.96	85.47

The design verifications of the gantry beam to column connection .are presented in Table 3.23.

Table 3.23. Bolt design verifications

Designation	**Verification**	**Value**	**Observation**
Shear resistance for one bolt,	$\mathrm{F_{v,Ed}} \leq F_{v,Rd}$	60.96 KN< 120.6 KN	Verified
Traction resistance for one bolt,	$\mathrm{F_{v,Ed}} \leq F_{t,Rd}$	85.47 KN < 90.43 KN	Verified
Bearing resistance for the plate,	$\mathrm{F_{v,Ed}} \leq F_{b,Rd}$	85.47 KN < 154.8 KN	Verified
Shear and traction of the bolt interaction,	$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4F_{t,Rd}} \leq 1$	0.92 < 1	Verified

3.5.2. Serviceability limit state verification of the structure without bridge crane

The SLS verification of the steel structure consists of checking the deflection of the structural members. Deflection is the displacement within a structural member under the influence of loads, ignoring the displacements of the rest of the structure. The deflection due to the load combination of SLS is shown in table 3.24.

Table 3.24. Deflection check of structural members

Designation	**Verification**	**Value**	**Observation**

Purlin	$\delta < \frac{l}{200}$	7.57x10^-6 mm< 25 mm	Verified
Gantry	$\delta < \frac{l}{200}$	2.89 x10^-3 mm< 69 mm	Verified
Bracing system	$\delta < \frac{l}{200}$	8.57 mm< 29 mm	Verified
Column	$\delta < \frac{h}{300}$	1.34 mm < 16.82 mm	Verified

3.5.3. Ultimate limit state design and verification of the structure with bridge crane

In addition to the building configuration above, the bridge crane was modelled by drawing the running beams in Sap2000 longitudinally and transversally along the columns. Its loads were applied as moving loads. In order to improve the performance of the bridge crane, the results of the influence lines in terms of stresses due to moving loads of the crane trolley obtained in terms of moment and shear force is used to design the longitudinal crane and the transversal crane is designed using the shear stress.

The maximum bending moment in the running beams of span 19.6 m and 34 m subjected to a couple of moving loads 73.7 KN at a spacing of 2.575 m is presented in figure 3.5. This is the critical arrangement for vertical bending in order to obtain maximum moment.

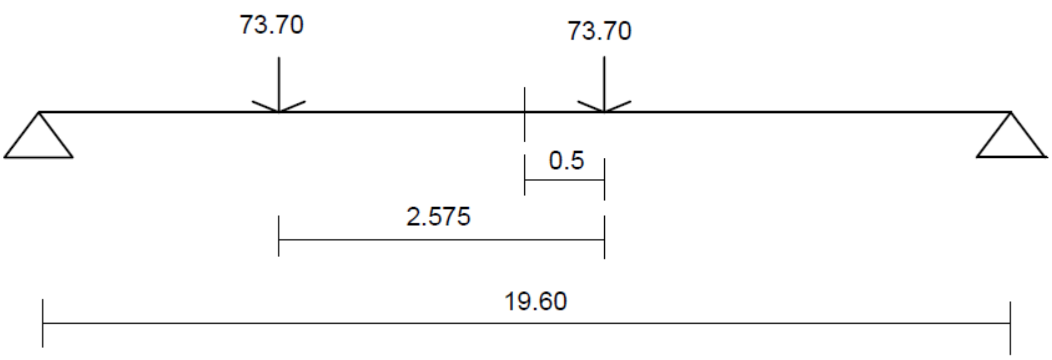


Figure 3.5. Critical arrangement of vertical bending for maximum moment

The position of these loads produces the influence line diagram for maximum moment as shown in figure 3.6.

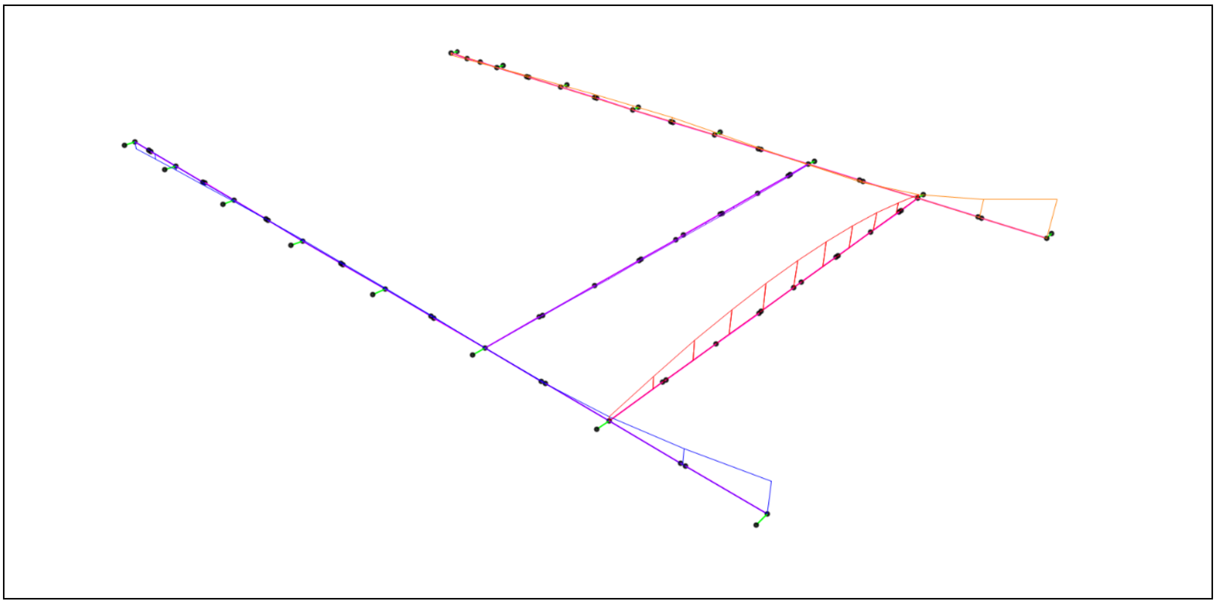


Figure 3.6. Influence line diagram for Bending moment

The maximum shear force in the running beams of span 19.6 m and 34 m subjected to a couple of moving loads 73.7 KN at a spacing of 2.575 m is presented in figure 3.7.

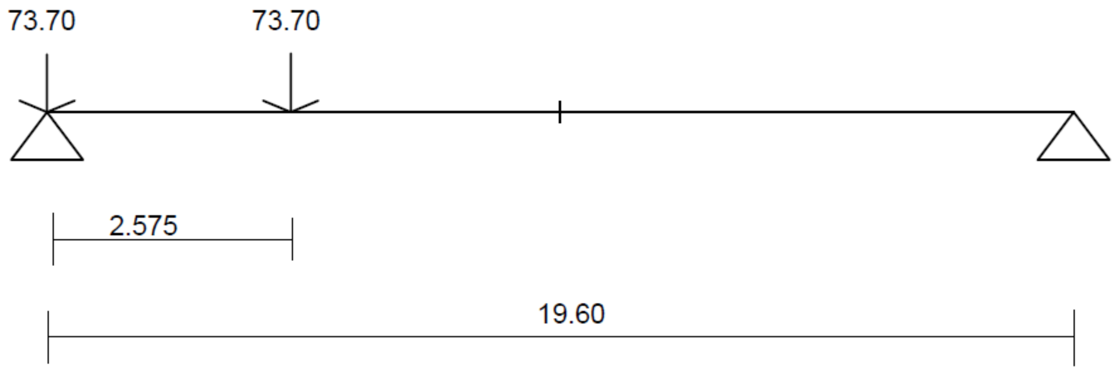


Figure 3.7. Critical arrangement of vertical shear for maximum shear

With the position of first load being directly on the support, this produces the influence line diagram of maximum shear force as shown in figure 3.8.

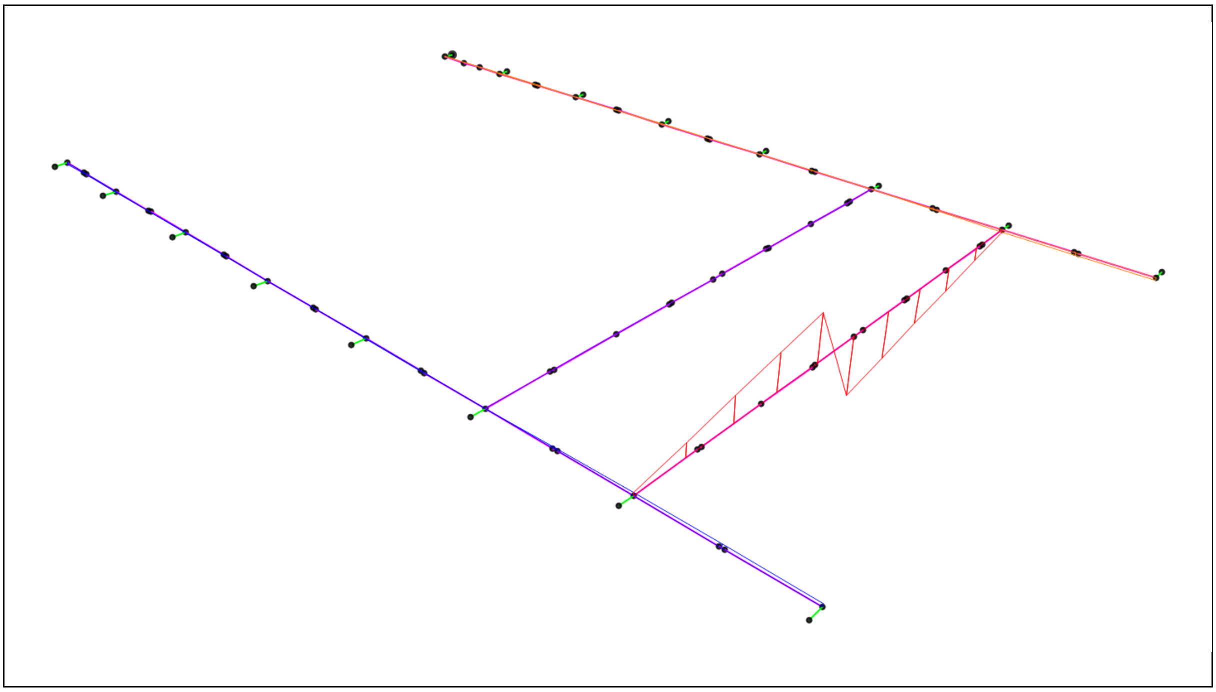


Figure 3.8. Influence line diagram for Shear force

In order to obtain maximum horizontal moment and deflection, the critical arrangement for horizontal loading is done as shown in figure 3.9.

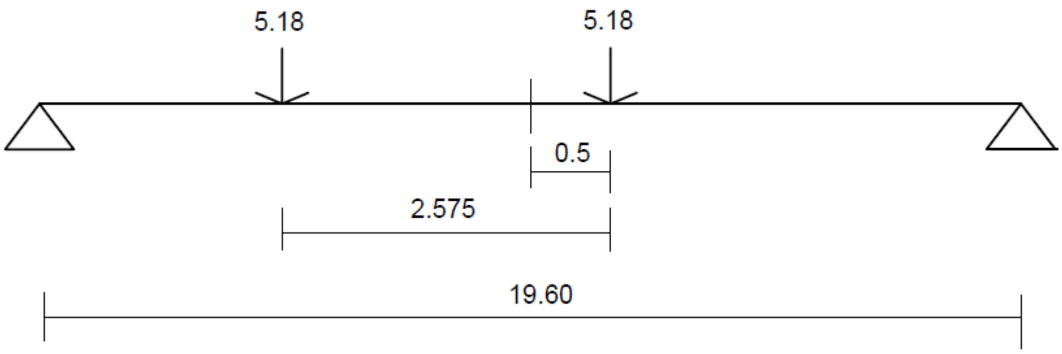


Figure 3.9. Critical arrangement for maximum horizontal moment and deflection

The position of these loads produces the influence line diagram as shown in figure 3.10. With the spacing between the loads been less than half the span, the maximum deflection develops when one load is at an eccentricity from the middle of the span.

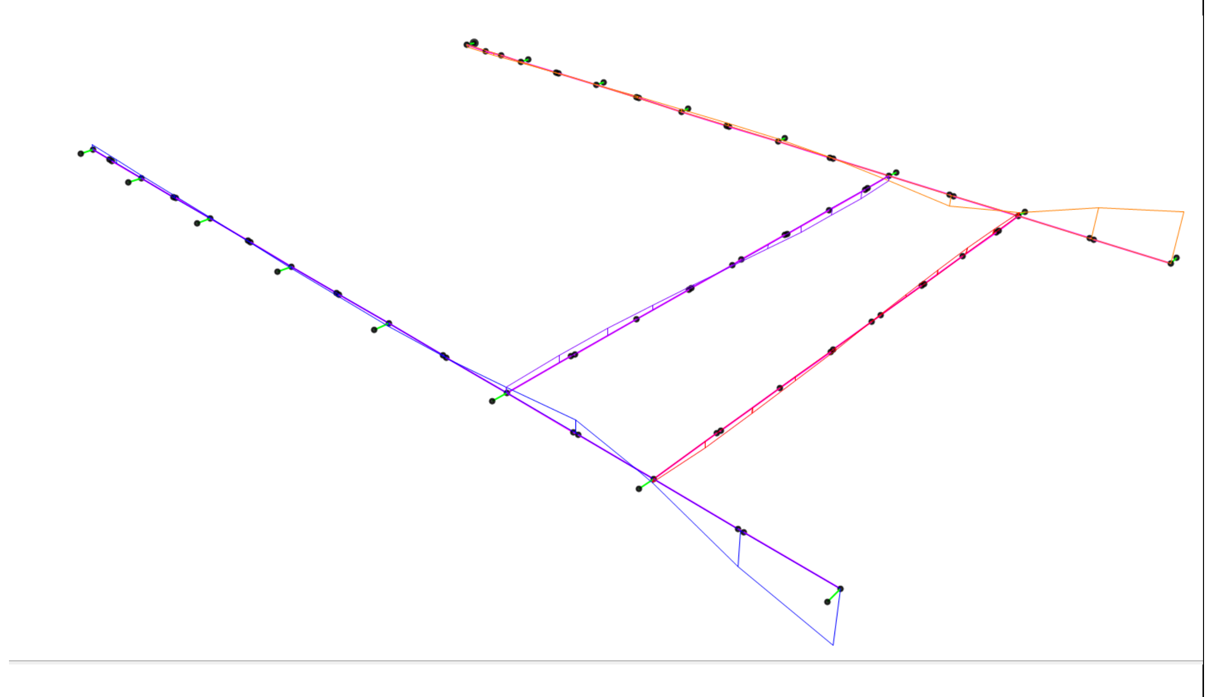


Figure 3.10. Influence line diagram for Shear

3.4.3.1. Preliminary design

The admissible deflection limit obtained permits to obtain the section which verifies the deflection condition.

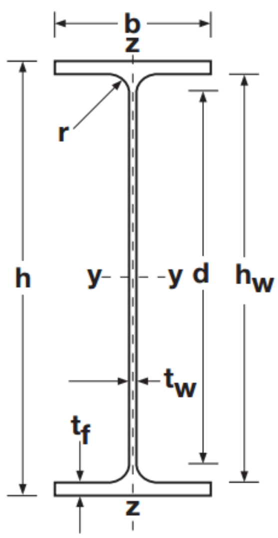
f_{adm}=26.13 mm

$I_y \geq 182020.77 \times 10^4$ mm^4

HE700B section is chosen and its characteristics are shown in Table 3.25.

Table 3.25. Properties of HE700B for the running beam

	G	241	kg/m
	h	700	mm
	b	300	mm
	t_w	17	mm
	t_f	32	mm
	r	27	mm
	A	306.4	cm²



h_w	636	mm
d	582	mm
I_y	256900	cm^4
I_z	14440	cm^4
i_y	28.96	cm
i_z	6.87	cm
$W_{pl,y}$	8327	cm^3
$W_{pl,z}$	1495	cm^3

Table 3.26. Crane loads specifications

Nominal capacity	10 t = 100 kN
Beam weight:	60 kN
Trolley weight	9 kN
Transversal Crane span:	19.6 m
Longitudinal Crane span:	34 m
Spectral load class	Q3
Total number of cycles class	U8
Trolley nominal speed	0.333 m/s
Number of rails	2
Number of wheels on each side	2
Lifting height	4 m
Rail size	A100
Power supply	380 V, 50 Hz, 3 Phase system

3.4.3.2. Proper design verifications

The verifications of the structure with a bridge crane are performed under the design load F_{Ed}.

$$F_{Ed} = 1.3G + 1.5Qkw + 1.5Qkr + 1.5\gamma Qmax = 274.62\ KN$$

a. Running beam design verifications

For the double girder bridge crane of 10 Tons, and a span of 19.6 m the maximum reactions per wheel is Qmax= 73.7 KN and Qmin= 18.2 KN. This is gotten statically using the equations (2.72) and (2.73) in accordance with the double girder bridge crane Abus catalogue.

From this maximum vertical loads the horizontal load can be calculated by applying 5% of the vertical load.

Hwmax=5.18 KN

Hwmin=1.82 KN

These maximum vertical and horizontal loads were applied in sap2000 as moving loads and the solicitations in Table 3.27 were obtained:

Table 3.27. Solicitations on the running beam

Bending moment, M[kNm]	**Shear force, V[kN]**	**Axial force, N[kN]**
1796.01	563.61	608.56

Firstly, the section is classified and then the design verifications of the running beam. are presented in Table 3.28.

Table 3.28. Running beam design verifications

Designation	**Verification**	**Value**	**Observation**
Web in bending	$\frac{d}{tw} \leq 72\varepsilon$	34.24 < 66.24	Class 1
Flange in compression	$\frac{c}{tf} \leq 9\varepsilon$	3.34 < 8.28	Class 1

Under Vertical Loads			
Resistance in bending,	$M_{Ed} \leq M_{pl,Rd}$	1796.01kNm<2289.93kNm	Verified
Resistance in shear	$V_{Ed} \leq V_{pl,Rd}$	563.61kN< 1885.13kN	Verified
Moment and shear interaction	$V_{Ed} \leq 0.5V_{pl,Rd}$	563.61kN< 1885.13kN	No bending moment reduction
Under Horizontal Loads			
Resistance to buckting,	$M_{Ed} \leq M_{b,Rd}$	44.7 kNm < 1854.81 kNm	Verified
Under Transversal Loads			
Resistance to crushing	$Qr_{max} \leq R_{y,Rd}$	92.125 KN< 937.17 KN	Verified
Resistance to local bending	$M_{Ed} \leq M_{pl,Rd}$	1796.01KNm< 2289.9kNm	Verified
Resistance to local bending	$f_{sd} \leq R_{a,Rd}$	5.12 KN < 1807.29 kN	Verified
Resistance to local bending	$\frac{f_{sd}}{R_{a,Rd}} + \frac{M_{Ed}}{M_{pl,Rd}} \leq 1.5$	0.36 < 1.5	Verified
Web buckling resistance	beff$\leq b$	707.11 ≥ 300	Not verified
Resistance in compression	$N_{Ed} \leq N_{pl,Rd}$	608.56 kN < 8426 kN	Verified

Bending and axial interaction	$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} \leq 1$	0.86 < 1	No bending axial interaction
	$N_{Ed} \leq 0.25 N_{pl,Rd}$	608.56 kN < 2106.5 kN	Axial forces doesn't affect moments
Flange buckling resistance	$\frac{d}{tw} \leq k \times \frac{E}{fyk} \times \sqrt{\frac{Aw}{Afc}}$	34.24< 243.12	Verified
Fatigue verification	$\Phi < 1$	0.21<1	Verified

b. Running beam support design verifications

The running beam is supported by the running beam support which is stressed by the solicitations given in Table 3.29.

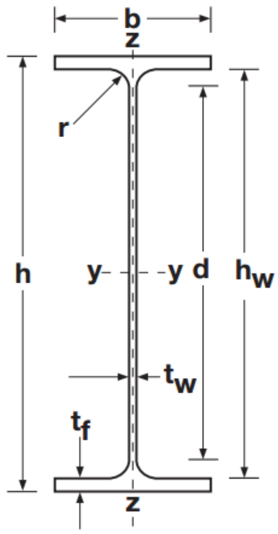
Table 3.29. Solicitations on the running beam support

Bending moment, M[kNm]	**Shear force, V[kN]**	**Axial force, N[kN]**
1003.87	813.9	-332.91

HE450B section is selected and has the characteristics shown in Table 3.30.

Table 3.30. Properties of HE450B for the running beam support

	G	171	kg/m
	h	450	mm
	b	300	mm
	t_w	14	mm
	t_f	26	mm
	r	27	mm



A	218	cm²
h_w	398	mm
d	344	mm
I_y	79890	cm^4
I_z	11720	cm^4
i_y	19.14	cm
i_z	7.33	cm
$W_{pl,y}$	3982	cm^3
$W_{pl,z}$	1495	cm^3

The design verifications of the running beam support.are presented in Table 3.31.

Table 3.31. Running beam support design verifications

Designation	Verification	Value	Observation
Web in bending	$\frac{d}{tw} \leq 69\varepsilon$	24.57< 66.24	Class 1
Flange in compression	$\frac{c}{tf} \leq 10\varepsilon$	4.23< 9.2	Class 1
Resistance in bending,	$M_{Ed} \leq M_{pl,Rd}$	836.49 kNm < 1095.05 kNm	Verified
Resistance in shear	$V_{Ed} \leq V_{pl,Rd}$	1003.87 kN < 1264.77 kN	Verified
Moment and shear interaction ,	$V_{Ed} \leq 0.5V_{pl,Rd}$	1003.87 kN < 632.39 kN	Bending moment reduction
Resistance in compression,	$N_{Ed} \leq N_{pl,Rd}$	332.9 kN < 5995 kN	Verified
Bending and axial interaction	$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} \leq 1$	0.82 < 1	No bending-axial interaction

	$N_{Ed} \leq 0.25N_{pl,Rd}$	332.9 kN < 1498.75 kN	Axial forces doesn't affect moments
Flange buckling resistance	$\frac{d}{tw} \leq k \times \frac{E}{fyk} \times \sqrt{\frac{Aw}{Afc}}$	24.57< 163.60	Verified

c. Design of connection

This is the connection between the existing column and the running beam using bolts. This is a moment connection which allows the transfer of bending moment between the two members and provides an additional stability to the steel structure.

This is thus the design of a bolted cover plate running beam splice that connects a HE700B S275 section to a HE340B column. The splice carries a vertical shear, an axial force and bending moment.

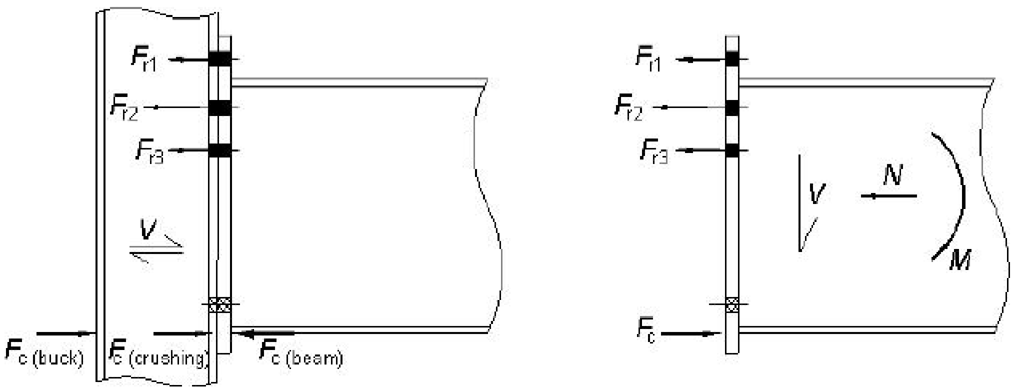


Figure 3.11. Effective resistances of bolt rows

The effective resistances of each of the three bolts in the tension zone are

F_{t1Rd}= 292.05 KN

F_{t2Rd}= 292.05 KN

F_{t3Rd}= 292.05 KN

The effective resistances of the tension rows expressed in kilonewtons [kN] may be summarized in Table 3.32 after verification of the column flange in bending, column web in tension, end plate in bending and beam web in tension.

Table 3.32. Effective resistances in the tension rows

	Column flange[kN]	**Column web[kN]**	**End plate[kN]**	**Beam web[kN]**	**Minimum [kN]**	**Effective resistance [KN]**
Row 1	292.05	464.4	382.97	-	292.05	**292.05**
Row 2 alone	292.05	484.4	406	1031.02	292.05	
Row 2 with row 1	812	1038.18	-	-	812	
Row 2					812-519.95	**292.05**
Row 3 alone	292.05	464.4	406	840.1	292.05	
Row 3 with row1 and 2	1218	1854.27	-	-	1218	
Row 3					1218-925.95	**292.05**
Row 3 with row 2	1101.67	1005.18	812	812	812	
Row 3					812-519.9	

With the calculations done for each row at a time, followed by the combination of the next row, the effective resistances of each row alone must be calculated in order to obtain the suitable dimensions of the flange and cover plate. Also after verifications, the buckling resistance of the flange cover plate in compression is adequate with the dimensions presented in Table 3.33.

Table 3.33. Flange cover plates

Dimensions and bolt spacing		
Thickness t_{fp}	20	mm
Length h_{tp}	640	mm
Width b_{tp}	210	mm
End distance e_{1tp}	80	mm
Edge distance e_{2tp}	40	mm
Spacing in the direction of the force p_{1f}	100	mm
Spacing in the transverse direction of the force p_{2f}	140	mm
Spacing in the direction of the force p_{1fj}	140	mm

The full bearing resistance was greater than the resultant force of the most highly loaded bolt (393.6 KN > 315.98 KN) and was shown to be adequate if the edge and end distances are sufficiently large that they do not limit the bearing resistance. These minimum distances have been achieved in the configuration of table 3.34, hence the bolt resistance is adequate.

Table 3.34. Web cover plates

Dimensions and bolt spacing		
Thickness t_{wp}	30	mm
Length h_{wp}	5601	mm
Width b_{wp}	1120	mm

End distance e_{1wp}	80	mm
Edge distance e_{2wp}	80	mm
Spacing in the direction of the force p_{1w}	200	mm
Spacing in the transverse direction of the force p_{2w}	200	mm
Spacing in the direction of the force p_{1wj}	200	mm

The construction details are as follows:

The plate on the running beam-column connection contains bolts of nominal diameter 39 mm with hole diameter 42 mm. The bolts' class 8.8 are globally subjected to shear and bending. The edge and end distances are of 80 mm and the spacing dimensions are 200 mm. These are on a flange cover plate of dimensions 210 X20 X 720 mm and a web cover plate of dimensions 1120 X 30 X560 mm which were selected to obtain full bearing resistance and strength.

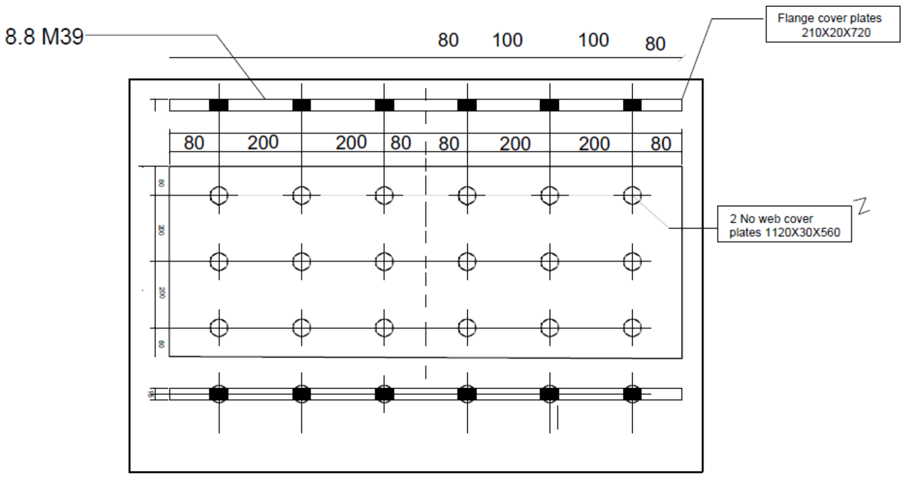


Figure 3.12. Joint configuration between running beam and column

For the flange splice, the full bearing resistance is satisfactory when using 9 M39 bolts in shear and three vertical lines of 3 bolts at a spacing of 200 mm.

For the web splice, three lines of 3 bolts are used on either side of the centerline for the full bearing resistance to be attained. This is at a vertical spacing of 200 mm from the centerline of the splice. All these have gone through the following verifications; column flange in bending, column web in transverse tension, end plate in bending, beam web in tension and were ok. The bolt connection designs are shown in figure 3.13 and figure 3.14.

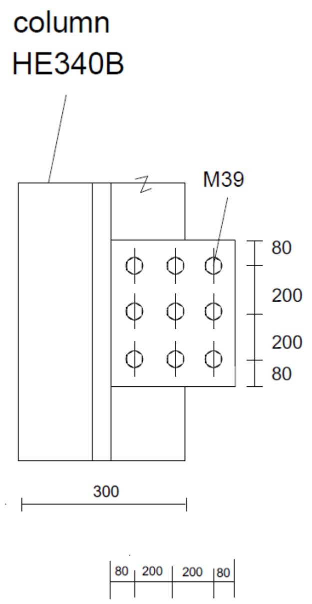


Figure 3.13. Joint column connection

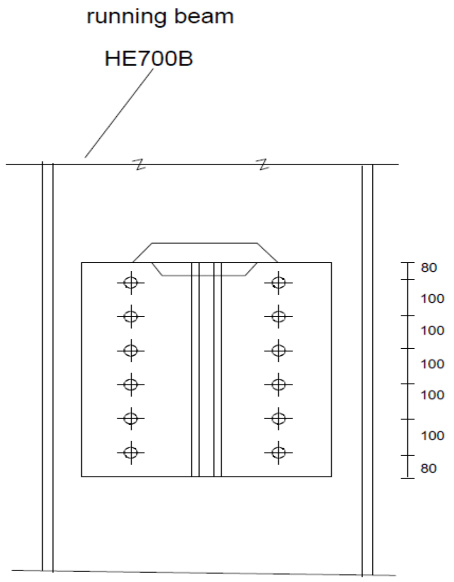


Figure 3.14. Running beam Bolt configuration

The rail chosen is the rail A100 crane rail of mass 74.3 kg/m and standard length of 12 m. The rail is a kind of steel rail with lower height of 95mm, head width of 100 mm and bottom width 200 mm. The thickness of the web is 60mm and has a tensile strength of 880 N/mm² suitable for a good resistance against tension.

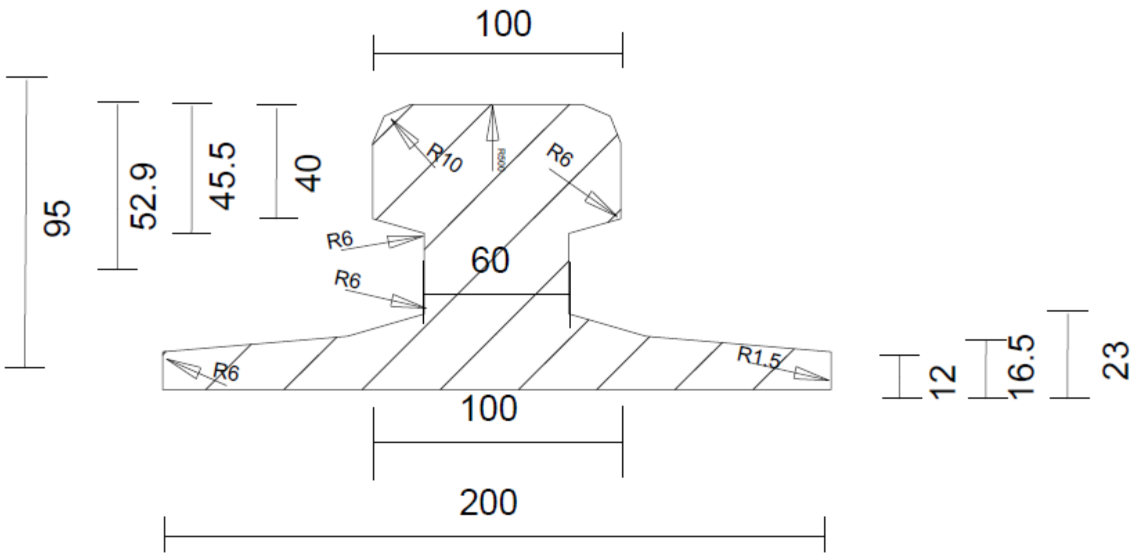


Figure 3.15. Rail with a large bolted section

The rail is used for laying the tracks of crane carts and trolley of bridge cranes. The rail should be continuous over the joints of the running beams. The joints are oblique to the rail direction at a distance from the running beam ends. The fixing of the rails on the running beams is rigid with the connection of the bolts passing through the flange of the rail. This connection permits to resist against longitudinal forces between the rail and the beam due to bending, in addition to the lateral forces applied by the wheels

The running beam in figure 3.14 is a HE700B profile of mass 241 kg/m with a height of 700 mm, width 300 mm and cross sectional area of 306.4 cm². This is the longitudinal load-bearing element of the path, hot rolled section which is connected to a set of end carriages. The bridge crane runs on the running beam, which is then fitted with a special rail. The bridge crane runs directly on the lower flanges of the running beam

The transversal running beam is what the hoist unit traverses along with below it running beam supports. It is a double girder configuration and with a bolted connection types between the end carriages.

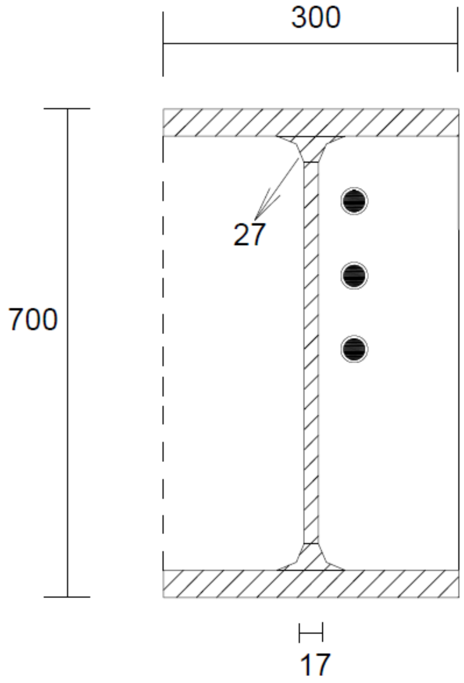


Figure 3.16. Running beam section

Indicative details of the running beam-column connection are presented in Figure 3.17. Runway beams are bolted to the supporting elements through their lower flange.

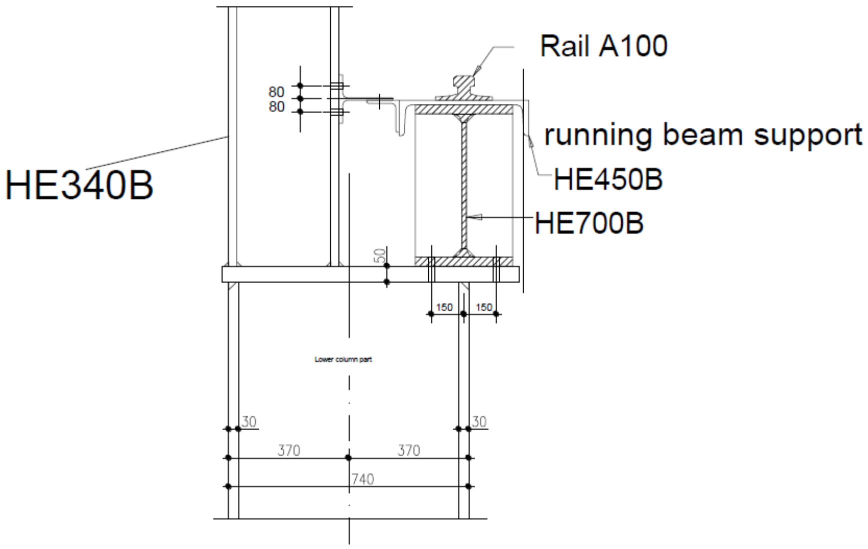


Figure 3.17. Crane supporting beams

The connection of the running beam to the column should be realized in such a way as to ensure that:

- The cross-section at the support cannot twist in order to fulfil the assumptions for the verification against lateral-torsional buckling
- The beam should be free to develop vertical and horizontal deformations as well as the corresponding end rotations within limits.

d. Design verifications of the purlin

Here, the design is the same as that done above but with the addition of the forces of the bridge crane been added. Our interest will be to design the purling element with maximum axial, shear forces and bending moment. The solicitations obtained are given in Table 3.35.

Table 3.35. Solicitations on the purlin

Bending moment, M[kNm]	**Shear force, V[kN]**	**Axial force, N[kN]**
72.33	7.64	295.76

IPE 160 was used for the purlin's section.

The design verifications of the purlin.are presented in Table 3.36.

Table 3.36. Purlin design verifications

Designation	**Verification**	**Value**	**Observation**
Web in bending	$\frac{d}{tw} \leq 72\varepsilon$	25.44 < 66.56	Class 1
Flange in compression	$\frac{c}{tf} \leq 9\varepsilon$	3.99 < 8.28	Class 1
Resistance in bending,	$M_{Ed} \leq M_{pl,Rd}$	72.33 kNm < 34.1 KNm	Not Verified
Resistance in shear	$V_{Ed} \leq V_{pl,Rd}$	7.64 kN < 153.37 kN	Verified

Moment and shear interaction	$V_{Ed} \leq 0.5V_{pl,Rd}$	7.64kN < 76.69 kN	No bending moment reduction
Resistance in compression,	$N_{Ed} \leq N_{pl,Rd},$	295.76 kN < 552.75 kN	Verified
Bending and axial interaction	$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} \leq 1$	2.66 >1	Not Verified
	$N_{Ed} \leq 0.25N_{pl,Rd}$	295.76 kN >276.375 kN	Bending moment reduction
	$N_{Ed} \leq \frac{0.5\, h_w t_w f_y}{\gamma_{M0}}$	295.76kN > 110.55 kN	bending moment reduction

To solve this non verification the purlin section is increased to IPE240 such that:

$$M_{Ed} = 72.33KNm \leq M_{pl,Rd} = 112.75\ KNm$$

$$N_{Ed} = 295.76KN \leq N_{pl,Rd} = 1262.25\ KN$$

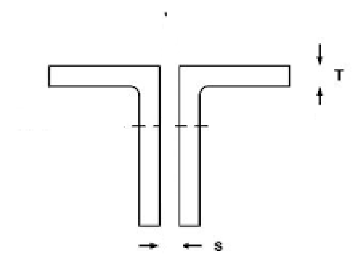
$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} = 0.88 \leq 1 \text{ which is OK}$$

$$\frac{N_{Ed}\gamma_{M1}}{\chi_{min}Af_y} + \frac{M_{eq,Ed}\gamma_{M1}}{W_{pl}f_y\left(1-\frac{N_{Ed}}{N_{cr}}\right)} = 0.75 \leq 1 \text{ which is OK}$$

e. Design verifications of the gantry frame

To design the gantry frame, the elements in compression of the roof will be verified. To design the elements in compression of the roof, the element with the maximum compressive force will be verified. The section of the element under study is 2L30X30X4. Its characteristics are given in Table 3.37.

Table 3.37. Properties of 2L30X20X4



t	4	mm
B=h	30	mm
A	2.27	cm^2
I	1.80	cm^4
W	0.85	cm^3

The solicitations obtained are given in Table 3.38.

Table 3.38. Solicitations on the gantry

Bending moment, M[kNm]	**Shear force, V[kN]**	**Axial force, N[kN]**
44.84	34.6	334.01

The axial force solicitations obtained $N_{Ed} = 334.01KN$. The design verifications of the element in compression are presented in table Table 3.39.

Table 3.39. Design verifications of the element in compression for the gantry frame

Designation	**Verification**	**Value**	**Observation**
Web in bending	$\frac{d}{tw} \leq 72\varepsilon$	7.5 < 30.36	Class 1
Resistance to compression,	$N_{Ed} \leq N_{b,Rd}$	106.28kN < 203 kN	Verified
Resistance in bending,	$M_{Ed} \leq M_{pl,Rd}$	44.84KNm < 0.47kNm	Not Verified

To cater for this non-verification, the section is changed to 2L150X150X15 for the bending moment to be verified. Thus the new value of design moment resistance is obtained.

$$M_{Ed} = 44.84KNm \leq M_{pl,Rd=}45.94KNm$$

f. Design verifications of the brace of the roof

To design the brace of the roof, the elements in compression of the roof will be verified. The axial force solicitations obtained is $N_{Ed} = 201.65KN$. The section of the element under study is L30X20X4.

The design verifications of the roof’s bracing system are presented in Table 3.40.

Table 3.40. Design verifications of the roof's bracing system

Designation	**Verification**	**Value**	**Observation**
Web in bending	$\frac{d}{tw} \leq 72\varepsilon$	15< 66.56	Class 1

Resistance in tension,	$N_{Ed} \leq N_{pl,Rd}$	201.65 KN > 62.43 KN	Not Verified

For the resistance in tension to verify, the section is changed to L80X80X55. Thus the new value of design axial force resistance is obtained.

$$N_{Ed} = 201.65KN \leq N_{pl,Rd=}216.15KN$$

g. Design verifications of the column

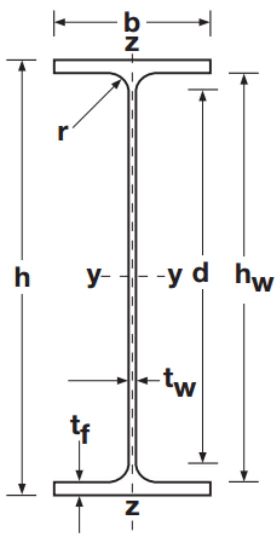
Here, the main interest will be to design the column element with maximum axial, shear forces and bending moment. The solicitations obtained are given in Table 3.41.

Table 3.41. Solicitations of the column

Bending moment, M[kNm]	**Shear force, V[kN]**	**Axial force, N[kN]**
574.16	319.65	834.1

HE200A is selected with the cross-section and dimensions shown in Table 3.42.

Table 3.42. Properties of HE200A for the column



G	42.3	kg/m
h	190	mm
b	200	mm
t_w	6.5	mm
t_f	10	mm
r	18	mm
A	53.8	cm^2
h_w	170	mm
d	134	mm
I_y	3692	cm^4
I_z	1336	cm^4
i_y	8.28	cm

	i_z	4.98	cm
	$W_{pl,y}$	429.5	cm^3
	$W_{pl,z}$	203.8	cm^3

The design verifications of the column are presented in Table 3.43.

Table 3.43. Column design verifications

Designation	**Verification**	**Value**	**Observation**
Web in bending	$\frac{d}{tw} \leq 72\varepsilon$	36.13 < 66.56	Class 1
Flange in compression	$\frac{c}{tf} \leq 9\varepsilon$	5.06 < 8	Class 1
Resistance in bending,	$M_{Ed} \leq M_{pl,Rd}$	574.16 kNm >188.94 kNm	Not Verified
Resistance in shear	$V_{Ed} \leq V_{pl,Rd}$	319.65 kN < 417.87 kN	Verified
Moment and shear interaction	$V_{Ed} \leq 0.5V_{pl,Rd}$	319.65 kN < 417.87 kN	No bending moment reduction
Resistance in compression,	$N_{Ed} \leq N_{pl,Rd}$	834.1KN < 1471 kN	Verified
Bending and axial interaction	$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} \leq 1$	2.99>1	Bending-axial interaction
	$N_{Ed} \leq 0.25N_{pl,Rd}$	834.1 kN < 367.75 kN	Axial forces doesn't affect moments
	$N_{Ed} \leq \frac{0.5\, h_w t_w f_y}{\gamma_{M0}}$	834.1kN < 270.5 kN	
Resistance to Buckling	$\chi \leq 1$	0.80 < 1	verified
Stability of the column	$\frac{N_{Ed}\gamma_{M1}}{\chi_{min}Af_y} + \frac{M_{eq,Ed}\gamma_{M1}}{W_{pl}f_y\left(1-\frac{N_{Ed}}{N_{cr}}\right)} \leq 1$	2.33>1	Not verified

To solve this non-verification, the column section can be increased to HE340B such that:

$$\mathrm{M_{Ed}} = 574.16KNm \leq \mathrm{M_{pl,Rd}} = 662.2KNm$$

$$\mathrm{N_{Ed}} = 834.1KN \leq \mathrm{N_{pl,Rd}} = 4699.75KN$$

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} = 0.92 \leq 1 \text{ which is OK}$$

$$\frac{\mathrm{N_{Ed}}\gamma_{\mathrm{M1}}}{\chi_{\mathrm{min}}\mathrm{A}f_{\mathrm{y}}} + \frac{\mathrm{M_{eq,Ed}}\gamma_{\mathrm{M1}}}{\mathrm{W}_{pl}f_{\mathrm{y}}\left(1-\frac{N_{Ed}}{N_{cr}}\right)} = 0.75 \leq 1 \text{ which is ok}$$

h. Design verifications of foundation

The footing of the column with the maximum axial force shall be designed. For the foundation, the section of footing used is a 150 cm x 200 cm x 50 cm the effective depth, d=50 cm. The longitudinal reinforcement for the top and bottom mesh is 16 Φ12 for each mesh. The steel plate used has as dimension 430 x 260 x 15 mm. The solicitations obtained are given in Table 3.44.

Table 3.44. Solicitations of the foundation.

Bending moment, M[kNm]	Shear force, V[kN]	Axial force, N[kN]
337.91	216.52	945.21

The design verifications of the foundation are presented in Table 3.45.

Table 3.45. Foundation design verifications

Designation	Verification	Value	Observation
Bearing capacity	$\sigma_{sol} \leq \sigma_{adm}$	0.11 MPa < 0.15 MPa	Verified
Compressive resistance,	$\sigma \leq f_{ck}$	3.33 MPa < 25 MPa	Verified
Resistance in bending,	$M_{Ed} \leq M_{lim}$	337.91 kNm < 1293.75 kNm	Verified

Longitudinal reinforcement of the footing	$\frac{M_{Ed}}{0.87 f_{yk} z} \leq A_s$	149.71 mm^2 < 1650 mm^2	Verified
	$A_{s,min} \leq A_s$	540 mm^2 < 1650 mm^2	Verified
Thickness of the plate,	$t_p \geq \mu \sqrt{\frac{3\sigma}{f_y}}$	15 mm > 10.8 mm	Verified

3.5.4. Serviceability limit state verification of the structure with bridge crane

The SLS verification of the steel structure consist of checking the deflection. The deflection due to the load combination of SLS of the structural members with bridge crane is as shown in table 3.46.

Table 3.46. Deflection check of structural members

Designation	Verification	Value	Observation
Running beam	$\delta < \frac{l}{600}$	25.16 cm < 26.13 cm	Verified
Running beam support	$\delta < \frac{l}{600}$	0.132 mm < 1mm	Verified
Purlin	$\delta < \frac{l}{200}$	0.71 mm < 25 mm	Verified
Gantry	$\delta < \frac{l}{200}$	0.83 mm< 69 mm	Verified
Bracing system	$\delta < \frac{l}{200}$	1.55mm< 29mm	Verified
Column	$\delta < \frac{h}{300}$	5.1 mm < 16.82 mm	Verified

3.6. Comparison of the results of the two structures

The structures are going to be compared based on the following criteria.

3.6.1. Comparison of the M, V, N solicitations

Table 3.47 shows the summary of the results obtained for each member of the structure.

Table 3.47. M, V, N Solicitations of the two structures

Without bridge crane				With bridge crane			
Member	**M(KNm)**	**V(KN)**	**N(KN)**	**Member**	**M(KNm)**	**V(KN)**	**N(KN)**
Purlin	6.43	1.45	26.36	Purlin	72.33	7.64	295.76
Frame	44.81	27.21	22.8	Frame	44.84	34.6	334.01
Brace	6.15	1.02	21.61	Brace	4.03	0.63	201.65
Pillar	587.21	108.60	60.55	Pillar	574.16	319.65	834.1
Foundation	6.81	11.1	75	Foundation	337.91	216.52	945.21

There is an increase in the moment, shear and axial force in the, purlin, gantry, roof brace, column and foundation. This increase is due to the presence of moving loads.

These comparisons are represented on histograms in order to summarize the data obtained on a scale. The diagrams in blue represent the structural members without the bridge crane while those in orange represent the bridge crane's effect on the structural elements.

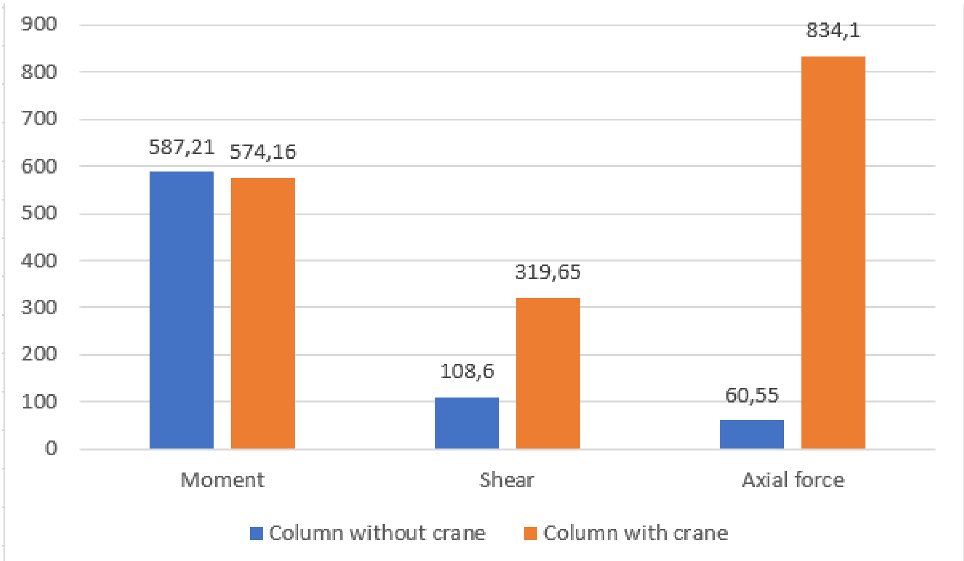


Figure 3.18. M, V, N, on column

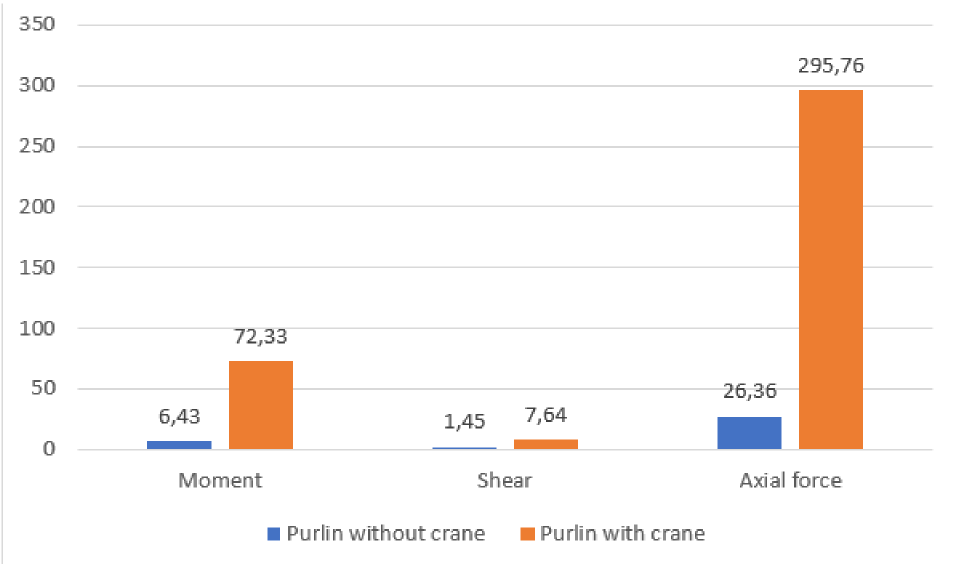


Figure 3.19. M, V, N on Purlin

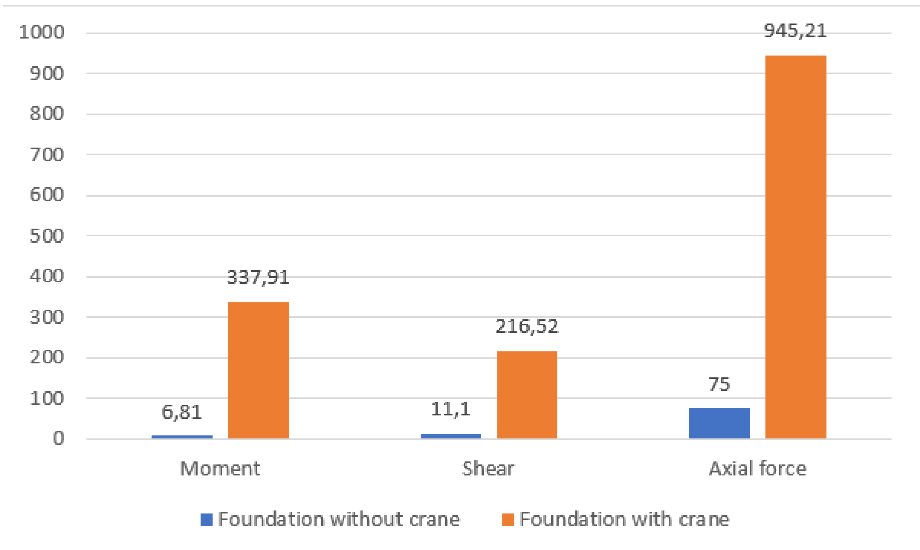


Figure 3.20. M, V, N on foundation

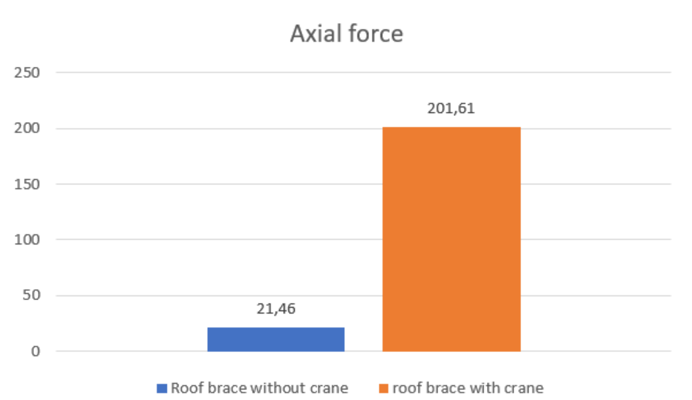


Figure 3.21. Axial force on roof brace

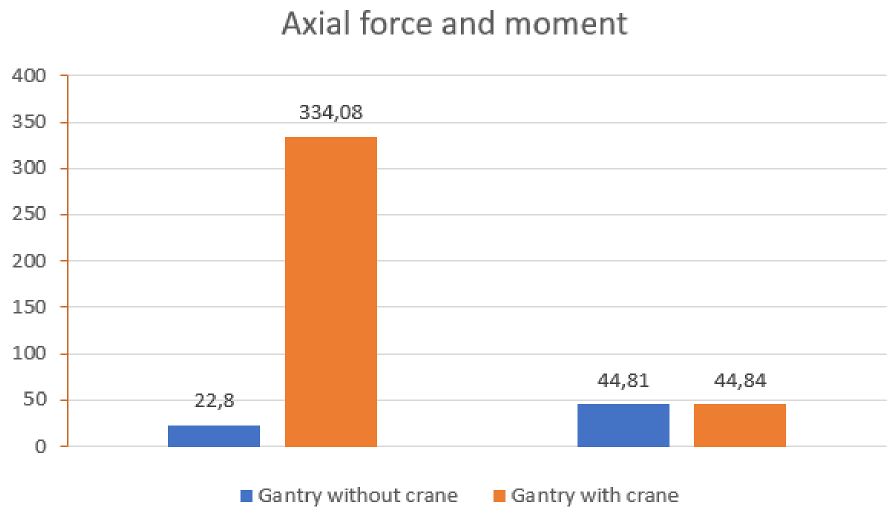


Figure 3.22. Axial force and Moment on gantry

3.6.2. Comparison of the structural performance

The structural performance is evaluated using the load carrying capacity of the elements. The load carrying capacity is calculated as the sum of the ratio of the individual efforts to their resistances. The load carrying capacity will be computed with the most loaded (critical) member of the structure.

For the steel structure without bridge crane

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} = 0.83$$

For the steel structure with bridge crane

$$\frac{N_{Ed}}{N_{c,Rd}} + \frac{M_{Ed}}{M_{c,Rd}} = 0.92$$

Comparing the load carrying capacity, there is an increase of 10.84 % as seen in Figure 3.23 and implies that the steel structure with bridge crane is more utilised than the steel structure without bridge crane. Hence the steel structure with bridge crane has a greater structural performance.

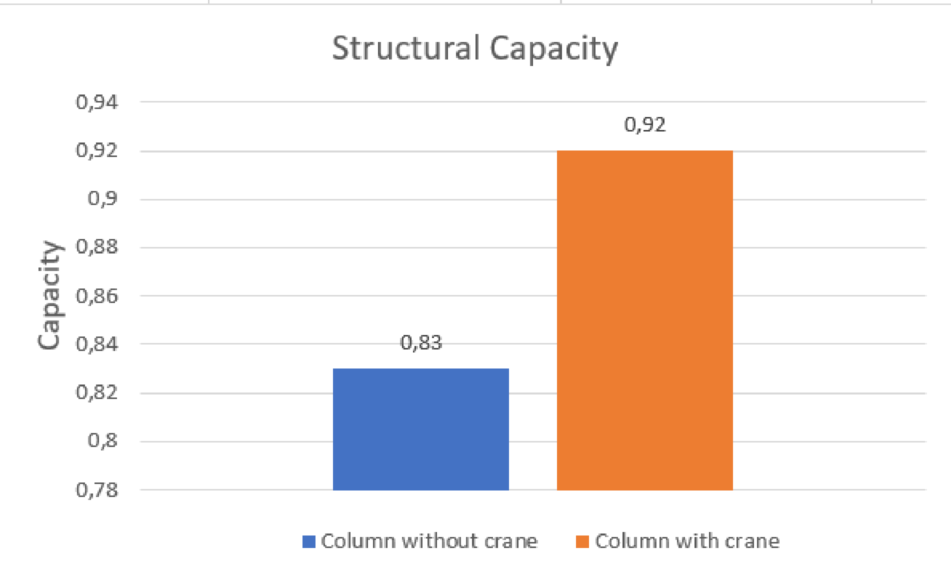


Figure 3.23. Comparison between the structural capacity of both structures.

3.6.3. Comparison of the functional performance

The functional performance of the structure will be assessed with the deflection. The deflection measured from the linear static analysis of the steel structure without a bridge crane was 0.003 mm near the midway of the structure compared to 0.83 mm for the steel structure with a bridge crane. The steel structure without bridge crane presents a better solution to reduce the deflection of the structure. Nonetheless both structures satisfy the SLS requirements

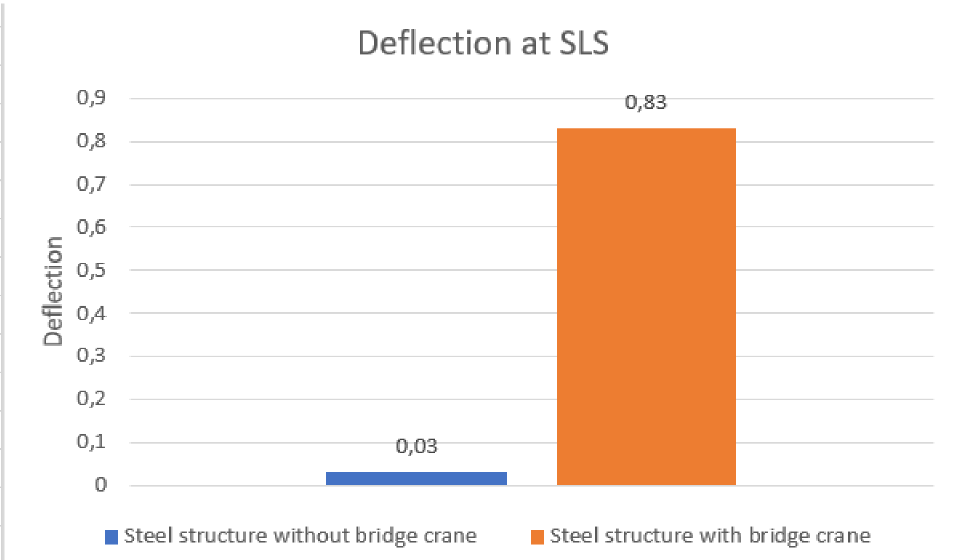


Figure 3.24. Comparison between the deflection at SLS

3.6.4. Comparison of the weight

From the masses obtained, the weight of the steel structure without bridge crane is 346.99 KN and the weight of the steel structure with bridge crane is 629.16 KN. As the bridge crane is made up of running beams which increase considerably the weight of the steel structure.

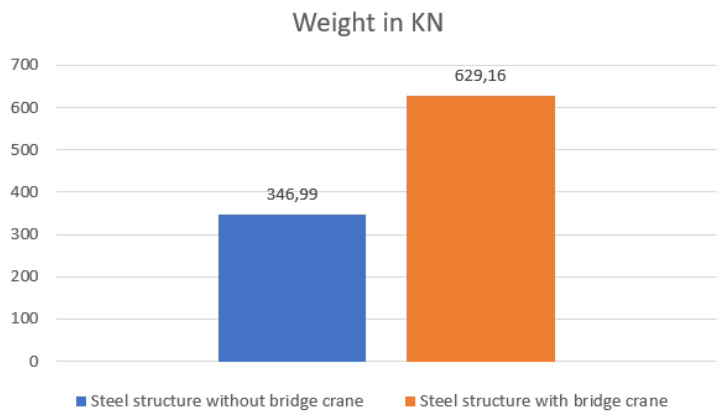


Figure 3.25. Comparison of the weight of the two steel structures

3.6.5. Comparison of the foundation volume

The foundation isolated footing with tie beams was designed with, the first for the steel structure without bridge crane and the second for the steel structure with bridge crane. The steel structure without bridge crane column footing was 1 m x 1.5 m x 0.5 m with 8 M27 anchors on a 430 x 260 x 15 mm steel plate. whereas the steel structure with a bridge crane structure column footing was 1.5 m x 2 m x 0.5 m with 8 M27 anchors on a 430 x 260 x 15 mm steel plate. It can be observed that the footing size with bridge crane is twice that of the footing without bridge crane. This is due to the presence of moving loads which increases the transmission of axial forces used in the design of the foundation.

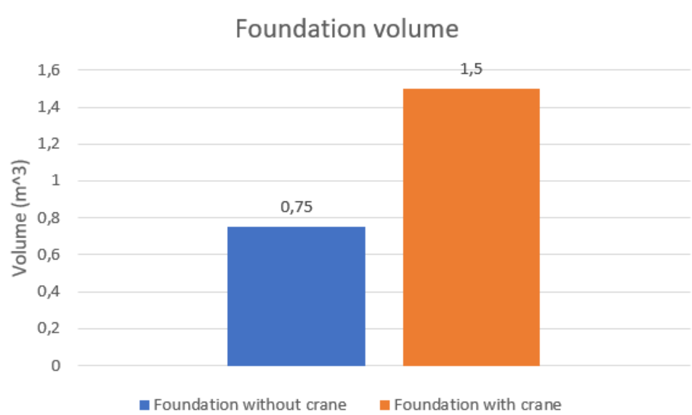


Figure 3.26. Comparison between the foundation volume of both steel structures

3.6.6. Comparison of the cost estimations (mass criteria)

The cost estimation of the two steel structures modelled is done using simplified criteria of mass. Here, the masses of the two structures are measured and the amounts compared. The mass is used to compute the cost of the two steel structures' material for the different projects. The mass of the two steel structures is obtained by computing the mass of every individual sections of all the steel structures. The structure without bridge crane has a mass of 28131 kg whereas that of the steel structure with a bridge crane is 46131 kg. The costs are presented in Table 3.48 with the cost analysis presented graphically in Figure 3.27.

Table 3.48. Cost of the mass of the steel structures

Designation	Mass(kg)	Price per kilogram (kg/FCFA)	Total (FCFA)
Steel structure without bridge crane	34699	656	22,763,000
Steel structure with bridge crane	62916	656	41,210,263

Price obtained on an average of international standards since a well-defined price is not common in Cameroon.

By multiplying, it can be noticed that the steel structure with a bridge crane is costlier (41,210,263FCFA) than the steel structure without a bridge crane (22,763,000 FCFA) with the cost of the bridge crane plus installation been 18,447,263 FCFA.

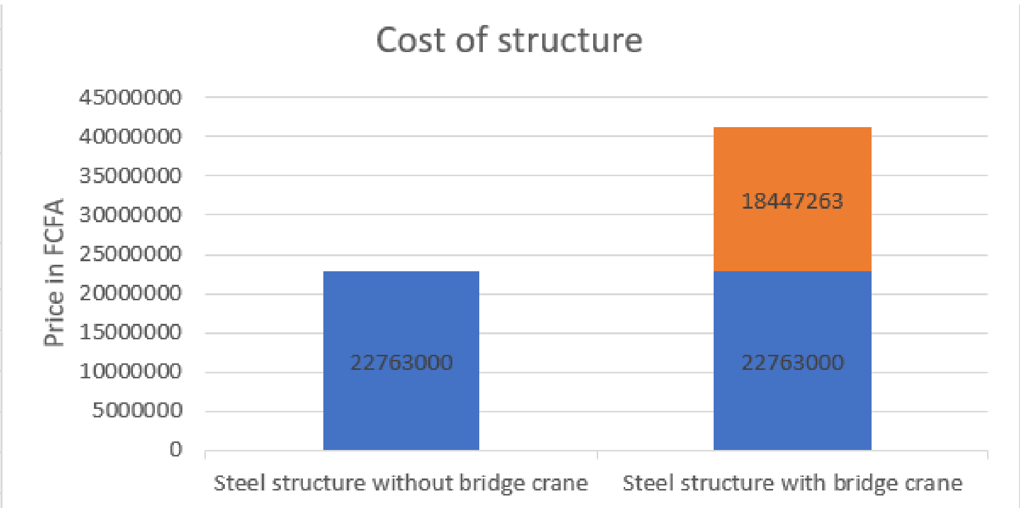


Figure 3.27. Comparison of the cost of both structures.

A detailed view of the quote is as shown in Table 3.49.

Table 3.49. Quote for the bridge crane

	Quote for 10T double girder bridge crane				
No	Designation	Quantity		Unit price	Total price
1	Running beam (main girder)	2	sets	5850000	11700000
2	End trucks	2	sets	650000	1300000
3	Hook	1	set	13000	13000
4	Trolley	1	set	433333	433333
5	Lifting hoist	1	set	507000	507000
6	Wheel	4	sets	73125	292500
7	Lifting motor	1	set	162500	162500
8	Reducer(Box springs)	1	set	66950	66950
9	Rails	2	sets	4900	9800
10	Electrics	1	set	25480	25480
11	Remote Control	1	set	33800	33800
12	Drum	2	sets	78000	156000
13	Wire rope	20	m	31655	633100
14	Lifting limit switches	1	set	3900	3900
15	Traveling limit switches	1	set	3900	3900
16	Overload limiter	1	set	130000	130000
17	Buffer	2	sets	13000	26000
18	Sub-total bridge crane				15497263
18	Installation		L/S	100000	100000
19	Bridge crane testing and commissioning		L/S	65000	50000
20	Renting of mobile crane	1	set	850000	850000
21	Crane runway	2	sets	325000	650000
22	Pillars	2	sets	650000	1300000
	Total bridge crane				18447263

3.7. Some possible solutions of reinforcement of steel structure

From the analysis results presented in section 3.5.1, the observations made showed that the presence of the bridge crane increases the shear and axial force on the column. Thus the column must be reinforced by other independent columns or mounting a bayonet upright or lattice with support for the running beam, so as to support the new loads. In addition to the fact that some of the structural members did not verify the following are possible solutions that can be taken into account when designing the structure to resist to crane loads and also serve as reinforcement of these members are described as follows.

3.7.1. The bracing system

This is generally achieved by vertical longitudinal bracing. This can be done by considering the two cases shown in figure 3.28.

The bracing is in a vertical plane passing through the running beam, this is the case when the running track constitutes an independent structure as in figure 3.28 (a);

The bracing is eccentric in relation to the bearing beam, this is generally the case if the longitudinal bracing of the hall is used as for this thesis as in figure 3.28 (b).

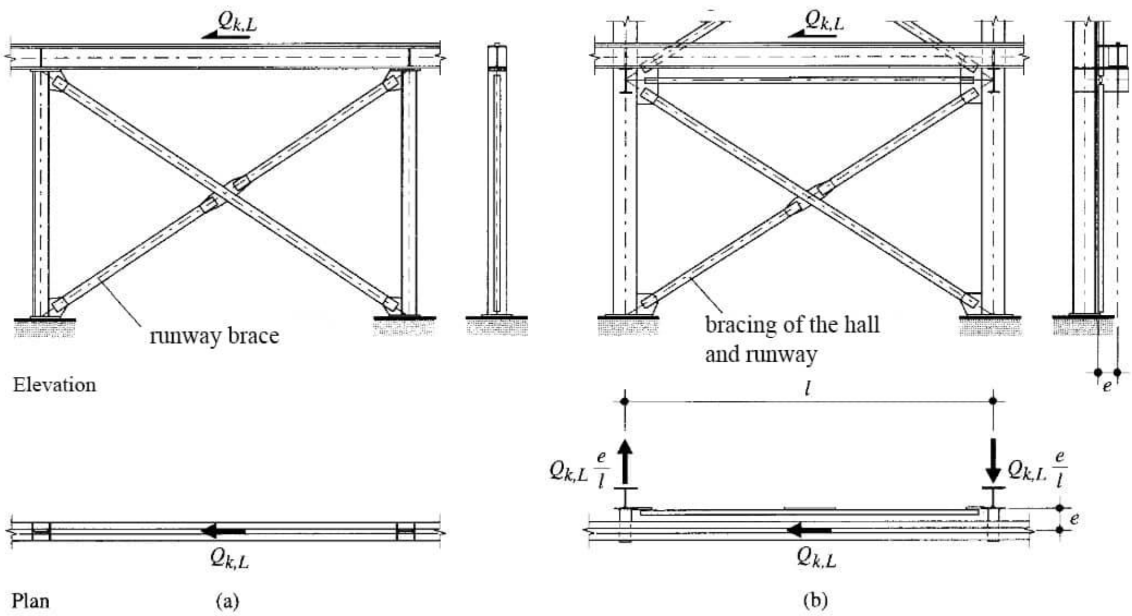


Figure 3.28. Path for horizontal forces.

In the first case, the normal force acting in the running beam is transmitted directly into the bracing. In the second case, the eccentricity of the longitudinal horizontal force, generates a moment which must be balanced by a couple of forces acting in the two columns adjacent to the bracing. These can also be stressed in torsion if there is no horizontal bracing. It must also be checked that the corbels are able to transmit the longitudinal horizontal force. If this is not the case, horizontal braking bracing must be provided.(Hirt, 1949).

3.7.2. Reinforcemecnt of column

The vertical reactions of the bearing beam are transmitted directly to the foundations via the posts: independent or not of those of the hall: as shown in the examples in Figure 3.29. In case (a) the bearing beam is placed on a corbel embedded in the frame upright. In case (b), the post of the beam of the bearing beam is independent of that of the hall. Case (c) is a combination of two posts, called a bayonet post. Cases (b) and (c) are particularly suitable for high capacity overhead cranes, the path of the forces being the most direct.

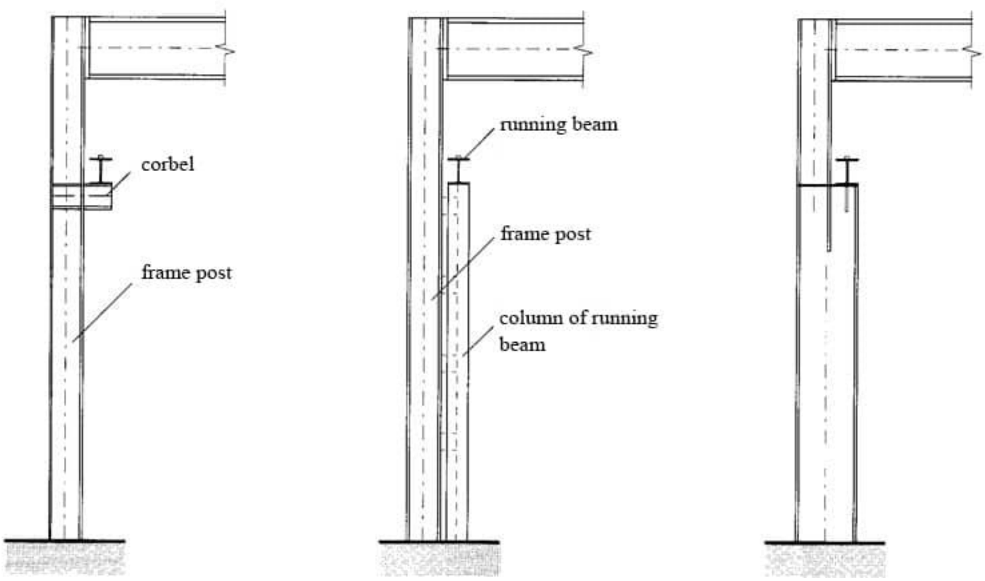


Figure 3.29. Example of vertical support for a running beam

3.7.3. Member strengthening

This is done by creating a beam-column joint between existing beam and existing column. Welding additional plates on both sides of existing columns is used for strengthening. Also haunches can be used to increase the resisting moment.

Conclusion

The aim of this chapter was to design and verify the members of the two steel structures with the results obtained been compared to appreciate the differences. The chapter started with the presentation of the site and the case study, followed by the design and verification of the structural elements and connections in our structures. Equally actions and load combinations were chosen and defined ideally in both structures in compliance with Eurocode 3. The last part of the chapter presented the characteristic features of the comparison for the different criteria listed in the methodology. Reporting the performance of each structure and comparing them, a considerable higher structural capacity in the structure with a bridge crane although having a larger footing size was noticed. Also it has a relatively higher cost with respect to the machine introduced. All these results were made possible due to the modelling of the structure on the software SAP2000 v23.2.0. Moreover, Microsoft Excel enabled the easy assessment of the different results and the elaboration of different histograms explaining the variation of solicitations, stresses, and deflections between the two models.

GENERAL CONCLUSION

Haven come to the end of the study entitled "Analysis and verification of the behavior of industrial steel structures with and without bridge cranes.", whose main aim is recalled was to design and compare the structural and functional performances of two identical steel structures of which one is equipped with a bridge crane to carry heavy loads and the other is without the bridge crane. In order to achieve this objective, the work was partitioned in three chapters. The first chapter was a state of art on the different concepts of steel, its processes, characteristics, types and uses, followed by bridge cranes, types, its mechanisms and functionalities, how structural analysis and design can be made for both bridge cranes and industrial steel buildings and a review on the latter as the bridge crane is indoor to the industrial building. The second chapter was the methodology, where the two steel structures were modelled with the help of the computer program SAP2000 v23.2.0 which were afterwards analysed and the results and verifications done using Excel 2016 were shown in chapter 3. This was later on designed and compared on the basis of their structural and functional performance and cost. All the members present in the case study were classified based on their cross section and then were proven to be statically verified according to Eurocode 3 prescription.

The following conclusions could be drawn from the study: the bridge crane of nominal capacity 10 tons has a section of HE700B and is made up of 2 rails of section A100 and a running beam support of HE450B.This machine has a lifting capacity of 4 m with the trolley situated at 9.6 m of the transversal crane, that is at its centre. Also the bridge crane is of spectral load class Q3, that is, the traveling crane is not lifting the nominal load quite frequently but regularly lifting loads between 1/3 and 2/3 of the nominal load and has the total number of cycles class U8, that is, has a regular use in intermittent service.

The bridge crane added indoor below the roof at 3.7 m in the steel structure brought about a considerable increase in the solicitations and stresses of the different structural members present. This was thus seen by a 10.84 % increase in structural capacity of the steel structure with a bridge crane; the footing and column showed a greater section in the steel structure with a bridge crane with the footing size of the bridge crane steel structure being twice that of the footing without bridge crane. The deflection in the roof's steel structure with bridge crane was 0.83 mm largely greater than the 0.003 mm of the steel structure without. Furthermore, there is an increase of 81.32 % in weight of the steel structure with bridge crane and finally, from the cost estimation, the steel structure with a bridge crane showed to be costlier than the steel structure without a bridge crane. The steel structure with a bridge crane has a better overall structural performance and functional reliability but comes with a considerable cost. The work also brought forth certain perspectives to be considered for further studies in order to improve this work:

- The effect of an addition of seismic loads.
- Study the behavior of the steel structure using high strength steel materials.
- Perform a local analysis on the joint connection between the bridge crane and the steel structure.

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ANNEXES

Annex A

1) ZIN INDUSTRIES est-elle une PME de fabrication des tôles de couverture et de bardage en aluminium et en aluzinc

- [x] Oui
- [ ] Non

2) Combien de tôles produisez-vous par jour ?

Environ 180 tôles de 6m (6m x 180)

3) Quelle masse d'une tôle produisez-vous ?

Environ 11 kg

4) Quel nombre de tôle soulevez-vous par jour ?

Environ 180

5) Quel type de pont roulant voudriez-vous ?

- [ ] Mono poutre
- [x] Bipoutre
- [ ] Autre

6) Quelle est la capacité de levage du pont roulant:

10 t ?

7) Quelle est la portée du pont roulant :

18 m ?

8) Quelle est la hauteur de levage :

4-5 m ?

9) Quelle est la distance parcourue :

19 m ?

10) Quel type de produit sera levé ? intérieur ou extérieur ?

Bobine de tôle et tôles, intérieur

1/2

11) Combien d'heures le pont roulant fonctionne-t-~~elle~~ il par jour ?

03 heures

12) Combien de fois le pont roulant travaille-t-il par heure ?

02 fois

13) Quelle charge sera soulevée à chaque fois ?

500kg – 1 tonne

14) Quelle est la méthode de contrôle :

- [x] Télécommande
- [ ] Pendante
- [ ] Cabine

15) Quelle est le type d'alimentation : 380V 50Hz triphasé ?

- [x] Oui
- [ ] Non

16) Autres indications jugez utiles par vous si possible

Directeur Général

2/2

Annex A1 : List of questions for bridge crane characteristics

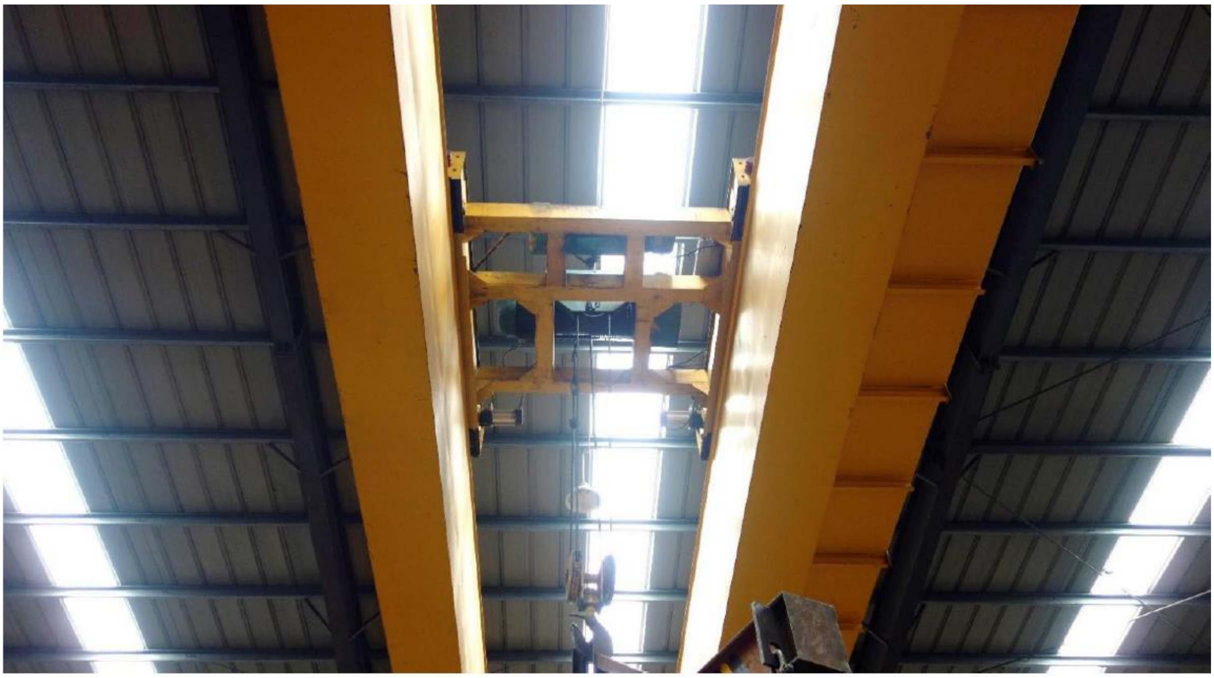
Annex A2: Double girder bridge crane Abus catalogue

Characteristics of double-girder bridge cranes ZLK

Capacité, Type de treuil[1]	S[1] m	A1 mm	K1 mm	C1 mm	L1 mm	L2 mm	Z min mm	Hmax[1] mm	R mm	LK mm	R.V./galet kN R max	R.V./galet kN R min
	10	200	770	-50	660	660	150	9000	2700	1605	30.6	6.9
5000 kg	14	300	770	-50	660	660	150	9000	2700	1605	33.5	8.7
Chariot treuil	16	300	770	-50	660	660	150	9000	2700	1630	35.5	10.4
GM 1050 H6	18	400	770	-50	660	660	150	9000	2700	1630	37.5	12.1
FEM 2m	20	500	770	-50	660	660	150	9000	2900	1730	39.6	14.0
	22	460	810	-90	660	660	170	9000	3200	1895	42.7	17.0
VL =	24	560	810	-90	660	660	170	9000	3800	2230	45.7	19.7
0.8/5 m/mn	26	500	870	-150	660	660	180	9000	4600	2650	50.7	24.4
	28	700	870	-150	660	660	180	9000	4600	2650	53.2	26.8
	30	700	870	-150	660	660	180	9000	4600	2650	57.2	30.7
	32	650	920	-200	660	660	180	9000	5100	2965	66.3	39.5
	34	660	920	-200	660	660	180	9000	5100	2965	71.7	44.9
	10	200	770	-30	660	660	150	9000	2700	1605	36.9	7.4
6300 kg	14	300	770	-30	660	660	150	9000	2700	1630	40.5	9.7
Chariot treuil	16	400	770	-30	660	660	150	9000	2900	1730	42.6	11.4
GM 2063 H6	18	500	770	-30	660	660	150	9000	2900	1730	44.7	13.3
FEM 1Am	20	500	770	-30	660	660	150	9000	2900	1730	46.0	14.3
	22	560	810	-70	660	660	170	9000	3200	1895	49.0	17.0
VL =	24	500	870	-130	660	660	180	9000	3800	2250	55.0	22.7
0.8/5 m/mn	26	500	870	-130	660	660	180	9000	3800	2250	58.7	26.3
	28	700	870	-130	660	660	180	9000	4600	2650	61.9	29.3
	30	700	870	-130	660	660	180	9000	4600	2650	66.1	33.3
	32	660	920	-180	660	660	180	9000	5100	2965	76.0	42.9
	34	900	920	-180	660	660	180	9000	5100	2965	78.7	45.6
	10	300	860	10	760	760	150	10000	2700	1605	45.7	9.0
8000 kg	14	400	860	10	760	760	150	10000	2700	1630	49.7	11.1
Chariot treuil	16	460	900	-30	760	760	170	10000	2900	1745	52.4	13.1
GM 3080 H6	18	460	900	-30	760	760	170	10000	2900	1745	53.9	14.2
FEM 3m	20	460	900	-30	760	760	170	10000	2900	1745	56.6	16.4
	22	560	900	-30	760	760	170	10000	3200	1930	59.3	18.9
VL =	24	500	960	-90	760	760	180	10000	3800	2250	65.8	25.0
0.8/5 m/mn	26	700	960	-90	760	760	180	10000	3800	2250	68.5	27.5
	28	700	960	-90	760	760	180	10000	4600	2650	71.0	29.6
	30	650	1010	-140	760	760	180	10000	4600	2715	79.2	37.7
	32	900	1010	-140	760	760	180	10000	5100	2965	85.6	43.7
	34	900	1010	-140	760	760	180	10000	5100	2965	87.9	45.9
	10	260	900	-30	760	760	170	10000	2700	1620	55.6	10.5
10 000 kg	14	360	900	-30	760	760	170	10000	2700	1645	60.0	12.5
Chariot treuil	16	460	900	-30	760	760	170	10000	2900	1745	62.8	14.6
GM 3100 H6	18	460	900	-30	760	760	170	10000	2900	1745	64.5	15.7
FEM 2m	20	500	960	-90	760	760	180	10000	2900	1765	67.8	18.6
	22	500	960	-90	760	760	180	10000	3200	1950	71.0	21.4
VL =	24	700	960	-90	760	760	180	10000	3800	2250	76.0	26.0
0.8/5 m/mn	26	700	960	-90	760	760	180	10000	3800	2250	78.0	27.8
	28	700	960	-90	760	760	180	10000	4600	2650	82.7	32.1
	30	660	1010	-140	760	760	180	10000	4600	2715	91.7	40.8
	32	900	1010	-140	760	760	180	10000	5100	2965	95.1	43.9
	34	900	1010	-140	760	760	180	10000	5100	3005	101.0	49.2
	10	300	1090	40	790	790	180	10000	2700	1665	70.4	13.2

Capacité, Type de treuil[1]	S[1] m	A1 mm	K1 mm	C1 mm	L1 mm	L2 mm	Z min mm	Hmax[1] mm	R mm	LK mm	R.V./galet kN R max	R.V./galet kN R min
	10	250	1330	-130	820	820	180	10000	2900	1830	109.0	19.7
20 000 kg	14	360	1330	-130	820	820	180	10000	2900	1830	116.0	21.8
Chariot treuil	16	460	1330	-130	820	820	180	10000	2900	1865	119.0	23.8
GM 6200 L6	18	650	1330	-130	820	820	180	10000	2900	1865	123.0	26.1
FEM 2m	20	650	1330	-130	820	820	180	10000	3200	2015	127.0	29.7
	22	900	1330	-130	820	820	180	10000	3200	2015	131.0	32.6
VL =	24	900	1330	-130	820	820	180	10000	3800	2315	134.0	35.0
0.8/5 m/mn	26	900	1330	-130	820	820	180	10000	3800	2315	139.0	39.0
	28	860	1380	-180	820	820	190	10000	4100	2515	146.0	44.9
	30	860	1380	-180	820	820	190	10000	4600	2805	152.0	51.0
	32	1100	1380	-180	820	820	190	10000	5100	3055	159.0	56.8
	34	1100	1380	-180	820	820	190	10000	5100	3055	162.0	59.7
	10	350	1330	-130	820	820	180	10000	2900	1830	132.0	22.4
25 000 kg	14	600	1380	-180	820	820	190	10000	3000	1930	142.0	26.3
Chariot treuil	16	600	1380	-180	820	820	190	10000	3200	2065	146.0	29.0
GM 6250 L6	18	610	1380	-180	820	820	190	10000	3200	2065	151.0	31.9
FEM 1Am	20	850	1380	-180	820	820	190	10000	3200	2065	154.0	33.6
	22	850	1380	-180	820	820	190	10000	3200	2065	158.0	37.2
VL =	24	860	1380	-180	820	820	190	10000	3800	2365	164.0	41.7
0.66/4 m/mn	26	860	1380	-180	820	820	190	10000	3800	2365	167.0	44.0
	28	860	1380	-180	820	820	190	10000	4600	2765	174.0	50.0
	30	860	1380	-180	820	820	190	10000	4600	2805	181.0	56.9
	32	1110	1380	-180	820	820	190	10000	5100	3055	190.0	64.4
	34	930	1560	-360	820	820	270	10000	5100	3055	201.0	75.1
	10	400	1460	40	1080	1080	190	8000	3400	2130	166.0	32.8
32 000 kg	14	600	1460	40	1080	1080	190	8000	3600	2265	179.0	34.7
Chariot treuil	16	610	1460	40	1080	1080	190	8000	3600	2265	184.0	36.7
GM 7320 H6	18	850	1460	40	1080	1080	190	8000	3600	2265	189.0	39.4
FEM 2m	20	850	1460	40	1080	1080	190	8000	3600	2265	193.0	41.0
	22	860	1460	40	1080	1080	190	8000	3600	2265	198.0	44.6
VL =	24	680	1640	-140	1080	1080	270	8000	3600	2305	207.0	51.7
0.66/4 m/mn	26	920	1640	-140	1080	1080	270	8000	3800	2405	213.0	56.5
	28	930	1640	-140	1080	1080	270	8000	4300	2655	220.0	62.4
	30	930	1640	-140	1080	1080	270	8000	4600	2805	225.0	65.8
	32	930	1640	-140	1080	1080	270	8000	5100	3055	234.0	73.9
	34	1180	1640	-140	1080	1080	270	8000	5100	3055	246.0	85.5
	10	220	1660	-160	1080	1080	270	8000	3600	2265	206.0	41.6
40 000 kg	14	430	1660	-160	1080	1080	270	8000	3600	2265	220.0	42.5
Chariot treuil	16	430	1660	-160	1080	1080	270	8000	3600	2265	227.0	45.0
GM 7400 H6	18	680	1660	-160	1080	1080	270	8000	3600	2265	233.0	48.0
FEM 1Am	20	680	1660	-160	1080	1080	270	8000	3600	2305	239.0	52.2
	22	680	1660	-160	1080	1080	270	8000	3600	2305	244.0	54.6
VL =	24	920	1660	-160	1080	1080	270	8000	3800	2405	251.0	59.8
0.66/4 m/mn	26	930	1660	-160	1080	1080	270	8000	3800	2405	258.0	65.2
	28	930	1660	-160	1080	1080	270	8000	4300	2655	267.0	72.8
	30	930	1660	-160	1080	1080	270	8000	4600	2805	272.0	76.7
	32	1180	1660	-160	1080	1080	270	8000	5100	3055	282.0	84.9
	34	1180	1660	-160	1080	1080	270	8000	5100	3055	291.0	93.6
	10	230	1890	240	1310	1310	270	10000	4300	2615	254.0	57.0

Annex A3: Double girder bridge crane



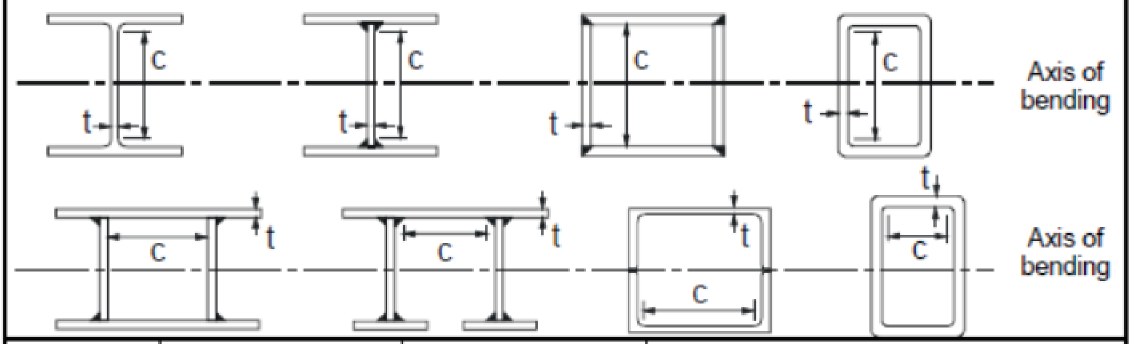
Annex A4: Bridge crane's trolley

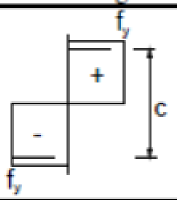
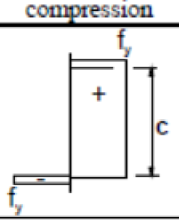
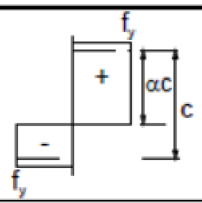
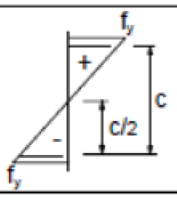
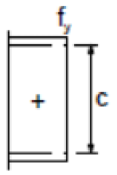
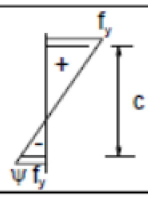
Annex B

Annex B1. Steel cross section's classification

Maximum width-to-thickness ratios for compression parts(da Silva et al., 2010)

Internal compression parts



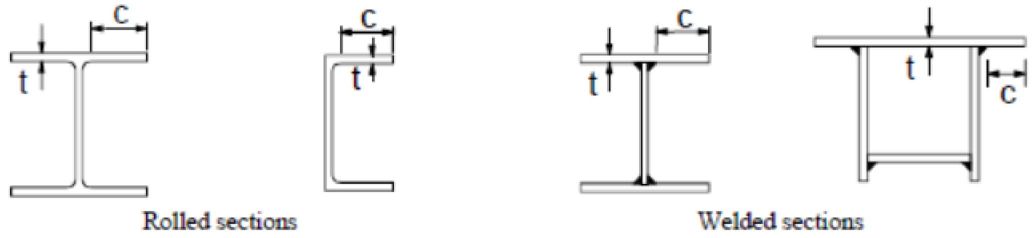
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \leq 72\varepsilon$	$c/t \leq 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{36\varepsilon}{\alpha}$
2	$c/t \leq 83\varepsilon$	$c/t \leq 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \leq \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \leq 0{,}5$: $c/t \leq \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 124\varepsilon$	$c/t \leq 42\varepsilon$	when $\psi > -1$: $c/t \leq \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \leq -1^{*)}$: $c/t \leq 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

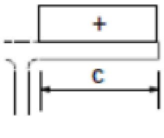
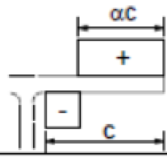
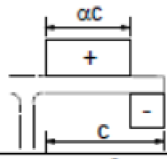
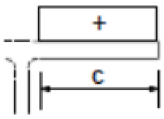
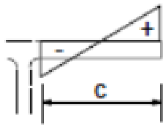
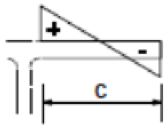
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

*) $\psi \leq -1$ applies where either the compression stress $\sigma \leq f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Maximum width-to-thickness ratios for compression parts (outstand flanges)

Outstand flanges

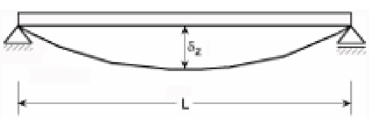
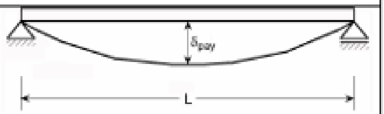


Class	Part subject to compression	Part subject to bending and compression	
		Tip in compression	Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \le 9\varepsilon$	$c/t \le \frac{9\varepsilon}{\alpha}$	$c/t \le \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \le 10\varepsilon$	$c/t \le \frac{10\varepsilon}{\alpha}$	$c/t \le \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \le 14\varepsilon$	$c/t \le 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

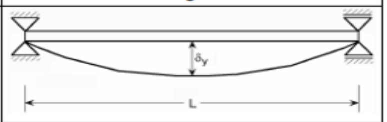
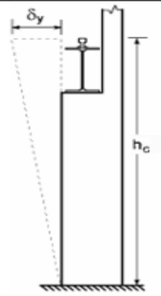
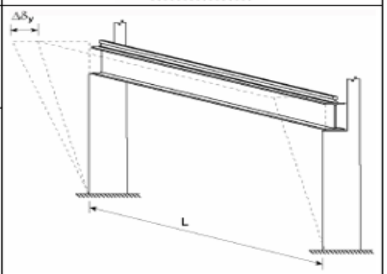
$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

Annex B2: Limiting value for vertical and horizontal deflections of bridge crane

Limiting values for vertical deflections

Description of deflection (deformation or displacement)	Diagram
a) Vertical deformation δ_z of a runway beam: $\delta_z \le L/600$ and $\delta_z \le 25$ mm. The vertical deformation δz should be taken as the total deformation due to vertical loads, less the possible pre-camber, as for δmax in figure A1.1 of EN 1990.	
b) Difference Δh_c between the vertical deformations of two beams forming a crane runway: $\Delta h_c \le s/600$	
c) Vertical deformation δ_{pay} of a runway beam for a monorail hoist block, relative to its supports, due to the payload only: $\delta_{pay} \le L/500$	

Limiting values for horizontal deflections

Description of deflection (deformation or displacement)	Diagram
a) Horizontal deformation δ_y of a runway beam, measured at the level of the top of the crane rail: $\delta_y \leq L/600$	
b) Horizontal displacement δ_y of a frame (or of a column) at crane support level, due to crane loads: $\delta_y \leq h_c/400$ where: h_c is the height to the level at which the crane is supported (on a rail or on a flange)	
c) Difference $\Delta\delta_y$ between the horizontal displacements of adjacent frames (or columns) supporting the beams of an indoor crane runway: $\Delta\delta_y \leq L/600$	
d) Difference $\Delta\delta_y$ between the horizontal displacements of adjacent columns (or frames) supporting the beams of an outdoor crane runway: - due to the combination of lateral crane forces and the in-service wind load: $\Delta\delta_y \leq L/600$ - due to the out-of-service wind load $\Delta\delta_y \leq L/400$	
e) Change of spacing Δs between the centres of crane rails, including the effects of thermal changes: $\Delta s \leq 10$ mm [see Note]