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Dimensionamento de estruturas metálicas: Aplicações práticas

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Dissertação apresentada à Universidade de Aveiro para o cumprimento dos requisitos necessários à obtenção do grau de Mestre em Engenharia Civil realizada sob a orientação científica do Doutor Nuno Filipe Ferreira Soares Borges Lopes, Professor Auxiliar do departamento de Engenharia Civil da Universidade de Aveiro.

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Steel structures design: Practical applications

Report submitted to the University of Aveiro to fulfill the requisites necessary for completing the Master's degree in Civil Engineering. This report shall be completed with the scientific supervision of Doctor. Nuno Filipe Ferreira Soares Borges Lopes, Auxiliar Professor of the Department of Civil Engineering at the University of Aveiro.

Aos meus pais

To my parents

o júri

presidente

Prof. Doutor Paulo Barreto Cachim
Professor associado da Universidade de Aveiro

Prof. Doutor José Miguel de Freitas Castro
Professor Auxiliar da Faculdade de Engenharia da Universidade do Porto

Prof. Doutor Nuno Filipe Ferreira Soares Borges Lopes
Professor Auxiliar da Universidade de Aveiro

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palavras-chave Dimensionamento, estruturas metálicas, projectos, turbinas de gás, execução.

resumo

O presente trabalho está inserido num estágio realizado na empresa Alstom e aborda o dimensionamento e a execução de estruturas metálicas para o suporte de tanques de arrefecimento de turbinas de gás. No presente trabalho é referido o dimensionamento da estrutura metálica no seguimento da execução de um projecto (Carrington).

No dimensionamento da estrutura metálica tem-se como bastante relevante os seguintes dados: a velocidade do vento, actividade sísmica, tipo de acesso para manutenção dos tanques de arrefecimento, movimentos dos tanques, tipos de normas foram utilizadas no projecto e ainda saber se a estrutura esta situada dentro ou fora do complexo.

Os movimentos dos tanques de arrefecimento da turbina de gás tem uma enorme importância no dimensionamento porque, quando conectamos os tanques à estrutura metálica é necessario implementar amortecedores para evitar o choque dos tanques com a estrutura metalica. Estes amortecedores evitam o movimento rápido quando existe actividade sismica travando o movimento dos tanques.

Depois do dimensionamento da estrutura estar concretizada, é necessário proceder aos detalhes, neste caso todas as peças desta estrutura têm que ser detalhados em desenho criado em AutoCAD, de maneira a que o fabricante da estrutura saiba toda a informação necessaria para a manufactura da mesma.

Também é criado, para isto, uma lista detalhada juntamente com a instrução de montagem com todos os materiais usados na construção da estrutura metálica.

A tese fará uma descrição mais aprofundada dos referidos assuntos.

keywords Design, steel structures, projects, gas turbines, execution.

abstract

This thesis presentation is in association to an internship program at Alstom, and undertakes the design and execution of metallic structures used to support gas turbine cooling vessels.

Specifically mentioned in this thesis is the design and execution of a steel structure for a specific project (Carrington).

When designing a metallic structure, the following inputs are extremely relevant; wind velocity, seismic activity, access options for the maintenance of the vessels, the movements of the vessels, subsequent norms or criteria to be used on the project as well as whether the structure is to be located indoor or outdoor of the plant.

The movement of the vessels of the gas turbine has an enormous importance on the design of the structure. When connecting the tanks to the steel structure it is necessary to install shock absorbers/ snubbers to avoid any clashes. These shock absorbers prevent any rapid movement of the tanks due to seismic activity.

When the steel structure has been designed, it is necessary to finalize any remaining details. In this case, all the specific pieces pertaining to the structure have to be detailed and illustrated on the AutoCAD program, giving the supplier all the specific information necessary for the manufacturing process.

A detailed list, called bill of material, is also put together along with an instruction assembly manual, of all materials used in the construction of the steel structure.

The thesis will make a more detailed description of these subjects.

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Abbreviations list

BOQ – Bill of Quantity

CAD – Computer aided design

CPL – Carrington Project

ESP – Engineering service provider

GA – General Arrangement

GT26– Gas Turbine 26

HP OTC – High Pressure Once Through Cooler

HW – Hardware

I&T Plan – Inspection and Test Plan

LP OTC – Low Pressure Once Through Cooler

MW – Mega Watts

NCR – Non Conformance Report

NUHX – Nuclear business heating exchanger

OTC – Once Through Cooler

OTC SS - Once Through Cooler Steel Structure

PDMS - Plant Design Management System

SLS – Serviceability Limit State

SRI – Steel Recycling Institute

STS – System time Schedule

TMG – Turbo Machinery Group

TOC – Top of Concrete

TOG – Top of Grating

TOS – Top of Steel

ULS – Ultimate Limit State

WI - Work Instruction

WP – Work Package

Latin letters

A_d – *Design value of an accidental action*

G_K - *Characteristic value of a single permanent action*

Q_K – *Characteristic value of a single variable action*

Fu – Ultimate strength

Fy – *yield strength*

t – Nominal thickness

v - *Displacement*

Greek letters

γ_G – *Partial factor for permanent action*

γ_Q – *Partial factor for variable action*

ε_y– *Yield strain*

ψ_1, ψ_2, ψ_3 – *Combination factors*

E - Modulus of elasticity

σ – *Stress*

Φ - *Rotation*

1. Introduction

1.1. General considerations

Steel structures are associated with modern architecture and engineering. Throughout the 20th century this material has inspired architects and engineers for its combined strength and efficiency with unique opportunities for sculptural expression. Today, in an era of engineering innovation, steel construction plays a relevant role with accomplished examples of modern building and structural design. This is partly due to the improvements that have been made in the metallurgic industry, though structural analysis as well as fabrication and construction in itself.

The primary purpose of a steel structure is to form a skeleton for the building or structure – essentially the steel will hold everything up and together. Steel has many advantages when is compared to other structural building materials such as concrete, timber, plastic and the newer composite materials (Berman, 2013).

Steel as a building material has been studied and tested for a long period of time. It is allowed to say that we understand the steel behavior, more than any other construction material.

The evolution of steel design brought us from the theory that the stiffer the structure the better. Today, flexibility and ductility is the key. The most significant attributes of the steel structure include high strength and low weight which gives the structure a high ratio of load carrying ability. One good example is the roof of the Zurich train main station (*Figure 1)* and the Beijing Olympic stadium *(Figure 2)* seen below.

Figure 1 – Zurich Main Train Station (www.stockpicturesforeveryone.com)

Up until the 1970's, structures were designed using proven formulas and calculations were done manually. Today however, calculations are done by using modern computer software enabling a steel structure to be designed rather quickly in one day if necessary, in comparison to what could have taken a structural engineer a much longer time to complete using previous methods of calculations, mainly using the traditional pencil and paper (Berman, 2013).

The days of drafting are almost gone and barely used. Digitizing a structure on the computer ensures quality and saves an enormous amount of time, thereby reducing the cost. One of the key instruments in this evolution of steel structure design is the Auto CAD program (AutoCAD).

The steel industry is well organized. There are several codes provided by steel industry, most local and national building codes that address steel issues. Teaching institutions are constantly studying the steel design, its constructional uses and capabilities.

Figure 2 – Beijing Olympic Stadium (www.telegraph.co.uk)

Steel is 100% recyclable and in fact, according to the Steel Recycling Institute, steel is the most recycled material around the world. Steel, unlike wood, does not warp or twist and does not substantially expand and contract with the weather. Unlike concrete, steel does not need time to cure and is immediately at full strength. Steel is versatile, it provides more strength with less weight, has an attractive appearance, can be erected in most weather conditions, has a uniform quality, durability and low life cycle cost. These advantages make steel a really good material for building. (Berman, 2013)

A steel structure has an additional important advantage as it can be manufactured in advance in factory locations and then be assembled at the construction site (ARCELOR, 2004).

This assignment is in reference to an internship job program at a multinational company named Alstom. Here, along with a group or co-workers, during the following academic year.

1.2. Brief Introduction to Alstom

Alstom is a multinational enterprise with the main objective of designing machines for the production of energy in many different fields, such as gas, nuclear, renewable and hydro. Alstom is also the world leader in very high speed transports. Alstom Power is one of the main and most advanced technologic fields at Alstom its main purpose is the production of gas turbines *(Figure 3)* for energy purposes.

For over 100 years, Brown Boveri & Cie (BBC), then from 1998 Asea Brown Boveri (ABB) and now Alstom Power Ltd. have significantly contributed to the achievements of todays advanced gas turbines. Brown Boveri & Cie (BBC) manufactured the very first industrial gas turbine in 1939 for a power plant in Neuchatel, Switzerland. This machine included a compressor and a turbine, each in its own casing and with one combustor for both.

Since then, Alstom sold more than one thousand gas turbines with total in production time and service exceeding 35 million hours. Alstom gas turbines have gained recognition in more than 60 countries, in diverse and extreme locations ranging from artic climates to deserts, maritime environments, and industrial areas with a high degree of air pollution, steel plants, oil and gas refineries, chemical industries (fertilizer, pulp and paper), with a continuous objective to improve and optimize the use of steel structures, taking it to a level of standardization and by reducing installation time.

Figure 3 – Gas turbine blades (www.hexagonmetrology.com)

Figure 4 –Alstom Gas Power Station with three gas turbines (www.alstom.com)

1.3. Problem Statement

When a power plant is created it is necessary to build all the necessary equipment for this new power plant.

After the structural and connection designs are completed, the structure is assembled on site, but sometimes is necessary to alter the design. This can happen due to project changes or manufacturing methods, when this happens it is necessary to find a feasible solution to improve the overall time and cost impact of the project.

This internship seeks to describe basic knowledge regarding steel structures and their many constructional capabilities emphasizing the difference between the theoretical aspect of my formal instruction to the practical approach of steel structures and their assembly on location.

This internship at Alstom has consisted on the design steel structures for OTC, pipe racks, platforms and ladders for accessing gas turbines.

Figure 5 – Power Plant in Terga, Algeria (gastoday.com.au)

1.4. Objectives

The main objectives of this work according to the internship in Alstom are:

- Improve the knowledge on specific designs of steel structures and the design of structural connections for gas turbine supports and their auxiliaries.
- Understand and create a process and work instruction for an OTC steel structure.
- Use directives and international standards, such as the “Eurocode” being integrated on a multidisciplinary team project.
- Cost reduction and overall project delivery time improvement are seen as the ultimate goal to be created by the achievement of the above objectives.
- Create a technical specification document for the design of platforms, stairs and ladders with standard measurements and dimensions for future projects.
- Achieve an Alstom environment, with all of its applicable processes, procedures and products, as well as order materials from suppliers.
- Utilize new informative tools for steel design, such as Hilti, Tekla, Fischer, RSTAB and P3DM.

1.5. Thesis organization

This thesis is divided in two main parts.

The first main part is referred to the design of steel structures as well as the connection design (bolted and welded).

The second main part will refer to how the design is made at Alstom, presenting project design (Carrington), including the hardware order.

To conclude, through this internship at Alstom, it was possible to help develop and create ongoing projects and WIs alongside co-workers involved with the steel structure, piping design and plant group thereby reducing overall time and costs for an OTC SS design in accordance with our supplier.

1.6. Tasks executed at Alstom

The tasks carried out throughout the current internship at Alstom were mainly structural steel designs as well as small and medium assignments. These were done according to ongoing projects and basic standards procedures.

The following are some of the projects:

- OTC steel structure work package, owner of North Bangkok II, Tzafit and Carrington;
- Steel structure design for pipe supports;
- Prepare order input for the different projects;
- Part of the team responsible for the technical specifications for platforms and ladders;
- Part of the team for the lead time reduction, responsible for the OTC scope;
- Creation of I&T Plan for the production of the hardware.

2. Basis for steel structural design

2.1. Steel behavior

Steel is a metal alloy carbon formed from iron minerals, whose main components are iron and carbon. However, there are other components of this kind of material, each considered impurities resulting from the manufacturing process (such as sulfur, phosphorus and silicon), others metal additives are added in well-defined proportions to improve some property, such as corrosion resistance or fire resistance.

The steel commonly used in metal construction are hot rolled, characterized by low amounts of about 0.2% carbon.

The important mechanical properties of most structural steel under static load are indicated in the idealized tensile stress-strain shown in the *Figure 6*. Initially the steel has a linear stress-strain curve whose slope is the Young's modulus of elasticity E. The values of E vary in the range 200 000 – 210 000 MPa, and the approximate value of 200 000 MPa is often assumed. The steel remains elastic in this linear range, and recovers perfectly on unloading. The limit of the linear elastic behavior is often closely approximated by the yield stress f_y and the corresponding yield strain $\varepsilon_y = f_y/E$. Beyond this limit the steel flows plastically without any increase in stress until the strain-hardening strain $\varepsilon_y$is reached. This plastic range is usually considerable, and accounts for the ductility of the steel. The stress increases above the yield stress $f_y$when the strain-hardening strain $\varepsilon_y$is exceeded, and this continues until the ultimate tensile stress f_u is reached. After this, large local reductions in the cross-section occur, and the load capacity decreases until tensile fracture takes place (Trahair et al., 2008).

The yield stress $f_y$is the most important strength characteristic of a structural steel. This varies significantly with the chemical constituents of the steel, the most important of which are carbon and manganese, both of which increase the yield stress. The yield stress also varies with the heat treatment used and with the amount of working which occurs during the rolling process.

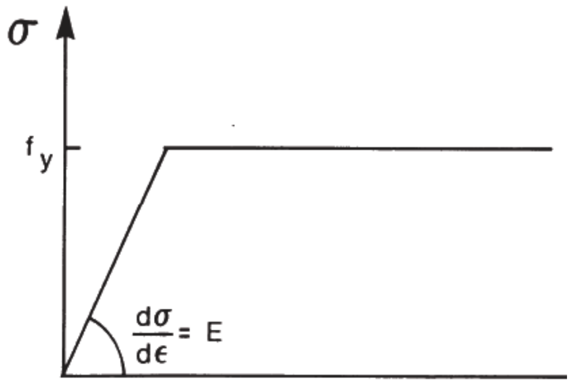


Figure 6 - Stress strain relationship considered in Eurocode 3 (EN1993-1-1)

Therefore thinner plates which are more worked have higher yield stresses than thicker plates of the same constituency. The yield stress is also increased by cold working. The rate of straining affects the yield stress, and high rates of strain increase the upper or first yield stress, as well as the lower yield stress f_y. The strain rates used in tests to determine the yield stress of a particular steel type are significantly higher than the nearly static rates often encountered in actual structures. (Trahair et al., 2008)

For design purposes, a "minimum" yield stress is identified for each different steel classification. In Europe, these classifications are according to European standards meanwhile for the mechanical classification of steel we use the EN10025-2.

The strain of a hyper static structure can be determined through an analysis of global elastic or a plastic global analysis.

The global elastic analysis is based on the hypothesis that the stress/strain relationship is linear at any point of the structure, whatever the amount of the actuating tension. In practical terms, taking into account the behavior of the soft steel chain, the analysis of global elasticity assumes that the tension caused by the strain shall be inferior to the yield in any given point of the structure (Duggal. 2008).

2.2. Actions

2.2.1. Determination of fundamental load cases

Before any detailed sizing of structural elements can start, it is necessary to start to estimate the loads that will act on a structure. Once the designer and the client have agreed on the purpose, size and shape of a proposed structure and what materials it is to be made of, the process of load estimation can begin. Loads will always include the self-weight of the structure *(Figure 7)*, called the "dead load." In addition there may be "live" loads due to people, traffic, service, etc., that may or may not be present at any given time, and also loads due to wind, snow, pipe loads, earthquakes etc. The required sizes of the members will depend on the weight of the structure but will also contribute to the weight. So load estimation and member sizing are to some extent an iterative process in which each affects the other. As the designer gains experience with a particular type of structure it becomes easier to predict approximate loads and member sizes, thereby reducing the time taken in trial and error. However the inexperienced designer can save time by intelligent use of some short cuts. For example the design of structures carrying heavy dead loads such as concrete slabs or machinery may be dominated by dead load. In this case it may be best to size the slabs or machinery first so the dead loads acting on the supporting structure can be estimated. On the other hand many steel-framed industrial buildings in warm climates where snow does not fall can be designed mainly on the basis of wind loads, since dead and live loads may be small enough in relation to the wind load to ignore for preliminary design purposes. The wind load can be estimated from the dimensions of the structure and its location. Members can then be sized to withstand wind loads and then checked to make sure they can withstand combinations of dead, live and wind load. Where snowfall is significant, snow loads may be dominant. Earthquake loads are only likely to be significant for structures supporting a lot of mass, so again the mass should be estimated before the structural elements are sized (Berman, 2013).

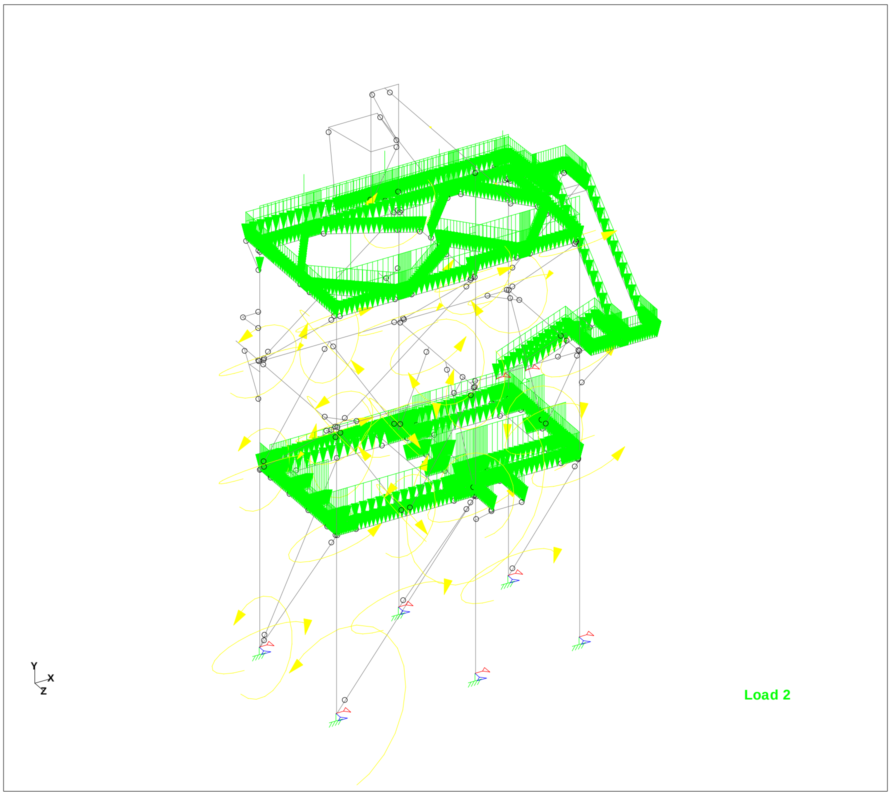


Figure 7 – STAAD PRO Dead load acting in the OTC SS

2.2.2. Dead Load

The dead loads acting on a structure arise from the weight of the structure including the finishes, and from any other permanent construction or equipment used in the structure. The dead loads are completely different when the construction is started then when it is concluded, but when is concluded they will remain constant, unless changes will be implemented on the structure itself, or the equipment changes as well (Trahair et- al., 2008).

The dead load may be assessed from knowledge of the dimensions and the specific weight of the material, and all the equipment. Guidance on specific weight needs to be considered, the dimensions used in the estimative of the load is also an estimative for the design. By making these assumptions in most of the cases when a structure is designed, the weights are over estimated causing an over design of the structure, making a waste of material (Trahair et al., 2008).

2.2.3. Live Load

The live loads are those loads on the structure which result from its use by the man, and these usually vary both in time and in the area of the structure. Live loads are usual consider the most sever spatial distribution, and this can only be determined by using both maximum and minimum values of this load. When the distribution of the live loads is considered in big areas the maximum live loads specified, are often reduced in order to make some allowance for the decreased probability that the maximum live load acts on the areas at the same time (Trahair et al., 2008).

2.2.4. Seismic activity

Experience shows that steel structures subjected to earthquake behave well. Global failures and huge numbers of casualties are mostly associated with structures made from other materials. This may be explained by some of the specific features of steel structures.

The structure’s global behavior is ductile. The structure can dissipate a significant amount of energy (ARCELOR. 2004).

A ductile behavior, which provides extended deformation capacity, is generally the better way to resist earthquakes. One reason for this is that because of the many uncertainties which characterize our knowledge of real seismic actions of the analyses that we make, it may be that the earthquake action and its effects are greater than expected. By ensuring ductile behavior, any such excesses are easily absorbed simply by greater energy dissipation due to plastic deformations of structural components (ARCELOR. 2004).

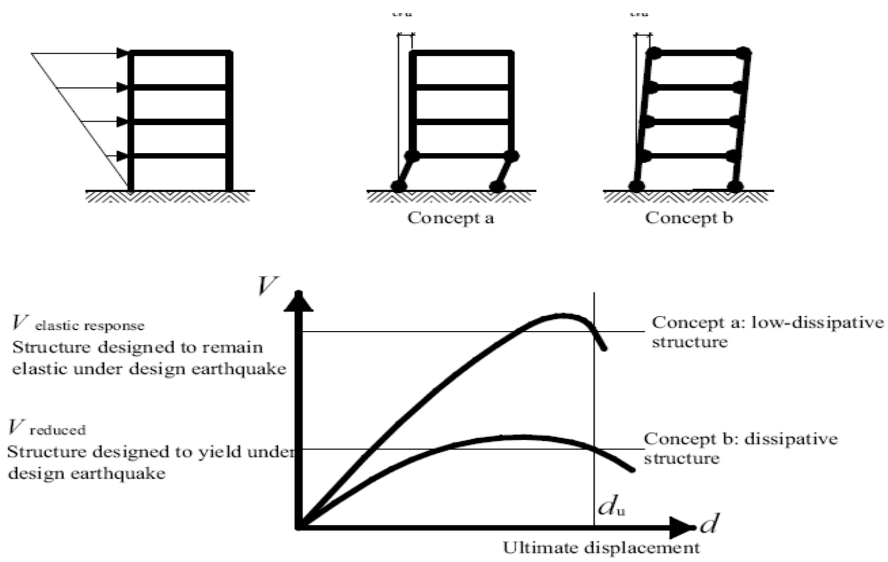


Figure 8 - Examples of global "Dissipative" and "non dissipative" behaviors of structures (ARCELOR, 2004)

There are other advantages of steel structures in a seismic zone, namely their flexibility and low weight. Stiffer and heavier structures attract larger forces when an earthquake hits. Steel structures are generally more flexible than other types of structure and lower in weight. Forces in the structure and its foundations are therefore lower. This reduction of design forces significantly reduces the cost of both the superstructure and foundations of a building (ARCELOR. 2004).

Steel structures are generally light in comparison to those constructed using other materials. As earthquake forces are associated with inertia, they are related to the mass of the structure and so reducing the mass inevitably leads to lower seismic design forces. Indeed some steel structures are sufficiently light that the seismic design is not critical, this is, that the building designed for gravity and wind loads implicitly provides sufficient resistance to earthquakes (ARCELOR. 2004).

Buildings and structures is seismic zones shall be designed for the effects of the seismic forces in both horizontal directions and vertical directions (when required). The structures can be considered as multi-degree-of-freedom systems with lumped masses. The lumped masses should be provided at the floor elevations and at specific nodes, where high weighted structural parts or equipment are located.

The design seismic forces shall be assumed as equivalent static forces applied to the lumped masses.

2.2.5. Wind activity

The wind is a movement initiated by an air mass transport of air in Earth's atmosphere. As it moves over the surface of the earth, the air hits and escapes trough all kinds of obstacles found along the path, including structures. In most of the cases the forces acting on these structures due the wind action have to be taken into account in its analysis and its design (Tranvanca. 2010).

The need to build higher, particularly in dense urban areas, brought about advancements in engineering and construction techniques that saw the skyscraper boom of 1920's and 30's and the revival in the 1960's. Building big meant spending big and subsequent advancements were made in the form of lighter, stronger materials. In turn, building large and light led to lightly damped and more flexible structures. Consequently wind was suddenly an important issue in the design of structures (Berman, 2013).

Throughout the years innovations have been made in how the structures are designed for the effects of wind loads, how wind loads are determined and applied, and how the limits of wind loads can be applied (Berman, 2013).

The estimation of wind loads is a complex problem because they vary greatly and are influenced by a large number of factors.

In nature, dynamic phenomenon varies in time. Represent this phenomenon in all its complexity is one of the major challenges for a structural engineer. Due to the difficulties to make the quantification of time-varying loads and subsequent verification of structural responses, several simplifying assumptions are generally assumed. To describe the effects caused by wind gusts normally is admitted the equivalent static loads. In situations where the structure presents a moderate dynamic response, the adoption of static loads can lead to overdesign of the structure.

2.2.6. Snow

Snow loads are prevalent in northern and/or mountainous regions all over the world. In colder regions, the peak snow load does not result of a single event of the winter season. Between storms, the snow load can varies due to the wind or the snow melting.

The derivation of snow load requires the designer to make judgments on the environment that the structure is implanted in as well the form of the structure itself. The potential for the buildup of snow must be allowed for when determining the magnitude of the resulting persistent snow load on top of a structure

The characteristic values are determined using the characteristic ground snow load for the site and roof shape coefficient. Ground snow load maps for European climatic regions are given by National Annex of the Eurocode

2.2.7. Temperature

Where the structures are exposed to daily and seasonal climatic changes in temperature, the effects of thermal actions should be accounted for in the design.

In general temperature variations cause deformations in single structural elements as well as in the overall structure itself. If the structure is hyperstatic, an additional consequence of the temperature variation will have a bigger influence in the stress states. Therefore, the effects of temperature variations may involve aspects of structures functionality as well as its safety.
In the case of this study the temperature will be very important because temperatures in the pipes and vessels structures will vary.

2.2.8. Fire

The possibility of a fire in a structure could be considered an accidental action, for this reason the designer needs to consider this situation during the design of the steel structure in order to prevent any accident caused by fire. Doing this the steel structure can resist a fire during a certain period of time instead of collapsing without notice.

In a prescriptive approach, using a more simplistic and conventional method, the nominal curve that is used as reference for the fire resistance is the worldwide standard fire curve ISO 834. However, according to 1991-1-2 (EC1) there are two more curves; the nominal external curve elements to fire, and fire curve hydrocarbons (Reis, 2012).

The effect of fire in a structure will increase the temperature of the elements that constitute it, thereby altering its stiffness and strength, as well as alter the displacements and loads of the support structures. These changes may cause the ruin or collapse of the structure, making it essential to determine the evolution of the thermal field in the transient regime and at the same time determine the time of resistance to the request of fire. Much of the existing codes resort to analysis by finite differences and finite elements (Vila Real, 2003).

The fact that the stress-strain relationship becomes highly non-linear at elevated temperatures, plus the fact that heating leads to thermal expansion with possible restraint forces, make the rules derived for ambient temperature inaccurate in a fire situation (Franssen and Vila Real, 2010).

The fire resistance of a structure is normally attributed by time, according to the hydrocarbon curve (Figure 9) and the ISO 834 curve, it is possible to classify the fire resistance of a structure by different types of categories R30, R60, R90, R120, R180 and R240, the R stands for the resistance of the structure following the amount of minutes that the structure resists.

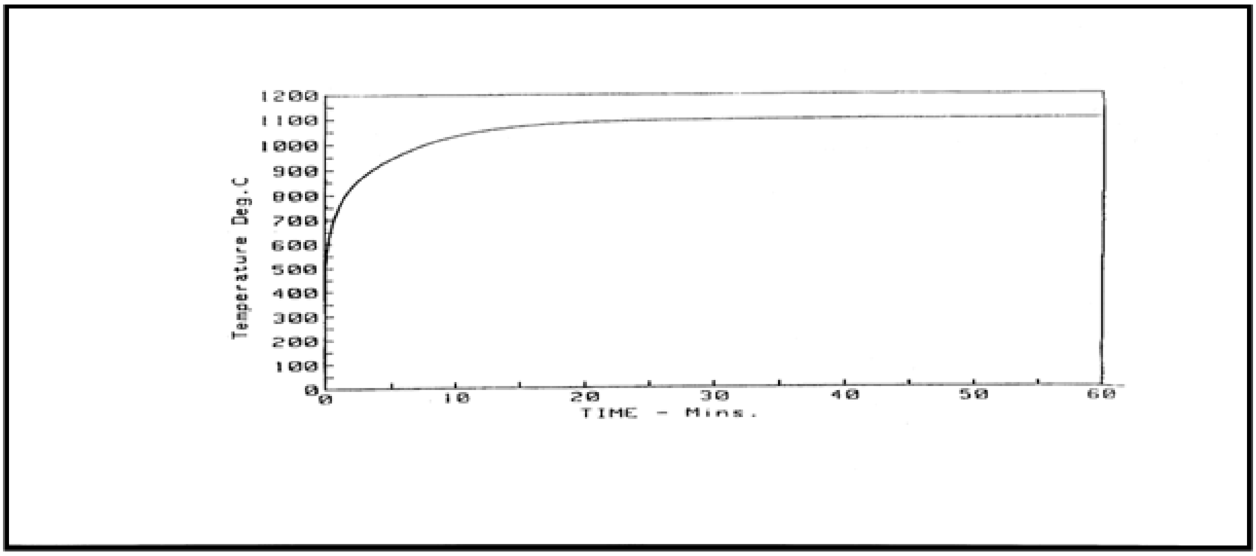


Figure 9 – Hydrocarbon Curve (rvrfireproof.com)

2.3. Determination of load combination

To check the safety in relation to different limit states, combinations of simultaneous actions whose performance is likely to produce in the structure the most unfavorable effects, should be considered.

It is not considered correct the overload acting simultaneously on the same element even if they are primarily due to the concentration of people with the actions of the snow or wind.

The different loads discussed above are not acting alone in a structure, but in combinations between them, where the designer has an important role to determine which of the combinations are most essential to the structure.

A limit State Design is a state that should be considered in a structure begins to show damage or partial damage, in the main function of the same structure.

In the Limit State Design we can consider three types:

- Ultimate Limit State, where the damage is severe to the structure
- Serviceability Limit State, where the damage isn’t that sever to the structure
- Accident State, such as fire and seismic events, when an undesired incident or condition acts in the structure

2.3.1. Ultimate Limit State

This state is associated with collapse or with similar forms of structural failure. The Ultimate Limit State represents only a simple happening of a certain incident, this will correspond to a limited situation, independent of the duration.

According to the Eurocode, the following load combination shall be adopted for the structure verification at the Ultimate Limit State (ULS):

For normal conditions: $\gamma_G . G_K + \gamma_{Q1} . Q_{K1} + \quad \psi_0 . \gamma_{Qi} . Q_{Ki}$

For exceptional conditions: $\gamma_{GA} . G_K + A_d + \psi_{,1,1} . Q_{K1} + \quad \psi_2 . Q_{Ki}$

2.3.2. Serviceability Limit State

The Serviceability Limit State are defined having in mind the duration of the load, this means that the behavior of the structure will only correspond to a Serviceability Limit State when this will have a duration part in the structure. This State also corresponds to conditions beyond which specified service requirements for a structure or structural member are no longer met.

According to the EN1991, the following load combinations shall be adopted for verifications at serviceability Limit States:

Rare combinations: $G_k + \quad \psi_{0,i} . Q_{k,i}$;

Common combinations: $G_k + \psi 1 . Q_{k,i} \quad \psi_{2,i} . Q_{k,i}$;

Permanent combinations: $G_k + \quad \psi_{2,i} . Q_{k,i}$;

On the safe side, a factor ≤1 can be adopted for the combination factor ψ_1, ψ_2, ψ_3 and so only one combination will be adapted for SLS will be $G_k + \quad Q_{k,i}$.

2.4. Structural design-background

2.4.1. Design of structural member for design loads

The analysis of global strain and displacement in a structure depends fundamentally on the characteristics of their deformability and rigidity, but also global stability and the stability of its elements, the behavior of the cross sections, the behavior of the connections, the imperfections and deformability of the support system. Therefore the definition of the type of analysis to be adopted in any given situation should take into account all of these characteristics (Simões. 2007).

In general, the various structural design requirements relate to corresponding limit states, and so the design of a structure to satisfy all the appropriate requirements is often referred to as a limit state design, the requirements are commonly presented in a deterministic fashion, by requiring that the structure shall not fail, or that its deflections shall not exceed prescribed limits. However, it is not possible to be completely certain about the structure and its loading, and so the structural requirements may also be presented in a probabilistic form, or in a deterministic form derived from probabilistic consideration. This may be only done by defining an acceptably low risk of failure within the design life of the structure, after reaching some sort of balance between the initial cost of the structure and the economic and human losses resulting from the failure. In many cases there will be a number of structural requirements which operate at different load levels, and are not usual to require a structure to suffer no damage at one load level, but to permit some minor damage to occur at a higher load level, provided there is no catastrophic failure (Trahair et al.. 2008).

The structural design criteria may be determined by the designer, by using the implemented standards of the country. The stiffness design criteria adopted are usually related to the serviceability limit state of the structure under the service loads, and are concerned with ensuring that the structure has sufficient stiffness to prevent excessive deflections such bending, distortion and settlements, and excessive motions under dynamic load including shaking and vibrations (Duggal. 2008).

The strength limit state design criteria are related to the possible methods of failure of the structure under overload and under strength conditions, and so these design criteria are concerned with yielding, buckling, brittle fracture, and fatigue. Also important is the ductility of the structure at and near failure. Ductile structures give a warning of the impending failure and often redistribute the load effects away from the critical regions, while ductility provides a method energy dissipation which will reduce the damage due to earthquake and blast loading. On the other hand, a brittle failure is more serious, as it occurs with no warning of failure with a catastrophic consequence the energy will be released increasing damage. Other design criteria may also be adopted, such as those related to corrosion and fire (Trahair et al., 2008).

A global plastic analysis assumes plastic lamination of some sections of the structure, generally through the formation of plastic hinges (plasticization by flexion) and the redistribution of tension to areas less stressed. In this type of analysis, it is fundamental that the sections where plastic hinges are formed have a higher capacity for rotation. This type of analysis fundamentally depends on the rheological behavior of the material beyond its elastic limitation. In the plastic analysis of steel structures when using steel, the use of elastic-plastic behavior is generally used (Trahair et al., 2008).

The analysis of strains and displacements can either be of first or second order. On analyzing the first order, the internal strains and displacements are obtained from the initial un-deformed geometry of the structure in contrast the analyses of the second order internal strains are influenced by the deformed configuration of the structure *(Figure 10)*. In a frame structure, where elements are submitted to axial strains, the effects of the second order are generally referred to as global effects or local effects at the element level these effects, because of the frame displacements generate additional strain, therefore altering the results on the dislocation themselves. This dependency between straining and displacements imply that, to obtain an analysis of the second order, one needs to resort to an interactive process, only possible with the aid of adequate computerized tools (Trahair et al., 2008).

The elastic analysis of the first order is the most utilized in calculating common structures however on occasion this may not be the correct option. In specific situations, it may be imperative to obtain the strain and shift dislocations of a structure through the analysis of the second order (Duggal. 2008).

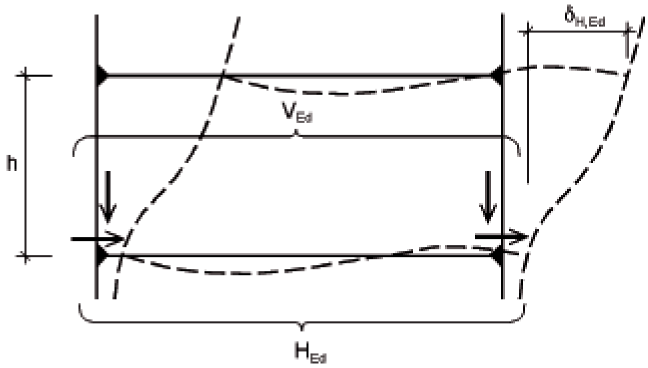


Figure 10 – Imperfections of 2nd order according EN1993-1-1

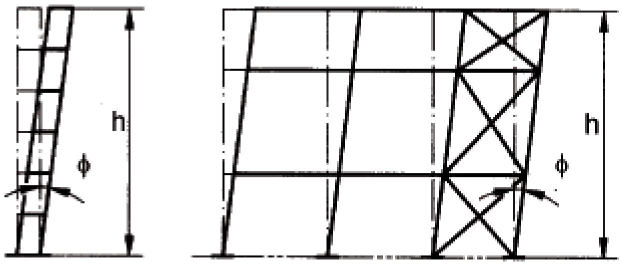


Figure 11 – Equivalent sway of imperfections according EN1993-1-1

2.4.1.1. Classification of cross-sections

The classification of cross-sections is related to the plastic calculation requirements imposed to the cross-sections. For a plastic global analysis, it is necessary that the bars allow the formation of plastic hinges, capable of deforming so that there is a redistribution of effort required for this type of calculation. For an elastic analysis, this requirement is no longer imposed any section and can be considered, since it has a bearing capacity sufficient given the possible (Simões. 2007).

The classification of cross-sections depends on the proportions of each of its compression elements as follows *(Figure 12):*

- Class 1- Cross section which allows formation of the plastic hinge, with the rotation capacity required for the plastic analysis.
- Class 2 – Cross section which allows formation of the plastic hinge, with a limit rotation capacity.

- Class 3 – Cross-section in which the calculated stress in the extreme compression fiber of the member can reach the steel yield strength.
- Class 4 – Cross-section partially active in which it is necessary to take into account the effects of local buckling when determining their bending moment resistance.

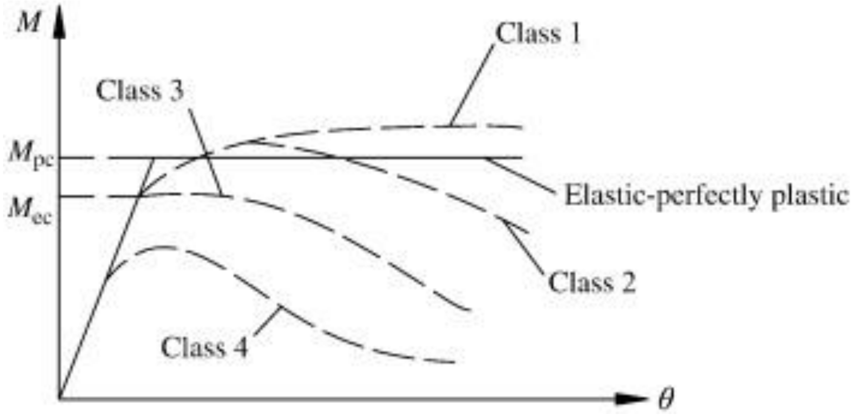


Figure 12 – Four behavioral classes of cross-sections (www.sciencedirect.com)

2.4.1.2. Tension resistance

A tension member is the one which is intended to resist axial tension. Tension members are also called ties or hangers. In contrast to compression members, the disposition of material in a tie has no effect on its structural efficiency so that compact sections such as rods may be used without reduction in allowable stress. For tensile force to be axial, it is necessary that the load is applied through the center line of the section of the member. However, in spite of all precautions, the fabrication work may cause some initial crookedness, due to which the so called axially tensioned members may have small eccentricity. The axial tensile force applied to the member has a tendency to straighten the member, which in turn reduces the initial eccentricity. Therefore, small unknown eccentricities are neglected in the design.

The gross area of a cross section is defined in the usual way and utilizes nominal dimensions. No reduction to the gross area is made for fasteners hole, but allowance should be made for large openings, such as those services. Below in the *Figure 13* it is possible to see non-staggered arrangement of fasteners according to the Eurocode 3

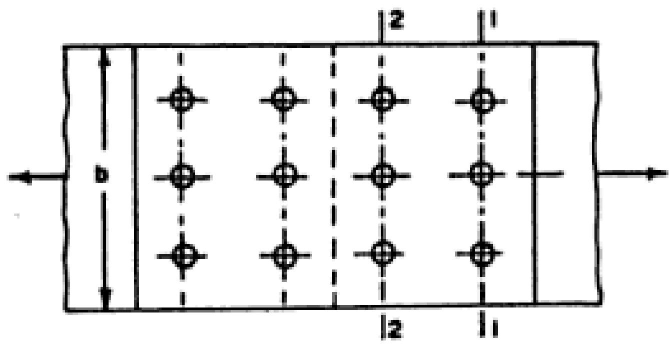


Figure 13 – Plate with bolt holes

2.4.1.3. Compression resistance

Buckling instability is a phenomenon which is characterized by the occurrence of large deformations in elements subject to transverse compression forces. In steel structures, this and other instability phenomena are of particular importance, due to the high strength steel elements generally have high slenderness.

Based on the theory of elastic stability, the elastic critical load (the value of the axial force to which the display element becomes not limited axial deformation). In the *Figure 14* below it is possible to see the buckling phenomenon in a compressed piece free from imperfections (Simões, 2007).

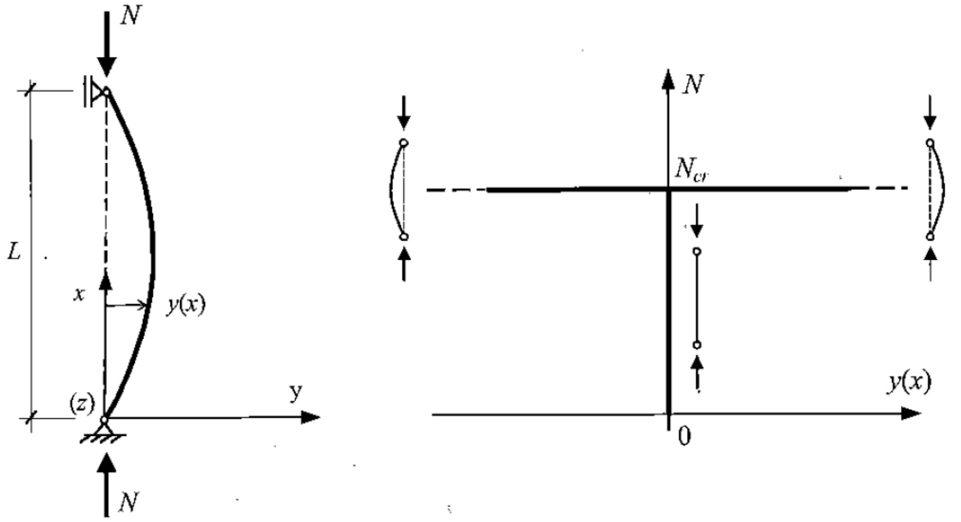


Figure 14 – Compression buckling, Euler column (Simões, 2007)

2.4.1.4. Bending moment and Shear resistance

The bending strength of a metal element may be conditioned by the resistance of cross sections or the occurrence of the phenomena of lateral instability. Failing phenomenon of lateral instability, designed elements subjected to bending can be made only on the basis of resistance of the cross sections. The main situations where it is not necessary to consider the effects of lateral instability in the design of metallic elements subjected to bending are:

- Sections flexed around the axis z of lower inertia;
- Elements restricted laterally by means of metal side by any method that prevents the compressed lateral areas of the sections;
- Sections with high torsional rigidity and lateral bending, such as circular hollow sections chains.

In steel elements subjected to shear and bending the sections most commonly used is H or I-sections or rectangular hollow sections, for presenting a high flexural rigidity in the y-axis. For all classes of sections can be used a scan elastic tensions, making use of criteria for assignment when required. However, the dimensioning of steel elements, where possible should be based on the strength of the plastic section, as it leads to more economical solutions (Simões. 2007).

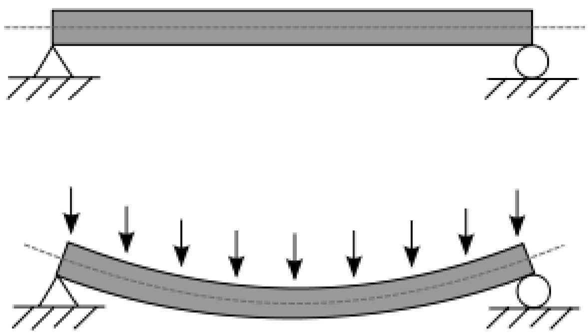


Figure 15 – Shear force acting in a beam (Wikipedia.org)

2.4.1.5. Bending in elements laterally non-restrained

The design of elements laterally non- restrained induced by bending forces, especially composed by open sections with thin walls such as I or H profiles, are generally influenced by lateral-torsional buckling (Simões. 2007).

The lateral-torsional buckling deformation is the side of the compressed portion of a section, an element for bending around the y axis (axis of higher inertia). Under these conditions, the compressed part behaves as a linear element compressed steadily restrained by the tensile, which will match and has no tendency to move laterally (Simões. 2007).

Below in the *Figure 16* it is possible to see a beam with a deformation caused by bending and torsional force.

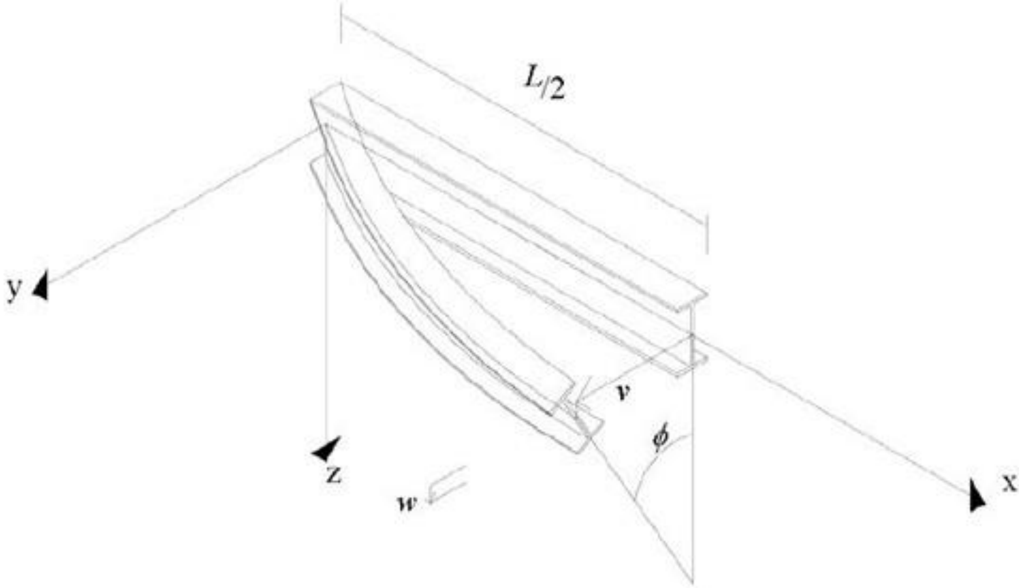


Figure 16 – Bending of a beam laterally non-restrained (Lopes, 2009)

The strength of a beam lateral buckling will depend fundamentally on the critical moment, which is the maximum deformation at a beam in ideal conditions can withstand without bending laterally (Simões. 2007).

2.4.1.6. Bending and axial compression

The behavior of structural elements subjected to composed bending, results from the combined effects of bending and axial force. Elements in high slenderness, with submitted to bending and compression, tends to collapse on itself by bending or buckling or by lateral buckling (Simões. 2007).

A component subjected to composed bending in addition to the displacement and first order moments, there are additional secondary moments and displacements that must be taken into account in the analysis and design (Simões. 2007).

In the *Figure 17* below we can see axial and bending acting in a member, here it is possible to see the initial displacement $e0$ it is also possible to see the additional bending moment introduced to the second order displacement, ending in a bigger transversal deformation.

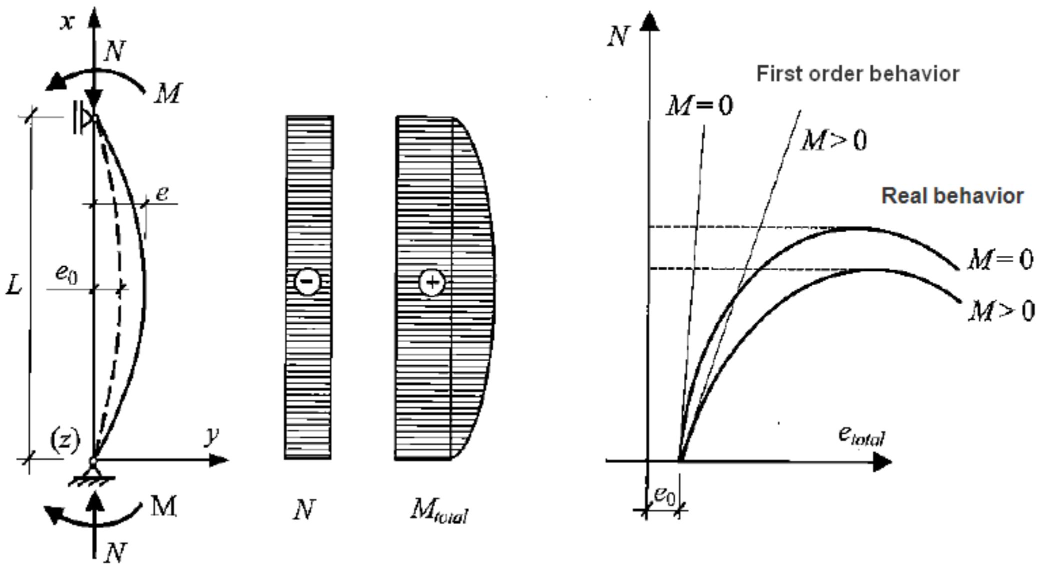


Figure 17 – Element behavior submitted to bending and axial force (Simões, 2007)

The behavior of a component subjected to composed bending compression results from iterations among phenomena of instability and plasticity this can also be greatly influenced by the geometrical and material imperfections.

According to the EN1993-1-1, the safety verification of an element submitted to compose bending is effected in two different stages:

- Cross-sections verification
- Element verification of the resistance for buckling

2.4.2. Connection analysis for design loads

Buildings in steel structures are constituted by different types of elements and each of these elements must be conveniently neighboring parts themselves, so that they can fulfill the primary goal of designing a structure: safety and functionality.

Connections can be made with several different types of materials, but the most normal kind of connections in steel structures is:

- Bolted connections
- Welded connections

2.4.2.1. Bolted Connections

The design of steel joints represents a crucial aspect in the design of steel structures. Steel joints account for a substantial proportion of the total cost of the structure, besides influences significantly its overall behavior. Recent advances in research over the past twenty years resulted in new design approaches to the analysis of steel joints. It is the purpose of the present paper to establish the basis of the current procedures for analysis and design of steel joints, in the context of the Eurocode 3 (Simões. 2007).

Bolting is nearly always the preferred method to connect the structures, unless special circumstances dictate otherwise. Using standard bolts and nuts to make connections to structural hollow sections is difficult because there is normally no access to the inside of the section to tighten them. Unless on-site welding has been adopted, this has usually meant that some form of additional fabrication, and therefore cost, has been necessary to overcome the problem (Simões. 2007).

In the *Figure 18* below we can see an example of a bolted example

Figure 18 – Bolted connection (wiki.mech.uwa.edu.au)

In order for a structure to behave as expected, the connections must behave as assumed in the beam and column design. Connections can be classified as nominally pinned, joints that are capable of transmitting internal forces without developing significant moments, and capable of accepting the resulting rotations under the design loads. Rigid and full strength joints that have sufficient rotational stiffness to justify analysis based in full continuity. Semi-Rigid joints that lie somewhere between nominally pinned and rigid (Simões. 2007).

Bolts have different kinds of classes, depending in the strength of the bolt and depending on the design of the structural engineer. In the *Table 2* below are described the bolt class resistance and the nominal values of yield stress f_{yb} and tensile strength f_{ub} commonly used in steel construction.

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

Table 1 – Bolt Class according the EN1993-1-8

The standard EN 1993-1-8 defines five categories of bolted connections. These categories distinguish between connections loaded in shear or tension, and connections containing preloaded or non-preloaded bolts.

For these two types of connections (shear and tension), EN1993-1-8 makes reference for the following types of categories. Below, it is possible to see types of categories needed for the bolted connections as well as the definitions of each category.

Shear connections:

- Category A – A bearing-type connections is the most common type of bolted connection. It is used in most simple-shear connections and in situations when loosening or fatigue due vibration or load fluctuations are not design considerations.
- Category B – A slip-critical connection is one in which loosening due to vibration or load reversals are to be considered. Also, holes that are oversize or slotted shall be designed as slip-critical connections. This category is used for connections where a loss of stiffness at ULS is not important
- Category C – This category of connection is required where slip is not acceptable under any circumstances.

Tension connections

- Category D – In this type of connections normal bolts are used with low percentage of carbon. In this category pre-loaded bolts are not necessary.
- Category E – In this type of category preloaded bolts are used, this is to ensure a better resistance to fatigue.

Minimum edge and end distances and bolt spacing are given in terms of the diameter of the bolt hole as we can see in the *Figure 19*.

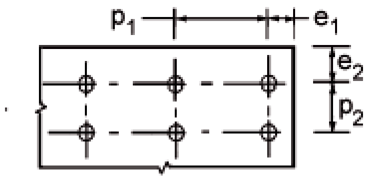


Figure 19 - Spacing of fasteners according EN1993-1-8

For a group of bolts subject to shear, the design resistance depends on the shear and bearing resistance calculated for each bolt within the group. If the design shear resistance of each bolt is greater than or equal to the bearing resistance the design resistance for the group of bolts may be taken as the sum of the bearing resistances of the individual bolts. Otherwise the design resistance of a group of bolts should be taken as the smallest shear or bearing resistance for any individual bolt multiplied by the number of bolts in the group (Simões. 2007).

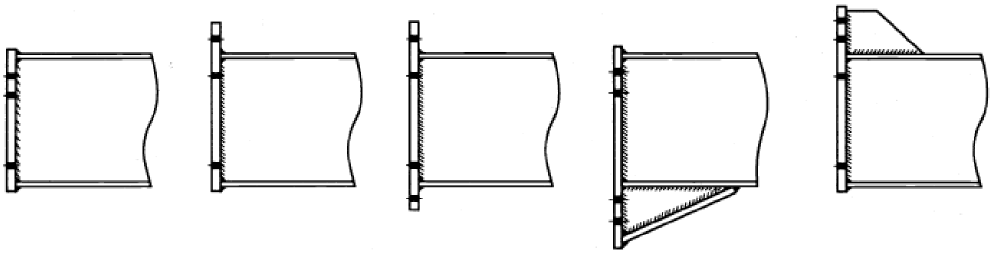


Figure 20 – Typical end-plate connections (structuremag.org)

2.4.2.2. Welded Connections

When two structural members are joined together by means of welds the connection is called a welded connection. Welded connections are mainly of two types, namely fillet welds and butt welds.

Weld is a way to execute connections between plates and resistant metallic sections which form a structure. A connection is made by melting the metal sheet or profile (base metal) while adding melted metal to join them. The melted metal in the weld metal is a mixture of metal base with steel electrode. This mixture after dry or solidified has the same minimum yield strength and a tensile strength specified for the base metal.

When welding is needed to be applied, it is the most economical way to make connections in steel structures. Welds must be performed preferably in the workshop can be executed on the spot if the specification allows.

The objective of the designer is to provide an assembly with adequate strength for the specific application with the least amount of weld metal and the minimum number of joints. This requires the designer to plan for a smooth flow of stresses through the joint, to compensate for any strength loss due to welding, to design the component such that there is sufficient access for welding and to select the metal to be welded with optimum weld ability in mind (Duggal. 2008).

The arrangements described here in this work are for welds which the base metal in this case profiles for beams and columns have a thickness less than 4mm. The weld connections must be performed using methods already proven, in particular welding processes by arc and oxyacetylene flame and should be in accordance with the corresponding standards for us is EN1993-1-3.
Welding should be performed preferably in the workshop, but may be performed on site if the specifications are possible. The provisions described in this paper are for welding in which the throat thickness is equal to or greater than 3 mm. Welded connections should be performed using proven processes, in particular the electric arc welding processes, and must comply with the standards (Duggal. 2008).

The basic weld types are:

- Fillet welds

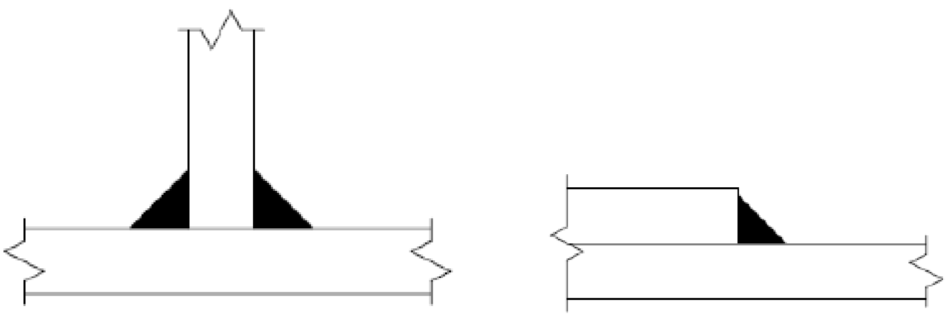


Figure 21 – Fillet Weld (Mark, 2012)

- Complete Penetration Joint

Figure 22 – Complete Penetration Joint (Mark, 2012)

- Partial Joint Penetration

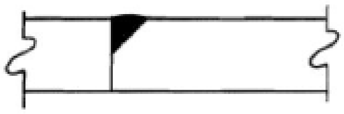


Figure 23 – Partial Joint Penetration (Mark, 2012)

Weld connections is determined by the mechanical characteristics and the chemical composition. However, there is no criterion which defines the weld for the different kind of welding processes, since the behavior of the steel during and after the welding is not dependent only on the material but also the size and shape as well as manufacturing and conditions of service of building elements (Duggal. 2008).

In the context of the mechanical qualities of steel EN10025 the choice of filler metal may be generally governed by the following principles:

- The welding consumables should be appropriate to the welding process chosen, the type of steel to be welded and the type of weld chosen
- The referred consumable should be stored and handled carefully following the instructions of the manufacturer
- The electrodes for manual metal arc welding must be kept within the original packaging and in a warm, dry place protected from the weather
- The flux must be stored and transported in containers to protect against moisture.

3. Structural Design – Practical Example (Carrington Project)

3.1. Introduction

3.1.1. General arrangement - Carrington Power Plant

The new 880 MW power plant will be located on the site of the old Carrington power station in Trafford, next to the Manchester Shipping Canal and the River Mersey, United Kingdom.

The power station will be capable of providing enough power for approximately one million homes

In the *Figure 24* and *Figure 25* below it's possible to see the general arrangement of the Carrington project with the Gas Turbine in the middle and all the auxiliaries connect to it.

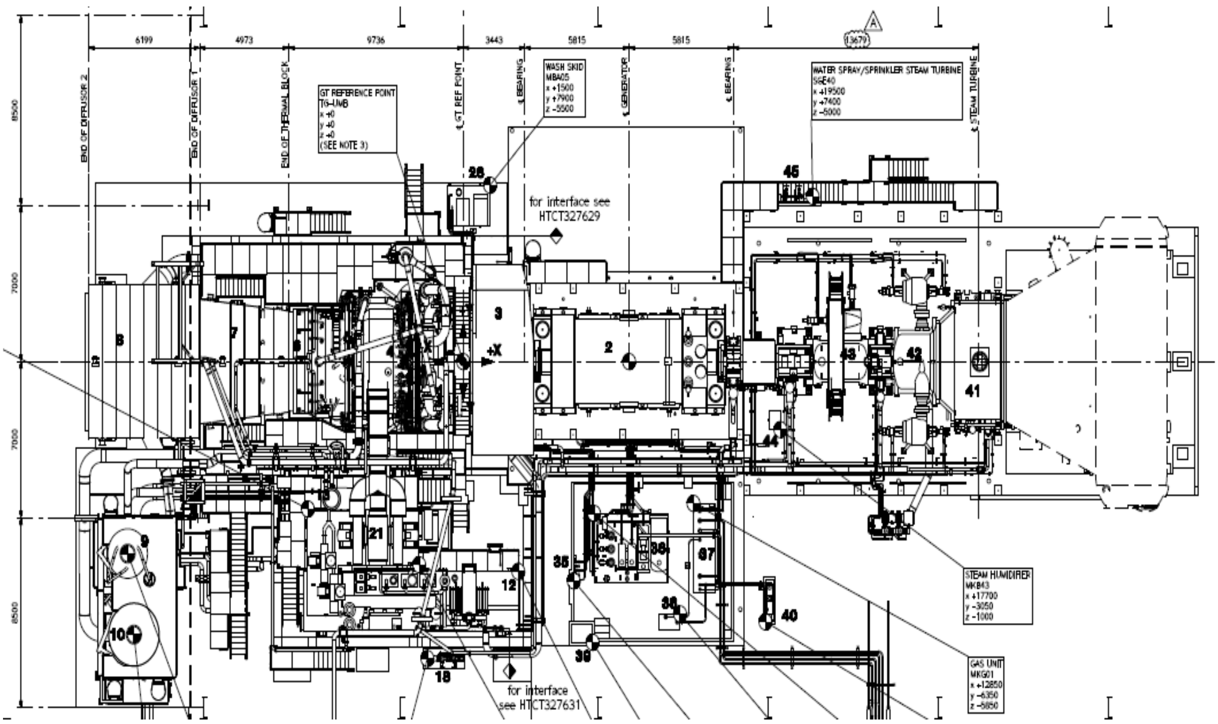


Figure 24 – GT 26 Plan view of the General Arrangement (HTCT023188 Rev. B)

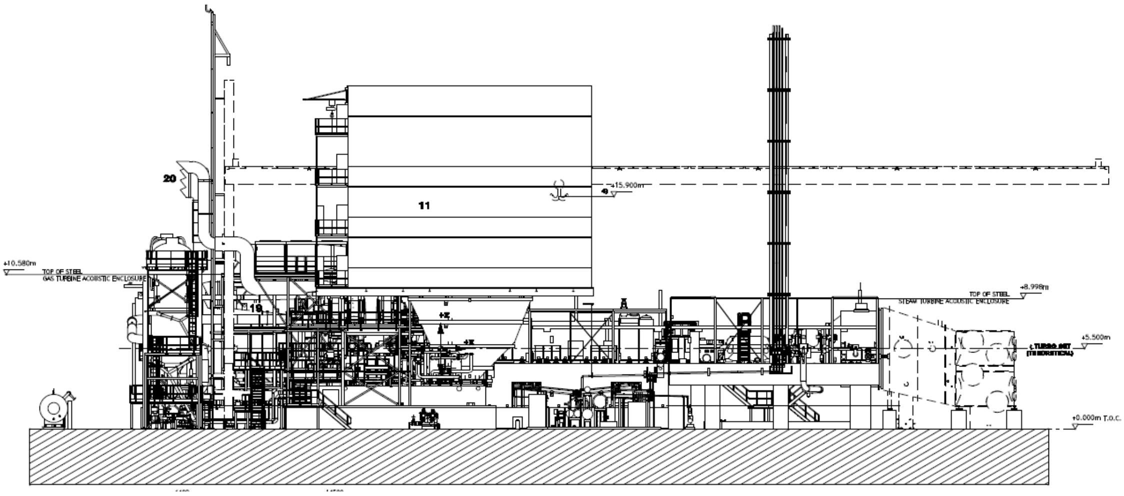


Figure 25 – GT26 Front view of the General Arrangement (HTCT023188 Rev. B)

According with the General arrangement above, the most important parts of the Gas Turbine are assembled in a continuous line called Gas Turbine Power Train Line, in the *Figure 26*, below it is possible to distinguish all the different parts and all the groups that work for Alstom Gas Power Business.

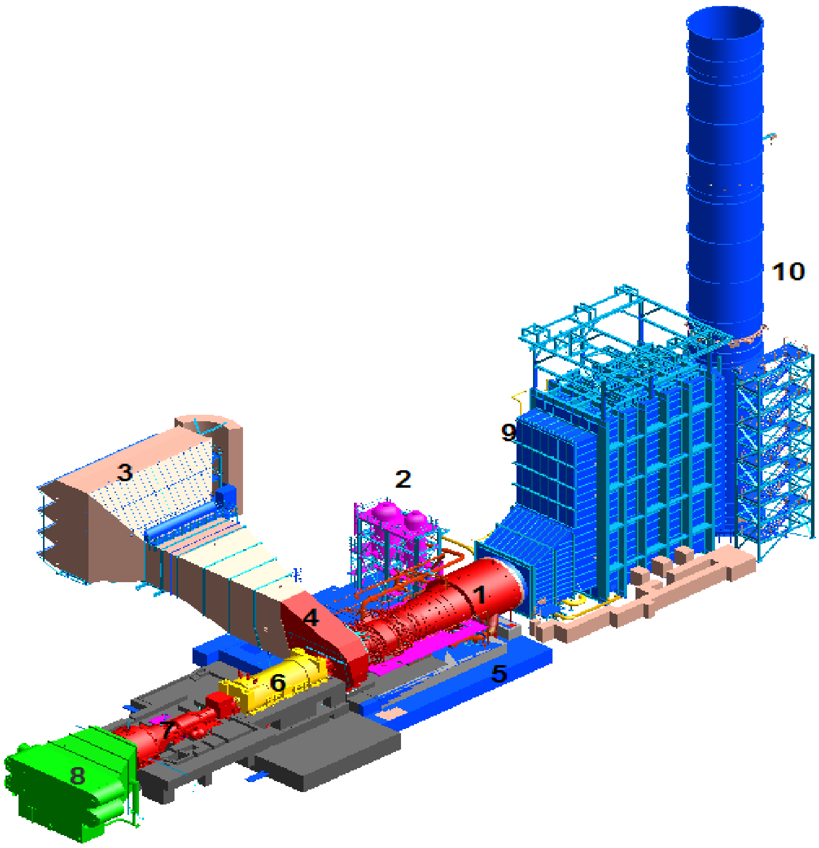


Figure 26 – GT26 power train (PDMS model)

1. Gas Turbine
2. Once Through Cooler (OTC)
3. Air Intake
4. Manifold
5. Main foundation
6. Generator
7. Steam Turbine
8. Condenser
9. Boiler
10. Exhaust stack

The project group that currently is working in the design of the OTC steel structure in Alstom is called Piping & Arrangement. The group is responsible for the arrangement of electrical cables, the piping arrangement and the design of the once through cooler (OTC) steel structure. The OTC steel structure is used to cool the gas turbine. In this specific project I will work on the OTC steel structure work package.

3.1.2. Once Through Cooler (OTC)

Some of the air that is compressed in the gas turbine compressor is used to cool the components in the gas turbine. However, air is heated up with compression, and in consequence its temperature has to be reduced for the gas turbine.

Such cooling is done by Once-Through Coolers (OTC). The design of this component is a tube heat exchanger, with the compressed air on the shell side, and feed water on the tube side. The tube side acts as once-through steam generator, producing superheated steam.

There are two once through coolers:

- HP OTC (*Figure 27*)
- LP OTC (*Figure 27*)

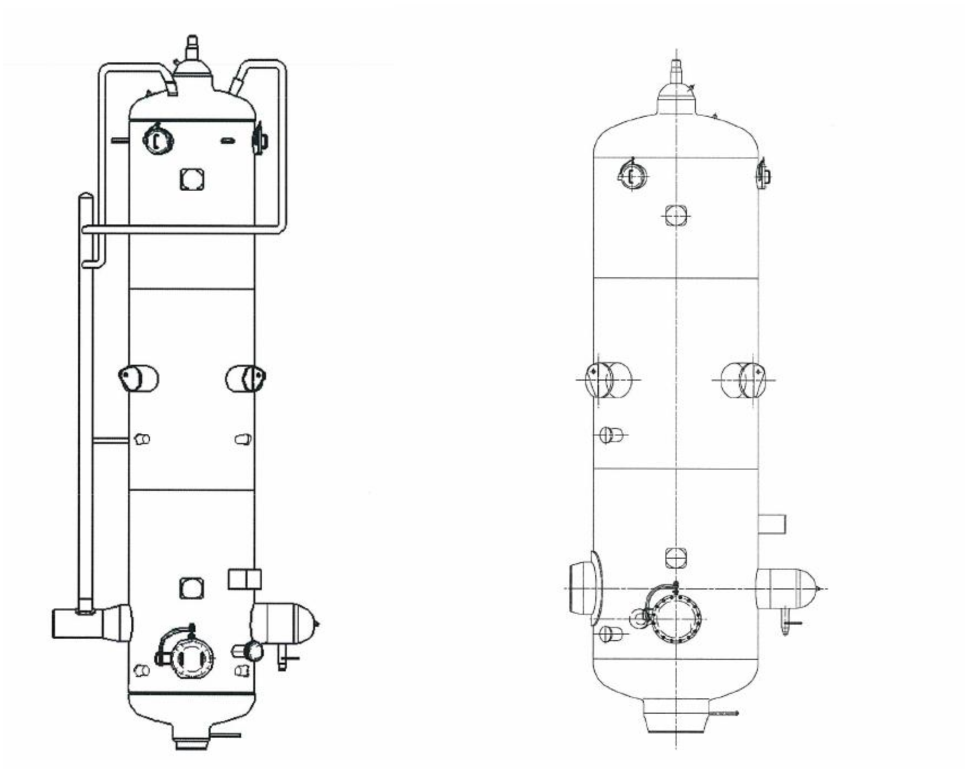


Figure 27 – HP & LP OTC vessel (1AHA090154 Rev. A)

As a recall, the air flowing through the compressor of a gas turbo-machine loses pressure, and is compressed. The potential energy of air obtained thanks to the compressor is then increased when ignition in the combustion chamber, and the released and transformed to mechanical energy in the turbine. When flowing through the turbine, there is a decrease in the pressure of the air. This means that pressure is variable in both Compressor and turbine. The LP-OTC (low pressure OTC) treats the air out of the low pressure stages of the compressor, and uses it to cool down the low pressure stages of the turbine, while the HP-OTC (high pressure OTC) does the same duty on the high pressure stages of the turbo-machine.

The primary task of the OTC is to cool the cooling air for gas turbine rotor blade and stator vanes. To increase the hot gas temperature of gas turbines to an economical level, the first expansion stage of turbine blade is air-cooled. The cooling air is taken from the compressor part of the gas turbine. Because of the sequential stage firing of the gas turbine, usually two expansion stages at different pressure levels must be cooled. The secondary task is to recuperate heat from the cooling air by producing steam.

The OTC is a surface heat exchanger of helix type. The hot air enters the vessel as shown in the *Figure 28* below, at the bottom and passes through the central channel (4) to the top where its flow direction is reversed. The air flows down through the helix bundle (2) on the outside of the tubes and transfers heat to the water and steam inside the tubes. The air leaves the cylindrical vessel through the radial outlet nozzle (8) at the lower end.

The cooling water enters the water chamber (5) through the inlet orifices (9) at the bottom of the vessel and is distributed to single tubes (2). These tubes are helical coiled from the bottom to the top of the vessel. While flowing upward through the tubes the water is heated up, evaporated and superheated. The generated steam is collected in the steam header (6) and leaves the vessel through the outlet nozzle (15) at the top.

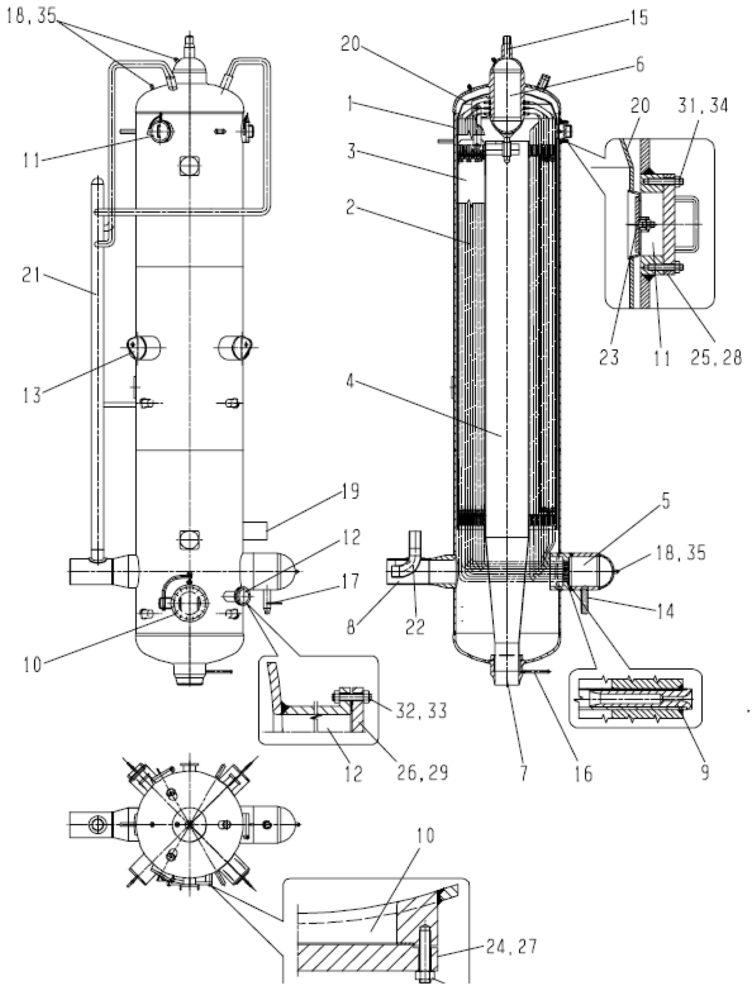


Figure 28 – OTC Vessels description (1AHA090154 Rev. A)

3.1.3. OTC - Steel Structure

The OTC steel structure supports the HP-OTC and LP-OTC vessels. Both vessels are supported at the top of the structure via spring hangers and shock absorbers. Other weights applied to structure include piping and platforms (*Figure 29*).

Figure 29 – OTC steel structure with the vessels assembled (www.alstom.com)

All major beams and columns are modelled using STAAD Pro and PDMS. These consist of a series of bracing frames pinned at the foundation.

Three access options are available to the maintenance of the OTC in general, one with stairs, another with ladders or with both access ways, stairs and ladders, three options will be decided by the costumer, when the contract is signed *(Figure 30)*.

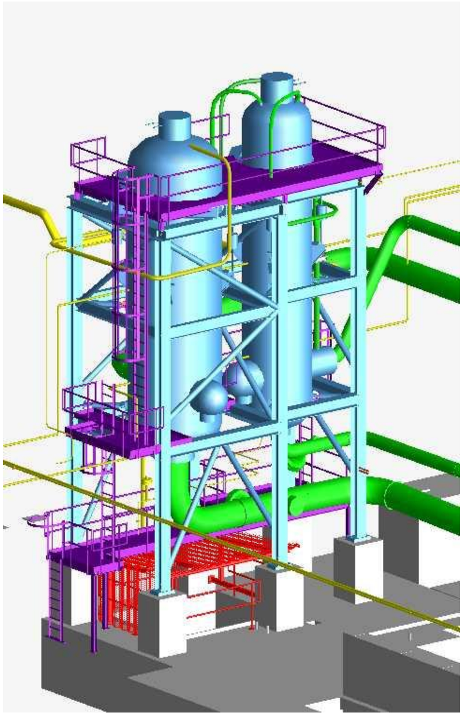


Figure 30 – OTC steel structure with the ladder access option (PDMS model)

3.1.4. Shock Absorbers and Spring Hangers

The shock absorbers commonly named snubbers allow for gradual horizontal movements of the OTC vessels, at which they do not transmit any axial forces. This is the case for movements induced by wind or thermal expansion of the connecting GT piping. It is only when the snubbers experience fast movements that they lock and then transmit axial forces. This is the case during an earthquake.

The structural model shall account for the fact that the snubber loads during an earthquake can be transmitted into the main columns eccentrically, thus inducing torsion of the main columns. The eccentricity can be determined with the plan view drawings of the snubbers. *(Figure 31)*

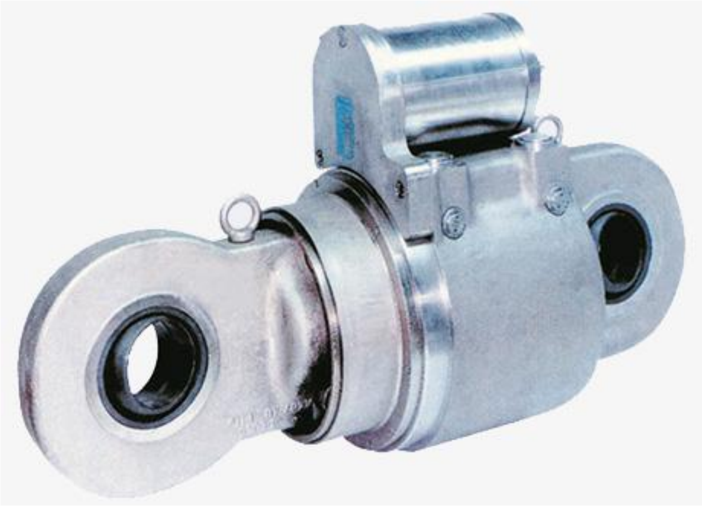


Figure 31 – Lisega Shock Absorber (www.lisega.de)

Wind forces that are collected by the OTC vessels are transmitted via the spring hangers into the support beams. They are not transmitted via the snubbers. One possibility to approach the eccentric transmission of these horizontal forces via the weld-on lugs into the support beams is to design the bottom flanges of the support beams for these transverse forces. This equally applies to the horizontal hanger forces induced by the possible inclination of the hangers. *(Figure 32)*

Figure 32 – Lisega Vertical Spring Hanger (www.lisega.de)

Stubbs or weld-on brackets welded into the sides of the main columns should be avoided, as they most likely cause distortion of the columns due to welding heat input.

An elastic design should be pursued for the reason that a dissipative design is not worth the enormous effort and wind loads govern the design.

3.2. Kick off meeting

After the contract is signed all the parts that are involved in the design of the power plant need to determine who will be the responsible work package engineer. The work package engineer then leads a kick off meeting. This meeting is composed by all the parties responsible for the project such as the lead engineer, civil works, aero-systems, electrical arrangement, piping, etc.

At the meeting all the project details will be discussed. Vital information such as which the client is, the location where the project will be executed, project schedules, or any contract agreements that the client may want or require that needs to be implemented on location or in the machine itself.

Every single project has special specifications signed in the contract these types of specifications normally demand standards, type of profiles, coatings, kind of erection etc. The project specifics are accorded with Alstom along with the client and should be delivered as an Alstom document to every group by the lead engineer after the kick off meeting has been presented.

All the relevant details for the Carrington Project presented during the kick-off meeting for the OTC Steel Structure were:

- Wind
- Seismicity
- Standards to be used in the design and in the profiles
- Corrosivity category
- Type of access (ladders or stairs)
- Type of machine (Rating)
- Noise outside the GT enclosure
- Location where the OTC will be positioned

3.3. Input to design the OTC Steel Structure analysis

After the presentation of the kick-off meeting and the annex part of the contract of the OTC Steel Structure scope has to been read, it is necessary to know all the inputs to the resolution of this design. According to the research made in the appendixes of the contract and dealt with all the relevant information we could conclude that the input data are:

- Load combinations
- Site conditions (seismicity, wind, environmental risk)
- Access option
- Vessel movements
- Type of standard used on site (profiles and design)

3.3.1. Load Combinations

The Load Combinations need to be reflected with the inputs given in the kick-off meeting, in the contract and in the technical specification for Carrington Project.

Here is necessary to consider the:

- Wind speed – 120 Km/h
- Seismic acceleration – 0.1 g
- Dead Loads:
    - HP OTC – 268 KN
    - LP OTC – 363 KN
- Live load – 5 KN/m2
- Piping Load
- Thermal Load

The self-weight of the steel structure is automatically calculated when the material is chosen in the STAAD Pro program.

For the carbon steel structures, we are mainly concerned with the Ultimate Limit States, which potentially could lead to loss of the structure.

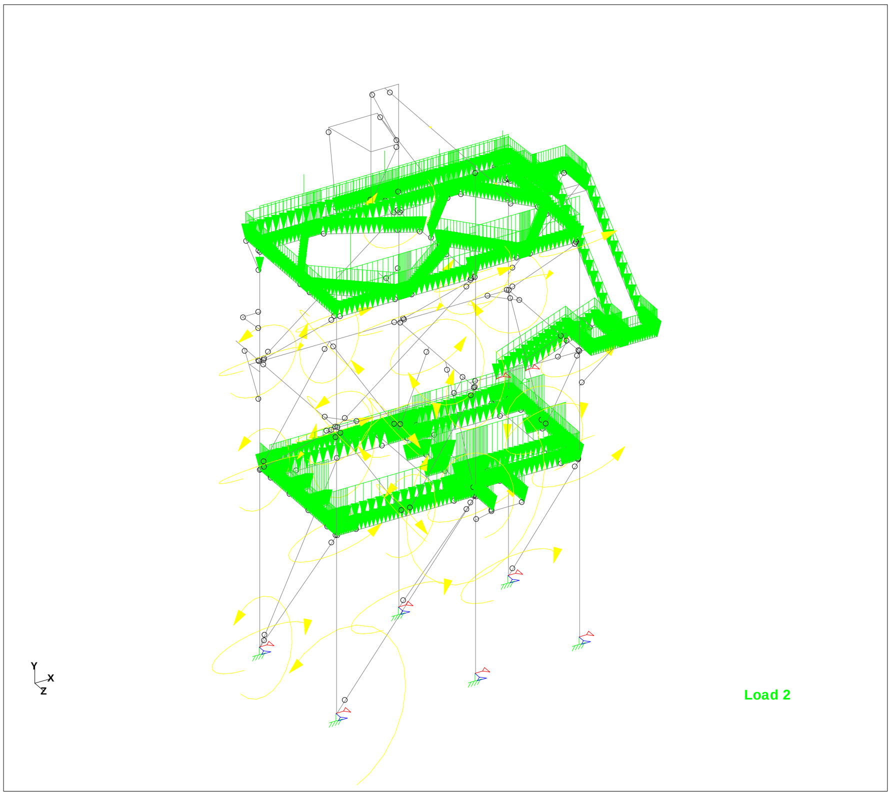


Figure 33 – OTC steel structure with dead loads (STAAD pro)

For Steel structures the sections shall be designed to resist the greatest demands corresponding to the most unfavorable of the combinations.

3.3.2. Site conditions

The reduction in structural performance of carbon steel building structures due to corrosion is not usually specifically considered by the structural designer, reliance instead being placed upon paint or other marine or offshore structures, the use of a sacrificial corrosion allowance on the thickness or by using cathodes protection.

Even though the Gas power plant is in a radius inferior to 50 km from the coast, this is a maritime environment, the OTC steel structure will be located indoor so according to these criteria the painting or coating to the steel structure shall be suitable to an environmental risk of a C3, with a high durability, the painting or coating need to ensure at least 15 years of protection.

3.3.3. Vessel Movements

The OTC vessels have a thermal movement, so in a plan view the vessels will have movements due to piping thermal expansion and contraction. For this reason the piping engineer needs to give the calculation of the vessel movements in order to know which type of shock absorbers and spring hangers will support those kind of movements, one other important and relevant criteria is, that the design of these shock absorbers and spring hangers should support the vessels forces.

Additional important information also needed for the design of the structure will be the seismic concept, with this document we will know the seismic activity acting in the steel structure.

The OTC was designed for a low seismicity location, with an acceleration of 0,1g so the structure only needs four top it snubbers and not four in the top and another 4 in the lower part like the OTC designed in a high seismicity location.

Figure 34 illustrates the position of the weld on brackets as well as the displacements of the OTC vessels.

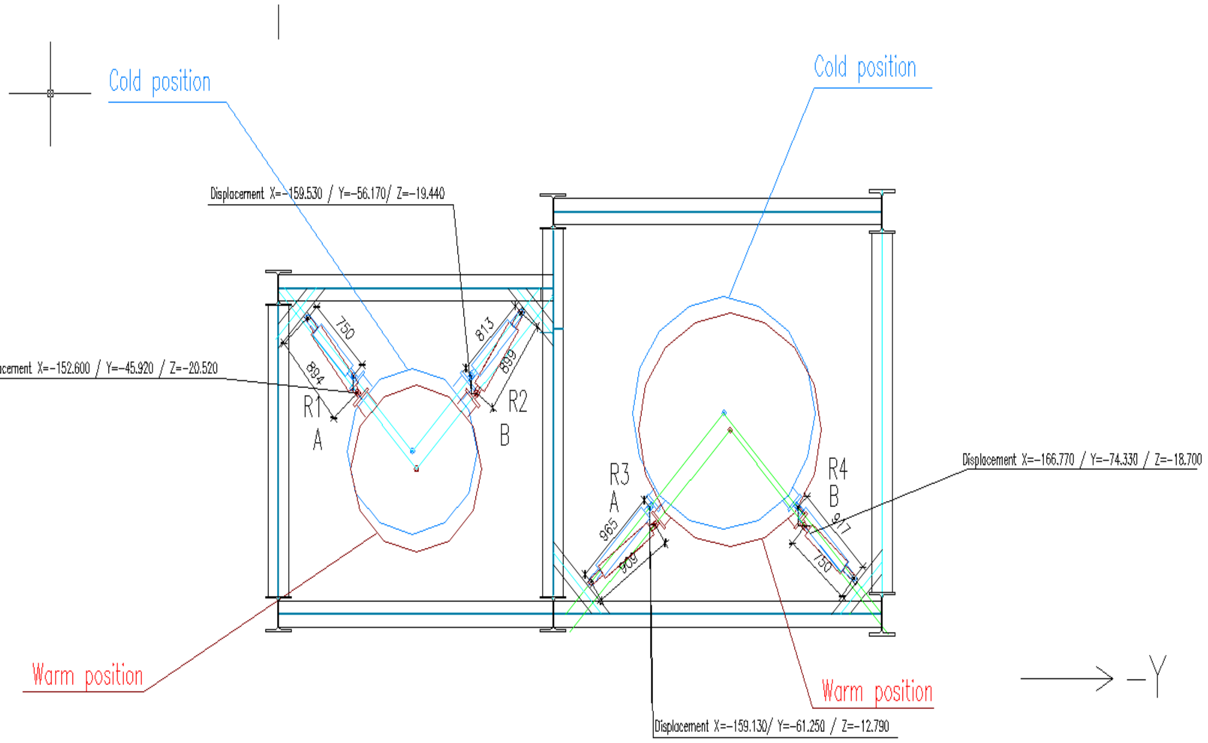
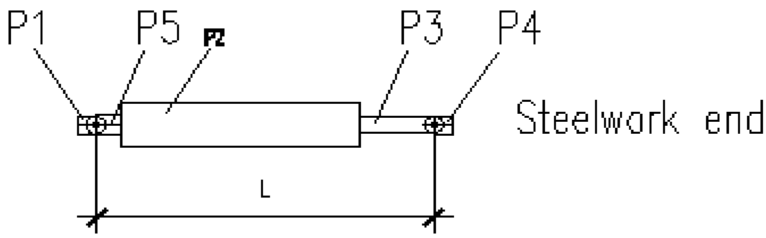


Figure 34 – OTC vessels warn and cold position (AutoCAD)

In the *Figure 35* below we can see all the necessary snubbers in the 4 positions to handle the OTC vessel as well as the range of the shock absorbers with and without the extension.

Parts list				
Rubric	P1	P2	P3	P4
	Weld-on bracket 1 pc.	Shock absorber 1 pc.	Installation extension 1 pc.	Weld on bracket 1 pc.
R1	[illegible]	305313	[illegible], L=150 mm	[illegible]
R2	[illegible]	305313	[illegible], L=150 mm	[illegible]
R3	[illegible]	[illegible]	[illegible], L=1[illegible]0 mm	[illegible]
R4	[illegible]	[illegible]	[illegible], L=1[illegible]0 mm	[illegible]

P 4 = 50 mm
P 5 = 65 mm



min P 2 = 595-2*65 = 465

min P 2 = 895-2*65 = 765

Without extension:
min E = 595 + 10 = 605
max E = 895 - 10 = 885

With extension L = 150:
min E = 65 + 465 + 150 +10 = 690
max E = 65 + 765 + 150 - 10 = 970

Figure 35 – Snubbers part list and stroke detail (AutoCAD)

3.3.4. OTC Steel Structure General Arrangement and access option

According with the contract all of the access ways for the OTC maintenance should be done by ladders.

With this type of access option is not necessary to design an auxiliary structure to the main steel structure. (*Figure 36)*

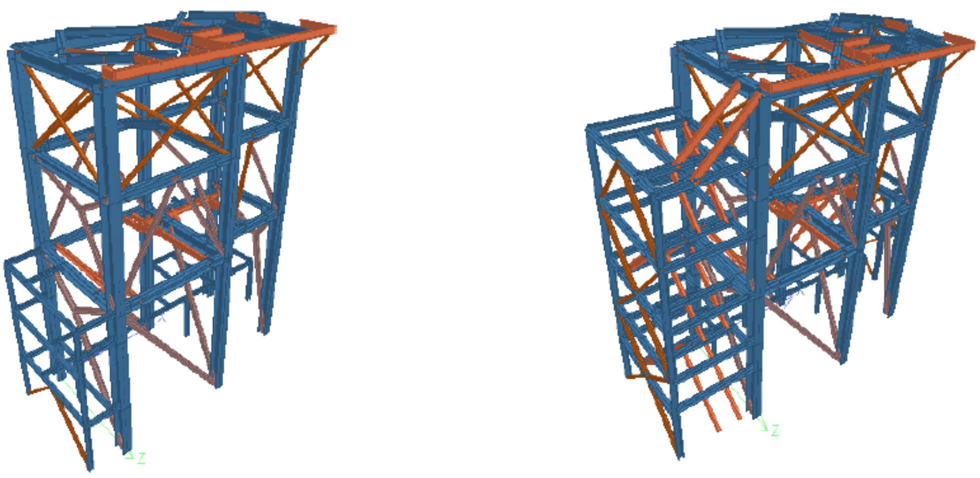


Figure 36 – OTC Steel Structure with ladder and with stair respectively (TN042902)

All of the guard-rails, gratings and ladders need to be in accordance with Alstom technical specification, with the use of this specification is possible to harmonize the design of this scope. In this same specification in the Appendices are also all the drawings details for this subject.

One important thing that is possible to save some time from our busy time schedule will be the General Arrangement drawing (*Figure 37*), with this drawing we will know all the necessary dimensions of the OTC Steel Structure and how this steel structure will look like. In this drawing will also appear the distances of the anchor bolts to connect the main columns of the structure.

Since Alstom has created standard OTC's Steel structure, the General arrangement of the structure can be used for several different project, only designing the steel, so in this case, we can used the general arrangement drawing as an input for the project.

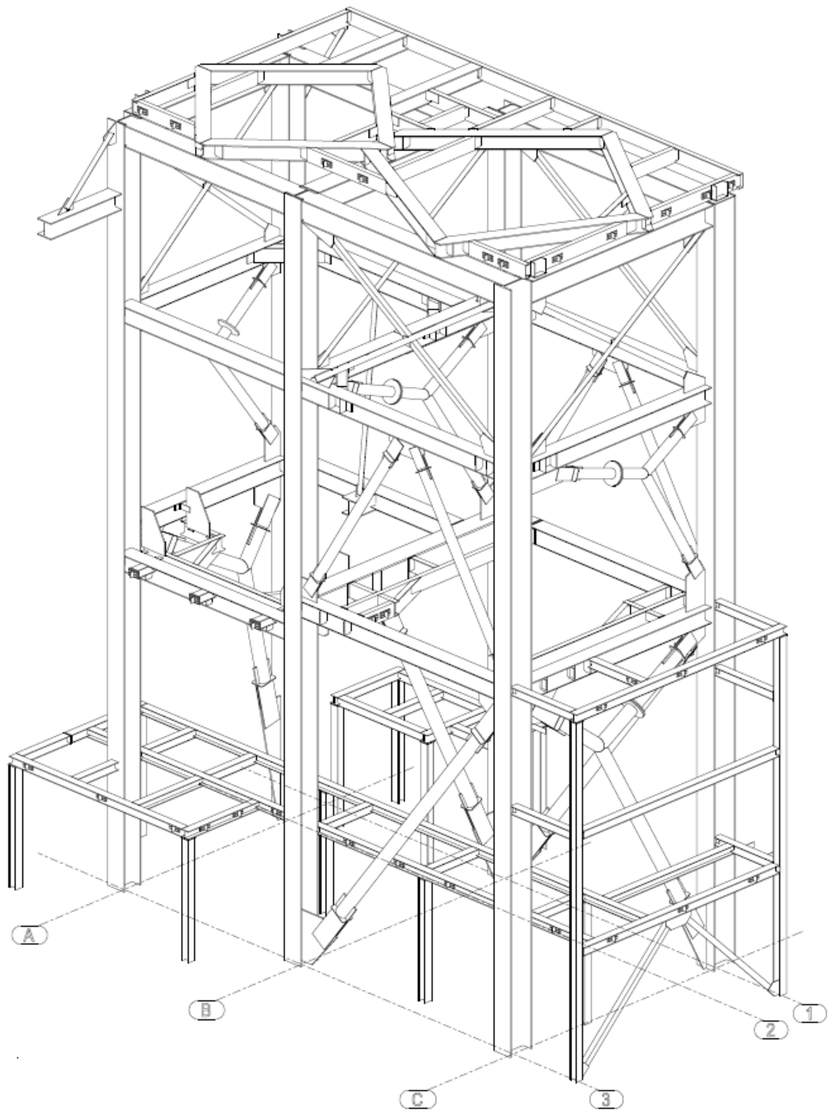


Figure 37 – OTC Steel Structure general arrangement (HTCT461705 Rev. C)

3.3.5. Standards & special agreements.

For this Project (Carrington) the Project Specific for the OTC steel structure designed shall be based on steel section materials with yield strength corresponding S275 according with the British Standard. The reason is to allow a subsequent up-rating of the structures for higher earthquake requirements of other projects by using higher grades without changing member sizes. If the material has a higher grade, it needs to be advised in advance to Alstom.

Based in the contract, the hardware supplier must clearly inform the origin of manufacturing.

All of the steel structure and the access option needs to be in compliance with the last technical specification in order to increase the time and cost

Arrangements for access, lifting, handling and associated lighting for operation and maintenance shall be considered at the time of designing the Plant, to ensure that practical, safe and cost effective arrangements are provided for persons, materials, tools and components.

The Suppliers shall ensure that all features necessary for safe operation are incorporated into each equipment design.

Safety features shall be incorporated which enable operations to be performed with minimum risk to operations staff, and the design of which are based on established good practice.

The supplier shall not subcontract portions of the supply or services without prior written approval by ALSTOM. The supplier shall notify ALSTOM’s engineer which of the major packages he intends to subcontract and where the production facilities are located.

3.4. Output

After consulting with the order manager and with our supplier, the time necessary for supplies and deliverables that need to send to Alstom, and in order to input all the necessary documents in our company system, the supplier needs to send us, via e-mail or via mail several documents necessary for the design of the OTC Steel Structure, such as:

- Calculation note
- Support reactions
- Civil Interface Drawings
- PDMS design
- Full Tekla Model
- Manufacturing drawings
- Bill of material
- Erection Instruction

Since the company doesn’t have a process design, the output had some dates that weren’t accomplished, causing delays in the deliverables, and some problems with vessels clashes with the structural beams.

Putting all these small issues a part the deliverables necessary to the project were delivered in time to the hardware order.

3.4.1. Support Reactions and Civil Interface drawings

One of the deliverables of the designer to send to the costumers is the support reactions report. This document shall have all the necessary information needed for the calculation of the supports, such as the load combinations with all the reactions calculated by the STAAD PRO.

Since for the project a standard OTC steel structure arrangement is used, it was not necessary to design the support reactions because the reactions will be the same as former projects, therefore we used documents of a previous project with the same specifications by doing this we were able to save time on the theoretical aspect of the project.

In these drawings, the interface between the foundation and the steel structure detail the connection of the steel structure and the auxiliary foundation are made with the anchor bolts. *(Figure 38 and Figure 39)*

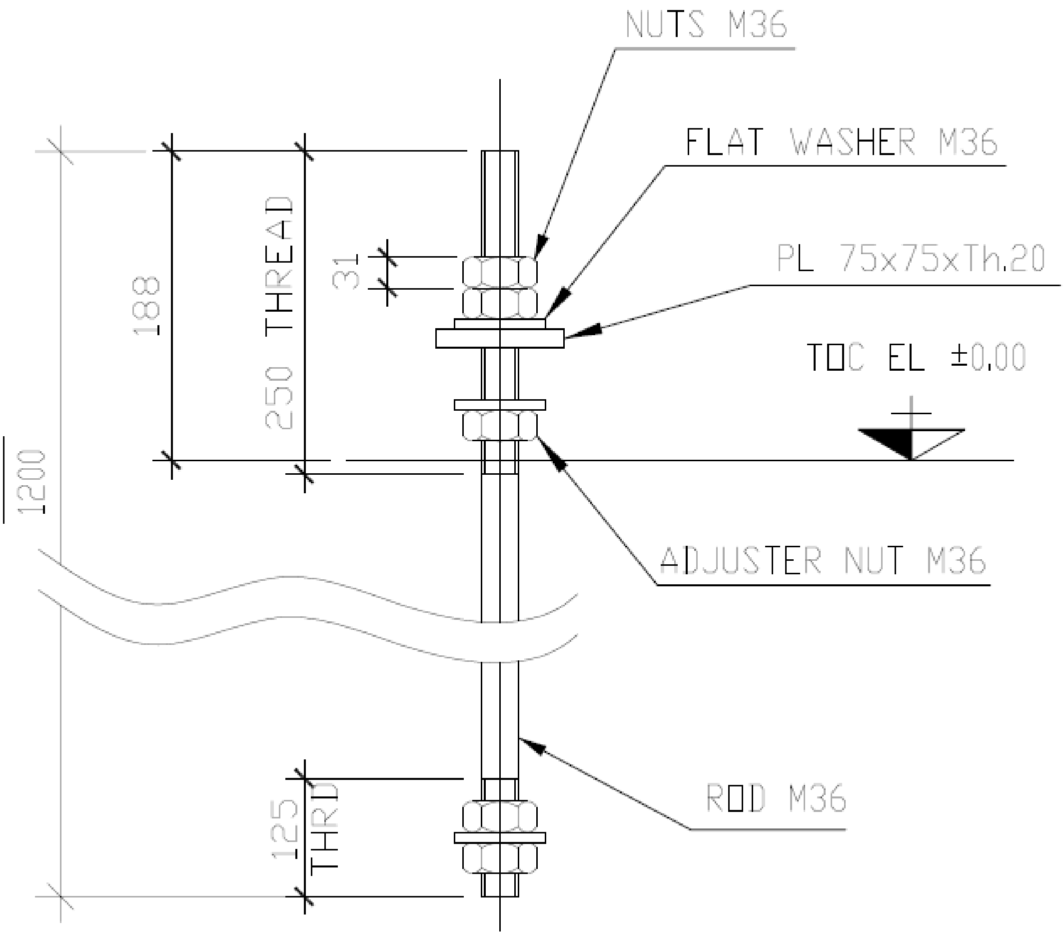


Figure 38 – Anchor detail (HTCT328787)

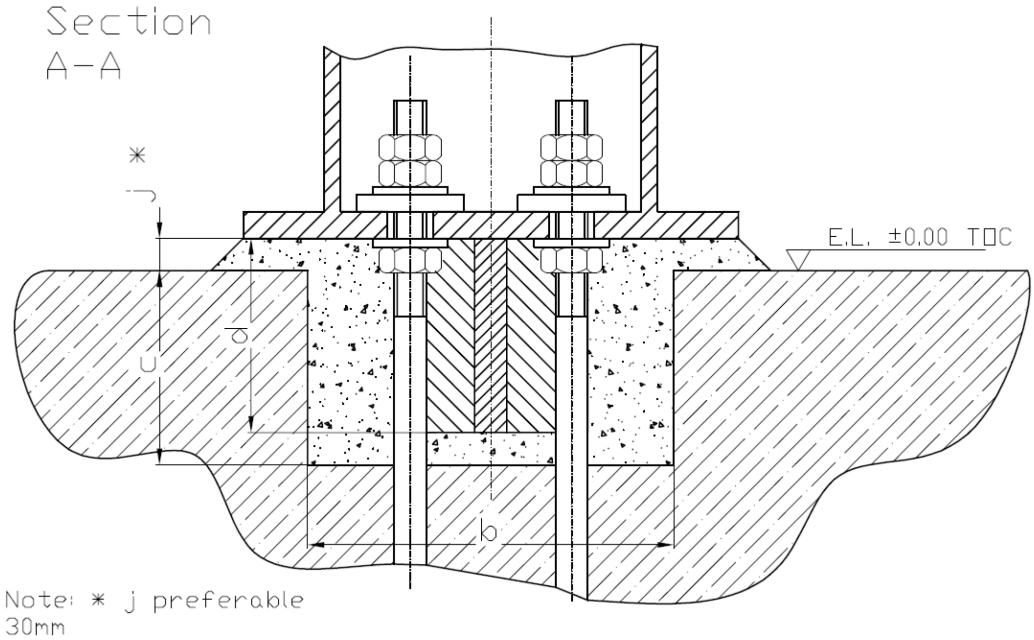


Figure 39 – Civil interface drawings (HTCT328787)

3.4.2. Calculation Note

The calculation note is the document where all the designed steel structure is presented to the costumer. The connection calculation is made only for bolted connections according to the Carrington contract. The connections design of the vessel to the steel structure via weld-on brackets is also described in this document.

For the steel structure the EN1993-1-1 is applicable but is used supplemented by certain national requirements. Seismic calculations are based on EN1998-1. The seismic loads from the vessels are included in the cooler loads and are derivative from the piping stress calculation which model the OTC vessels and air side piping.

The OTC steel structure is composed by columns beams and bracings. The main columns were designed with a HEB400 and HEA320 profiles and HEA240 and HEB220 for the main beams, for bracings are used circular hollow sections CHS 139.7x6.0.and CHS 177.8x12.0, all the remaining profiles are not presented in the STAAD PRO because the overall design it is not really necessary to implement all the secondary steel.

Calculations are presented with all load combinations, verification of the members and connections. Calculations also include all the inputs and outputs of the program STAAD PRO in an editable format.

The steel structure was verified for the two limit state design, ULS and SLS.

For the ULS all the main profiles were verified for the resistance of cross-sections, such as tension, compression, bending moment, shear, torsion, bending and shear, bending and axial force, and bending, shear and axial force. The steel structure was also designed in the Staad pro for the buckling resistance of members as uniform members in compression, uniform members in bending, uniform members in bending and axial compressions.

For the connection design we can separate them in two different categories, the trusses and the beams and columns.

In the *Figure* 40 is possible to see a bracing connection detail. For the most unfavorable bracing connection design are used six rows of bolts, the diameter of the bolts are M20 with an 8.8 class they have a transversal spacing of 70 mm and longitudinal spacing of 60 mm.

This was designed according to the EN1993-1-8 and the bolts were verified for shear resistance per share plane, bearing resistance, tension resistance, punching resistance and combined shear and tension.

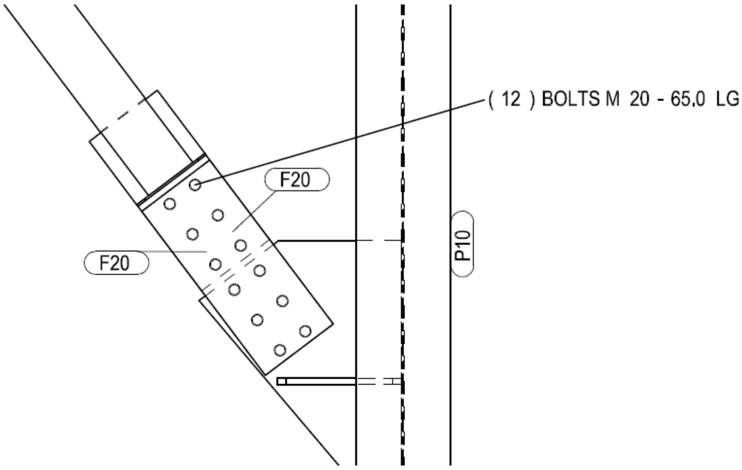


Figure 40 – Bracing connection detail (HTCT023395 Rev. A, sheet 3)

For the connection of the beams to the columns *(Figure 41)*, the forces acting are vertical shear force, axial force and in some cases we also have moments.

For this connection design the necessary verifications to be made are bolt tension, end plate bending, column flange bending, column web tension, column web panel shear, beam flange compression, column web crushing, column web buckling, bolt shear and bolt bearing.

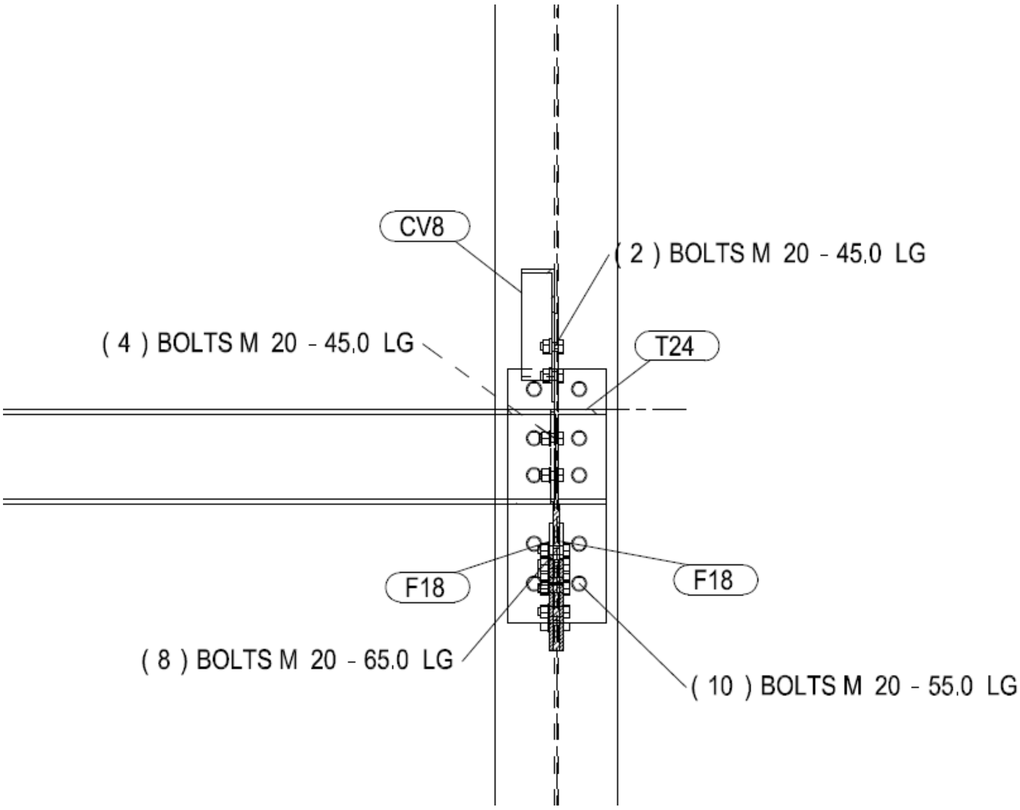


Figure 41 – Beam and column connection detail (HTCT023395 Rev. A, sheet 4)

The only welded connections that we have in the structure are the weld-on brackets that will support the Lisega shock absorbers, the end plates and the gusset plates that are connected to the columns and beams this is used to connect the bracings. For the weld-on brackets connection it wasn't necessary to design the web thickness because it is a standard value provided by Lisega (the weld-on bracket supplier), in this specific case the web thickness is 6mm. For the end plates web thickness was designed and the web thickness depends of the force that is acting on the element.

In the *Figure 42* below, it is possible to see a beam with the weld-on bracket and two end plates attached to the supporting beam.

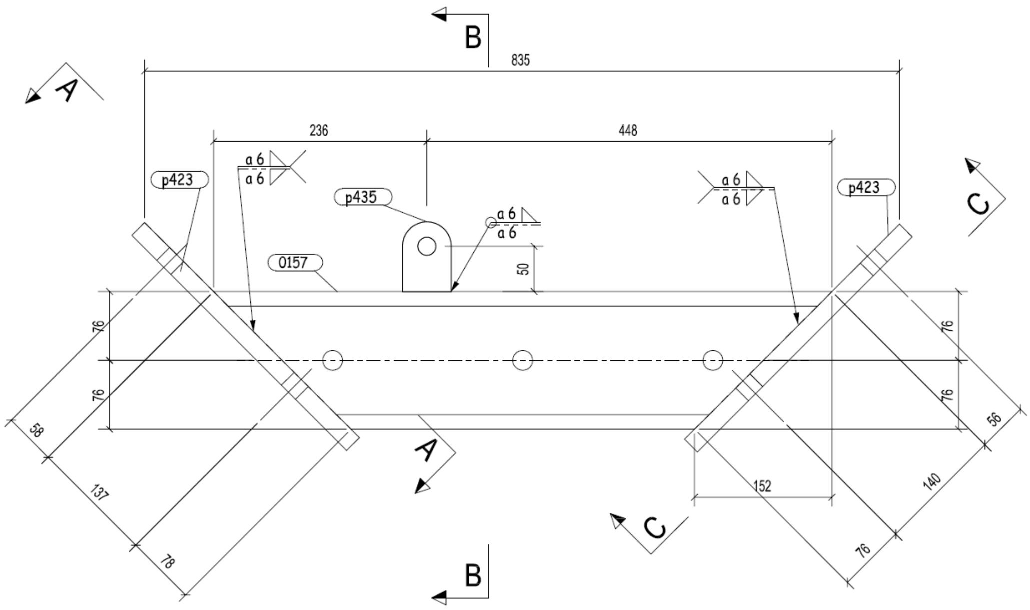


Figure 42 – Weld-on bracket detail drawing. (HTCT022770 Rev. A)

3.4.3. PDMS design

PDMS is a 3D CAD software program used for engineering design and construction projects

The designer of the OTC steel structure will need to perform the design on this computer program. This program is dedicated to modeling components of the power plant, after all the groups modeled every single component in their scope, we will be able to see in this program the entire power plant.

In the *Figure 43* it's possible to see the Carrington power plant modeled in PDMS, with all the details necessary to the accomplish of the project.

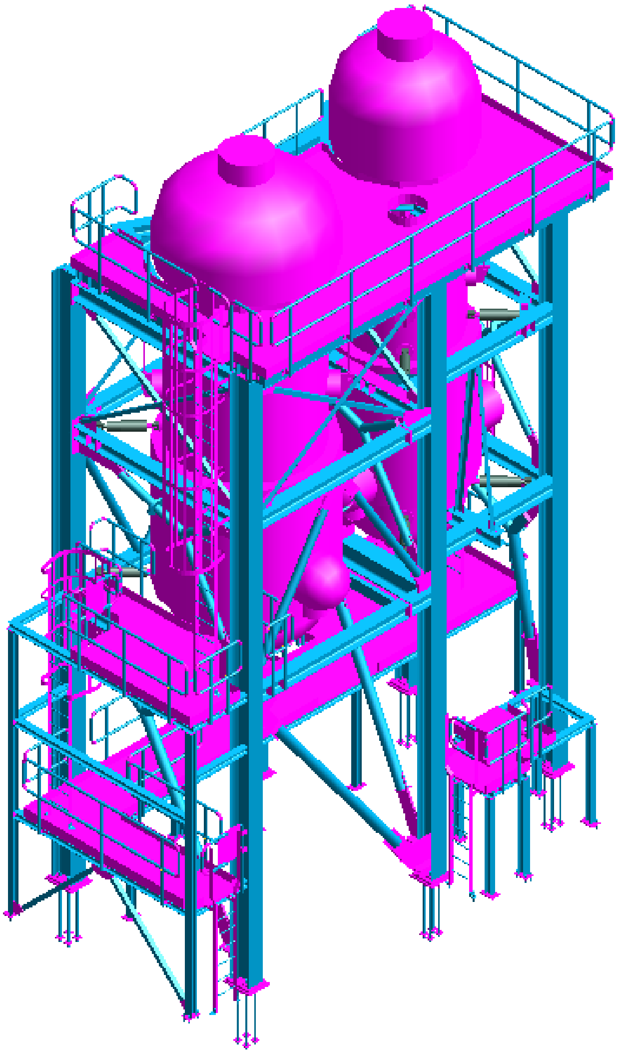


Figure 43 – Carrington PDMS model of the OTC Steel Structure with the vessels and ladder access option (CPL model, PDMS)

3.4.4. Tekla Model

Tekla BIMsight (Building Information Modeling) is a software to model project that enables the creation and management of accurately detailed, highly constructible 3D models of enumerable types of materials or structural complexity. Tekla model can be used to cover the entire building process from conceptual design to fabrication, erection and construction management (www.tekla.com).

The Tekla model is a good way to check all the small details that the designer makes in the structures making our job easier to check. Here we can find all the bolted details, welded details, distances, lashes etc. *(Figure 44, Figure 45 and Figure 46)*

With the Tekla model it is really easy to check all the connections mistakes, it is also easy to check the access ways, this being fundamental to improve time between the cross checking from Alstom and the changes by the designer.

The Tekla model is only used for the internal use of the company and the responsible of the steel structure therefore it is not necessary to send to the hardware supplier.

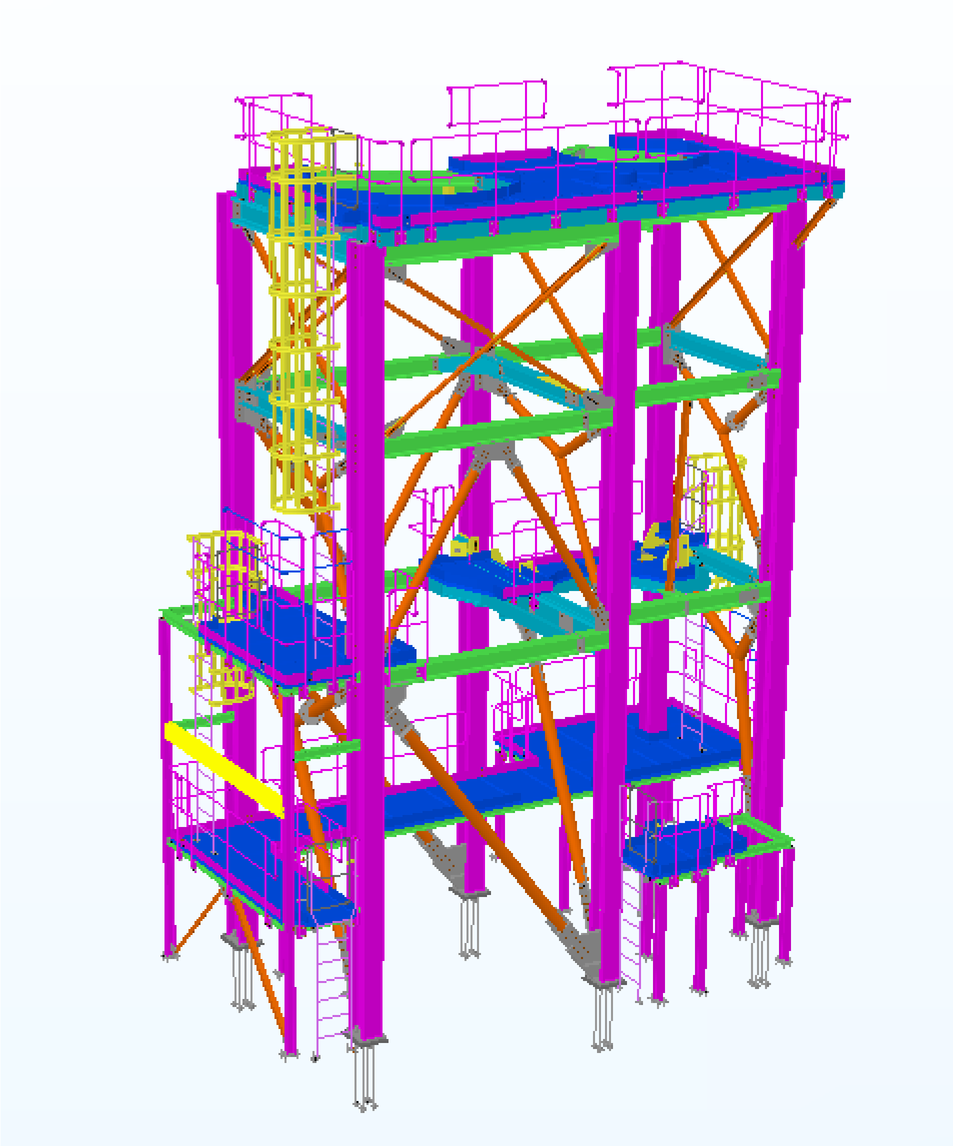


Figure 44 – OTC steel structure (Tekla BIMsight)

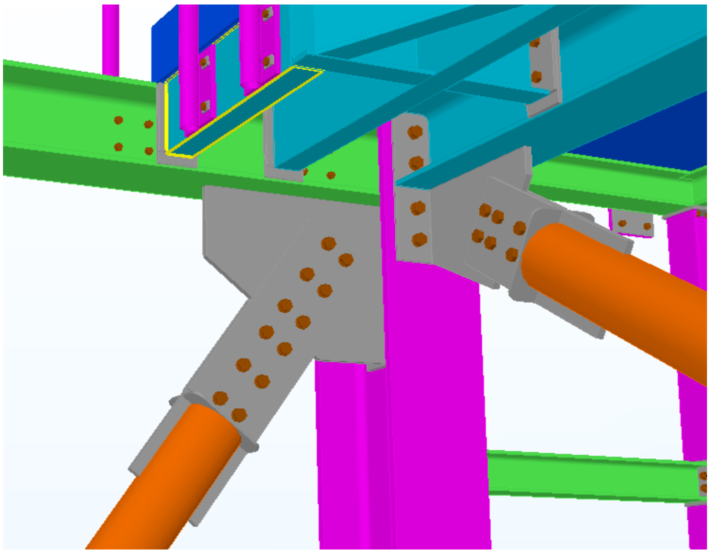


Figure 45 – Bracing connection detail (Tekla BIM Sight)

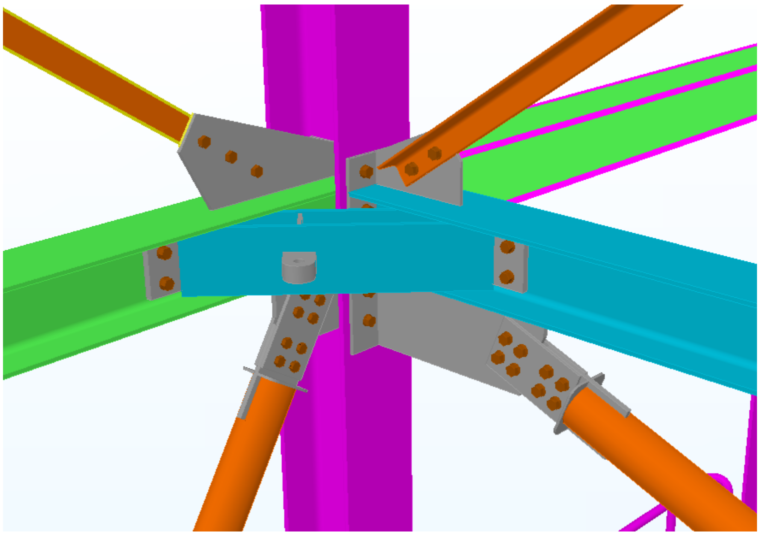


Figure 46 – Support stub beam detail (Tekla BIM Sight)

3.4.5. Erection and Detailed Drawings

Erection drawings provide the field erection crew with the roadmap of how to erect the steel assemblies after they are delivered to the field. Essentially, they are a set of instructions on how to put the puzzle pieces together. The erection drawings look very similar to the structural drawings produced by the designer with a few major distinctions. First, every assembly shipped to the field is given a shipping piece number to identify it. This number is noted in the drawing and is also stenciled into the actual assembly of the steel. In the *Figure 47* below we can see the OTC erection drawing will all the labels in the elements (Berman, Gary).

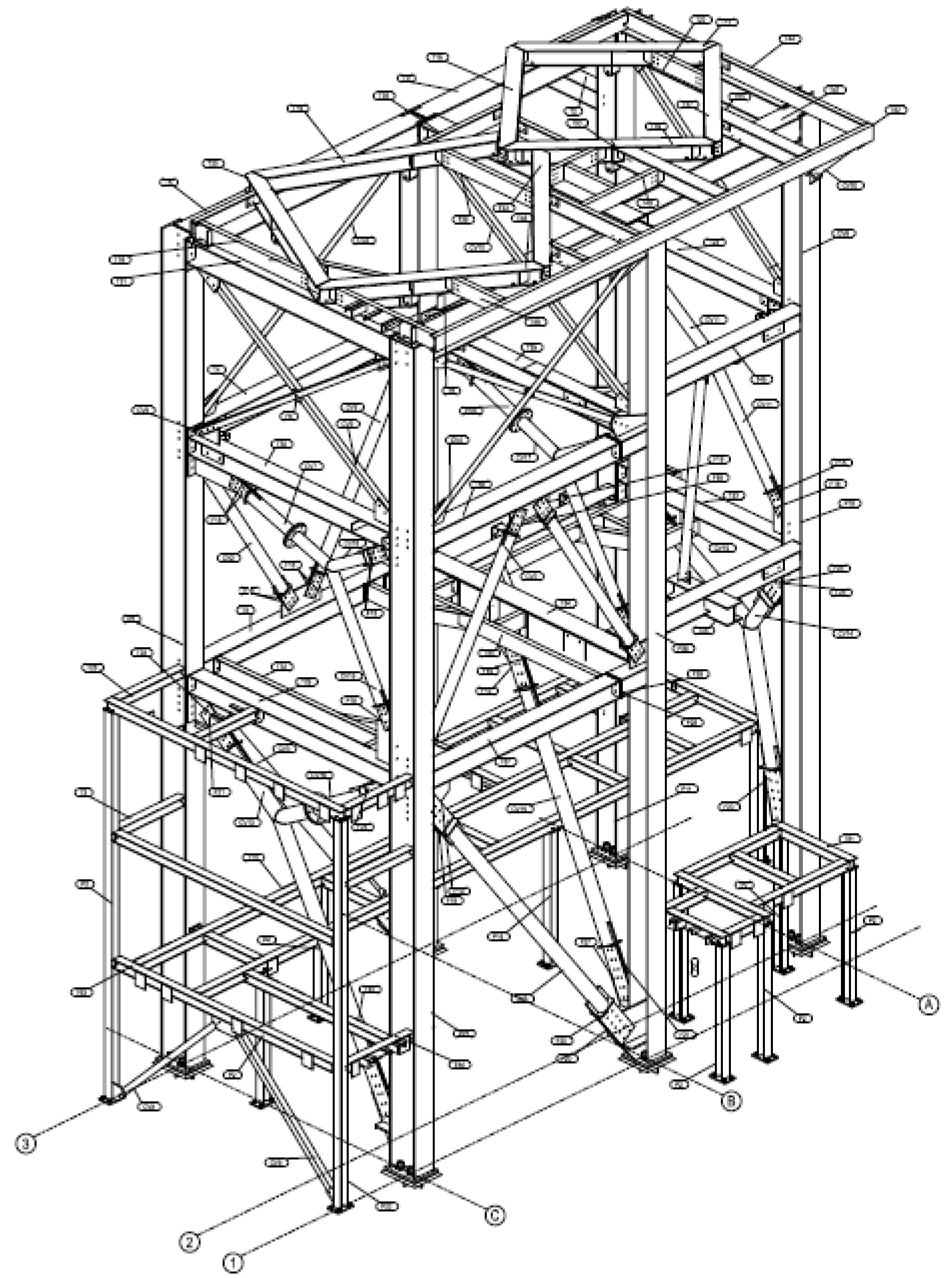


Figure 47 – Erection drawing detail with all the labeling parts (HTCT023395 Rev. A)

On the erection drawings, every assembly of steel it is shown, no matter how small the pieces are. The erection drawings need to exactly illustrate the location of the members so that the geometry of the connections can be designed and detail drawings produced also illustrate the parts of the assembly.

Detail drawings depict the components of each assembly. On these drawings, the designer give the fabricator step by step instructions on how to fabricate each and single piece of the structure. Each detail pieces is given a distinct number so that the persons that adjust the structure know how to put the assembly the whole structure (Berman, Gary).

The detail drawings usually show a final bill of material for the steel used on a particular drawing.

With these drawings we also know where we need to connect each element (columns bracings, beams).

In the *Figure 48* below we have an example of a detail drawing of a construction column with the base and end plate belonging to the OTC steel structure. In this same picture is possible to see the labels in all the different parts.

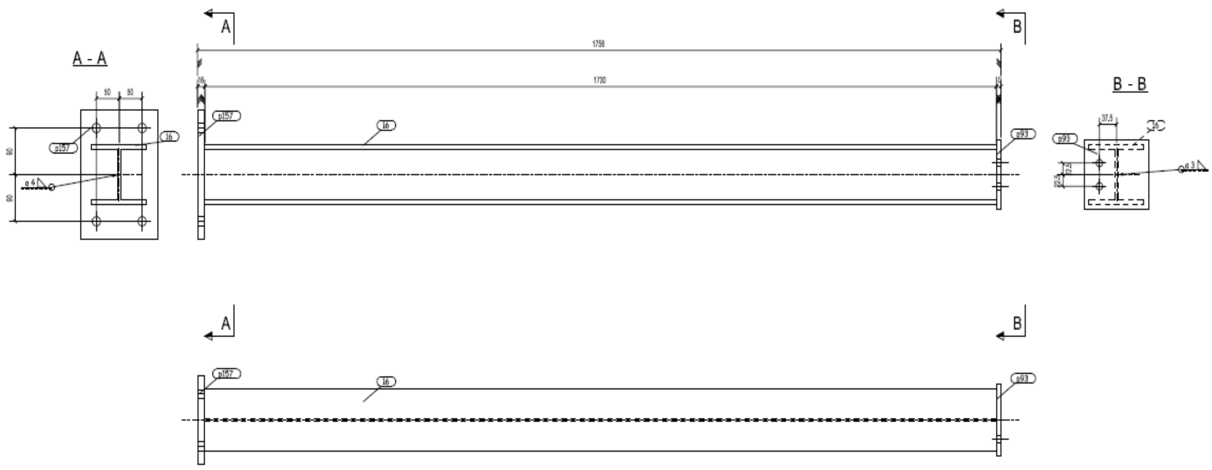


Figure 48 – Detail drawing (HTCT022668 Rev. C)

3.4.6. Bill of Material

When all the steel structure is designed and agreed by the company, we need to make the order to the hardware supplier. Taken directly from the structural plans produced by the designer, the order managers prepare the steel orders according to the bill of material.

The bill of material is no more than a detailed quantity takeoff of the steel in the job. Besides ordering standard shapes, the designers determine the quantities, sizes and thicknesses or flat plates that are required for the structure.

These plates are most commonly used to the various assemblies, including end plates, stiffeners and base plates (Berman, 2013).

Below in the *Table 4* is possible to see an example of the bill of material

Profile	Material	Q.ty	Length (mm)	Area(m^2) un.	Area(m^2) tot.	Weight (kg) un.	Weight (kg) tot.
PD139.7X6	S235JRH	1	381.17	0.15	0.15	6.69	6.69
PD139.7X6	S235JRH	1	582.55	0.25	0.25	10.99	10.99
PD139.7X6	S235JRH	1	629.94	0.27	0.27	11.92	11.92
PD139.7X6	S235JRH	1	704.83	0.31	0.31	13.48	13.48
PD139.7X6	S235JRH	1	722.72	0.30	0.30	13.24	13.24
PD139.7X6	S235JRH	1	732.69	0.32	0.32	14.02	14.02
PD139.7X6	S235JRH	1	736.49	0.32	0.32	14.10	14.10
PD139.7X6	S235JRH	1	914.37	0.39	0.39	16.94	16.94
PD139.7X6	S235JRH	1	1213.54	0.52	0.52	23.00	23.00
PD139.7X6	S235JRH	1	1637.13	0.70	0.70	31.16	31.16
PD139.7X6	S235JRH	1	1675.23	0.72	0.72	31.81	31.81
PD139.7X6	S235JRH	1	1867.65	0.80	0.80	35.56	35.56
PD139.7X6	S235JRH	2	2512.57	1.09	2.19	48.42	96.84
PD139.7X6	S235JRH	2	2512.86	1.09	2.19	48.43	96.86
	Total	**16**	**21849.00**		**9.43**		**416.61**

PD= Pipe Diameter, i.e. Circular Hollow Section according EN 10210-2 (hot-formed)

Table 2 - Bill of material (HTCT473246 Rev. E)

4. Final Considerations

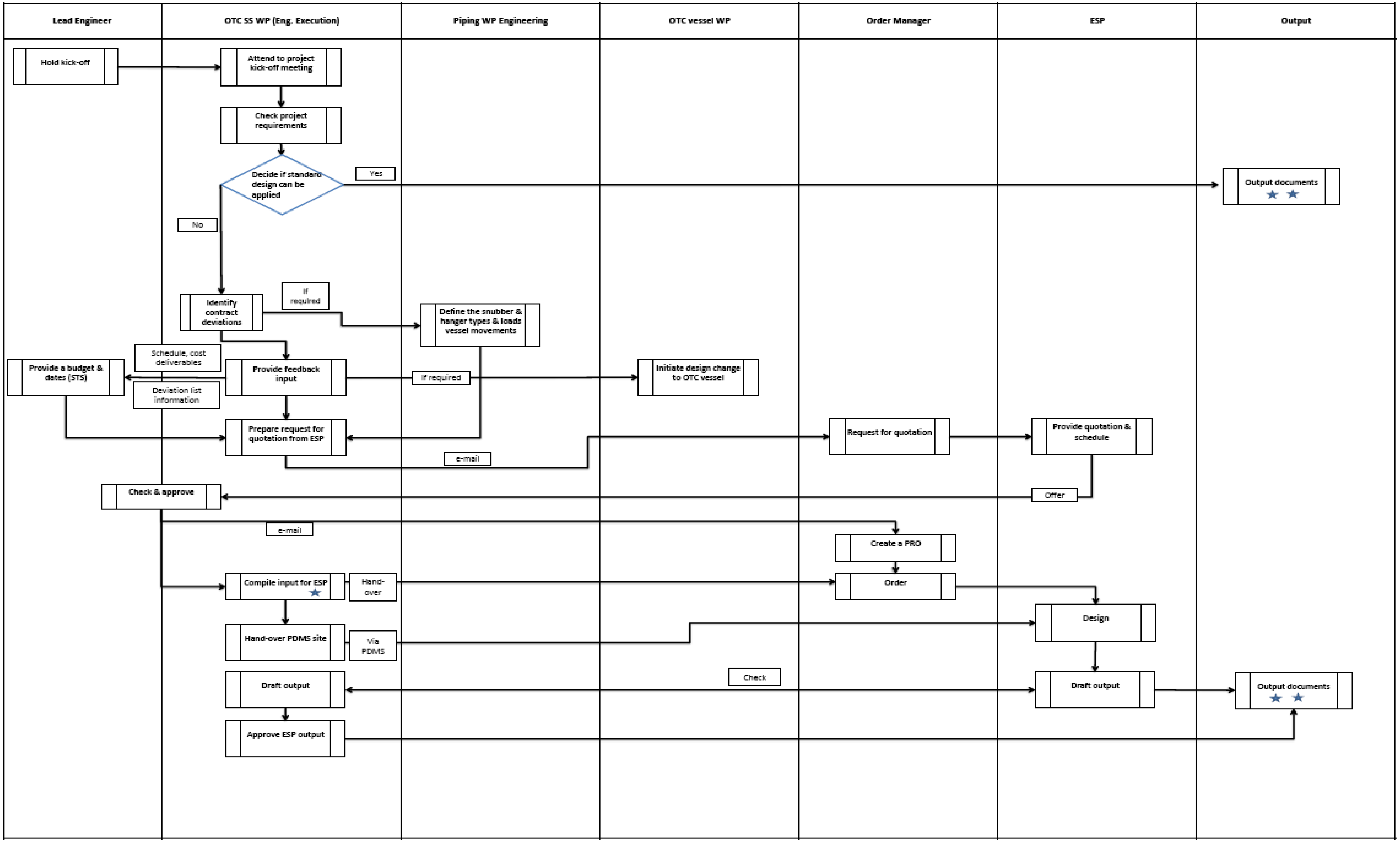
4.1. Conclusion

On this OTC Steel Structure Work Package for Carrington project, a process of the engineering service package including the work instruction was created. The objective of this WI was to define the process and the deliverables for the design of the OTC SS with the partners and suppliers.

The WI is applicable to all GSTEM, and must be applied to all engineering projects.

With this process it is ensured efficient project execution time, high quality of the delivered project documents, clear definition of the process steps, discussion the fundamental load cases and the load combinations to be applied to the structure with the ESP.

According with this work instruction, the creation of a chart flow is also necessary to follow the steps for the design with all the deliverables are in *Annex A.*


Lead Engineer
OTC SS WP (Eng. Execution)
Piping WP Engineering
OTC vessel WP
Order Manager
ESP
Output
Hold kick-off
Attend to project kick-off meeting
Check project requirements
Decide if standard design can be applied
Yes
No
Output documents
Identify contract deviations
If required
Define the snubber & hanger types & loads vessel movements
Schedule, cost deliverables
Provide a budget & dates (STS)
Provide feedback input
If required
Initiate design change to OTC vessel
Deviation list information
Prepare request for quotation from ESP
e-mail
Request for quotation
Provide quotation & schedule
Check & approve
Offer
e-mail
Create a PRO
Compile input for ESP
Hand-over
Order
Design
Hand-over PDMS site
Via PDMS
Check
Draft output
Draft output
Output documents
Approve ESP output

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Annex A

The necessary inputs & outputs for the process.

★ When all the information necessary for the production of the OTC steel structure is available, the work package owner can decide if it is possible to use the standard structure. In order to solve the engineering tasks related to the OTC steel structure, we will deliver the following inputs to the ESP:

1. PDMS model handover;
2. Platforms and ladders specification HTCT625876;
3. OTC vessel outline drawings (by NUHX);
4. List of deliverables with project schedule and STS dates
5. Design requirements document (including material and shapes to be used)
6. PDMS model (column grids, TOSs & TOGs, bracings, snubbers & hangers, grating cut-outs, access options, diamond beam layout)
7. Lower & upper snubber elevation plan view drawings
8. Hanger loads (from piping WP engineer)
9. Snubber loads (from piping WP engineer)
10. Pipe support loads (from TMG)
11. Pipe support loads (from Plant)
12. Basic wind speed F_{wk}
13. Basic snow load Q_{Sn}
14. Live load on grating & stairs
15. Seismic concept
16. Thermal load
17. Any other relevant loading
18. Any relevant customer specific documents
19. Design standard(s) to be used

★★ Once the engineering tasks are completed by the engineering team, the deliverables should be:

Items	**Time**	**Best practice documents**
Civil interface drawing	within approx. 10 weeks	HTCT355818
Support reactions	within approx. 10 weeks	HTCT473791
Rough BOQ	within approx. 10 weeks	HTCT473792
GA drawing	within approx. 10 weeks	HTCT022890

Full Tekla model	within approx. 11 weeks	-
Parts list	within approx. 12 weeks	HTCT473794
Fabrication drawings	within approx. 14 weeks	HTCT125769 - HTCT125968
Erection drawings	within approx. 15 weeks	HTCT022891
Calculation note	within approx. 16 weeks	HTCT473793
Erection instruction	within approx. 17 weeks	HTCT473795
Tekla BIMsight	successively upon commencement of Tekla modeling	-