MASTER THESIS OF SCIENCE

STEEL QUANTITY AND COST COMPARISON OF MODULAR CONSTRUCTION OPTIONS FOR SEA-TRANSPORTED PIPE RACKS

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FLUOR

STEEL QUANTITY AND COST COMPARISON OF MODULAR CONSTRUCTION OPTIONS FOR SEA-TRANSPORTED PIPE RACKS

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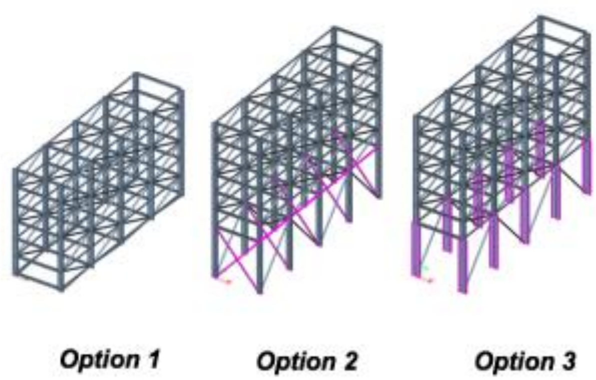
ABSTRACT

Steel Quantity and Cost Comparison of Modular Construction Options for Sea-transported Pipe Racks

This thesis is aimed at finding the most cost effective way of executing Modular Execution Strategy (MES) for building pipe racks of a project that an engineering company Fluor B.V. is currently executing in Kuwait. A pipe rack is a steel structure which is constructed to efficiently place and support multiple levels of pipelines for industrial plants such as refinery plants, chemical plants or power plants.

The Modular Execution Strategy aims at relocating parts of fabrication and assembly activities of a pipe rack construction to potentially low cost locations at which the conditions for fabrication and assembly activities are more favorable. The pre-assembled pipe racks will be transported to the onshore installation site by a vessel, which results in sea-transport design requirements (due to vessel motions) in addition to the in-place design.



Three options of different configuration for MES were considered. The first option is to transport only upper parts of the pipe racks without their bottom columns and assemble the bottom columns at the installation site. The second option is to transport the complete pipe racks including bottom columns which are stiffened by temporary bracings. The last option is to transport complete pipe racks with strengthened columns having a larger profile dimensions.

In order to consider various sizes of pipe racks, 27-representative configurations of pipe racks of the project were selected. These pipe racks were designed to withstand in-place loadings and sea-transport loadings with a quasi-static analysis method. The in-place loadings are weight of pipe lines and wind force. The sea-transport loadings are forces due to motions of a vessel and critical sea-transport loadings come from roll + heave and pitch + heave. Quantities of steel for each option were found after completion of the design. Subsequently, the quantities were translated into steel work cost which includes procurement, fabrication, assembly and installation costs of steel work.

As a conclusion, it was found that considering the quantities and costs of steel work for the project, option 1 (transport the pipe racks without columns) is the most cost effective solution. If pinned supports are used at the vessel deck, which are more favorable for the company, it was calculated that option 1 requires, on average, 15% and 30% less cost than option 2 and option 3 respectively. For clamped supported conditions, option 1 still requires 15% less cost than both option 2 and option 3.

Furthermore, it was demonstrated by performing a resonance check and a dynamic analysis for a tall two-dimensional frame, that a quasi-static analysis method could be used to assess the sea-transport loadings. It was found that there is very low possibility of resonance and only low dynamic amplification.

In this thesis, the focus has been on differences in the structural configurations. Other aspects, some of which may be difficult to express in cost terms such as logistical difficulty, safety/risk, and project schedule, were not taken into account. Therefore, in order to verify the attractiveness of each option in more detail, it is suggested to also make a complete assessment of those mentioned aspects.

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NOMENCLATURE

Symbols

a_h	Tangential acceleration of the roll motion
c	Damping coefficient
k	Stiffness coefficient
m	Mass
n	number of nodes
q_p	Wind pressure
u	Horizontal displacement
$\dot{u}$	Horizontal velocity
$\ddot{u}_r$	Relative horizontal acceleration
$\ddot{u}$	Horizontal acceleration
$\ddot{u}_t$	Total horizontal acceleration
$\ddot{v}$	Vertical acceleration
$\dot{v}$	Vertical velocity
v	Vertical displacement
θ	Rotational displacement
$\dot{\theta}$	Rotational velocity
$\ddot{\theta}$	Rotational acceleration
w	Vertical displacement
x_n	Transverse distance from center of a pipe rack
z_n	Height of the nodes

F_x	External horizontal force
F_y	External vertical force
M	External moment
$F_{h,portal}$	Horizontal load for the portal-side frame
$F_{v,portal}$	Vertical load for the portal-side frame
F_{HRT}	Horizontal load in case of roll + heave motion
F_{HRD}	Horizontal load from acceleration of a roll motion
F_{HHR}	Horizontal load from acceleration of a heave motion with a roll motion
F_{HRG}	Horizontal load from inclination of a roll motion
F_{VRT}	Vertical load in case of roll + heave motion
F_{VRD}	Vertical load from acceleration of a roll motion
F_{VHR}	Vertical load from acceleration of a heave motion with a roll motion
F_{VRG}	Vertical load from inclination of a roll motion
F_{HPT}	Horizontal load in case of pitch + heave motion
F_{HPD}	Horizontal force from acceleration of a pitch motion
F_{HHP}	Horizontal force from acceleration of a heave motion with a pitch motion
F_{HPG}	Horizontal force from inclination of a pitch motion
F_{VPT}	Vertical load in case of pitch + heave motion
C	Damping matrix
K	Stiffness matrix
M	Mass matrix
H	Height
L	Length
W	Width

Abbreviations

FEM	Finite Element Method
LC	Load Case
MES	Modular Execution Strategy
OSL	Other Side Loadings
PLE	Pipe line Load Empty condition
PLO	Pipe line Load Operating condition
RAO	Response Amplitude Operator
SLS	Serviceability Limit State
SW	Self-Weight
ULS	Ultimate Limit State
WL	Wind Load

1. INTRODUCTION

In this chapter, it is explained that what this thesis is about, why this thesis is needed and how this thesis was done, in order of introducing the background, problem statement, objectives, research questions and overview of research approach.

1.1 Background

Building complex or large structures at remote locations like in Figure 1-1 can be challenging due to e.g. a lack of production facilities, skilled labor or risk on a shortage in material supply; these risks can influence the total project costs as well as the scheduled construction time significantly. A modular execution strategy (MES) aims at relocating parts of the production and assembly process to potentially different locations at which the conditions for production and assembly are more favorable. After production and assembly, the modules will be transported to the installation site where the final completion will be conducted.

Figure 1-1 Typical example of a pipe rack [1]

This thesis is aimed at finding the most cost effective way of executing a modular execution strategy for building pipe rack structures of a project which an engineering company Fluor B.V. is currently executing in Kuwait. A pipe rack is a steel structure which is constructed to efficiently place and support multiple levels of pipelines and also provide ground clearance for industrial plants such as refinery plants, chemical plants or power plants. Figure 1-2 shows a typical example of pipe rack.

Figure 1-2 Typical example of a pipe rack [2]

As seen in Figure 1-2, to support long pipe lines, normally, pipe racks are also long and have certain clearance from the ground to pipelines for the purpose of maintenance and an ease of passage under it. This ground clearance is typically 6 meters and the distance between vertical columns are also typically 6 meters. Conventionally, these pipe racks are procured, fabricated, erected and installed at the construction site of the country where the construction project is executed. However, as an alternative way of a pipe rack construction, it is nowadays getting more popular that to procure, to fabricate and erect pipe racks at a fabrication yard located where sufficient well trained workers at lower costs are available. These pre-assembled pipe racks will be transported to the construction site over sea. This is called Modular Execution Strategy or MES. Figure 1-3 shows an example of transporting pipe racks by a barge.

Figure 1-3 Sea transport of pipe racks [3]

This strategy has advantages over the conventional way. For example, it can reduce the effort of securing labors in the country where the project is executed which can become a major difficulty for remote locations. Furthermore, well trained workers can provide fast and good quality of performance at a lower labor costs. For these reason, the MES can be attractive for a project which is executed in remote area.

1.2 Problem statement

This MES has also a disadvantage that it can lead a result in more use of steel than the conventional way which is constructing the pipe racks directly at the project site. The reason is that during transporting the pipe racks on a vessel, in most cases, the external loads from motions of a vessel are higher than loads of in-place (on-site) situation. It means the in-place designed pipe racks will fail due to the loads from sea transport. The most critical parts of the pipe racks from the sea transport loads are the bottom columns of the pipe racks because the bottom columns get the biggest stresses due to their position and length. Therefore, most of steel difference between an in-place situation and a sea transport salutation come from the additional stiffeners for the bottom columns. The bottom columns are shown in Figure 1-4.

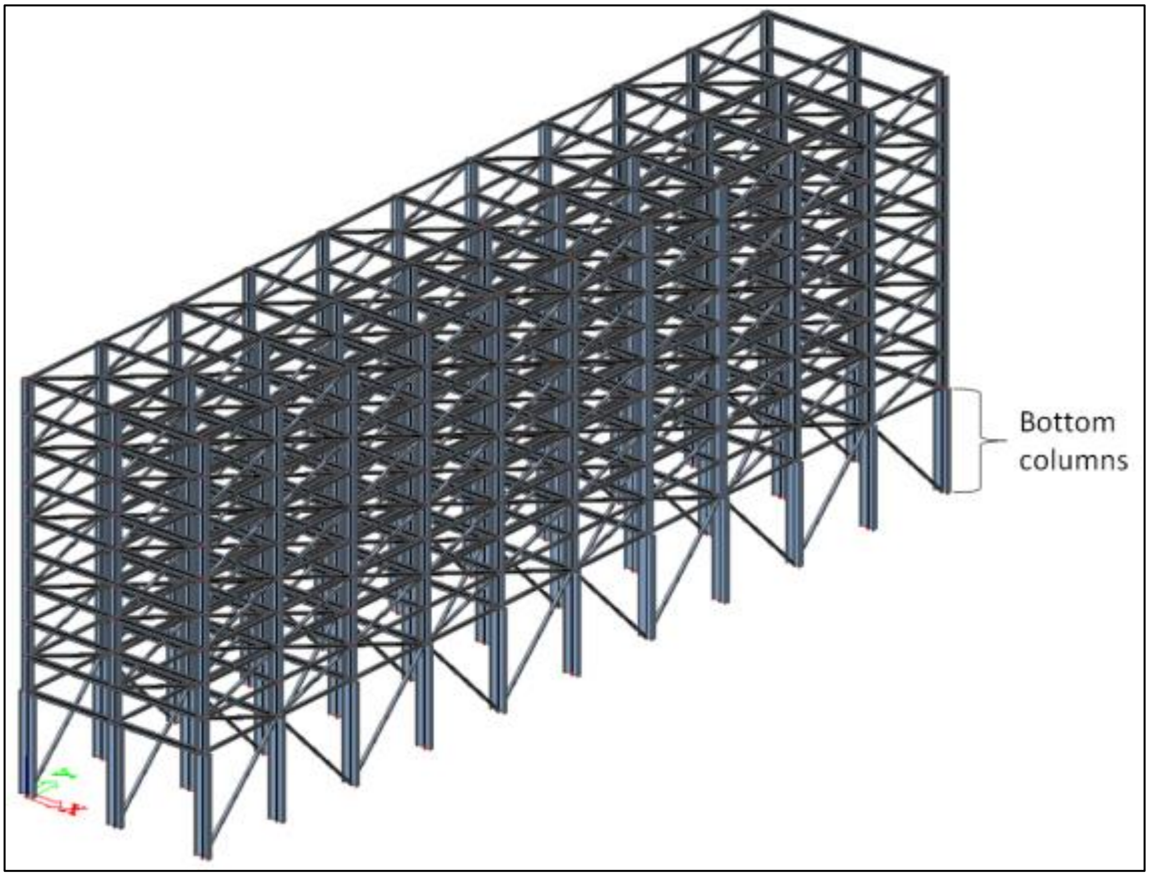

Figure 1-4 3D Model of a pipe rack

In order to reinforce the bottom columns, temporary bracings can be installed for sea transport or the columns can be replaced with bigger size of columns.

Alternatively, the height of the center of gravity of the pipe racks can be lowered by not assembling the bottom columns instead of stiffening. However, separating the bottom columns reduces the benefits of the use of the MES which is intended to reduce direct works and labors at the project site. Besides having to connect the upper parts to the columns, additional on-site work is required in erecting since the columns have to be installed before arrival of the upper parts.

The main problem here is that it is not certain that which way is the most effective way to reduce total project costs.

1.3 Objectives and research questions

The company, 'Fluor B.V.' is currently executing a project in Kuwait using a MES and wishes to find the most cost effective way to execute the project among possible options.

First option is that to transport the pipe racks without their bottom columns. Second option is that to transport the pipe racks with their bottom columns stiffened by temporary bracings. Third option is that to transport the pipe racks with replaced bigger and stronger columns.

In this thesis, knowing a difference steel quantity and cost for each option is aimed which is structural analysis required. Another study for other aspects is outside of the scope and needs to be done with a follow-up study to be able to make a final decision.

The loads used to generate designs for this comparison will be in-place loads as well as sea transport loads. Considering the sea loads, a quasi-static analysis will be used as an industry standard. However, the validity of using a quasi-static analysis should be checked because the dynamic effect can be risky to a high structure.

In this regard, the main questions of this thesis are:

1) What are the quantity of steel and cost of each option and the difference among them?

- Option 1: Transporting pipe racks without the bottom columns
- Option 2: Transporting pipe racks with their bottom columns and temporary bracings between the bottom columns
- Option 3: Transporting pipe racks with their bigger and stronger bottom columns

2) Is the quasi-static analysis for sea transport situation still valid for the design of high structure which will be affected by dynamic effect than shorter one?

1.4 Overview of research approach

In order to answer the first research question, representative configurations of the pipe racks which have to be analyzed were decided. An investigation of the project which the company is currently executing was performed and dimensions for every pipe rack were found as design database. Subsequently the minimum, middle, and maximum dimensions of width, height, and length of the pipe racks were used to create the 27 pipe racks.

With the 27 configurations of pipe racks, initial structural design for in-place loadings was done. For simpler analysis, two-dimensional finite element analysis of the structure was performed in which each the transverse and longitudinal plane. This approach is valid for the assumption of a rectangular geometry of the pipe racks and in plane loading. Therefore 9 of transverse plane structures which are called portal side frames and 9 of longitudinal plane structures which are called bracing side frames were set to be analyzed. These frames were checked in accordance with Eurocode 0: Basis of Design, Eurocode 1: Actions on structures and Eurocode 3: Design of steel structures assuming the loadings are static.

After the designs for in-place loadings were done, feasibility check of initial design subjected to sea transport loadings were performed. The initial design for each side frames were checked for sea transport loadings. The sea transport loadings are calculated from prescribed amplitude and period of a vessel by DNV-ST-N001: Marine operations and marine warranty as quasi-static loadings.

The initial design fails due to sea transport loading; so, the three MES options are applied and finalized the designs for each 18 frames. Subsequently, for each MES option, frames of equal height are combined into 27 distinct structural configurations.

Finally, comparison of steel quantity and steel work cost which includes procurement, fabrication and installation of steel were done from the design. The steel quantity was directly obtained from the design. For the steel work cost, an average of cost at the project location and at the fabrication yard was used.

In order to answer the second research question, one portal side frame which is 6m wide and 24m high was selected for comparing resonance frequencies with the prescribed period of roll motion which obtained from DNV-ST-N001.

After check of the resonance frequency, dynamic response of the structure was compared to quasi-static response of the structure. The assumed roll and heave motion of a vessel are transformed to horizontal and vertical inertial forces to the structure. The amplitude and period of motions are same as the quasi-static analysis, but the loadings vary in time for dynamic analysis and this was done with modal time-history analysis. With the results, comparison was available for the results from quasi-static and dynamic analysis approach. The results of the dynamic and quasi-static approach are used to perform a comparison of the maximum horizontal and vertical deflection of the top node, as well as comparison of the maximum stresses occurring in the bottom columns.

2. DESIGN PROCEDURE, CRITERIA AND METHOD

In order to find answers to the research questions, it is necessary to know a procedure to design pipe racks for the MES as well as structural design criteria and the analysis method for the pipe rack design.

2.1 Design procedure for MES

Prior to the main study, it is necessary to know a procedure of a pipe rack design for the MES. Overview of a pipe rack design procedure for a MES is shown in Figure 2-1.

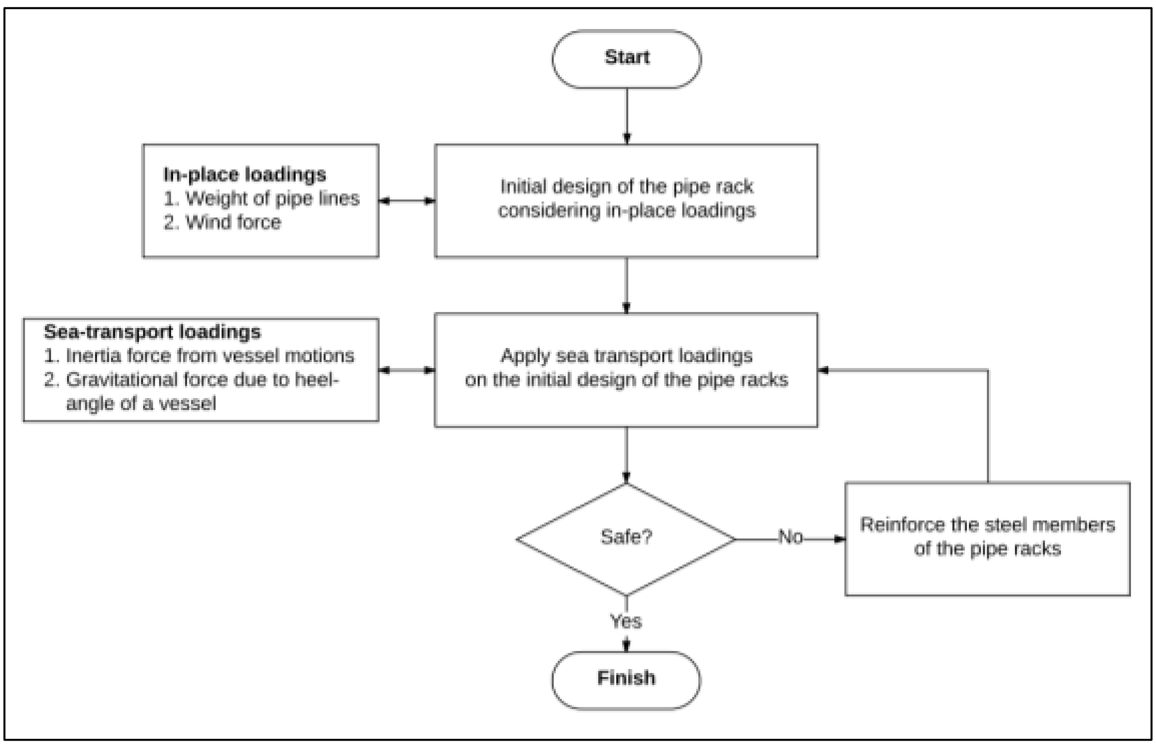

Figure 2-1 Schematic of a pipe rack design procedure for the MES

As indicated in the Figure 2-1, the first job is initially to design the pipe racks for in-place loadings. This is the initial in-place design which is done considering an in-place situation such as an arrangement of pipe lines, weight of the pipe lines and any other loadings that will be applied to the pipe racks. A configuration of a pipe rack is decided according to the arrangement of the pipe lines. Sizes of steel members are decided based on the weight of the pipe lines, wind load and any other loadings at the project installation site. After the completion of the initial in-place design, the second design step is assuming the designed pipe racks for the in-place situation are placed on a vessel. Stability of the pipe racks and integrity of the steel members have to be checked after applying the loadings from motions of the vessel. At this stage, if the in-place design fails due to the sea transport loadings, the three options mentioned in Chapter 1.3 are considered.

2.2 Structural design criteria

In order to design a structure, structural design criteria are needed. In this thesis, Eurocodes [4][5][6] are used for the design of pipe racks. However, Eurocodes do not contain criteria for a sea transport situation. Therefore, to consider the sea transport situation, DNV GL Rules and standards [7] are used.

2.2.1 Eurocodes for design of steel structures

According to Eurocode 3 [6], a steel structure has to satisfy two principal criteria. One is the ultimate limit state (ULS) and the other one is the serviceability limit state (SLS).

The ULS has to be used for several checks for the steel structure. There are two main checks for ULS checks, one for section and one for stability and each main check has several sub-checks. Table 2-1 shows the list of the checks based on Eurocode 3 [6].

Section Checks	Stability Checks
Compression check	Flexural Buckling check
Bending moment check	Torsional(-Flexural) Buckling check
Shear check	Lateral Torsional Buckling check
Torsion check	Bending and axial compression check
	Shear Buckling check

Table 2-1 List of ULS checks

A design for pipe racks has to satisfy each of the checks. Detail information for requirements of each check is explained in APPENDIX G.

Deflection check has to be done for SLS checks. There is no specific requirement of deflection limits from Eurocodes. It is up to decision of an engineer judgement. Therefore, in this thesis, allowable horizontal and vertical displacements were chosen based on a company design criteria for a project that the company is currently executing and it is shown in Table 2-2.

Description	In-place	Sea transport
Allowable horizontal displacement	H/180	H/100
Allowable vertical displacement	L/400	L/200

Table 2-2 Allowable horizontal and vertical displacement

Where H is the height of the pipe rack and L is length of the beam member. Note here that the allowable deflections for the sea transport design are higher than in-place design. It is because there will be no personnel on the pipe racks during the sea transport. Therefore, the restriction of the serviceability is less strict than the in-place design.

2.2.2 DNV GL Rules and standards for sea transport criteria

In order to know how the sea transport loading from motions of a vessel, knowing the motions of vessel is needed. There are six degrees of freedom for motions of a vessel as described in Figure 2-2.

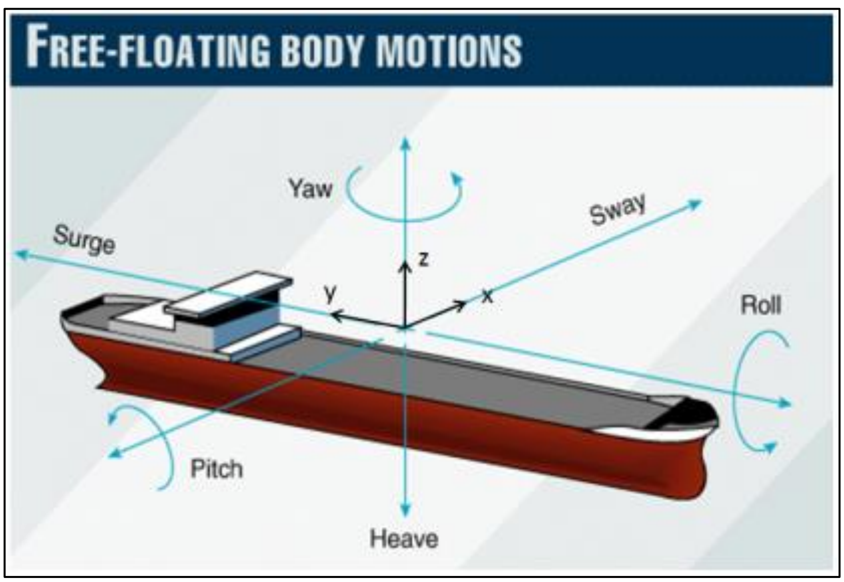

Figure 2-2 Motions of a vessel [8]

As shown in Figure 2-2, it is defined that sway is in x direction, surge is in y direction and heave is in z direction. According to DNVGL-ST-N001 [7], the DNV default motion criteria are used to find amplitude and full cycle periods of vessel motions if data of vessel motions from a naval architect is not available at the moment when a design of pipe rack starts. A detail explanation of the reason of the used of DNV default motion criteria is done in APPENDIX A. Table 2-3 shows the DNV default motion criteria.

Case	LOA (m)		B (m)	L/B	Block Coef.	Full cycle period (secs)	Single amplitude		Heave
							Roll	Pitch	
1	> 140	and	> 30	n/a	< 0.9	10	20°	10°	0.2g
2	> 76	and	> 23	n/a	any	10	20°	12.5°	0.2g
3	≤ 76	or	≤ 23	≥ 2.5	< 0.9	10	30°	15°	0.2g
4	≤ 76	or	≤ 23	≥ 2.5	≥ 0.9	10	20°	15°	0.2g
5	≤ 76	or	≤ 23	< 2.5	< 0.9	10	30°	30°	0.2g
6	≤ 76	or	≤ 23	< 2.5	≥ 0.9	10	25°	25°	0.2g

Table 2-3 DNV – Default motion criteria [7]

In this study, the case no. 2 from Table 2-3 is chosen for motions of a vessel. Therefore, 20 degrees of roll, 12.5 degrees of pitch and 5m of heave are chosen as amplitude of each motion. 10 seconds is chosen as full cycle period for all motions. There are only three motions, roll, pitch and heave in Table 2-3. It is because according to DNVGL-ST-N001 [7], the most severe combinations which decide steel member size are:

- Roll +/- Heave
- Pitch +/- Heave

In order to take the most severe case, it is assumed roll, pitch and heave are in same phase which means when acceleration of roll or pitch is the maximum, acceleration of heave is also the maximum.

2.3 Pipe rack design method

Theory behind a design of the pipe racks is basically establishing equations of motions for each degree of freedom for the pipe racks and to solve the equations. With their solutions, displacements and of the pipe racks and stresses in the steel are found.

2.3.1 Equation of motion for simplified pipe rack structure

Equation of motion for simple steel frame is shown in case that a vessel is experiencing a roll motion. Figure 2-3 describes a simple steel frame supporting pipe lines on the vessel.

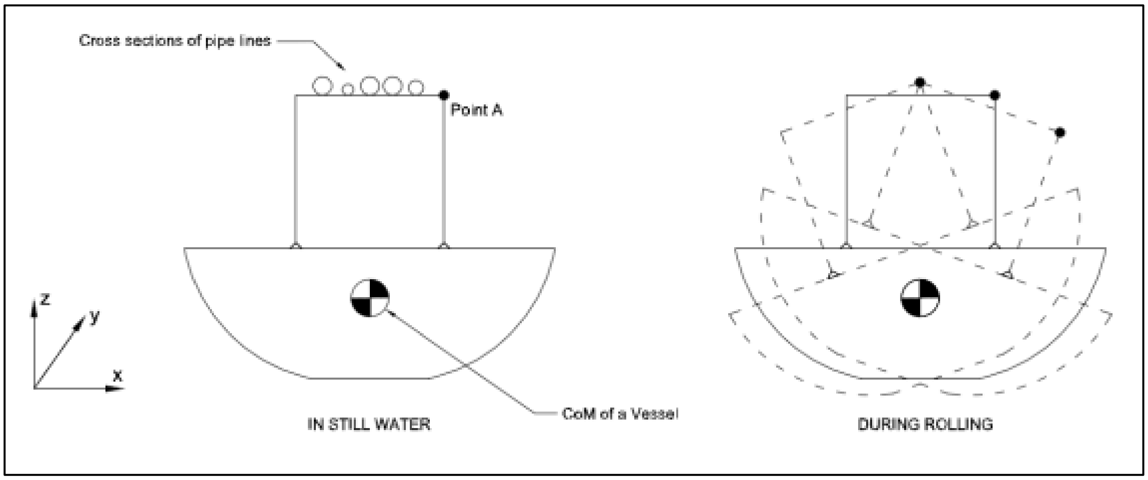

Figure 2-3 A vessel experiencing a roll motion

Point A is taken as an example. Because of the roll motion, mass of steel and pipe lines induce forces to the steel frame and the steel frame will deflect as shown in Figure 2-4 and there will be also stresses in the steel.

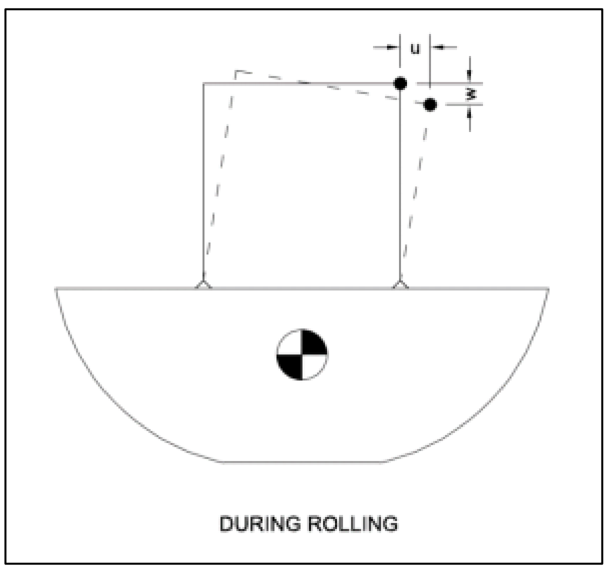

Figure 2-4 Displacements of the steel frame

The letter 'u' represents a horizontal displacement and 'w' represents a vertical displacement. Equations of the roll motion for horizontal displacement of point A is expressed as (Eq. 2-1).

$$m \cdot \ddot{u}_t(t) + c \cdot \dot{u}(t) + k \cdot u(t) = 0 \qquad \text{(Eq. 2-1)}$$

Where,

- m: Mass of point A
- $\ddot{u}_t(t)$: Total horizontal acceleration of point A
- c: Damping coefficient
- $\dot{u}(t)$: Horizontal velocity of point A
- k: Stiffness coefficient of the steel frame
- $u(t)$: Horizontal displacement of point A

The total horizontal acceleration is a summation of relative horizontal acceleration of point A and tangential acceleration of the roll motion.

$$m \cdot \ddot{u}_t(t) = \ddot{u}(t) + a_h(t) \qquad \text{(Eq. 2-2)}$$

Where,

- $\ddot{u}_r(t)$: Relative horizontal acceleration of point A
- $a_h(t)$: Tangential acceleration of the roll motion

Therefore, using (Eq. 2-2), (Eq. 2-1) is expressed as (Eq. 2-3).

$$m \cdot \ddot{u}(t) + c \cdot \dot{u}(t) + k \cdot u(t) = -m \cdot a_h(t) \qquad \text{(Eq. 2-3)}$$

This shows that the tangential acceleration with mass can be expressed as an external force to the steel frame. If the inertial term and the damping term are disregarded and the maximum value of tangential acceleration is used, (Eq. 2-3) will reduce to an equation for a quasi-static analysis as shown in (Eq. 2-4).

$$k \cdot u(t) = -m \cdot a_{h,max} \qquad \text{(Eq. 2-4)}$$

This is an approach which is the use of a quasi-static analysis how the computation is done for the first main question in Chapter 3. In Chapter 4, the inertial term and the damping term are considered to see a dynamic response of a steel frame.

2.3.2 Introduction of the used of Finite Element Method program

A steel structure has infinite number of degrees of freedom. In order to compute them, a computer program which can perform a finite element method (FEM) is needed. Brief study of the FEM is done in 0. In this thesis, two computer programs are used. Scia Engineer is used for the first main question in Chapter 3, because this program is used in practice and it uses a quasi-static analysis. SAP2000 is used for the second main question in Chapter 4, because it can perform a dynamic analysis. These programs can compute deflections and stresses and also check automatically whether the design of pipe racks satisfies the design criteria of Eurocodes or not.

3. DESCRIPTION OF MODELING AND LOADINGS

In this chapter, firstly, the configurations of pipe racks, a modeling approach and material data are presented. Secondly, load cases and combinations for an in-place situation are explained. Finally, load cases and combinations for both a sea transport situation are explained.

3.1 Pipe rack model description

In order to make the results of this thesis available for any other project in general, various configurations of the pipe racks are suggested as well as two-dimensional modeling and material data.

3.1.1 Pipe rack configurations

A configuration of a pipe rack depends on an arrangement of pipe lines and the arrangement of the pipe lines are various for projects. Therefore, the configuration of a pipe rack is also various. However, most of pipe racks have rectangular shapes and typical distance between columns is 6m and between beams is 2m for a pipe rack. Figure 3-1 shows a typical pipe rack as an example.

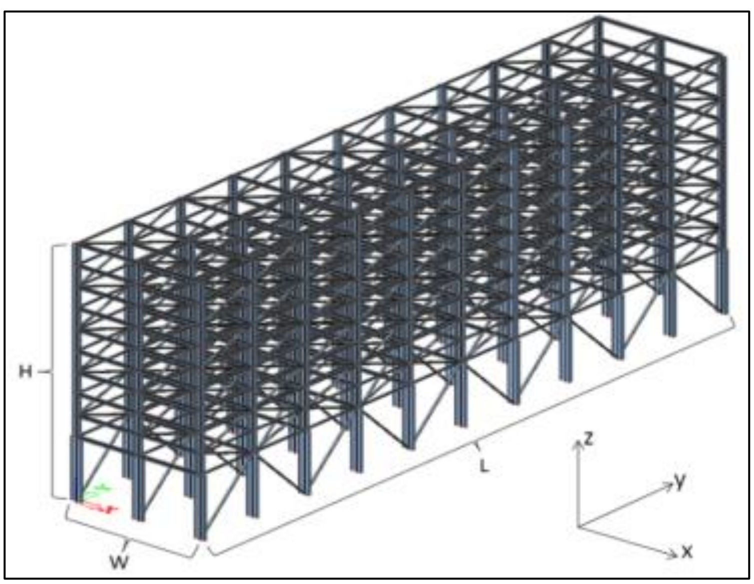

Figure 3-1 An example of a pipe rack

W, H, L is width, height and length respectively. In this thesis, it is defined that width is in x direction, length is in y direction and height is in z direction. In order to decide configurations for the thesis, firstly, the minimum and maximum dimensions of pipe rack for the MES are found based on a data base from a project which the company is currently executing. The results are shown in Table 3-1. The data base is shown in APPENDIX D.

Pipe rack	Width [m]	Height [m]	Length [m]
Minimum	5	4	20
Maximum	26	24	60

Table 3-1 Smallest and largest sizes of pipe rack

The width is between 5m to 26m, height is between 4m to 24m and the length is between 20m to 60m. Based on this data and considering the typical distance of columns and beams, the configurations of pipe rack which will be designed and study are determined. It is shown in Table 3-2.

Width [m]	Height [m]	Length [m]	Width [m]	Height [m]	Length [m]	Width [m]	Height [m]	Length [m]
6	12	24	6	12	36	6	12	60
6	18	24	6	18	36	6	18	60
6	24	24	6	24	36	6	24	60
12	12	24	12	12	36	12	12	60
12	18	24	12	18	36	12	18	60
12	24	24	12	24	36	12	24	60
24	12	24	24	12	36	24	12	60
24	18	24	24	18	36	24	18	60
24	24	24	24	24	36	24	24	60

Table 3-2 Configurations of pipe racks

For the length of the pipe racks, 24m, 36m and 60m are chosen. For the width of the pipe racks, 6, 12m and 24m are chosen. For the height of the pipe racks, 12m, 18m, 24m are chosen. Therefore, with the combinations of each dimension, 27 (=3x3x3) configurations of pipe racks are chosen to be designed. These configurations of pipe racks cover most of pipe racks which are able to be transported by a vessel.

3.1.2 2D modeling approach of a pipe rack

In practice, a design of a steel structure is done in three-dimensional space to take the shapes of the pipe lines into account. However, in this study, the pipe lines are assumed as uniformly distributed on pipe racks and the configurations of the pipe racks are assumed as symmetric. Therefore, it is easier to design in two-dimensional space for each frame of pipe racks and combine them at the end. A pipe rack has two kinds of frames. One is a 2D frame in transvers direction so-called portal side frame. The other one is a 2D frame in longitudinal direction so-called bracing side frame.

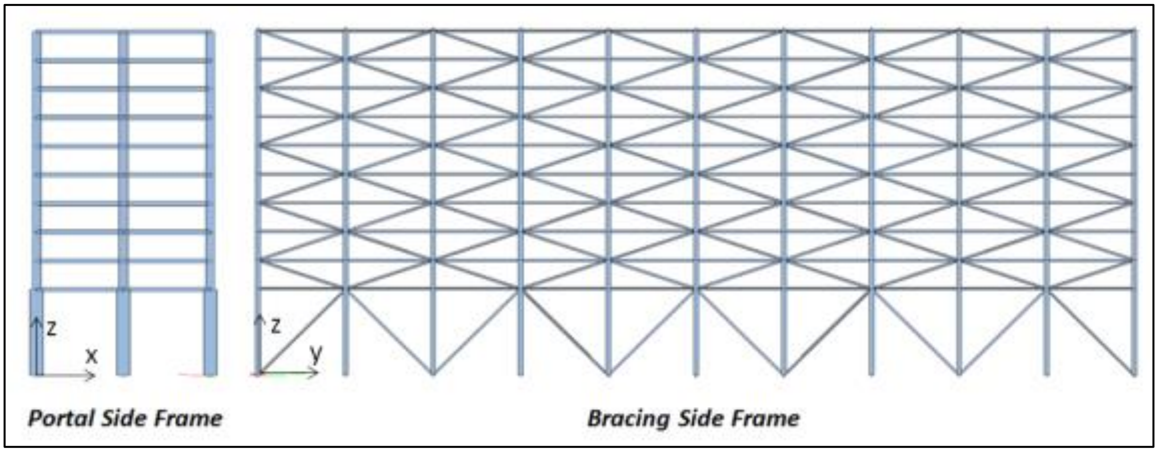

Figure 3-2 Examples of side frames

Figure 3-2 shows an example of each side frame. Portal side is in x-plane and bracing side is in y-plane by which x & y planes are defined in Figure 3-1. A portal side frame of a pipe rack allows that people and/or vehicles are able to pass under the structure for maintenance, which means there cannot be structural bracings between columns. A portal side frame has relatively higher slenderness than a bracing side frame. Therefore, beams and columns of a portal side frame are normally connected as moment connections which can resist moment efficiently. The pipelines are placed on portal frames so they are the dominant parts of a pipe rack design. Typically, I shape or H shape is used for steel members of pip racks. In this regard, it is difficult to make moment connections for both portal and bracing side. Moment connections are used for portal side frames so bracing side frames are connected as pinned connections so they need bracings to resist moment. It is assumed that connections to foundation are clamped for an in-place situation and pinned for a sea transport situation. The reason is that clamped connection to the foundation can reduce the use of steel because forces are delivered to the foundation. For the in-place situation, the foundation is usually made by concrete which is cheaper material than steel. Therefore, clamped connection to the foundation is preferable for the in-place situation. However, for the sea transport situation, connecting pipe racks to the deck of a vessel as clamed connection is difficult and takes time so pinned connection is used.

3.1.3 Material properties

There are various types for steel parameters. The design parameters used in this thesis are listed in Table 3-3.

Type description	British universal beam
Material	S 355
Unit mass of steel	7850 kg/m3
Young's modulus	210000 MPa
Ultimate strength	490 MPa
Yield strength	355 MPa

Table 3-3 Steel design parameters

Material type, unit mass, Young's modulus, ultimate strength and yield strength are determined by following British universal beam data.

There are also various types of steel profiles so it should be determined which will be used for the thesis. In Table 3-4, the types of steel are listed.

Name	Type	Material
C1	UC152/152/23	S 355
C2	UC152/152/30	S 355
C3	UC203/203/46	S 355
C4	UC254/254/73	S 355
B1	UB305/165/46	S 355
B2	UB406/178/74	S 355
B3	UB457/191/89	S 355
B6	UB610/305/149	S 355
B13	UB914/305/289	S 355

Table 3-4 Types of steel profiles

UB represents I beam and UC represents an H beam or an H column. The use of the steel members is limited to the profiles listed in Table 3-4. Suitable types of the steel profiles are chosen each of the configurations of pipe racks. The names of the types are created to distinguish the steel members easily. C represents columns and B represents beams. Here the beams referred the steel member of pipe racks which are horizontally formed and the column referred the steel member of pipe racks which are vertically formed.

3.2 In-place design condition

Design load are in accordance with Eurocodes [4][5]. For an in-place situation, loadings from self-weight, pipe lines and wind are chosen to be applied to the pipe racks.

3.2.1 Load cases for an in-place situation

- **Self-Weight [SW]**

For the design of the steel structure, an effect of the self-weight of the structure has to be taken into account. It is named as SW and the value is 78.5kN/m3 as in Table 3-5.

Name	Value	Direction
[SW]	78.5 kN/m^3	-z

Table 3-5 Steel self-weight

- **Pipe line load empty condition [PLE] and pipe line load operating condition [PLO]**

There are two conditions for load of pipe lines, empty and operating condition. They are named as PLE and PLO respectively. The values are listed in Table 3-6.

Name	Value	Direction
[PLE]	1.5 kN/m^2	-z
[PLO]	2.5 kN/m^2	-z

Table 3-6 Pipe loadings

The values for the pipe line loadings are empirical numbers of the industry. As mentions in 3.1.2, load of pipe lines is assumed as uniformly distributed as shown in Figure 3-3.

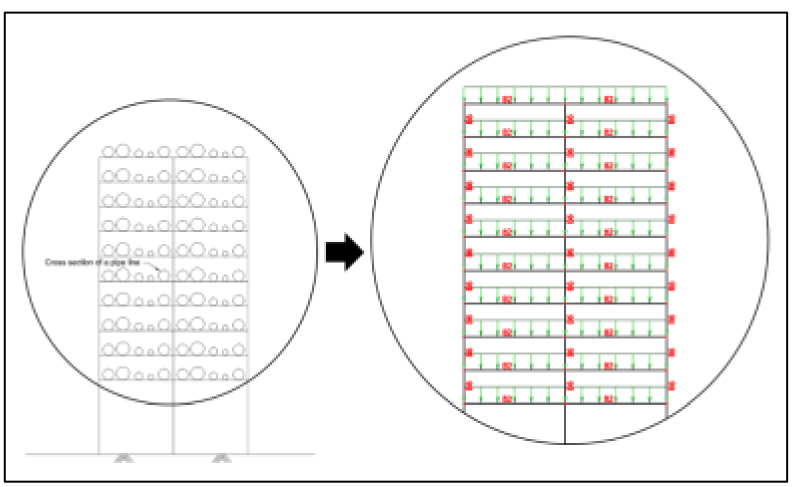

Figure 3-3 Loadings of pipe lines

- **Other Side Load [OSL]**

Due to the use of 2D modeling approach, when one side of frame is designed, weight of the other side has to be taken in to account. This weight is named as OSL. For example, weight of bracing side frame has to be applied as loadings to the portal side frame design. This is not the case for 3D modeling because in 3D modeling, the portal and bracing side frames are designed together at the same time not separated as 2D modeling. The values are listed in Table 3-7.

Description	Value	Direction
[OSL] for portal side frames	4.5 kN	-z
[OSL] for bracing side frames	4.5 kN	-z

Table 3-7 Other side loadings

These other side loadings are applied at the joints where beams and columns meet. The load to portal side frames is 4.5kN which is weight of 6m steel member C1 and C2. The load to bracing side frames is also 4.5kN which is weight of 6m steel member B6.

- **Wind Load [WL]**

Wind loads are also determined in accordance with Eurocode [5]. Wind loads are considered as two separate load cases for two orthogonal directions. The case that the wind blows in diagonal direction is neglected because in that case the wind load is smaller than the case wind blowing in orthogonal directions. In this thesis, 35 m/s of the basic 10-minute mean wind velocity is chosen to be converted to wind loads. The terrain category is assumed as 'II'. These basic assumptions for wind data will differ from location to location thus wind load of other projects will be different. However, it is not expected the wind load affects the final results of this thesis since the sea transport loadings are expected bigger than the wind load; the sea transport load will be the dominant load to determine structural design in the end. Only final wind loads are shown in the chapter and detailed methodology for the detail calculation of wind loads are shown in the APPENDIX D.

For the bottom columns, the height and the distance between the columns is 6m which means it is open so wind loads are applied to every column. Table 3-8 shows loads on the bottom columns.

Height	$q_p(z)$	Portal	Bracing
m	kN/m2	kN/m	kN/m
7	1.63	0.90	1.00

Table 3-8 Wind loads on the bottom columns

The height of the bottom columns is assumed 7m considering the height of the foundation. Wind pressure $q_p(z)$ is 1.63kN/m2. Therefore, wind load to the bottom columns for portal side frames is 0.9kN/m and for bracing side frames is 1.0 kN/m. Wind loads to the bottom columns are assumed as distributed loads.

It is assumed that the upper part of the pipe rack is closed structure because it is dense due to pipe lines, so wind load is applied only one side of the pipe rack. Table 3-9 shows wind loads on the upper part of the pipe racks.

Height	$q_p(z)$	Force
m	kN/m2	kN
9	1.75	24.0
11	1.85	25.0
13	1.93	26.0
15	2.00	27.0
17	2.07	28.0
19	2.12	29.0
21	2.18	29.0
23	2.23	30.0
25	2.27	31.0

Table 3-9 Wind loads on the body

Every 2m above the top of the bottom columns, the wind loads are applied at the joint nodes as concentrated point loads. Figure 3-4 shows how the wind loads are applied. The heights are including 1 m height of the foundation.

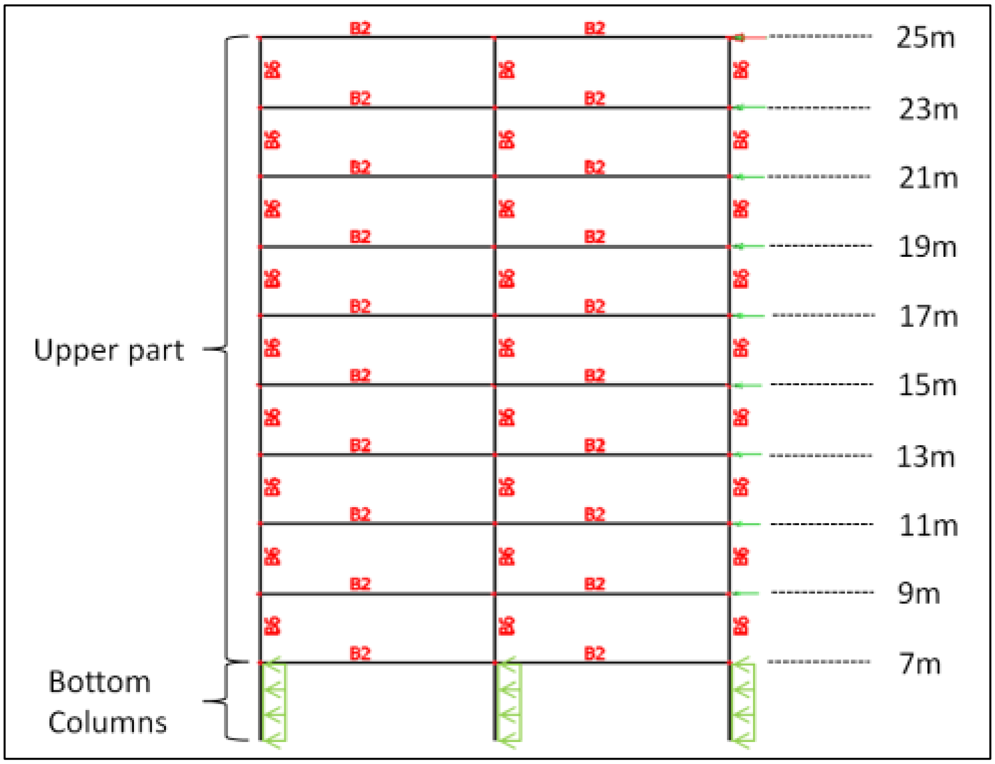

Figure 3-4 Wind load application

3.2.2 Load combinations for an in-place situation

In order to take all the loadings for a certain situation that the pipe racks experience into account, the combinations of the load cases are necessary. The load combinations should be categorized into two, one for ULS and one for SLS and they are listed in Table 3-10.

Limit state	Combination no.	Load Combination
ULS	LC 101	1.55[SW]+1.35[OSL]+1.35[PLO]+1.5[WL+T]
	LC 102	1.55[SW]+1.35[OSL]+1.35[PLO]+1.5[WL-T]
SLS	LC 201	1.15[SW]+1.0[OSL]+1.0[PLO]+1.0[WL+T]
	LC 202	1.15[SW]+1.0[OSL]+1.0[PLO]+1.0[WL-T]

Table 3-10 Load combination for in-place

For ULS, Load Combination 101 (LC 101) and Load Combination 102 (LC 102) are defined. The difference between these two is only direction of the wind load (WL). [WL+T] means the wind load applied to positive direction, +x direction for portal side frames and +y direction for bracing side frames. [WL-T] is the other way around. Load combinations for SLS are defined in same manner with ULS. The difference between ULS and SLS combinations is partial safety factors. These factors are chosen according to Eurocode 0 [4].

3.3 Sea transport design condition

Design load are in accordance with DNVGL-ST-N001 [7]. For a sea transport situation, loadings from roll + heave and pitch + heave is chosen to be applied to the pipe racks. For the first research question, a quasi-static analysis is used so the load cases for a sea transport situation have to be quasi-static loadings.

For the sea transport design, effects of wind are ignored. The effect that wind load contributes to motions of a vessel and the effect that wind load induce stress in the steel directly are assumed negligible.

3.3.1 Load cases for a sea transport situation

As mentioned in 2.2.2, the maximum loads from motions of a vessel are a summation of loads from roll and heave or pith and heave. Gravitational force has to be also added as load to the pipe racks. Therefore, final load combinations are:

- Roll inertial force +/- Heave inertial force +/- Gravitational force
- Pitch inertial force +/- Heave inertial force +/- Gravitational force

The load cases are defined as horizontal loads and vertical loads in case of the roll + heave motion and the pitch + heave motion. The load cases for a sea transport situation are defined in four cases in the thesis, horizontal and vertical loads of a roll motion and a pitch motion.

- **The horizontal loads in case of the + heave motion [HL ± R]**

The horizontal load (F_{HRT}) in case of roll + heave motion is a summation of three forces as (Eq. 3-1).

$$F_{HRT} = F_{HRD} + F_{HHR} + F_{HRG}\ [N] \qquad \text{(Eq. 3-1)}$$

Where,

- F_{HRD}: Horizontal load from acceleration of a roll motion
- F_{HHR}: Horizontal load from acceleration of a heave motion with a roll motion
- F_{HRG}: Horizontal load from inclination of a roll motion

- **The vertical loads in case of roll + heave motion [VL ± R]**

  The vertical load (F_{VRT}) in case of roll + heave motion is a summation of three forces as (Eq. 3-2).

$$F_{VRT} = F_{VRD} + F_{VHR} + F_{VRG} \; [N] \qquad \text{(Eq. 3-2)}$$

  Where,

  - F_{VRD}: Vertical load from acceleration of a roll motion
  - F_{VHR}: Vertical load from acceleration of a heave motion with a roll motion
  - F_{VRG}: Vertical load from inclination of a roll motion

- **The horizontal loads in case of pitch + heave motion [HL ± P]**

  The horizontal load (F_{HPT}) in case of pitch + heave motion is a summation of three forces as (Eq. 3-3).

$$F_{HPT} = F_{HPD} + F_{HHP} + F_{HPG} \; [N] \qquad \text{(Eq. 3-3)}$$

  Where,

  - F_{HPD}: Horizontal force from acceleration of a pitch motion
  - F_{HHP}: Horizontal force from acceleration of a heave motion with a pitch motion
  - F_{HPG}: Horizontal force from inclination of a pitch motion

- **The vertical loads in case of the pitch + heave motion [VL ± P]**

  The vertical load (F_{VPT}) in case of pitch + heave motion is a summation of three forces as (Eq. 3-4).

$$F_{VPT} = F_{VPD} + F_{VHP} + F_{VPG} \; [N] \qquad \text{(Eq. 3-4)}$$

  Where,

  - F_{VPD}: Vertical force from acceleration of a pitch motion
  - F_{VHP}: Vertical force from acceleration of a heave motion with a pitch motion
  - F_{VPG}: Vertical force from inclination of a pitch motion

Detail explanation how to calculate each force term can be seen in APPENDIX E.

- **Wind Load**

  Wind load for the sea transport situation is not considered. Interaction between wind, wave, and the structure is complex so it is not possible to find the effect in this thesis. Therefore, it is assumed that the wind force is resulted in motions of a vessel which means the direct wind effect on the pipe racks is ignored.

3.3.2 Load combinations for a sea transport situation

Limit state	Combination no.	Load Combination
ULS	LC 301	1.35[HL+R]+1.35[VL+R]
	LC 302	1.35[HL-R]+1.35[VL-R]
	LC 303	1.35[HL+P]+1.35[VL+P]
	LC 304	1.35[HL-P]+1.35[VL-P]
SLS	LC 401	1.0[HL+R]+1.0[VL+R]
	LC 402	1.0[HL-R]+1.0[VL-R]
	LC 403	1.0[HL+P]+1.0[VL+P]
	LC 404	1.0[HL-P]+1.0[VL-P]

Table 3-11 Load combination for sea transport

Partial safety factors and combination factors for sea transportation were chosen according to DNVGL-ST-001 [7] which are not different with the factors from Eurocode 0: Basis of Design.

3.3.3 Application of the sea transport loads

There are some assumptions as boundary conditions.

- Bottom elevation of the module is assumed 1.5m above the deck of the vessel and the deck elevation is assumed 4.3m above center of motion of the vessel.
- Due to the long length of the pipe racks are placed on a vessel in the longitudinal direction, which means the portal side frames experience a roll motion and the bracing side frames experience a pitch motion.
- Transverse distance from centerline of the vessel is assumed as 12m while the longitudinal distance from centerline of the vessel is assumed 30m.
- Connection types for sea transport, both clamped and pinned are considered while it is clamped type for in-place situation.

Figure 3-5 shows that weights which are summation of pipe line and steel member. A portal side frame with 6m width & 18m height and a bracing side frame with 24m length & 18m height are chosen to be examples for load application.

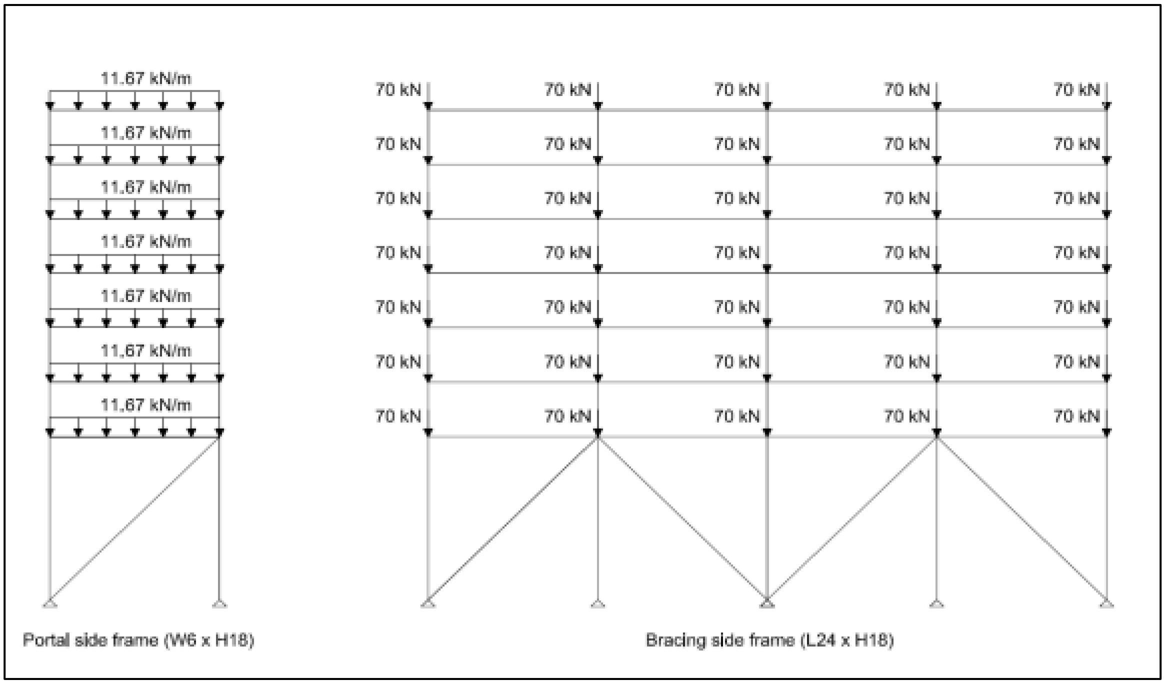

Figure 3-5 Weights in still water

11.67kN/m is weight of pipe line and steel member on the portal side frame. It is assumed that the pipe line weight is 9kN/m (=1.5kN/m2 x 6m) and the steel member weight is 2.67kN/m. These weights can be applied as loads to the structure. 70kN (=11.67kN/m x 6m) is applied to the bracing side frame as a point load. These weights are used to calculate the sea transport loads. Figure 3-6 shows sea transport loads that applied to the portal side and bracing side frames for option 2.

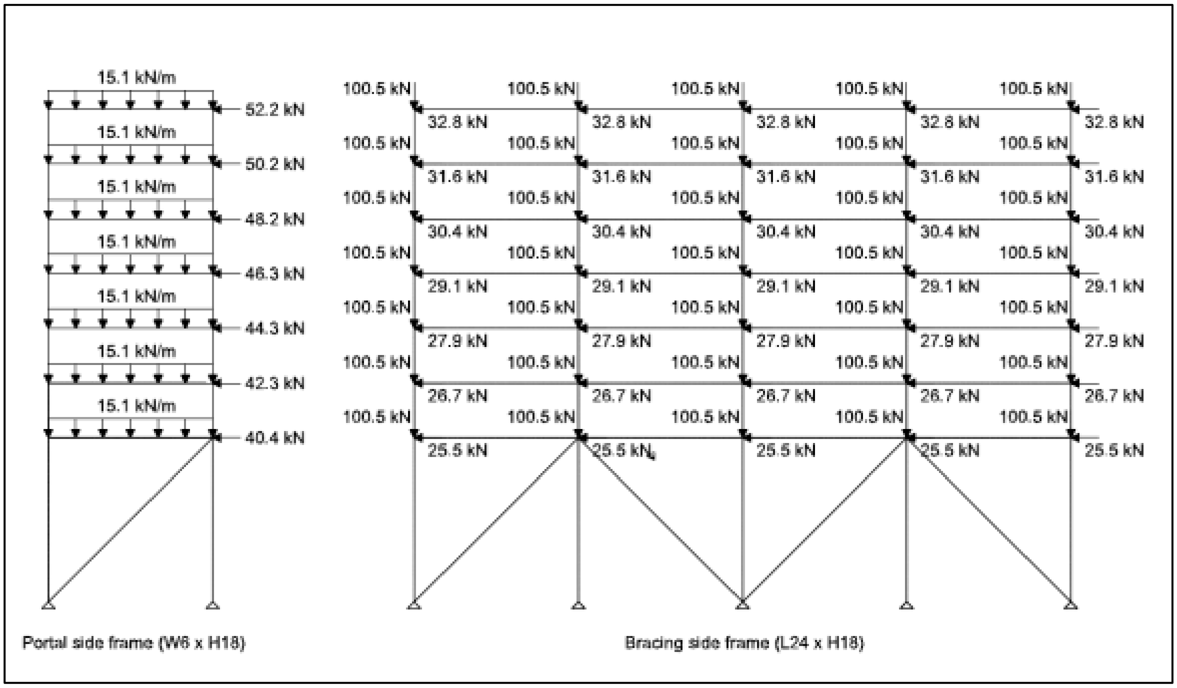

Figure 3-6 Sea transport loads of option 2

For the portal side frame, the vertical load is 15.1kN/m. This is almost same as the load of the in-place design. However, the horizontal loads are much bigger than the wind load of in-place design. It is same for the bracing side frame. Therefore, it can be known that the sea transport loadings are the dominant loads for the final decision of the steel member size. The detail sea transport loads of each option and the detail calculations are shown in APPENDIX E.

4. STEEL QUANTITY AND COST COMPARISON

In this chapter, the steel quantity and cost of steel work are found. Firstly, an example of design of a steel structure is explained and then the steel quantity and cost for each pipe rack are found for each option.

4.1 Structural design checks with quasi-static analysis method

In order to know steel quantity, the sizes of the steel members are decided in accordance with the structural design criteria.

4.1.1 Dimensions of each frame

Analysis is performed for 9 portal-side frames and 9 bracing-side frames. The configurations of each frame are listed in Table 4-1.

No.	Portal side frame	Bracing side frame
1	W6 x H12	L24 x H12
2	W6 x H18	L24 x H18
3	W6 x H24	L24 x H24
4	W12 x H12	L36 x H12
5	W12 x H18	L36 x H18
6	W12 x H24	L36 x H24
7	W24 x H12	L60 x H12
8	W24 x H18	L60 x H18
9	W24 x H24	L60 x H24

Table 4-1 Configurations of frames

When the analysis is done, steel quantity for each frame can be known. Total steel quantity for a complete pipe rack will be found by combining portal side frames and bracing side frames. For example, a pipe rack which has configuration of W6 x H18 x L24, the total steel quantity is a summation of 5 of W6 x H18 portal side frame and 2 of L24 x H18 bracing side frames.

In order to explain how the analysis is done, a portal side frame of W6 x H18 with pinned supports is chosen as an example. Analysis results for other frames can be seen in APPENDIX E.

4.1.2 Comparison of shear force, axial force and moment

Prior to check ULS and SLS, shear force, axial force and moment are checked. Figure 4-1 shows a moment comparison between the in-place design and the sea transport design. The sizes of the steel members are same for both.

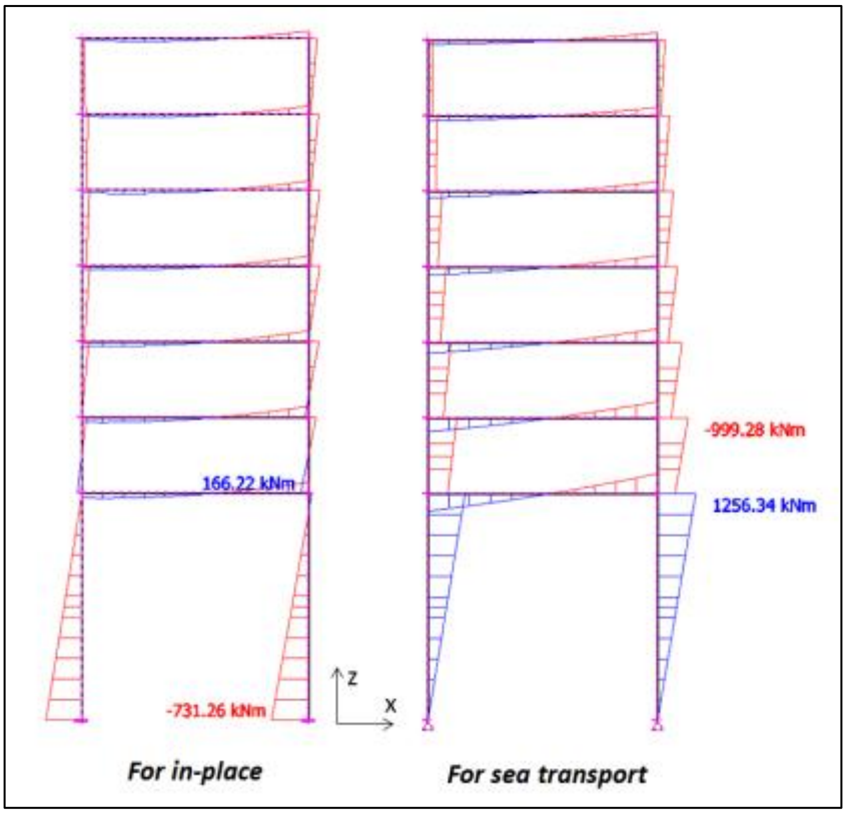

Figure 4-1 Comparison of moment

The maximum absolute moment value for the in-place design is much smaller than the maximum absolute moment value for the sea transport design. The maximum moment appears at the bottom of the columns for the in-place design whereas for the sea transport design it appears at the top of the bottom columns. Axial forces and shear forces as well as moments are listed in Table 4-2.

Description	Moment	Axial force	Shear force
In-place design	731.26kNm	801.48kN	149.47kN
Sea transport design	1256.34kNm	1253.30kN	265.53kN

Table 4-2 Absolute value of moment, axial force and shear force for W6 x H18 x L24

4.1.3 ULS and SLS check

Based on the values in Table 4-2, ULS and SLS are checked in accordance with Eurocode 3[6]. Figure 4-2 shows the result of ULS checks.

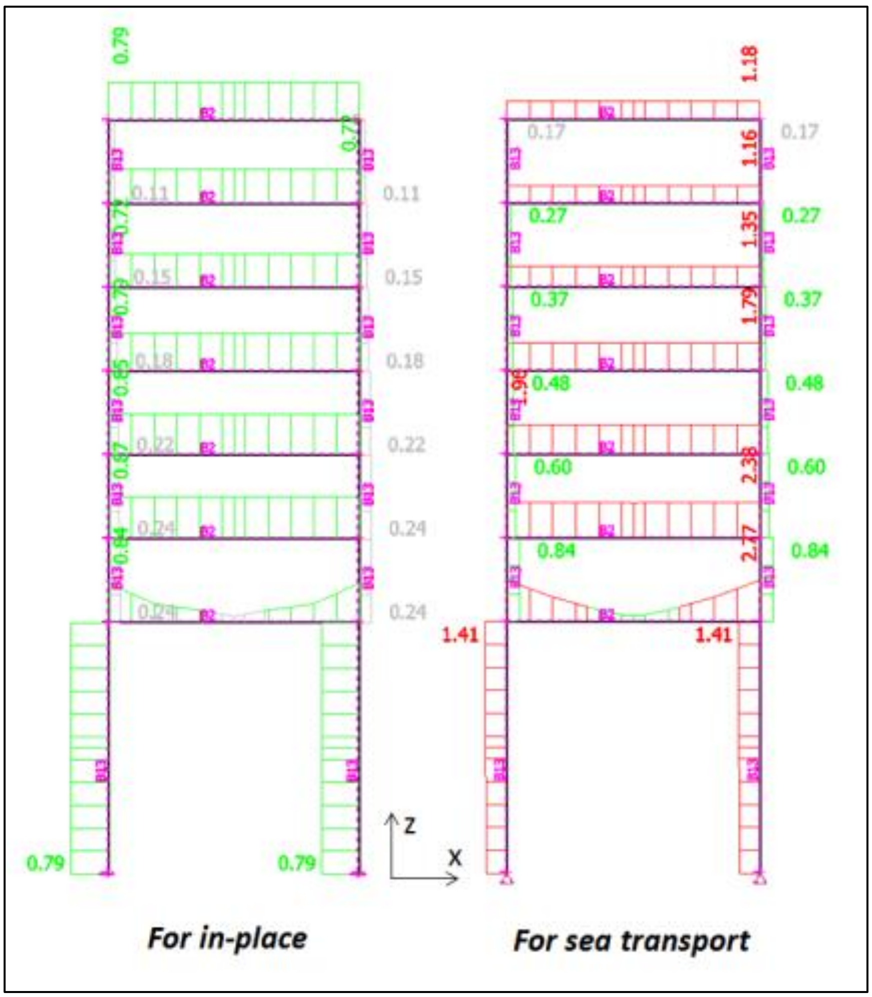

Figure 4-2 ULS check

ULS checks which are listed in Table 2-1 are performed. The numbers in Figure 4-2 represent the maximum number among the ULS checks. In order to satisfy the checks, these numbers have to be less than 1.0. It is obvious that this structure cannot resist the sea transport loads because most of numbers for ULS checks are exceeding 1.0. SLS check has to be done as well. Figure 4-3 shows the result of SLS check for lateral and vertical displacements of each joint.

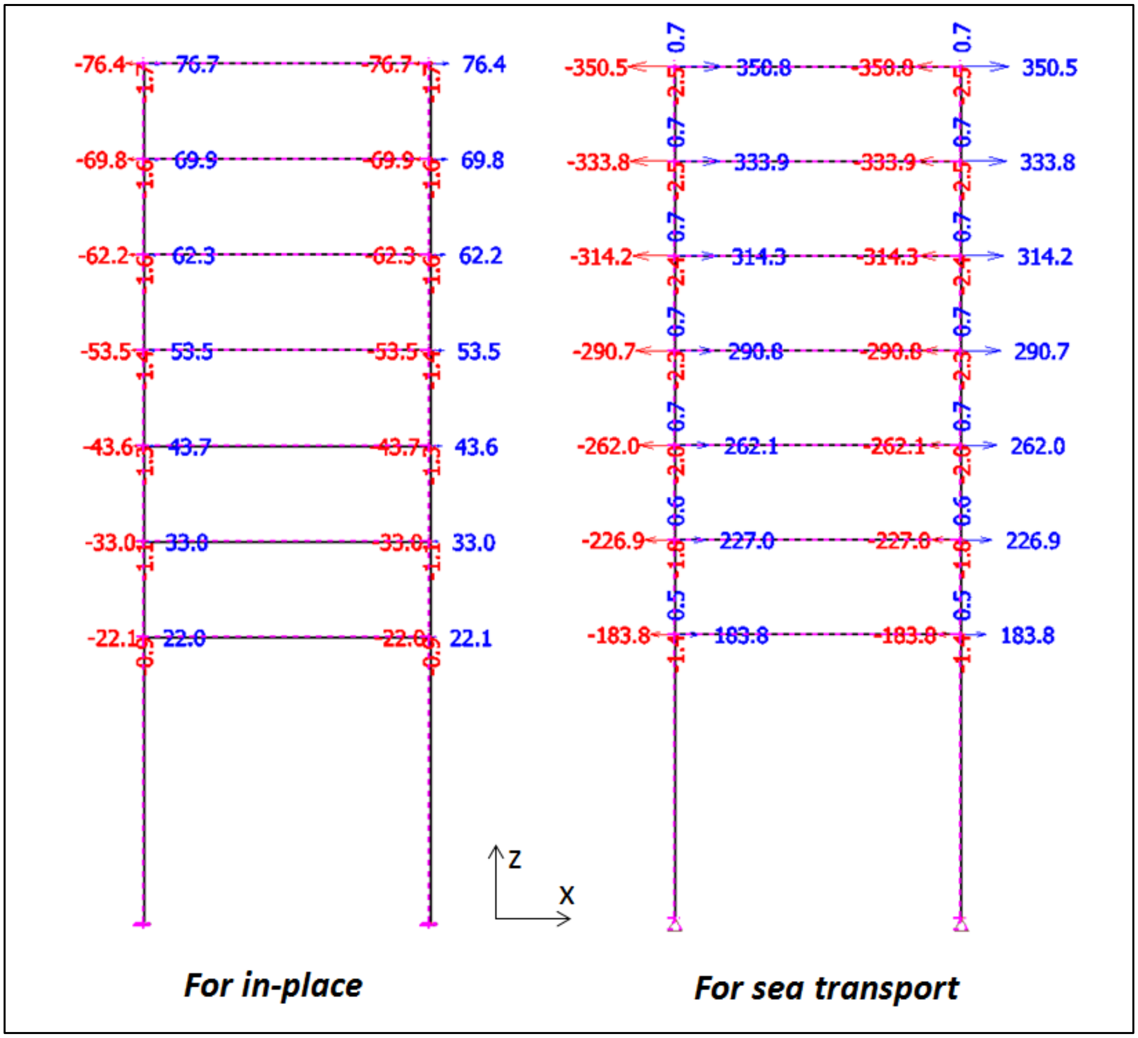

Figure 4-3 SLS check (units are in mm)

The allowable lateral displacements are H/180 for the in-place design and H/100 for the sea transport design according to Table 2-2. Therefore, for the height of 18m, it is 100mm and 180mm respectively. However, the maximum displacement is 350.5mm for sea transport situation from Figure 4-3; thus, as same as ULS checks, this structure does not satisfy the criteria for SLS checks.

In order to make the design of the structure satisfies both criteria for ULS and SLS, the options suggested in Chapter 1.3 are used. Figure 4-4 shows the stress difference of the sea transport design depending on the presence of the bracings between the bottom columns.

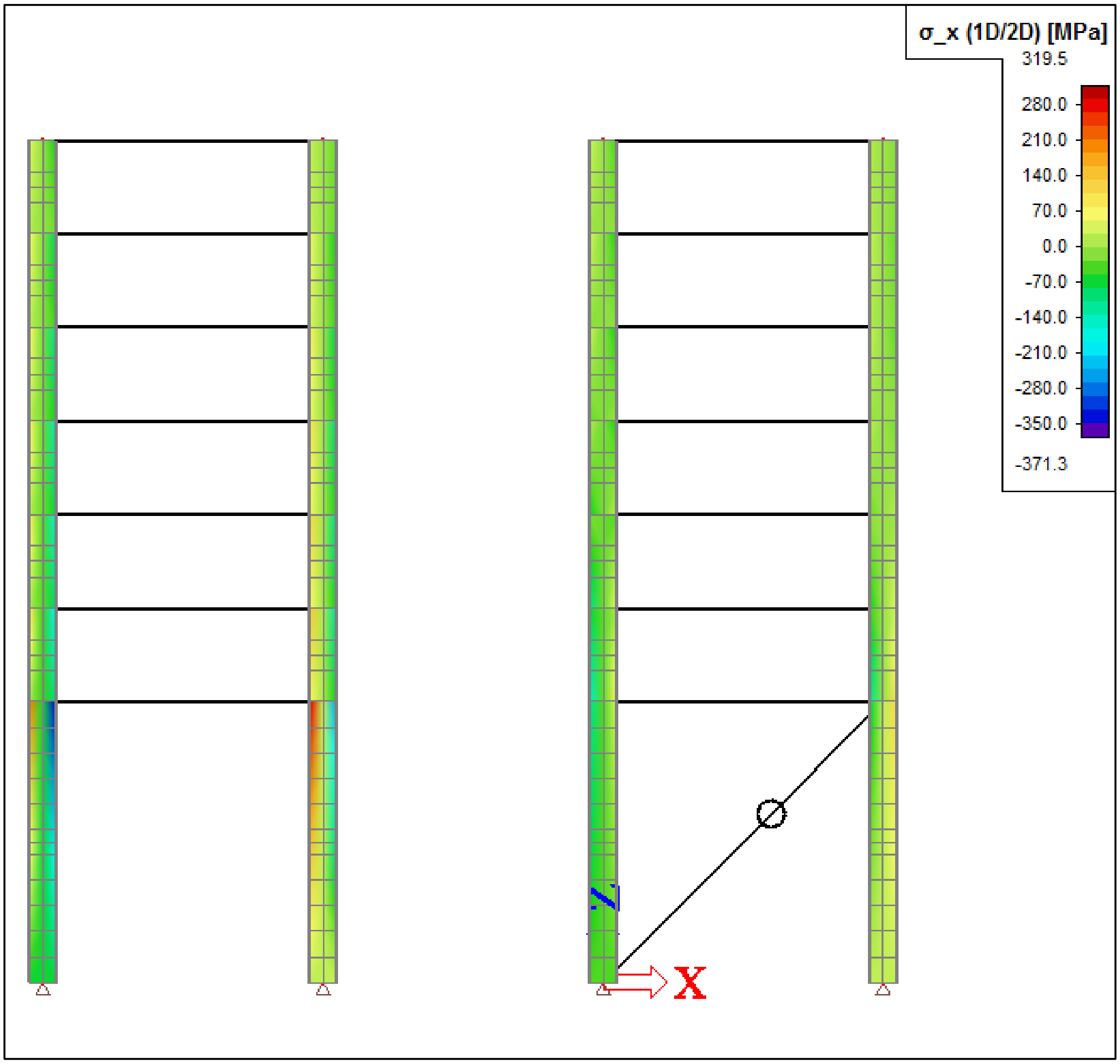

Figure 4-4 Comparison of stress in the steel (MPa)

As expected, the top of the bottom columns gets the most stress and it can be seen that the use of bracing helps to reduce the stress. The overall steel check, ULS checks have to be performed. The results of ULS checks and SLS checks for each option are shown in Figure 4-5 and Figure 4-6 respectively.

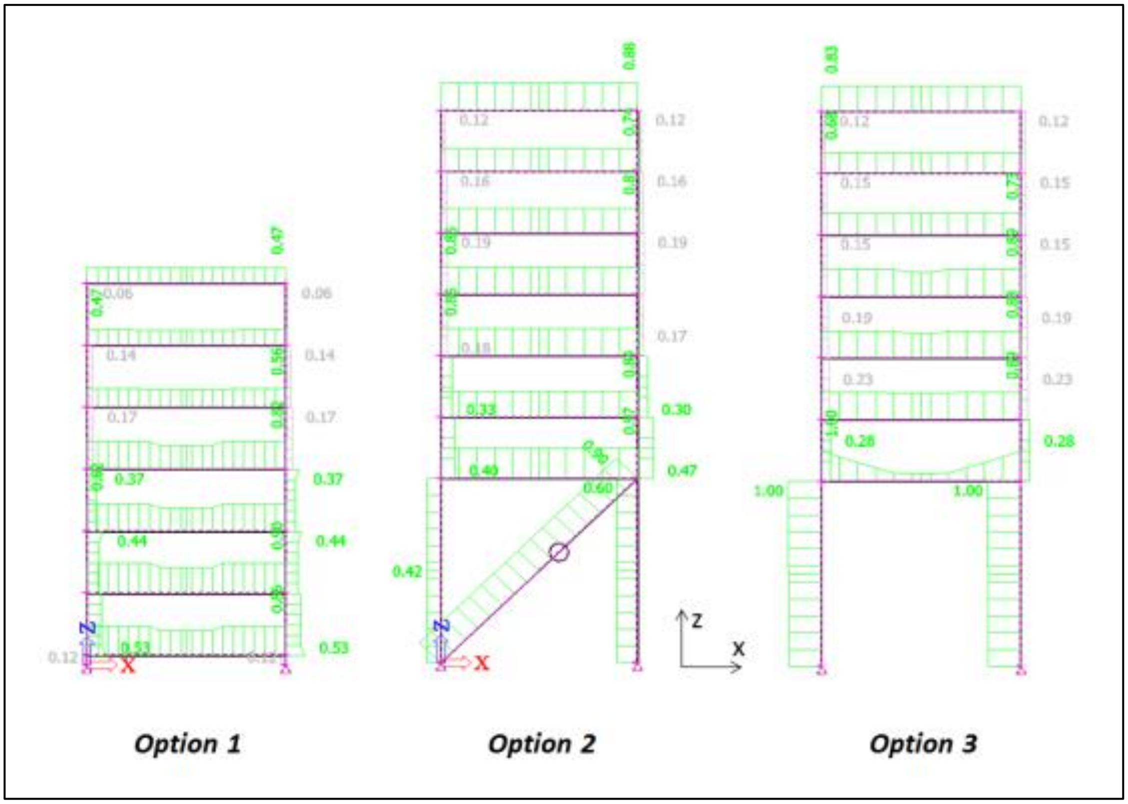

Figure 4-5 ULS checks for options

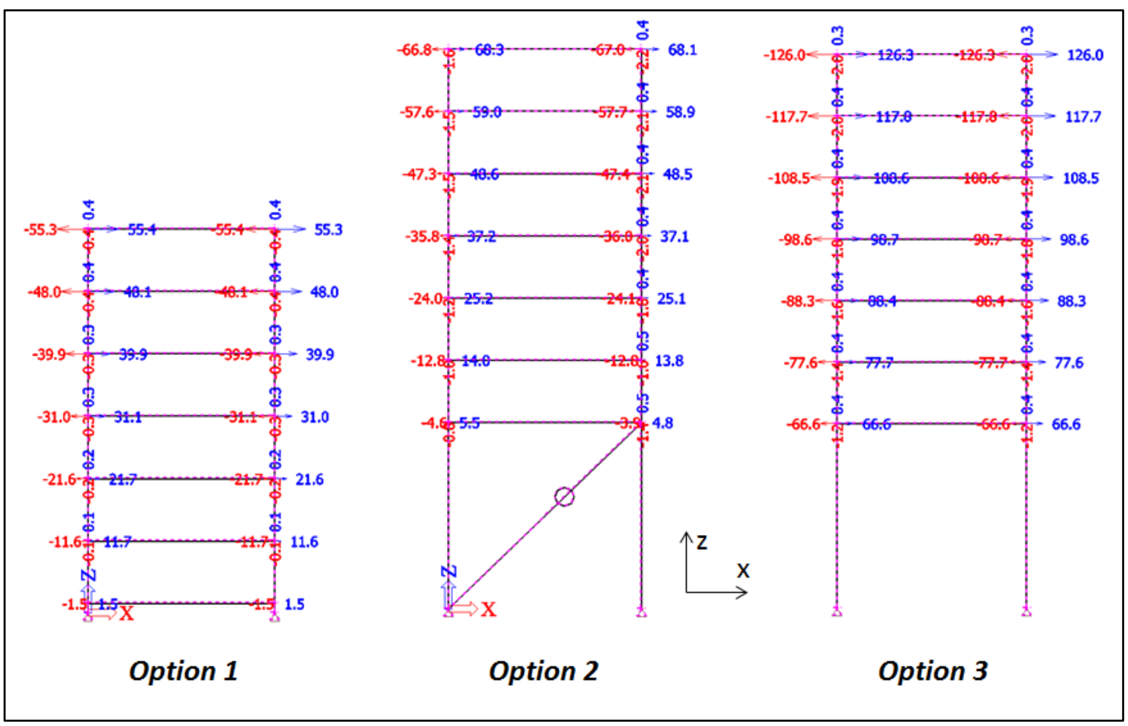

Figure 4-6 SLS checks for options (mm)

For options 1, the ULS checks are satisfied by lowering the height of the structure. For option 2 & 3, the ULS checks are satisfied by the bracings of options 2 and the larger bottom columns of options 3. It should be noted that the sizes of the lower parts of the beams are also increased for all of the options. A design for the bracing side frame is done in the same manner with the portal side frame.

It should be noted that due to the use of the 2D modeling, the steel check of the beams on portal side during a pitch motion and the beams on bracing side during a roll motion are omitted. In order to check whether this omission is permissible, the most critical case is checked. The smallest size beam, B1 is chosen to be checked with the biggest horizontal load 32.8kN. Therefore, 5.47kN/m (32.8kN / 6m) is applied to the weak axis of this beam and the ULS and SLS check was performed. The results are shown in Figure 4-7.

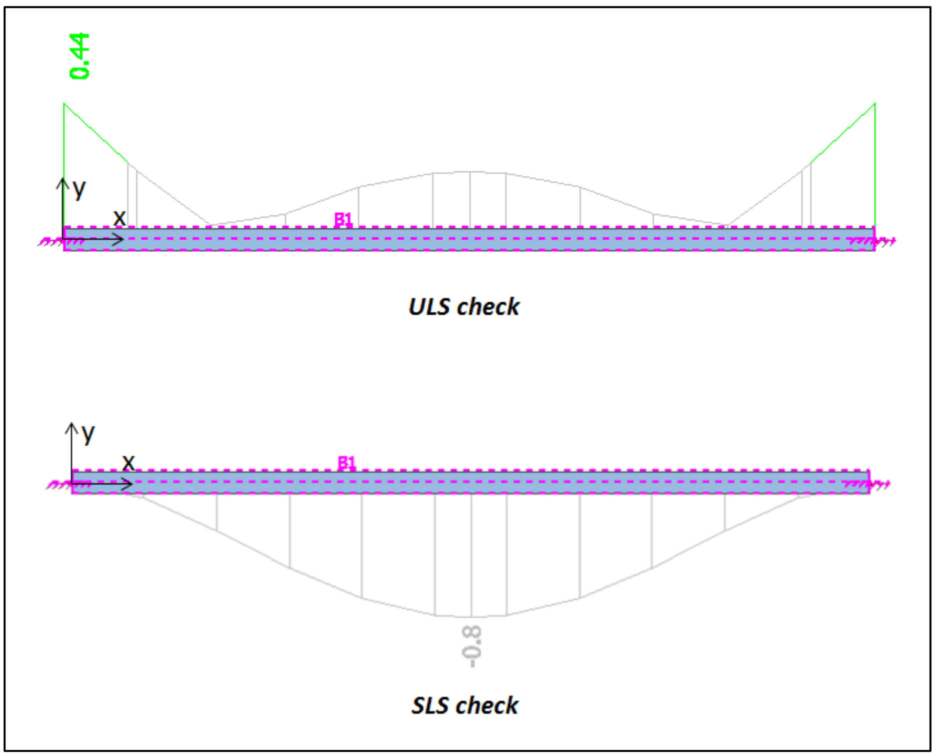

Figure 4-7 ULS and SLS (mm) check for weak axis beam

For ULS check the ratio is 0.44 and for SLS check the displacement is 0.8mm therefore the most critical case satisfies both ULS and SLS checks which means other cases will also satisfy the checks.

4.2 Results of the steel checks

As an example of steel quantity and cost comparison, a pipe rack of W6 x H18 x L24 with pinned supports is chosen to be presented. In the end, steel quantity and steel work cost of the 27 pipe racks for each option will be presented.

4.2.1 Steel quantity comparison

Table 4-3 shows the steel quantities of each option for the portal side and bracing side frame.

Description	In-place	Option 1	Option 2	Option 3
Portal-side frame W6 x H18	7.3 Ton	7.5 Ton	8.0 Ton	8.5 Ton
Bracing-side frame L24 x H18	8.9 Ton	8.9 Ton	11.6 Ton	11.6 Ton

Table 4-3 Steel quantities of options

In order to know the total steel quantity for the complete pipe rack (W6 x H18 x L24), the frames were combined with 5 portal-side frames and 2 bracing-side frames. The results are shown in Table 4-4.

Description	In-place	Option 1	Option 2	Option 3
Pipe rack W6 x H18 x L24	54.3 Ton	55.3 Ton	63.2 Ton	65.7 Ton

Table 4-4 Total steel quantities of options

The result is shown in Figure 4-8.

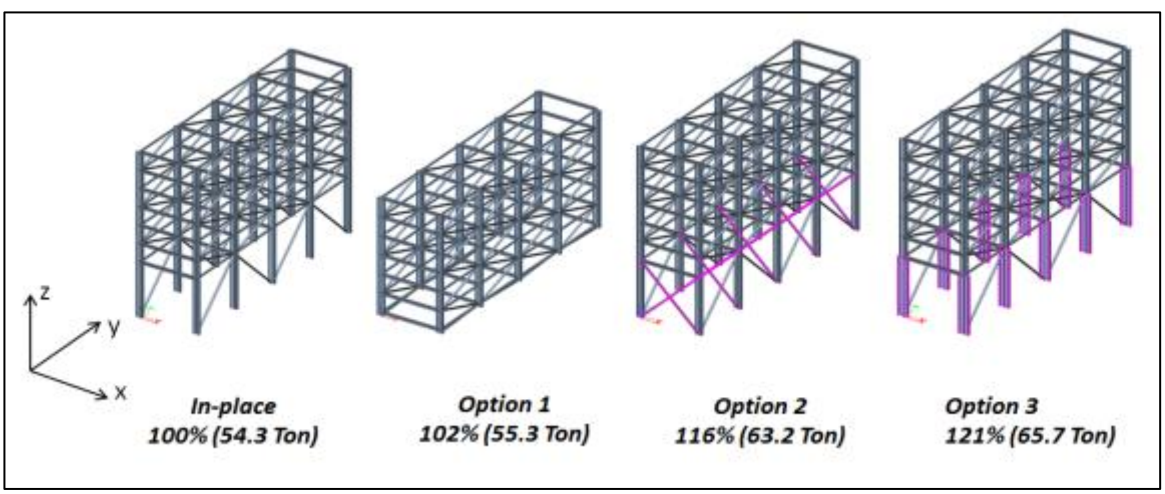

Figure 4-8 Comparison of total steel quantities of options

If the steel quantity of the in-place design is set as 100%, it is 102% for options 1, 116% for options 2 and 121% for option 3 respect to the in-place design. Steel quantities for other configurations are found in the same manner and they are summarized in APPENDIX G.

4.2.2 Steal work cost comparison

In order to see the differences of steel quantity between options, it is necessary to know what the difference of process for options are. Depending on the option, the process of the MES will be changed. Table 4-5 shows the process of each option.

Process	**For option 1**	**For option 2 & 3**
Procurement	• Procure steels for columns at local • Procure steels for a module at fab. yard	• Procure all steels at fab. yard
Fabrication	• Fabricate steels for columns at local • Fabricate steels for the module at fab. yard	• Fabricate all steels at fab. yard
Assembly	• Assemble steels for a module at fab. yard	• Assemble all steels as a complete module at fab. yard
Transport	• Deliver the module to the project site by a vessel	• Deliver the module to the project site by a vessel
Installation	• Install the columns first • Place & install the module on the columns afterwards	• Install the module directly on concrete foundations

Table 4-5 Process of the MES

The main difference is that for options 1, the bottom columns are made in the country where the pipe racks will be installed while for options 2 & 3, the bottom columns are made in the country where the pipe rack for the MES are made. The steel costs are different in different countries. In this thesis, it is assumed that pipe racks are made in China and delivered to Kuwait.

Table 4-6 shows the steel costs for both countries.

Description	MES (China) [USD/Ton]		Stick built (Kuwait) [USD/Ton]	
	Procurement + Fabrication	Assembly	Procurement + Fabrication	Installation
XX Heavy (> 120 kg/m)	1,216	320	1,159	380
X Heavy (90 - 120 kg/m)	1,273	350	1,203	410
Heavy (60 – 90 kg/m)	1,307	460	1,268	490
Medium (30 – 60 kg/m)	1,379	550	1,334	620
Light (0 – 30 kg/m)	1,496	660	1,443	820
Average	**1,334**	**468**	**1,281**	**544**

Table 4-6 Steel cost for MES and stick built [9]

Stick built steel means the steel built directly on site. Table 4-6 shows that the average costs is 1802 USD for MES and 1825 USD for stick built. Therefore, to compare the cost of each option, 1802 USD have to be used to find the cost of pipe rack and 1825 USD have to be used to find the cost of the bottom columns. However, the difference is not much so 1810 USD is used to calculate steel cost for every option. Based on this steel cost information and the steel quantities which found in 4.2.1, the total steel cost for the 27 pipe racks for each option can be calculated. Figure 4-9 shows steel work cost of each option for the W6xH18xL24 pipe rack.

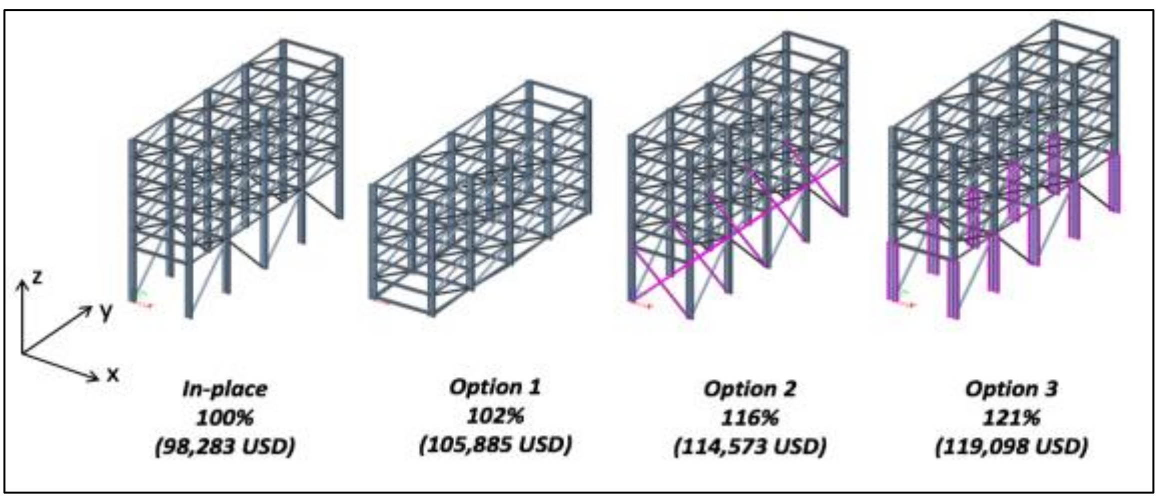

Figure 4-9 Steel work cost of each option

The ratios between options are same as steel quantity because the steel work cost is same for each option. The costs for every configuration and option are listed in APPENDIX G.

Figure 4-10 shows the ratios for each configuration of pinned supported pipe racks.

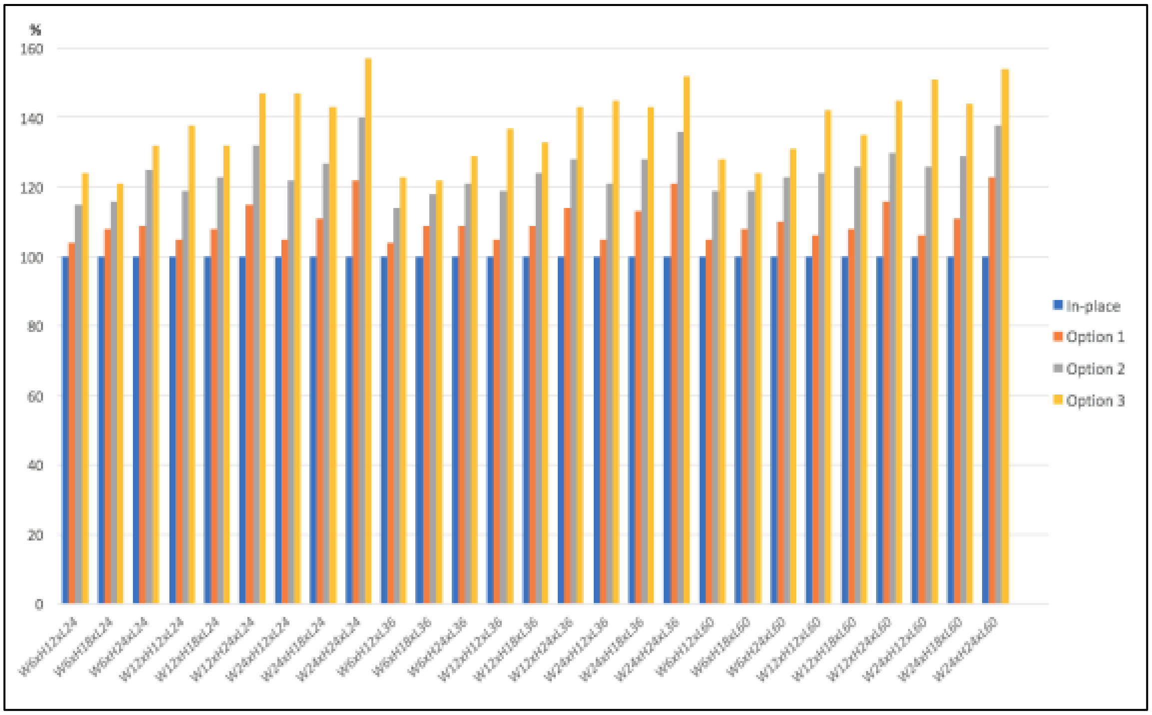

Figure 4-10 Summary of ratios in chart (pinned supported)

The steel work cost for in-place situation are set as 100%. It is found that in case of pinned supports the average difference of ratio between option 1 and 2 is about 14.5% and between option 1 and 3 is 27.9%. Therefore, if the company use pinned supports, the difference is about 15% between options 1 and 2 but it is about 30% between option 1 and 3 while with clamped supports both differences are about 15%. Detail data for clamped supported pipe racks are in APPENDIX G.

4.3 Discussion for the results of the first research question

The results show that option 1 is the most cost effective way of transporting the pipe racks. It was found that if option 1 is used, comparing the other options, the steel can be saved greatly. It is because by using option 1, the center of gravity can be lowered so the horizontal forces are also lower. Consequently, the moment, which produces most of the stress in the steel, is much less. Therefore, it is known that the horizontal loads are dominant forces for the design of the pipe racks. In the thesis, it is seen that sea transport loads are bigger than in-place loads. Therefore, the sea transport loadings are the dominant loadings for the design of the pipe racks. It was also found that, as expected, the bottom columns are the critical parts since they get biggest stresses from the external loadings. When the supports for sea transport are pinned, the steel quantity and cost difference between option 2 and 3 is, on average, about 15% bigger than those with clamped. It shows that the bracings of option 2 help more efficiently to withstand the forces than the larger columns of option 3 when the deck of the vessel does not take any moment force. In practice, pinned supports are more favorable because pinned supports on a vessel is easier and quicker to install than clamped supports; thus, if options 1 is not available, option 2 with pinned supports will be most favorable.

In this thesis, steel work cost data of Kuwait and China is taken to be used to find the steel work cost for each option. Coincidently, the cost of both Kuwait and China is approximately same, so total steel quantity of a pipe rack determined the cost of the pipe rack. However, if the cost is much different for example the installation site location is somewhere very isolated the stick built cost would be very expensive and it will result in more expensive option 1. Figure 4-11 shows the average of steel work ratios for each option for pinned supported pipe racks.

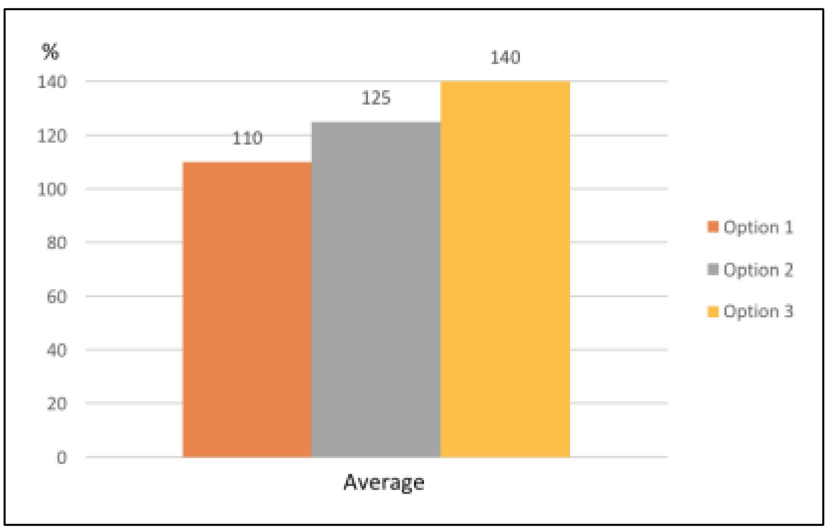

Figure 4-11 Average of steel work ratios of each option (pinned supported)

It shows there is 15% difference for each option in average. This is when the steel work cost is same in Kuwait and China. If the cost is different, the ratios will be different. The average steel quantity for bottom columns for option 1 which will be produced as stick built is 20% of total steel quantity of a pipe rack. Therefore, if the steel work cost in Kuwait is 1.7 times more expensive than in China, the chart will be changed as in Figure 4-12.

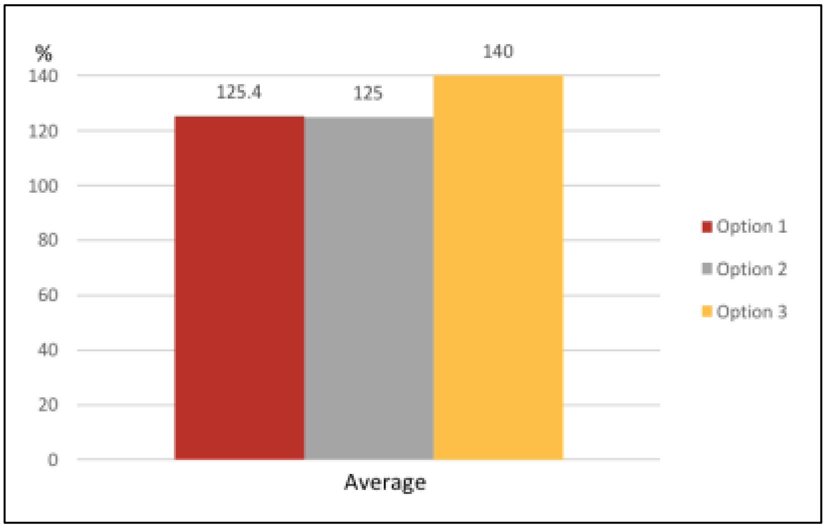

Figure 4-12 Average of steel work ratios of each option (pinned supported) with different steel work cost

20% of 110% (22%) steel quantity is multiplied with 1.7 times more expensive cost and the cost ratios of bot option 1 and option 2 is almost same. This means if the stick built cost is 1.7 times more expensive than MES cost, option 2 will be a better choice.

There are some facts that can change the result. A location of the construction project affects the most. Based on the location, the location of a fabrication yard, the loads of in-place design, the loads of sea transport and steel price of MES and stick built will be changed. Other variations of the results are a type of a vessel, the use of transport beam, the re-use of the bracings, the pipe rack placement on a vessel and etc. Therefore, to derive more accurate results, a specific project data is necessary. However, the conclusion that option 1 is the most cost effective way of transporting is expected to be applicable for most of onshore project. It is because the ratio of the bottom columns steel quantity is relatively small so for option 2&3 to be more beneficial, the stick built cost have to be much more expensive than MES.

5. VERIFICATION OF USE OF QUASI-STATIC ANALYSIS

A quasi-static analysis method was used in Chapter 4 to obtain estimates of the steel quantity and steel work costs of three different configuration options for MES. In this chapter, the validity of the use of the quasi-static analysis method was verified by comparing displacements of a structure and stress in the steel member from both the quasi-static and dynamic analysis method. For the dynamic analysis, a methodology of the dynamic analysis was introduced first.

5.1 Methodology of dynamic analysis

In order to perform a dynamic analysis, equations of motion, external forces have to be set as well as model description. Furthermore, basic setting like solution method, time step, initial condition and damping effect has to be determined.

5.1.1 Model description

A portal side frame of W6 x H24 is chosen for the dynamic analysis because it is expected that this frame will have the biggest dynamic effect due to its slenderness and height.

5.1.2 Equations of motion

In Chapter 2.3.1, an equation of motion for a single degree of freedom was explained as an example. However, the portal-side frame has multiple degrees of freedom as seen in Eq. 5-1.

$$[\mathrm{M}]\begin{bmatrix}\ddot{u}_1\\ \ddot{v}_1\\ \ddot{\theta}_1\\ \vdots\\ \ddot{u}_n\\ \ddot{v}_n\\ \ddot{\theta}_n\end{bmatrix}+[\mathrm{C}]\begin{bmatrix}\dot{u}_1\\ \dot{v}_1\\ \dot{\theta}_1\\ \vdots\\ \dot{u}_n\\ \dot{v}_n\\ \dot{\theta}_n\end{bmatrix}+[\mathrm{K}]\begin{bmatrix}u_1\\ v_1\\ \theta_1\\ \vdots\\ u_n\\ v_n\\ \theta_n\end{bmatrix}=\begin{bmatrix}F_{x_1}\\ F_{y_1}\\ M_1\\ \vdots\\ F_{x_n}\\ F_{y_n}\\ M_n\end{bmatrix} \qquad \text{(Eq. 5-1)}$$

Where,

- M: Mass matrix
- C: Damping matrix
- K: Stiffness matrix
- $\ddot{u}$: Horizontal acceleration
- $\ddot{v}$: Vertical acceleration
- $\ddot{\theta}$: Rotational acceleration
- $\dot{u}$: Horizontal velocity
- $\dot{v}$: Vertical velocity
- $\dot{\theta}$: Rotational velocity
- u : Horizontal displacement
- v : Vertical displacement
- θ : Rotational displacement
- F_x: External horizontal force
- F_y: External vertical force
- M: External moment
- n: number of nodes

FEM program will automatically produce the mass matrix, damping matrix and stiffness matrix, but external loading has to be defined by user. For the analysis 2D beam element is used which has 6 degrees of freedom for each element. More information about the element as well as the finite element method is explained in APPENDIX C.

5.1.3 Dynamic loading induced by motions of a vessel

Not like the quasi-static analysis, the loads of the dynamics analysis are time dependent. For example, the dynamics loads in case of a roll motion are explained and it is shown as (Eq. 5-2) and (Eq. 5-3). The total load in vertical as well as horizontal direction is composed out of contributions due to the roll motion, heave motion and gravitational force. These individual load contributions are presented in the remainder of this chapter. In order to obtain the load for the simulation it is assumed that roll and heave motions are in same phase which will cause the maximum load on the structure.

$$F_{h,portal} = m \cdot \ddot{\theta}(t) \cdot z_n + m \cdot \ddot{h}(t) \cdot sin(\theta(t)) + m \cdot g \cdot sin(\theta(t)) \qquad \text{(Eq. 5-2)}$$

$$F_{v,portal} = m \cdot \ddot{\theta}(t) \cdot x_n + m \cdot \ddot{h}(t) \cdot cos(\theta(t)) + m \cdot g \cdot cos(\theta(t)) \qquad \text{(Eq. 5-3)}$$

Where,

- $F_{h,portal}$: Horizontal load for the portal-side frame
- $F_{v,portal}$: Vertical load for the portal-side frame
- $\theta(t) = \theta_a \cdot cos(\omega \cdot t)$: Rotational (roll) displacement
- $\ddot{\theta}(t) = -\omega^2 \cdot \theta_a \cdot cos(\omega \cdot t)$: Rotational (roll) acceleration
- $\theta_a = 0.349\ rad\ (= 20°)$: Amplitude of rotational (roll) displacement
- $h(t) = h_a \cdot cos(\omega \cdot t)$: Vertical (heave) displacement
- $\ddot{h}(t) = -\omega^2 \cdot h_a \cdot cos(\omega \cdot t)$: Vertical (heave) acceleration
- $h_a = 5m$: Amplitude of vertical (heave) displacement
- $\omega = \frac{2 \cdot \pi}{t} = 0.6283 \frac{rad}{s}\ (t = 10s)$: Circular frequency
- z_n: Height of the nodes
- x_n: Transverse distance from the center of a pipe rack

- **Load from a roll motion**

The first terms of (Eq. 5-2) and (Eq. 5-3) are the forces caused by a roll motion. These terms can be expressed as (Eq. 5-4) and (Eq. 5-5).

$$m \cdot \ddot{\theta}(t) \cdot z_n = -m \cdot \omega^2 \cdot \theta_a \cdot z_n \cdot cos(\omega \cdot t) \qquad \text{(Eq. 5-4)}$$

Calculated horizontal dynamic loadings from the roll motion are listed in Table 5-1.

Height	6m	8m	10m	12m	14m	16m	18m	20m	22m	24m
Amplitude [kN]	11.6	13.6	15.5	17.5	19.5	21.4	23.4	25.4	27.3	29.3
Time function	$cos(\omega \cdot t)$									

Table 5-1 Horizontal dynamic loadings from the roll motion

$$m \cdot \ddot{\theta}(t) \cdot x_n = -m \cdot \omega^2 \cdot \theta_a \cdot x_n \cdot cos(\omega \cdot t) \qquad \text{(Eq. 5-5)}$$

Calculated vertical dynamic loadings from the roll motion are listed in Table 5-2.

Height	6m	8m	10m	12m	14m	16m	18m	20m	22m	24m
Amplitude [kN]	11.7									
Time function	$cos(\omega \cdot t)$									

Table 5-2 Vertical dynamic loadings from the roll motion

The vertical loadings from the roll motion is the transvers distance dependent. Since the transvers distance form center of the pipe racks is assumed 12m in Chapter 3.3.3, the vertical loadings are same for all height.

- **Load from a heave motion**

The second terms of (Eq. 5-2) and (Eq. 5-3) are the forces caused by a heave motion which can be expressed as (Eq. 5-6) and (Eq. 5-7).

$$m \cdot \ddot{h}(t) \cdot sin\big(\theta(t)\big) = -m \ \cdot \omega^2 \cdot h_a \cdot cos(\omega \cdot t) \cdot sin(\theta_a \cdot cos(\omega \cdot t)) \qquad \text{(Eq. 5-6)}$$

Calculated horizontal dynamic loadings from the heave motion are listed in Table 5-3.

Height	6m	8m	10m	12m	14m	16m	18m	20m	22m	24m
Amplitude [kN]	14.1									
Time function	$cos(\omega \cdot t) \cdot sin(\theta_a \cdot cos(\omega \cdot t))$									

Table 5-3 Horizontal dynamic loadings from the heave motion

$$m \cdot \ddot{h}(t) \cdot cos\big(\theta(t)\big) = -m \ \cdot \omega^2 \cdot h_a \cdot cos(\omega \cdot t) \cdot cos(\theta_a \cdot cos(\omega \cdot t)) \qquad \text{(Eq. 5-7)}$$

Calculated vertical dynamic loadings from the heave motion are listed in Table 5-4.

Height	6m	8m	10m	12m	14m	16m	18m	20m	22m	24m
Amplitude [kN]	1.97									
Time function	$cos(\omega \cdot t) \cdot cos(\theta_a \cdot cos(\omega \cdot t))$									

Table 5-4 Vertical dynamic loadings from the heave motion

- **Load from gravitational force**

The second terms of (Eq. 5-2) and (Eq. 5-3) are the gravitational forces acting on the structure, these terms can be expressed as (Eq. 5-8) and (Eq. 5-9).

$$m \cdot g \cdot sin\big(\theta(t)\big) = m \cdot g \cdot sin(\theta_a \cdot cos(\omega \cdot t)) \qquad \text{(Eq. 5-8)}$$

Calculated horizontal dynamic loadings from the gravitational force are listed in Table 5-5.

Height	6m	8m	10m	12m	14m	16m	18m	20m	22m	24m
Amplitude [kN]	70									
Time function	$sin(\theta_a \cdot cos(\omega \cdot t))$									

Table 5-5 Horizontal dynamic loadings from gravitational force

$$m \cdot g \cdot cos\big(\theta(t)\big) = m \cdot g \cdot cos(\theta_a \cdot cos(\omega \cdot t)) \qquad \text{(Eq. 5-9)}$$

Calculated vertical dynamic loadings from the gravitational force are listed in Table 5-6.

Height	6m	8m	10m	12m	14m	16m	18m	20m	22m	24m
Amplitude [kN]	2.35									
Time function	$cos(\theta_a \cdot cos(\omega \cdot t))$									

Table 5-6 Vertical dynamic loadings from the gravitational force

For the heave motion and gravity force, both horizontal and vertical loading are height independent.

In the end sum of the horizontal and vertical loadings are applied on the portal-side frame as in Figure 5-1. Left side figure shows the quasi-static loadings and right side figure shows the dynamic loadings. The dynamic loads are varying in time because of the time dependent terms. The maximum dynamic loads are same as the quasi-static loads.

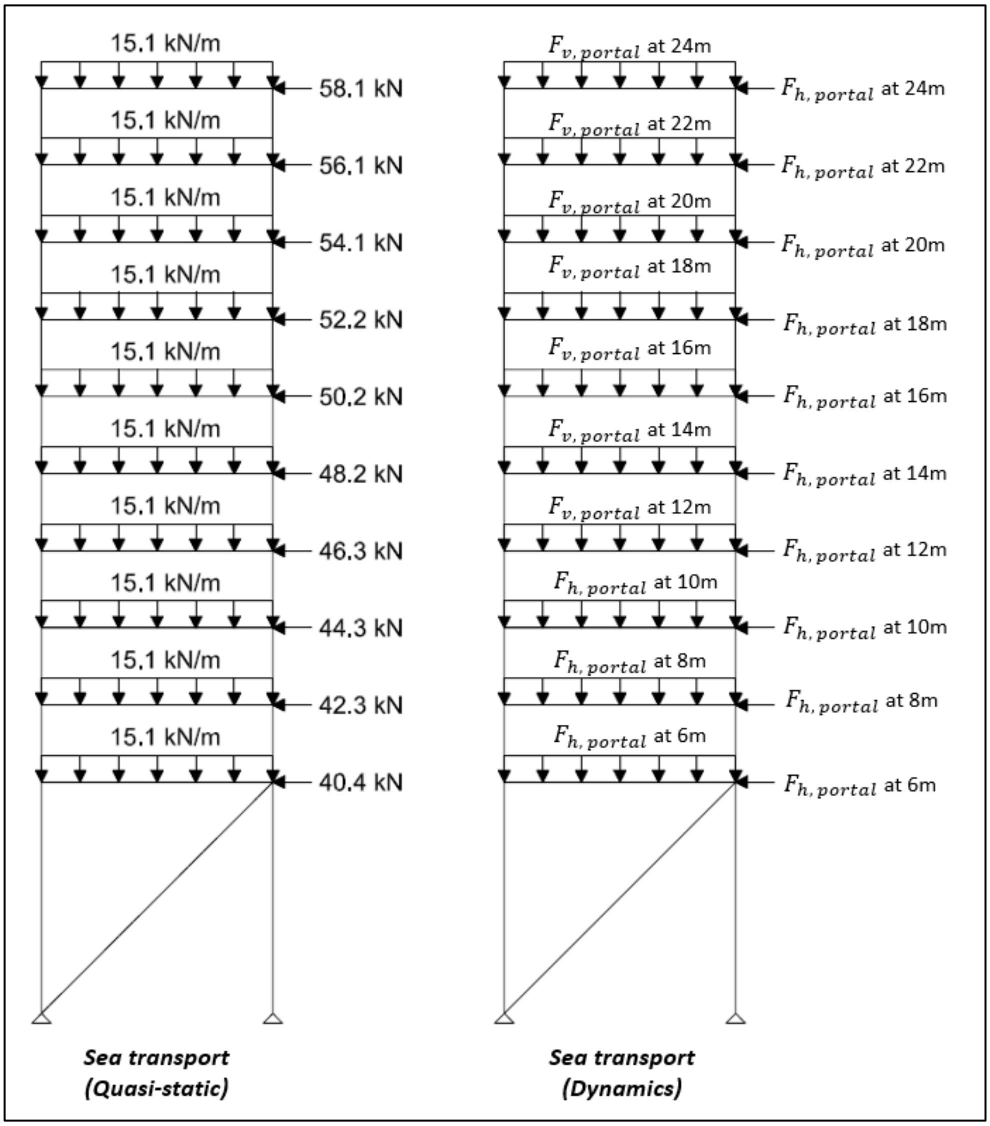

Figure 5-1 Load applications for each analysis method

Assuming the dynamic loads as harmonic loads does not reflect realistic sea transport situations. However, the purpose of performing the dynamic analysis is to check an inertial effect; thus, the analysis was performed only changing the quasi-static loadings to be time dependent.

5.1.4 Software settings for dynamic analysis

For the dynamic analysis, an analysis method of SAP2000, a linear modal time-history analysis is used. Time-history analysis is a step-by-step analysis of the dynamical response of a structure to a specified loading that may vary with time. Modal analysis which is used to determine the vibration modes of a structure is the basis for modal superposition in the linear modal time-history analysis.

Zero initial condition which means the analysis start from unstressed state is chosen for an initial condition.

Time-history analysis is performed at discrete time steps. 500 of output time steps and 0.1 of output time step size are used which results in 50 seconds of time.

As seen in Chapter 2.3.1, there is a damping term in the equation of motion. In this thesis, damping in the structure is not considered because the damping effect will reduce deflections and stresses of the structure. This chapter focuses on finding the maximum value of deflections and stresses with the dynamics analysis method so they can be compared with the maximum value of deflections and stresses with the quasi-static anlaysis method.

In order to validate the results of SAP2000, a dynamic anlaysis with 1000 seconds period cycle was done as well. If the computation result is same as a quasi-static analysis, it means the computation is performed correctly.

5.2 Resonance check with natural frequencies of the structure

Prior to check dynamic response, natural frequencies of the structure are found to check whether a resonance problem will occur or not. The natural frequencies are listed in Table 5-7 and Figure 5-2 shows the shapes of each mode.

Type	No.	Period [s]	Frequency [1/s]	Circ. Freq. [rad/s]
Mode	1	0.32	3.10	19.52
Mode	2	0.23	4.22	26.56
Mode	3	0.069	14.47	90.96
Mode	4	0.031	32.06	201.45
Mode	5	0.022	44.89	282.05

Table 5-7 Natural Frequencies for W6 x H24

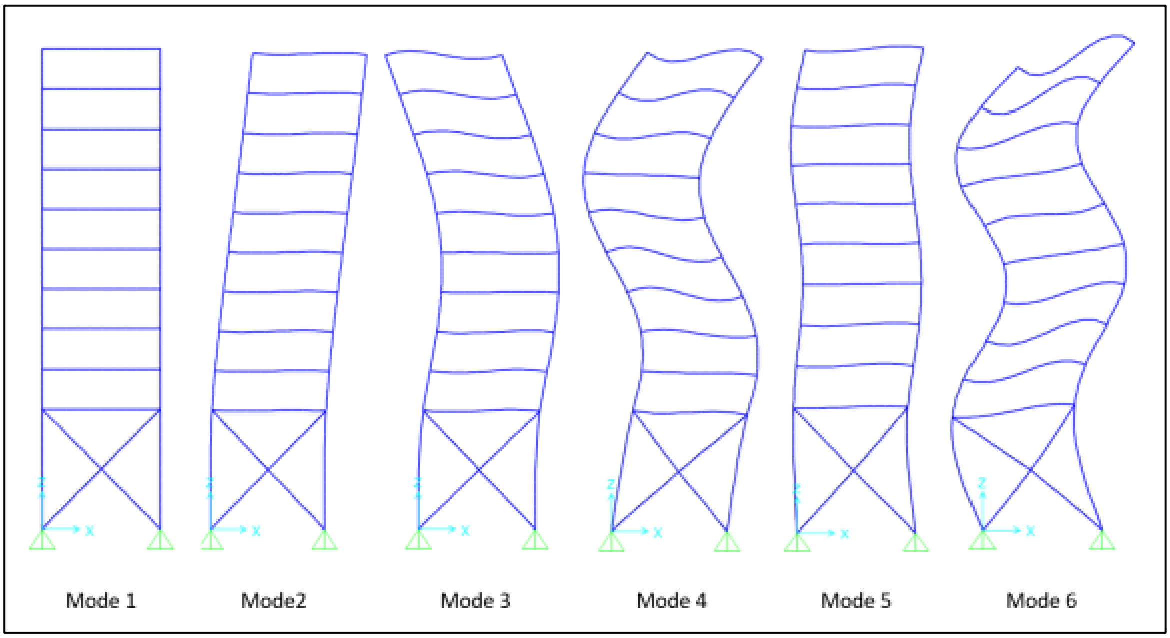

Figure 5-2 Shapes of each mode

Table 5-7 shows the natural period of the first mode of the structure is 19.52 rad/s. Figure 5-3 is an example of roll RAO of a containership in frequency domain.

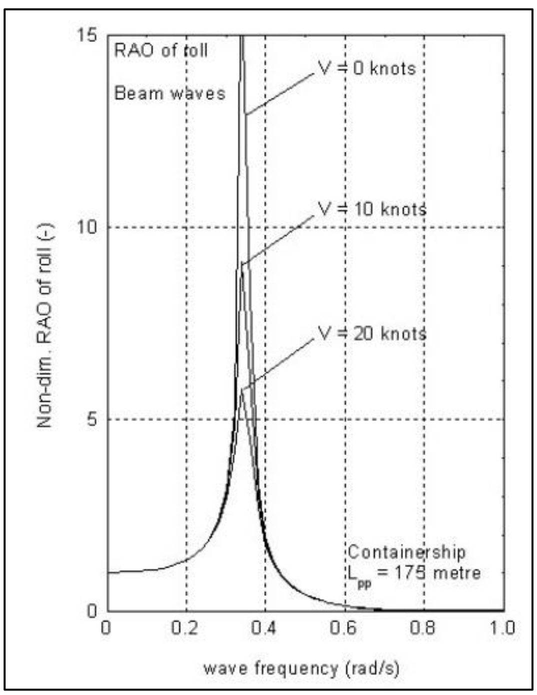

Figure 5-3 Example of Roll RAO **Error! Reference source not found.**

This figure show that after 0.8 rad/s, there is hardly roll response of the ship. This is just one example of Roll RAO of a ship but the gap between 0.8 rad/s and 19.52 rad/s is so big. It can be expected that other vessels' roll RAO will also not reach to 19.52 rad/s. Therefore, the resonance will not be happening because it is impossible that the frequency of the roll motion reach to the natural frequencies of the structure.

5.3 Comparison of the quasi-static and dynamic analysis results

Prior to perform a dynamic analysis, resonance check was done. As a result of the dynamic analysis, response of structure was found. The maximum displacement of the dynamics analysis was compared with the maximum displacement of the quasi-static analysis.

5.3.1 Displacement comparison of quasi-static and dynamic analysis

Displacement checks are done for the top node of the structure which deflects the most. Figure 5-4 shows horizontal response of the node for 50 seconds.

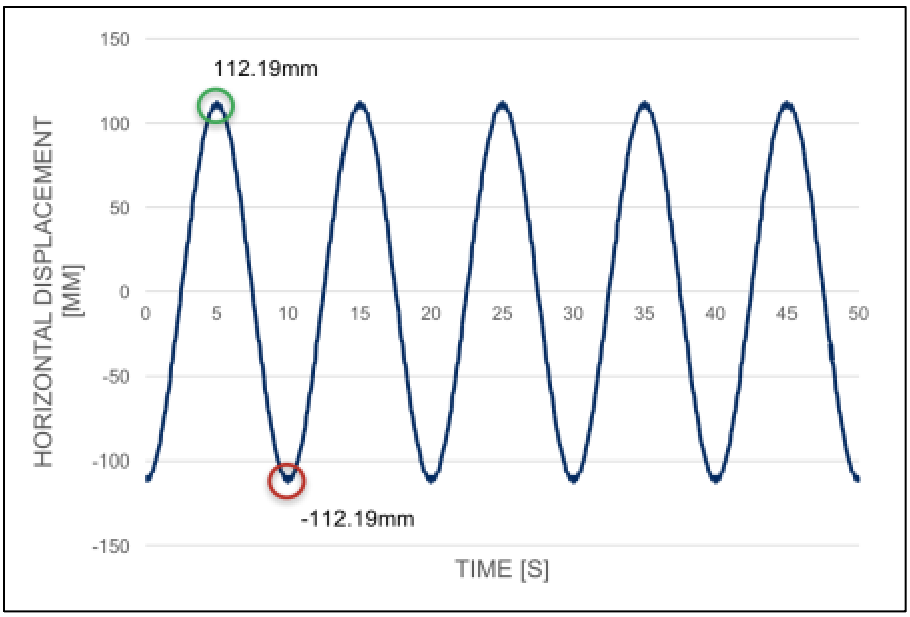

Figure 5-4 Horizontal response of the top node of the structure

Table 5-8 shows the comparison for the maximum horizontal displacement of the quasi-static and dynamic analysis methods.

Analysis Type	Horizontal Disp. [mm]
Quasi-Static Analysis	110.92
Dynamic Analysis with 10s period	112.19
Dynamic Analysis with 1000s period	110.92

Table 5-8 Absolute maximum displacements

The differences are less than 2 mm. The dynamic analysis result with 1000s period shows that if the period of motion is sufficiently long, there is no effect of inertial term; so, the result is same as the quasi-static analysis result.

5.3.2 Stress comparison of quasi-static and dynamic analysis

The maximum stress in the bottom column were checked for four analysis method. The results are listed in Table 5-9.

Analysis Type	Max. Stress [N/mm2]
Quasi-Static Analysis	141.45
Dynamic Analysis with 10s period	142.69
Dynamic Analysis with 1000s period	141.45

Table 5-9 Absolute maximum and minimum stresses

The maximum stress was found at the top of the bottom column since the top part of the bottom column gets highest moment. The difference of the maximum stress between the quasi-static and dynamic analysis is 1.24 N/mm2. As seen from this analysis results, there is not much difference between the presented results of the quasi-static analysis and the dynamic analysis. As same as Chapter 5.3.1, the 1000s dynamic analysis result is same as the quasi-static analysis result.

5.4 Discussion for the results of the second research question

It was found that there is no chance of resonance as well as a portion which inertial term and damping term contributes to deflection of the structure and stresses in the steel is very limited.

The main difference between the quasi-static analysis and the dynamic analysis is whether or not the inertial term and the damping term are considered. The inertial term causes more deflection whereas the damping term reduces the deflection. According to the results of this thesis, it was found that the dynamic analysis is not necessary for a pipe rack design for MES because the difference of deflection and stress between the quasi-static and dynamic analysis was negligible. It means the inertial and damping term are relatively much smaller than stiffness term so it has almost no effects on the deflection and stress for the pipe racks. It can be expected less stiffness causes more involving of inertia and damping effects.

In order to satisfy the design criteria for both USL and SLS as introduced from Table 2-2 in Chapter 2.2.1, the structure was designed with stiff steel members so the displacement was approximately 110mm to satisfy SLS criteria which H/100. The height of the structure is 24m; thus, the stiffness is rather high that inertial term and damping terms to the equations of motion have relatively a small effect on the displacement of the structure.

In fact, the effect of damping has not been considered well because of using prescribed motions, however, even without damping, the difference of maximum displacement and stress between the analysis is negligibly small; thus, it will not be a problem.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The first research question was: what are the quantity of steel and cost of each option and the difference between them?

In order to answer the first research question, firstly, representative configurations of the pipe racks which have to be analyzed were decided. 27 configurations of pipe racks were chosen to be analyzed. Secondly, initial structural design for in-place loadings was conducted. After the designs for in-place loadings were finished, feasibility check of initial design subjected to sea transport loadings were performed. The initial design for each side frames were checked for sea transport loadings and the three MES options were applied. Subsequently, for each MES option, frames of equal height are combined into 27 distinct structural configurations. Finally, comparison of steel quantity and steel work cost which includes procurement, fabrication and installation of steel were done from the design.

As a result, it was found that the option 1 is the most cost effective solution for both the pinned and clamped supported pipe racks. On average, for the pinned supported pipe racks, the option 1 requires 15% and 30% less cost than option 2 and option 3 respectively while the options 1 requires 15% less cost than both option 2 and 3 for the clamped supported pipe racks. This result shows that regardless of types of supports, the options 1 is most cost effective way of transporting the pipe racks.

The second research question is: is the quasi-static analysis for sea transport situation still valid for the design of high structure which can be affected by dynamic effect more than shorter one?

In order to answer the second research question, one portal side frame which is 6m wide and 24m high was selected for comparing resonance frequencies with the prescribed period of roll motion which obtained from DNV-ST-N001.

Prior to performing a dynamic analysis, the possibility of resonance was checked. It was found that there is a big gap between the frequency of motions of the vessel and the natural frequencies of the frame, so considering the excitation of the vessel there is no possibility of the resonance. After the check of the resonance frequency, dynamic response of the structure was compared to quasi-static response of the structure. The assumed roll and heave motion of a vessel are transformed to horizontal and vertical inertial forces to the structure. The amplitude and period of motions are same as the quasi-static analysis, but the loadings vary in time for dynamic analysis and this was done with modal time-history analysis. With the results, comparison was available for the results from quasi-static and dynamic analysis approach. The results of the dynamic and quasi-static approach are used to perform a comparison of the maximum horizontal and vertical deflection of the top node, as well as comparison of the maximum stresses occurring in the bottom columns. It was found that they are negligibly small. Therefore, the answer for the second research question is concluded as that a use of a quasi-static analysis for the pipe rack design is acceptable.

6.2 Recommendations

The thesis was done only in terms of different structural configuration. Cost of logistics, safety, and administrative aspects were not taken into account to the results of the study. Therefore, in order to verify the effectiveness of each option for the overall project in more detail, further work is required to identify the other aspects which can affect the cost of the project.

This thesis was done for a project which the company is currently executing in Kuwait. However, there will be many variables which are different for a different project like, installation site location, fabrication yard location, pipe rack configuration, in-place loadings, sea-transport loadings, and the use of different design criteria; so, for a different project, different variables have to be applied.

Furthermore, this thesis was focused on determining the best option among the options in terms of steel work with the company's standard that for massive production of pipe racks, conservative design approach is used and also was used for this thesis. This means that this thesis does not reflect the realistic structural response. In order to optimize the structural design itself, taking a single structure and performing a simulation with realistic sea state can be a good attempt.

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APPENDICES

APPENDIX A : STEEL ULS CHECKS

As mentioned in Chapter 2.2.1, there are several checks for ULS. Detail requirements for each check are shown in this appendix.

A.1 Compression check

According to EN 1993-1-1 article 6.2.4 and formula (6.9)

The design value of the compression force N_{Ed} at each cross-section shall satisfy:

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0 \qquad \text{(E. A-1)}$$

The design resistance of the cross-section for uniform compression $N_{c,Rd}$ should be determined as flows:

$N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}}$ for class 1, 2 or 3 cross-sections

$N_{c,Rd} = \frac{A_{eff} \cdot f_y}{\gamma_{M0}}$ for class 4 cross-sections

A.2 Bending moment check

According to EN 1993-1-1 article 6.2.5 and formula (6.12), (6.14)

The design value of the bending moment M_{Ed} at each cross-section shall satisfy:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0 \qquad \text{(E. A-2)}$$

The design resistance for bending about one principal axis of a cross-section is determined as follows:

$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}}$ for class 1 or 2 cross sections

$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,min} \cdot f_y}{\gamma_{M0}}$ for class 3 cross sections

$M_{c,Rd} = \frac{W_{eff,min} \cdot f_y}{\gamma_{M0}}$ for class 4 cross sections

A.3 Shear check

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

The design value of the shear force V_{Ed} at each cross section shall satisfy:

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1.0 \qquad \text{(E. A-3)}$$

A.4 Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.2 and formula (6.42)

A.5 Flexural buckling check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

A.6 Torsional (-Flexural) buckling check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

For the I-section the torsional (-flexural) buckling resistance is higher than the resistance for flexural buckling.

A.7 Lateral torsional buckling check

According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

A.8 Bending and axial compression check

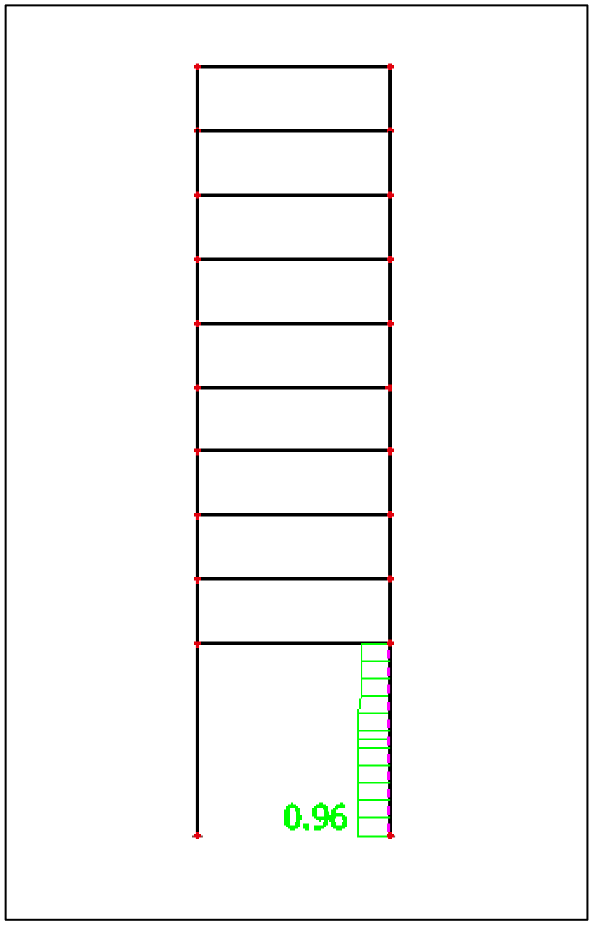
According to EN 1993-1-1 article 6.3.3 and formula (6.61), (6.62)

A.9 Shear buckling check

According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

A.10 Example of ULS check by Scia Engineer

Fg. A-1 shows a result of ULS check for the bottom column.



Fg. A-1 ULS check for the bottom column

The number 0.96 is the maximum value among the results of ULS checks. In this case, 0.96 is from bending and axial compression check. Fg. A-2, Fg. A-3 and Fg. A-4 show the results of USL check done by Scia Engineer. 0.96 can be found in Fg. A-3.

Check of steel

Linear calculation, Extreme : Member
Selection : B2025
Class : ULS for On Site

EN 1993-1-1 Code Check
National annex: Standard EN

Member B2025	6.000 m	UB914/305/224	S 355	LC 101_1U[SW]	0.96 -

Partial safety factors	
Gamma M0 for resistance of cross-sections	1.00
Gamma M1 for resistance to instability	1.00
Gamma M2 for resistance of net sections	1.25

Material		
Yield strength fy	355.0	MPa
Ultimate strength fu	490.0	MPa
Fabrication	Rolled	

...::SECTION CHECK::...

Classification for cross-section design
According to EN 1993-1-1 article 5.5.2
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	51.85
Class 1 Limit	44.40
Class 2 Limit	51.12
Class 3 Limit	72.08

=> Internal Compression parts Class 3
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	5.23
Class 1 Limit	7.32
Class 2 Limit	8.14
Class 3 Limit	11.20

=> Outstand Flanges Class 1
=> Section classified as Class 3 for cross-section design

The critical check is on position 0.000 m

Internal forces	Calculated	Unit
N,Ed	-1258.07	kN
Vy,Ed	0.00	kN
Vz,Ed	203.53	kN
T,Ed	0.00	kNm
My,Ed	-1576.49	kNm
Mz,Ed	0.00	kNm

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

A	2.8600e-02	m^2
Nc,Rd	10153.00	kN
Unity check	0.12	-

Bending moment check for My
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.14)

Wel,y,min	8.2690e-03	m^3
Mel,y,Rd	2935.49	kNm
Unity check	0.54	-

Shear check for Vz
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Eta	1.20	
Av	1.6458e-02	m^2
Vpl,z,Rd	3373.30	kN
Unity check	0.06	-

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.2 and formula (6.42)

Normal stresses		
Fibre	1	
Sigma,N,Ed	44.0	MPa
Sigma,My,Ed	190.6	MPa
Sigma,Mz,Ed	0.0	MPa
Sigma,tot,Ed	234.6	MPa
Unity check	0.66	-

The member satisfies the section check.

...::STABILITY CHECK::...
Classification for member buckling design
Decisive position for stability classification: 6.000 m
Classification of Internal Compression parts
According to EN 1993-1-1 Table 5.2 Sheet 1

Maximum width-to-thickness ratio	51.85
Class 1 Limit	26.85
Class 2 Limit	30.92
Class 3 Limit	50.50

=> Internal Compression parts Class 4
Classification of Outstand Flanges
According to EN 1993-1-1 Table 5.2 Sheet 2

Maximum width-to-thickness ratio	5.23
Class 1 Limit	7.32
Class 2 Limit	8.14
Class 3 Limit	11.20

=> Outstand Flanges Class 1
=> Section classified as Class 4 for member buckling design

Calculation effective area properties with direct method.

Properties					
sectional area A eff	2.4536e-02	m^2			
Shear area Vy eff	1.4536e-02	m^2	Vz eff	9.9997e-03	m^2
radius of gyration iy eff	389	mm	iz eff	68	mm
moment of inertia Iy eff	3.7070e-03	m^4	Iz eff	1.1231e-04	m^4
elastic section modulus Wy eff	8.1437e-03	m^3	Wz eff	7.3863e-04	m^3
Eccentricity eny	0	mm	enz	0	mm

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters	yy	zz	
Sway type	sway	sway	
System length L	6.000	6.000	m
Buckling factor k	1.79	1.00	
Buckling length Lcr	10.711	6.000	m

Fg. A-2 Results of ULS check-1

Buckling parameters	yy	zz	
Critical Euler load Ncr	68016.33	6468.94	kN
Slenderness Lambda	29.52	95.73	
Relative slenderness Lambda,rel	0.36	1.16	
Limit slenderness Lambda,rel,0	0.20	0.20	
Buckling curve	a	b	
Imperfection Alpha	0.21	0.34	
Reduction factor Chi	0.96	0.50	
Buckling resistance Nb,Rd	8392.77	4355.88	kN

Flexural Buckling verification		
Cross-section effective area Aeff	2.4536e-02	m^2
Buckling resistance Nb,Rd	4355.88	kN
Unity check	0.29	-

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: For this I-section the Torsional(-Flexural) buckling resistance is higher than the resistance for Flexural buckling. Therefore Torsional(-Flexural) buckling is not printed on the output.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters		
Method for LTB curve	Alternative case	
Cross-section effective modulus Weff,y	8.1437e-03	m^3
Elastic critical moment Mcr	4934.83	kNm
Relative slenderness Lambda,rel,LT	0.77	
Limit slenderness Lambda,rel,LT,0	0.40	
LTB curve	c	
Imperfection Alpha,LT	0.49	
LTB factor Beta	0.75	
Reduction factor Chi,LT	0.79	
Correction factor kc	0.81	
Correction factor f	0.90	
Modified reduction factor Chi,LT,mod	0.87	
Design buckling resistance Mb,Rd	2513.73	kNm
Unity check	0.63	-

Mcr parameters		
LTB length L	6.000	m
Influence of load position	no influence	
Correction factor k	1.00	
Correction factor kw	1.00	
LTB moment factor C1	1.54	
LTB moment factor C2	0.00	
LTB moment factor C3	1.00	
Shear center distance d,z	0	mm
Distance of load application z,g	0	mm
Mono-symmetry constant beta,y	0	mm
Mono-symmetry constant z,j	0	mm

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.
Note: The correction factor kc is determined from C1.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters		
Interaction method	alternative method 1	
Cross-section effective area Aeff	2.4536e-02	m^2
Cross-section effective modulus Weff,y	8.1437e-03	m^3
Design compression force N,Ed	1258.07	kN
Design bending moment (maximum) My,Ed	-1576.49	kNm
Design bending moment (maximum) Mz,Ed	0.00	kNm
Additional moment Delta My,Ed	0.00	kNm
Additional moment Delta Mz,Ed	0.00	kNm
Characteristic compression resistance N,Rk	8710.15	kN
Characteristic moment resistance My,Rk	2891.02	kNm
Reduction factor Chi,y	0.96	
Reduction factor Chi,z	0.50	
Modified reduction factor Chi,LT,mod	0.87	
Interaction factor k,yy	1.19	
Interaction factor k,zy	1.07	

Maximum moment My,Ed is derived from beam B2025 position 0.000 m.
Maximum moment Mz,Ed is derived from beam B2025 position 0.000 m.

Interaction method 1 parameters		
Critical Euler load N,cr,y	68016.33	kN
Critical Euler load N,cr,z	6468.94	kN
Elastic critical load N,cr,T	11712.40	kN
Cross-section effective modulus Weff,y	8.1437e-03	m^3
Second moment of area Iy	3.7652e-03	m^4
Second moment of area Iz	1.1236e-04	m^4
Torsional constant It	4.0636e-06	m^4
Method for equivalent moment factor C,my,0	Table A.2 Line 2 (General)	
Design bending moment (maximum) My,Ed	-1576.49	kNm
Maximum relative deflection delta,z	5.5	mm
Equivalent moment factor C,my,0	1.00	
Factor mu,y	1.00	
Factor mu,z	0.89	
Factor epsilon,y	3.78	
Factor a,LT	1.00	
Critical moment for uniform bending Mcr,0	3205.05	kNm
Relative slenderness Lambda,rel,0	0.95	
Limit relative slenderness Lambda,rel,0,lim	0.23	
Equivalent moment factor C,my	1.00	
Equivalent moment factor C,mLT	1.17	

Unity check (6.61) = 0.15 + 0.75 + 0.00 = 0.90 -
Unity check (6.62) = 0.29 + 0.67 + 0.00 = 0.96 -

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters		
Buckling field length a	6.000	m
Web	unstiffened	
End post	non-rigid	
Web height hw	863	mm
Web thickness t	16	mm
Yield strength fyw	355.0	MPa
Flange width bf	304	mm

Fg. A-3 Results of ULS check-2

Shear Buckling parameters		
Flange thickness tf	24	mm
Yield strength fyf	355.0	MPa
Material coefficient epsilon	0.81	
Shear correction factor Eta	1.20	

Shear Buckling verification		
Web slenderness hw/t	54.25	
Web slenderness limit	48.82	
Plate slenderness lambda,w	0.77	
Reduction factor chi,w	1.08	
Contribution of the web Vbw,Rd	3023.25	kN
Capacity of the flange Mf,Rd	1729.65	kNm
Flange factor c	1.641	m
Contribution of the flange Vbf,Rd	6.36	kN
Maximum resistance Vb,Rd,limit	3373.30	kN
Resistance Vb,Rd	3029.61	kN
Plastic resistance Mpl,Rd	3384.75	kNm
Shear ratio eta,3,bar	0.07	

Unity check (5.10) = 0.07 -

Note: The interaction between Bending and Shear Buckling does not need to be verified because the shear ratio does not exceed 0.5.

The member satisfies the stability check.

Fg. A-4 Results of ULS check-3

APPENDIX B : USE OF DNV PRESCRIBED MOTIONS

The reason of use of DNV prescribed motions are shown here as well as justification of why it is valid.

B.1 Justification of the use of prescribed motions

In order to design pipe racks for the sea transport situation, prescribed motion with a quasi-static analysis was used. In this way, the realistic behavior of pipe rack on a vessel cannot be simulated.

However, in fact, only the maximum load cases are needed to determine steel profiles for a structure. Therefore, once the maximum motions of a vessel are known as well as the minimum periods of the motions; thus other motion data is trivial.

Furthermore, during a sea transport, if a harsh sea state which could make bigger motions than prescribed motions, the vessel is planned to flee to the nearest harbor; so there will be no risk that the actual sea transport loading are bigger than designed loadings.

If the prescribed motions are bigger than actual motions, it means the designs are over-designed but this is better than the case that the prescribed motions are smaller than actual motions. The reason is explained in detail in Chapter B.2.

B.2 Reasons of the use of DNV default motion criteria

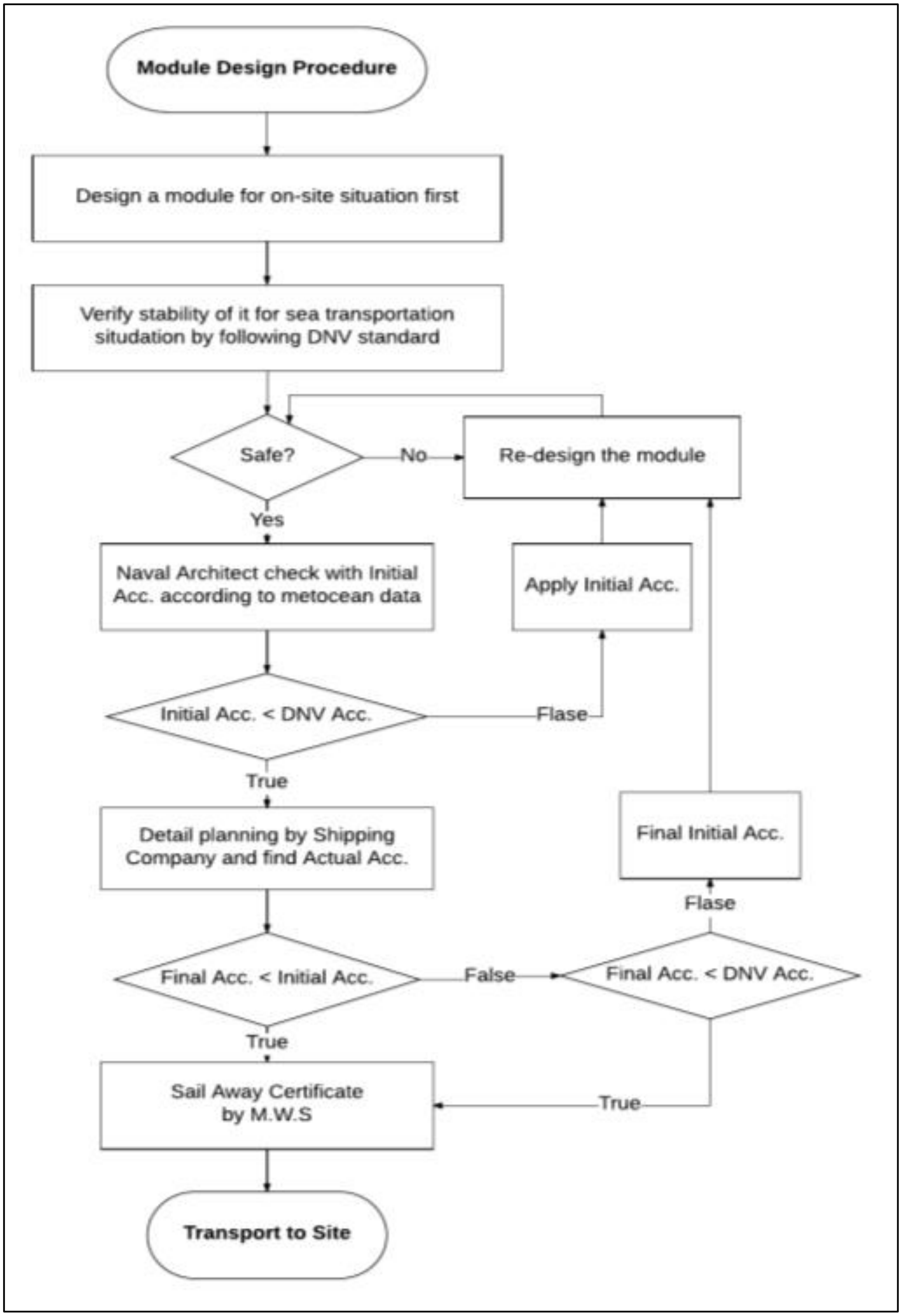
In order to find accurate effects of sea transport loading to the pipe racks, motions of a vessel, size of the vessel and locations of each pipe rack on the vessel are necessary.

However, the initial design of pipe racks starts in an early design stage of a project because there are dozens of pipe racks for one project so it is critical to start the design as soon as possible for a successful completion of the project.

Therefore, those sea transport data are not available at the initial design stage because finding a naval architect and a shipping company, and making a contract takes time. That is why, not like for the big offshore structure, simulating each and every pipe rack for the sea transport situation is not an option for the company.

Therefore, the company uses prescribed motions from DNVGL-ST-N001 to find sea transport loadings. Then, at the later stage, it is checked whether the reliability of using the motions from DNV criteria is okay or not by comparing with a simulation data from a naval architect.

If the used DNV motions are bigger than the simulation data from the naval architect, it means the initial design of pipe racks is acceptable. However, if the DNV motions are smaller than the data from the naval architect, re-design works have to be done which the company is most afraid of because re-work will make the project delayed and it will cause more cost. With experience of other projects the company performed, DNV motion criteria are more conservative than data from naval architects. Therefore, the company tends to use DNV motion, so they can avoid re-work. The procedure of the design of the pipe racks is shown in Fg. B-1.



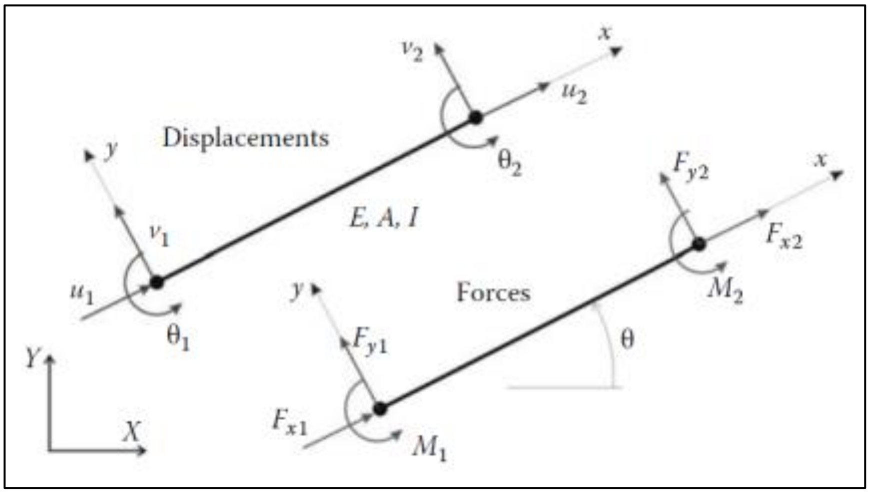
Fg. B-1 Schematic of pipe rack design procedure

APPENDIX C : FINITE ELEMENT METHOD

For easier understanding, a simple case like a SDOF system was studied in Chapter 2.3.1, but to design a steel structure, multi degrees of freedom (MDOF) have to be used because each part of the structure has their own degree of freedom. In order to solve MDOF, a help of a computer program which can perform Finite Element Method (FEM) is needed. FEM is a method with dividing members of the module as small elements and calculate the response of each element numerically. Theories of FEM are referred to R. W. Clough and J. Penzien, “Dynamics of Structures”, 3rd edition, 2003 [17], D. L. Logan, “A First Course in the Finite Element Method”, 4th edition, 2007 [18], A. Khennane, “Introduction to Finite Element Analysis Using MATLAB and Abaqus”, 2013 [19] were referred.

C.1 Stiffness Matrix and Mass Matrix

It should be noted that here, for the beam theory, Euler-Bernoulli beam theory was used. Furthermore, it is assumed that the material deformations are very small, and the material is linear elastic, then the axial displacements of the beam-column element do not interact with the bending deformations. Therefore, the principle of superposition can be applied, and the displacements, forces, and stiffness matrix of the beam–column element can be obtained by simply adding the respective matrices of a truss element and that of a beam element. A beam element has two nodes at each end of the element and each node has three degrees of freedom in 2D, axial, lateral and rotational displacement which means each element has six degrees of freedom.



Fg. C-1 Beam column element with six degrees of freedom [19]

Where:

u_n : Axial displacement

v_n : Lateral displacement

θ_n : Rotational displacement

For each element, there are a stiffness matrix and a mass matrix.

C.2 Local stiffness matrix

(E. C-1) is called local stiffness matrix. In order to compute MDOF, a global stiffness matrix is needed which reflects the structure with its global angle, θ_G. The way of transforming the local matrix to global matrix is explained in Fg. C-2 and Fg. C-3.

$$\mathrm{K_L} = \begin{bmatrix} \frac{EA}{l} & 0 & 0 & -\frac{EA}{l} & 0 & 0 \\ 0 & \frac{12EI}{l^3} & \frac{6EI}{l^2} & 0 & -\frac{12EI}{l^3} & \frac{6EI}{l^2} \\ 0 & \frac{6EI}{l^2} & \frac{4EI}{l} & 0 & -\frac{6EI}{l^2} & \frac{2EI}{l} \\ -\frac{EA}{l} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & -\frac{12EI}{l^3} & -\frac{6EI}{l^2} & 0 & \frac{12EI}{l^3} & -\frac{6EI}{l^2} \\ 0 & \frac{6EI}{l^2} & \frac{2EI}{l} & 0 & -\frac{6EI}{l^2} & \frac{4EI}{l} \end{bmatrix} \qquad \text{(E. C-1)}$$

This stiffness matrix represents stiffness of a beam element. It has six degrees of freedom, three for each node.

C.3 Local mass matrix

Lumped mass matrix is a default matrix of SAP2000; it is shown in (E. C-2).

$$M_L = \begin{bmatrix} \frac{\rho Al}{2} & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{\rho Al}{2} & 0 & 0 & 0 & 0 \\ 0 & 0 & \frac{\rho Al}{2} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{\rho Al}{2} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{\rho Al}{2} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{\rho Al}{2} \end{bmatrix} \qquad \text{(E. C-2)}$$

C.4 Matrix to change from local to global

Fg. C-2 shows a matrix which can change the local matrix to global matrix. For a beam member, local matrix and global matrix are same.

$$\begin{bmatrix} \cos\theta & -\sin\theta & 0 & 0 & 0 & 0 \\ \sin\theta & \cos\theta & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \cos\theta & -\sin\theta & 0 \\ 0 & 0 & 0 & \sin\theta & \cos\theta & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

Fg. C-2 Transformation matrix

Fg. C-2 shows a form of a transformation matrix. For the pipe racks, the angles between beams and columns are always right angles so 90 degrees in which used for left side columns and 270 degrees in which used for right side columns are needed.

$$\mathbf{C}_{90^\circ} = \begin{bmatrix} 0 & 1 & 0 & 0 & 0 & 0 \\ -1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & -1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \quad \mathbf{C}_{270^\circ} = \begin{bmatrix} 0 & -1 & 0 & 0 & 0 & 0 \\ 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & -1 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

Fg. C-3 Transformation matrix for 90 and 270 degrees

The transformation is carried out as follows:

$$[\mathrm{M_G}] = [\mathrm{C}][\mathrm{M_L}][\mathrm{C}]^{\mathrm{T}}, \quad [\mathrm{K_G}] = [\mathrm{C}][\mathrm{K_L}][\mathrm{C}]^{\mathrm{T}}$$

Where, $[\mathrm{M_G}], [\mathrm{K_G}]$ represent the element mass matrix and stiffness matrix in the global coordinate system respectively.

C.5 Equations of Motions for a steel frame structure

Now it is possible to make equations of motions for a steel frame structure. Equations below shows equations for n degrees of freedom system.

$$[\mathrm{M_G}]\begin{bmatrix} \ddot{u}_1 \\ \ddot{v}_1 \\ \ddot{\theta}_1 \\ \vdots \\ \ddot{u}_n \\ \ddot{v}_n \\ \ddot{\theta}_n \end{bmatrix} + [\mathrm{K_G}]\begin{bmatrix} u_1 \\ v_1 \\ \theta_1 \\ \vdots \\ u_n \\ v_n \\ \theta_n \end{bmatrix} = \begin{bmatrix} F_{x_1} \\ F_{y_1} \\ M_1 \\ \vdots \\ F_{x_n} \\ F_{y_n} \\ M_n \end{bmatrix} \qquad \text{(E. C-3)}$$

Where $[\mathrm{M_G}], [\mathrm{K_G}]$ represent the element mass matrix and stiffness matrix in the global coordinate system respectively. For the damping term, modal damping ratio is used.

APPENDIX D : PIPE RACK DIMENSIONS DATA BASE

Module Configuration Table - Pipe Racks

Revision F: 30-Sept-2016

Unit No.	Unit Name	Plant No.	Sub - Project (JV)	Module Number	Module Envelope Dimensions Length (m)	Width (M)	Height (m)	Volume (m3)
31	Hydrogen Recovery	N/A	DEC	3100BR01M	36.0	6.0	7.3	1,577
				3100BR02M	27.0	6.0	7.3	1,183
								2,759
32	Hydrogen Compression	N/A	AMS	3200BR01M	24.5	9.5	8.5	1,978
				3200BR02M	25.0	5.0	8.5	1,063
				3200BR03M	25.0	5.0	8.5	1,063
				3200BR04M	25.0	5.0	8.5	1,063
				3200BR05M	25.0	5.0	8.5	1,063
				3200BR06M	25.0	5.0	8.5	1,063
				3200BR07M	25.0	5.0	8.5	1,063
				3200BR08M	25.0	5.0	8.5	1,063
								9,416
33	Hydrogen Production	Plant 1	ND	3301AR01M	19.0	13.0	11.0	2,717
				3301BR01M	33.0	13.0	16.0	6,864
				3301BR02M	39.0	10.0	13.5	5,265
				3301BR03M	39.0	14.5	14.5	8,200
				3301BR04M	41.5	7.0	13.5	3,922
		Plant 2	ND	3302AR01M	19.0	13.0	11.0	2,717
				3302BR01M	33.0	13.0	16.0	6,864
				3302BR02M	39.0	10.0	13.5	5,265
				3302BR03M	39.0	14.5	14.5	8,200
				3302BR04M	41.5	7.0	13.5	3,922
		Plant 3	ND	3303AR01M	19.0	13.0	11.0	2,717
				3303BR01M	33.0	13.0	16.0	6,864
				3303BR02M	39.0	10.0	13.5	5,265
				3303BR03M	39.0	14.5	14.5	8,200
				3303BR04M	41.5	7.0	13.5	3,922
		Plant 4	ND	3304AR01M	19.0	13.0	11.0	2,717
				3304BR01M	33.0	13.0	16.0	6,864
				3304BR02M	39.0	10.0	13.5	5,265
				3304BR03M	39.0	14.5	14.5	8,200
				3304BR04M	41.5	7.0	13.5	3,922
		Common	ND	3305BR01M	39.0	19.0	19.0	14,079
				3305BR02M	45.0	15.0	16.0	10,800
								132,749
35	Sour Water Stripper	Plant 1	HHI	3501BR01M	48.0	19.0	23.5	21,432
				3501BR02M	42.0	17.5	19.5	14,333
		Plant 2	HHI	3502BR01M	48.0	19.0	23.5	21,432
				3502BR02M	42.0	17.5	19.5	14,333
		Plant 3	HHI	3503BR01M	48.0	19.0	23.5	21,432
				3503BR02M	42.0	17.5	19.5	14,333
								107,294

Module Configuration Table - Pipe Racks

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					Module Envelope Dimensions			
Unit No.	Unit Name	Plant No.	Sub - Project (JV)	Module Number	Length (m)	Width (M)	Height (m)	Volume (m3)
39	Amine Regeneration	Plant 1	ND	3901BR01M	36.6	15.5	19.0	10,779
				3901BR02M	42.0	15.0	19.0	11,970
				3901BR03M	42.0	15.0	19.0	11,970
				3901BR04M	41.5	15.0	19.0	11,828
		Plant 2	ND	3902BR01M	36.6	15.5	19.0	10,779
				3902BR02M	42.0	15.0	19.0	11,970
				3902BR03M	42.0	15.0	19.0	11,970
				3902BR04M	41.5	15.0	19.0	11,828
		Plant 3	ND	3903BR01M	36.6	15.5	19.0	10,779
				3903BR02M	42.0	15.0	19.0	11,970
				3903BR03M	42.0	15.0	19.0	11,970
				3903BR04M	41.5	15.0	19.0	11,828
								139,639
43	Sulfer Recovery Unit	Plant 1	ND	4301BR01M	33.5	15.0	14.0	7,035
				4301BR02M	42.0	15.0	14.0	8,820
				4301BR03M	42.0	15.0	19.0	11,970
				4301BR04M	34.0	15.0	24.0	12,240
		Plant 2	ND	4302BR01M	33.5	15.0	14.0	7,035
				4302BR02M	42.0	15.0	14.0	8,820
				4302BR03M	42.0	15.0	19.0	11,970
				4302BR04M	34.0	15.0	24.0	12,240
		Plant 3	ND	4303BR01M	33.5	15.0	14.0	7,035
				4303BR02M	42.0	15.0	14.0	8,820
				4303BR03M	42.0	15.0	19.0	11,970
				4303BR04M	34.0	15.0	24.0	12,240
								120,195
60	Steam Generation		ND	6000AR01M	42.0	11.0	7.0	3,234
				6000AR02M	48.0	11.0	11.2	5,914
				6000AR03M	42.0	11.0	7.0	3,234
				6000AR04M	31.0	11.0	7.0	2,387
								14,769
61	Air Systems		ALK	6100ER01M	48.0	10.0	7.0	3,360
				6100ER02M	48.0	9.5	3.5	1,596
				6100ER03M	48.0	9.0	3.5	1,512
								6,468
62	Water Systems		DEC	6200HR01M	24.0	9.0	6.9	1,490
				6200HR02M	48.0	9.0	6.9	2,981
				6200HR03M	42.0	9.0	4.9	1,852
				6200HR04M	42.0	9.0	4.9	1,852
				6200HR05M	42.0	9.0	4.9	1,852
				6200HR06M	42.0	9.0	4.9	1,852
								11,880

Module Configuration Table - Pipe Racks

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Unit No.	Unit Name	Plant No.	Sub - Project (JV)	Module Number	Module Envelope Dimensions Length (m)	Width (M)	Height (m)	Volume (m3)
74	Interconnecting Pipe Rack	EPC-2	ND/AMS	7412AR01M	36.0	23.0	14.6	12,105
				7412AR02M	36.0	23.0	14.6	12,105
				7412AR03M	42.0	23.0	14.6	14,123
				7412AR04M	42.0	18.0	14.6	11,053
				7412AR05M	42.0	23.0	14.6	14,123
				7412AR06M	42.0	18.0	14.6	11,053
				7412AR07M	42.0	24.0	14.6	14,737
				7412AR08M	42.0	18.5	14.6	11,360
				7412AR09M	30.0	25.5	14.6	11,184
				7412AR10M	54.0	19.0	14.6	15,000
				7412AR11M	25.0	18.0	14.6	6,579
				7412AR12M	39.5	23.0	14.6	13,282
				7412AR13M	43.5	22.0	17.0	16,269
				7412AR14M	24.0	15.0	14.6	5,256
				7412AR15M	42.0	12.0	7.00	3,528
				7412AR16M	42.0	12.0	7.00	3,528
				7412AR17M	42.0	12.0	7.00	3,528
				7412AR18M	42.0	12.0	7.00	3,528
				7412AR19M	42.0	12.0	7.00	3,528
				7412AR20M	36.0	12.5	7.00	3,150
				7412AR21M	55.0	12.5	7.00	4,813
				7412AR22M	51.5	13.0	7.00	4,687
				7412AR23M	60.0	14.5	7.00	6,090
				7412AR24M	46.0	14.5	7.00	4,669
				7412AR25M	46.0	14.5	7.00	4,669
				7412AR26M	34.5	12.0	16.0	6,624
				7412AR28M	57.0	16.5	16.5	15,518
				7412AR29M	29.0	14.5	11.0	4,626
				7412AR30M	46.0	13.0	19.0	11,362
				7412AR32M	46.0	12.0	12.0	6,624
				7412AR33M	40.0	17.0	10.00	6,800
				7412AR34M	40.0	17.0	10.00	6,800
				7412BR01M	50.5	16.0	13.0	10,504
				7412BR02M	24.5	16.0	13.0	5,096
				7412BR03M	38.0	17.0	13.0	8,398
				7412BR04M	35.5	16.0	13.0	7,384
				7412BR05M	41.5	14.0	13.0	7,553
				7412BR06M	27.1	14.0	13.0	4,932
				7412BR07M	36.5	16.5	13.0	7,829
				7412BR08M	34.0	21.0	13.0	9,282
				7412BR09M	36.3	17.0	13.0	8,011
				7412BR10M	42.0	20.0	13.0	10,920
				7412BR11M	51.7	16.5	13.0	11,090
				7412BR13M	52.5	16.0	9.0	7,553
				7412BR14M	43.5	13.0	10.0	5,655
				7412BR15M	43.5	13.0	10.0	5,655
				7412BR16M	43.5	13.0	10.0	5,655

Module Configuration Table - Pipe Racks

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Unit No.	Unit Name	Plant No.	Sub - Project (JV)	Module Number	Module Envelope Dimensions Length (m)	Width (M)	Height (m)	Volume (m3)
74	Interconnecting Pipe Rack	EPC-3	ND	7413AR01M	34.0	8.0	6.0	1,632
				7413BR01M	23.0	10.0	8.0	1,840
				7413BR02M	37.0	8.0	8.0	2,368
				7413BR03M	42.0	13.0	8.0	4,368
				7413BR04M	42.0	13.0	8.0	4,368
				7413BR05M	30.0	8.0	8.0	1,920
				7413CR01M	24.0	16.0	9.0	3,456
				7413CR02M	38.0	18.0	12.0	8,208
				7413CR03M	42.0	21.0	9.0	7,938
				7413CR04M	36.0	12.0	9.0	3,888
				7413CR05M	25.0	15.0	9.0	3,375
				7413DR01M	34.0	9.0	6.0	1,836
				7413ER01M	49.0	19.0	10.0	9,310
				7413FR01M	34.0	15.0	6.0	3,060
				7413GR01M	42.0	10.0	9.0	3,780
				7413GR02M	42.0	13.0	9.0	4,914
				7413GR03M	30.0	10.0	9.0	2,700
				7413GR04M	28.0	12.0	9.0	3,024
				7413HR01M	34.0	12.0	6.0	2,448
				7413JR01M	42.0	10.0	9.0	3,780
				7413JR02M	42.0	15.0	9.0	5,670
				7413JR03M	42.0	10.0	9.0	3,780
				7413JR04M	42.0	13.0	9.0	4,914
				7413JR05M	42.0	10.0	9.0	3,780
				7413JR06M	42.0	13.0	9.0	4,914
				7413JR07M	24.0	10.0	9.0	2,160
				7413JR08M	28.0	10.0	9.0	2,520
				7413JR09M	36.0	10.0	9.0	3,240
				7413JR10M	42.0	10.0	10.0	4,200
				7413JR11M	42.0	12.0	10.0	5,040
				7413JR12M	26.0	10.0	10.0	2,600
				7413JR13M	36.0	10.0	9.0	3,240
				7413KR01M	42.0	10.0	11.0	4,620
				7413KR02M	42.0	11.0	11.0	5,082
				7413KR03M	42.0	10.0	11.0	4,620
				7413KR04M	31.0	11.0	11.0	3,751
				7413LR01M	34.0	9.0	6.0	1,836
								925,959
75	Flare Recovery		ALK	7500AR01M	36.0	10.0	14.0	5,040
								5,040
				TOTALS:				

APPENDIX E : WIND LOAD CALCULATION

Wind load calculation is done accordance with EN 1991-1-4 [5]. In this study, the Basic 10 minute mean wind velocity (V10) used for the design shall be 35 m/s and terrain category shall be 'II'.

Parameter	Symbol	Value	Reference
Basic wind velocity	V_b	35 m/s	at reference height 10m
Orography factor	$C_0(z)$	1.0	
Turbulence factor	k_1	1.0	
Air density	ρ	1.25 kg/m^3	
Roughness length	$z_{0,II}$	0.05m	for terrain category II
Min. roughness length	$z_{0,min,II}$	2m	for terrain category II

Tl. E-1 Wind parameters

E.1 Mean wind velocity

$$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b \qquad \text{(E. E-1)}$$

Where:

- v_b: Basic wind velocity = 35m/s (10 minute mean velocity at reference height 10m)
- $c_r(z) = k_r \cdot ln\left(\frac{z}{z_0}\right) \quad for \quad z_{min} \le z \le z_{max}$
- $c_r(z) = c_r(z_{\min}) \quad for \quad z \le z_{min}$
- $c_r(z)$: Roughness factor
- $c_o(z)$: Orography factor

Where:

- z_0: Roughness length
- $k_r = 0.19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0.07}$: Terrain factor

Where:

- $z_{0,II}$: 0.05m (Terrain category II)
- z_{min}: Minimum height

- z_{max}: 200m

Tl. E-2 shows the description of the terrain category.

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

Tl. E-2 Terrain category

In this thesis, Terrain category II is used and it is decided following the company design criteria.

E.2 Wind turbulence

$$I_v(z) = turbulence\ intensity \qquad \text{(E. E-2)}$$

$$I_v(z) = \frac{k_l}{c_o(z) \cdot ln\left(\frac{z}{z_0}\right)} \quad for \quad z_{min} \leq z \leq z_{max} \qquad \text{(E. E-3)}$$

$$I_v(z) = I_v(z_{min}) \;\; for \quad z < z_{min} \qquad \text{(E. E-4)}$$

Where:

k_l: Turbulence factor

c_o: Orography factor

z_0: Roughness length

E.3 Peak velocity pressure

$$q_p(z) = [1 + 7 \cdot I_v(z)] \cdot \frac{1}{2} \cdot \rho \cdot v_m^2(z) \tag{E. E-5}$$

Where:

$q_p(z)$: Peak velocity pressure

ρ: Air density

E.4 Wind load on the bottom columns

The wind force F_w acting on a bottom column was determined using force coefficients according to equation 5.3 of EN 1991-1-4.

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \tag{E. E-6}$$

Where:

$c_s c_d$: Structural factor from Chapter 6 of EN 1991-1-4

c_f: Force coefficient for the structure or structural element to be determined from section 7 of EN 1991-1-4. For rectangular structures with h/d=1, c_f is equal to 1.3 (=0.8+0.5) according to table 7.1 of EN 1991-1-4.

$q_p(z_e)$: Peak velocity pressure at reference height

z_e: The reference height, as determined in section 7 of EN 1991-1-4.

E.5 Wind load on upper part of pipe rack

For a simplification of calculation, wind on an open structure such as pipe rack can be designed as wind force acting on a closed structure by using a solidity ratio method.

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_g \cdot \varphi \tag{E. E-7}$$

Where:

$\varphi = A_c/A$: Solidity ratio, Effective solid area divided by the gross or envelope area and it's assumed as 0.75 for pipe rack based on the company experience. The same force coefficient as a closed structure will be utilized, i.e., c_f=1.3.

z	$v_b(z)$	cr(z)	c0(z)	kr	z0	z0,min	vm(z)	k1	Iv(z)	$q_p(z)$
m	m/s						m/s			kN/m2

0.00	35.00	0.70	1.00	0.19	0.05	2.00	24.53	1.00	0.27	1.09
1.00	35.00	0.70	1.00	0.19	0.05	2.00	24.53	1.00	0.27	1.09
2.00	35.00	0.70	1.00	0.19	0.05	2.00	24.53	1.00	0.27	1.09
3.00	35.00	0.78	1.00	0.19	0.05	2.00	27.23	1.00	0.24	1.26
4.00	35.00	0.83	1.00	0.19	0.05	2.00	29.14	1.00	0.23	1.38
5.00	35.00	0.87	1.00	0.19	0.05	2.00	30.62	1.00	0.22	1.48
6.00	35.00	0.91	1.00	0.19	0.05	2.00	31.84	1.00	0.21	1.56
7.00	35.00	0.94	1.00	0.19	0.05	2.00	32.86	1.00	0.20	1.63
8.00	35.00	0.96	1.00	0.19	0.05	2.00	33.75	1.00	0.20	1.69
9.00	35.00	0.99	1.00	0.19	0.05	2.00	34.53	1.00	0.19	1.75
10.00	35.00	1.01	1.00	0.19	0.05	2.00	35.23	1.00	0.19	1.80
11.00	35.00	1.02	1.00	0.19	0.05	2.00	35.87	1.00	0.19	1.85
12.00	35.00	1.04	1.00	0.19	0.05	2.00	36.45	1.00	0.18	1.89
13.00	35.00	1.06	1.00	0.19	0.05	2.00	36.98	1.00	0.18	1.93
14.00	35.00	1.07	1.00	0.19	0.05	2.00	37.47	1.00	0.18	1.97
15.00	35.00	1.08	1.00	0.19	0.05	2.00	37.93	1.00	0.18	2.00
16.00	35.00	1.10	1.00	0.19	0.05	2.00	38.36	1.00	0.17	2.04
17.00	35.00	1.11	1.00	0.19	0.05	2.00	38.76	1.00	0.17	2.07
18.00	35.00	1.12	1.00	0.19	0.05	2.00	39.14	1.00	0.17	2.10
19.00	35.00	1.13	1.00	0.19	0.05	2.00	39.50	1.00	0.17	2.12
20.00	35.00	1.14	1.00	0.19	0.05	2.00	39.84	1.00	0.17	2.15
21.00	35.00	1.15	1.00	0.19	0.05	2.00	40.17	1.00	0.17	2.18
22.00	35.00	1.16	1.00	0.19	0.05	2.00	40.48	1.00	0.16	2.20
23.00	35.00	1.16	1.00	0.19	0.05	2.00	40.77	1.00	0.16	2.23
24.00	35.00	1.17	1.00	0.19	0.05	2.00	41.06	1.00	0.16	2.25
25.00	35.00	1.18	1.00	0.19	0.05	2.00	41.33	1.00	0.16	2.27
26.00	35.00	1.19	1.00	0.19	0.05	2.00	41.59	1.00	0.16	2.29
27.00	35.00	1.20	1.00	0.19	0.05	2.00	41.84	1.00	0.16	2.31
28.00	35.00	1.20	1.00	0.19	0.05	2.00	42.08	1.00	0.16	2.33
29.00	35.00	1.21	1.00	0.19	0.05	2.00	42.31	1.00	0.16	2.35
30.00	35.00	1.22	1.00	0.19	0.05	2.00	42.54	1.00	0.16	2.37
31.00	35.00	1.22	1.00	0.19	0.05	2.00	42.76	1.00	0.16	2.39
32.00	35.00	1.23	1.00	0.19	0.05	2.00	42.97	1.00	0.15	2.40
33.00	35.00	1.23	1.00	0.19	0.05	2.00	43.17	1.00	0.15	2.42
34.00	35.00	1.24	1.00	0.19	0.05	2.00	43.37	1.00	0.15	2.44
35.00	35.00	1.24	1.00	0.19	0.05	2.00	43.56	1.00	0.15	2.45
36.00	35.00	1.25	1.00	0.19	0.05	2.00	43.75	1.00	0.15	2.47
37.00	35.00	1.26	1.00	0.19	0.05	2.00	43.93	1.00	0.15	2.48
38.00	35.00	1.26	1.00	0.19	0.05	2.00	44.11	1.00	0.15	2.50
39.00	35.00	1.27	1.00	0.19	0.05	2.00	44.28	1.00	0.15	2.51
40.00	35.00	1.27	1.00	0.19	0.05	2.00	44.45	1.00	0.15	2.53

Tl. E-3 Wind pressure respect to height

E.6 Calculate wind load to the bottom columns

H	qp(z)	Profile	Width	Depth	Cf	CsCd	Solidity	Trans	Long
m	kN/m2		m	m				kN/m	kN/m
7	1.63	UC 254 x 254 x 167	0.27	0.29	2.00	1.00	1.00	0.90	1.00

Tl. E-4 Loads on the bottom columns

E.7 Calculate wind loads of the upper parts of the pipe racks

H	qp(z)	Length	Height	Cf	CsCd	Solidity	Force
m	kN/m2	m	m				kN
9	1.75	6.00	2.00	1.30	1.13	0.75	24.0
11	1.85	6.00	2.00	1.30	1.13	0.75	25.0
13	1.93	6.00	2.00	1.30	1.13	0.75	26.0
15	2.00	6.00	2.00	1.30	1.13	0.75	27.0
17	2.07	6.00	2.00	1.30	1.13	0.75	28.0
19	2.12	6.00	2.00	1.30	1.13	0.75	29.0
21	2.18	6.00	2.00	1.30	1.13	0.75	29.0
23	2.23	6.00	2.00	1.30	1.13	0.75	30.0
25	2.27	6.00	2.00	1.30	1.13	0.75	31.0

Tl. E-5 Loads on the upper parts

APPENDIX F : SEA TRANSPORT LOAD CALCULATION

Here, the detail explanation how to find the maximum acceleration and the load from the acceleration are presented for the quasi-static analysis.

F.1 Accelerations at the maximum motion

In order to calculate the quasi-static force from a dynamic motion, the acceleration from the motion has to be known. For example, the equation below shows a rotational acceleration function for a roll motion.

$$\ddot{\theta}(t) = -\omega^2 \cdot \theta_a \cdot sin(\omega \cdot t) \tag{E. F-1}$$

For the quasi-static analysis, the sinus term is assumed as one. Therefore, the absolute value of rotational acceleration for a roll motion is as below.

- **Roll Rotational Acceleration**
  The maximum pitch rotational acceleration is shown in (E. F-2).

$$\alpha_{Roll} = \ddot{\theta} = \omega_{Roll}^2 \cdot \theta_a = 0.1378\ [rad/sec^2] \tag{E. F-2}$$

Where, $\omega = \frac{1}{T} \cdot 2\pi$ and θ_a is an amplitude of a roll motion.

θ_a and *T* are taken from the DNV default motion criteria Table 2-3.

In a same manner, acceleration for a pitch motion and a heave motion are found as below.

- **Pitch Rotational Acceleration**
  The maximum pitch rotational acceleration is shown in (E. F-3).

$$\alpha_{Pitch} = \ddot{\phi} = \omega_{Pitch}^2 \cdot \phi_a = 0.0861\ [rad/sec^2] \tag{E. F-3}$$

- **Heave Acceleration**
  The maximum heave acceleration is shown in (E. F-4).

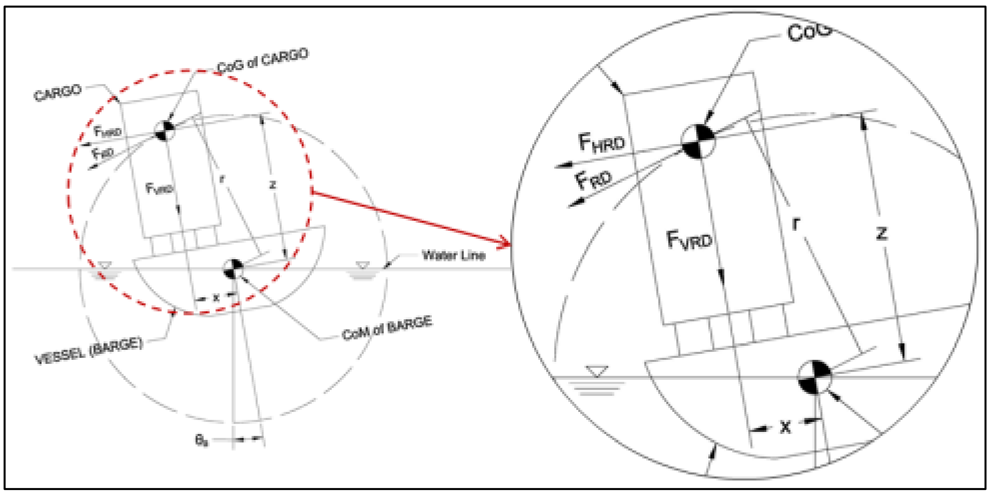
$$a_{Heave} = \ddot{z} = \omega_{Heave}^2 \cdot z_a = 1.9739\ [m/sec^2] \tag{E. F-4}$$

It should be noted the roll and pitch motions induce rotational accelerations [rad/s2]. In next subchapters, it is shown that how to calculate the forces from the motions.

It should be noted that for the gravity (self-weight) force calculation, the moment when the structure tilts with its maximum angle is chosen because it causes the maximum gravity force to the structure. Furthermore, for the heave motion, it is assumed that the heave occurs at the maximum roll or pitch angle to consider worst case which causes maximum force to the structure. In other word, the heave motion is assumed same phase with the roll and pitch motions.

F.2 Inertial force from roll motion

Fg. B-1 shows acceleration forces from a roll motion.



Fg. F-1 Acceleration forces from a roll motion

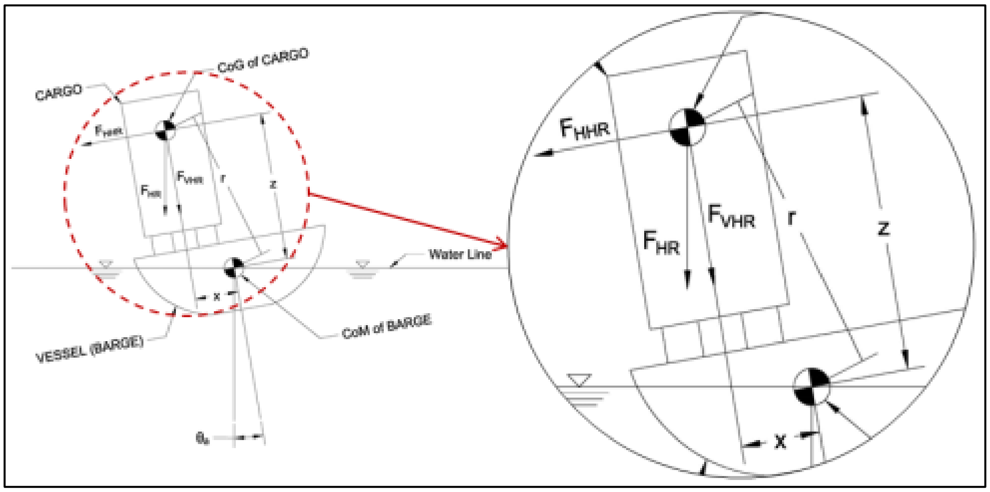
F_{HRD} is the horizontal roll acceleration force and F_{VRD} is the vertical roll acceleration force. They are expressed as in (E. F-5) and (E. F-6)(E. E-6 respectively.

$$F_{HRD} = m \cdot \ddot{\theta} \cdot z \; [N] \qquad \text{(E. F-5)}$$

$$F_{VRD} = m \cdot \ddot{\theta} \cdot x \; [N] \qquad \text{(E. F-6)}$$

F.3 Inertial force from heave motion at maximum roll angle

Fg. F-2 shows acceleration forces from a heave motion at maximum roll angle



Fg. F-2 Acceleration forces from a heave motion at the maximum roll angle

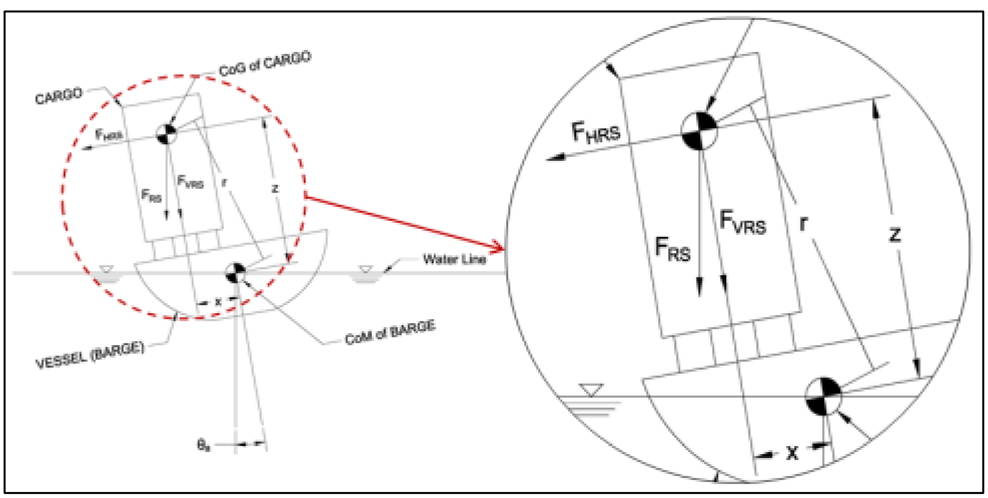
F_{HHR} is the horizontal heave acceleration force and F_{VHR} is the vertical heave acceleration force at the maximum roll angle. They are expressed as in (E. F-7) and (E. F-8) respectively.

$$F_{HHR} = m \cdot \ddot{z} \cdot sin(\theta_a)\ [N] \tag{E. F-7}$$

$$F_{VHR} = m \cdot \ddot{z} \cdot cos(\theta_a)\ [N] \tag{E. F-8}$$

F.4 Gravitational (self-weight) force at maximum roll angle

Fg. F-3 shows gravity forces from a roll motion.



Fg. F-3 Gravity (self-weight) forces from a roll motion

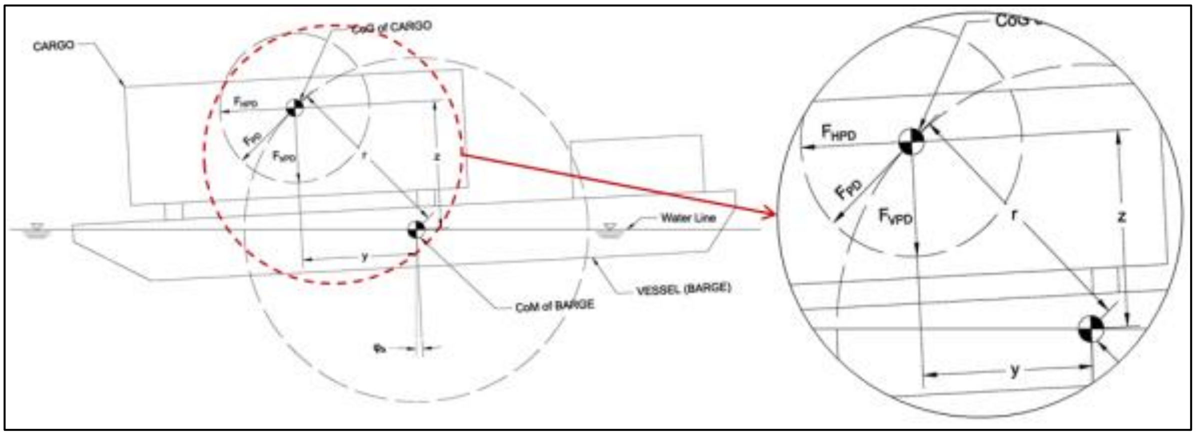
F_{HRG} is the horizontal roll gravity force and F_{VRG} is the vertical roll gravity force. They are expressed as in (E. F-9) and (E. F-10) respectively.

$$F_{HRG} = m \cdot g \cdot sin(\theta_a)\ [N] \qquad \text{(E. F-9)}$$

$$F_{VRG} = m \cdot g \cdot cos(\theta_a)\ [N] \qquad \text{(E. F-10)}$$

F.5 Inertial force from pitch motion

Fg. F-4 shows acceleration forces from a pitch motion.



Fg. F-4 Acceleration forces from a pitch motion

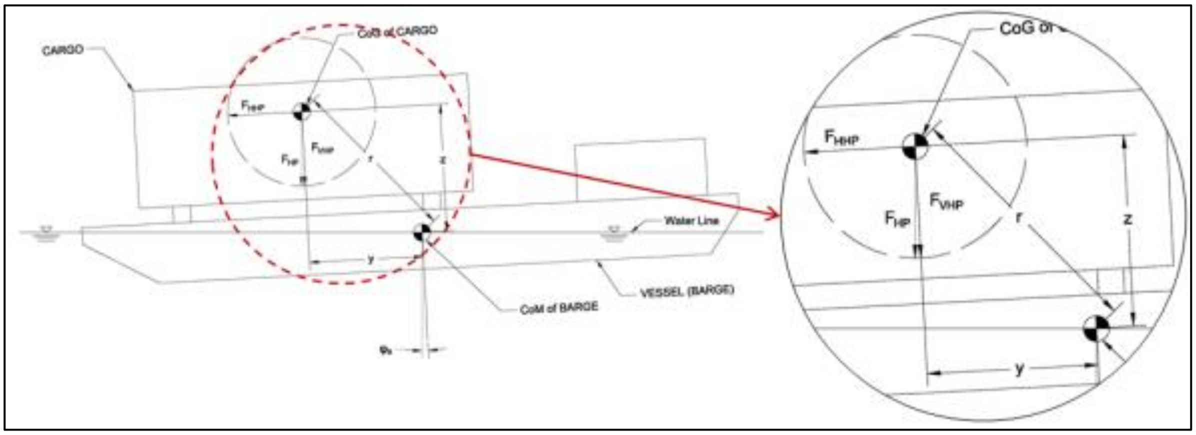
F_{HPD} is the horizontal pitch acceleration force and F_{VPD} is the vertical pitch acceleration force. They are expressed as in (E. F-11) and (E. F-12) respectively.

$$F_{HPD} = m \cdot \ddot{\varphi} \cdot z\ [N] \qquad \text{(E. F-11)}$$

$$F_{VPD} = m \cdot \ddot{\varphi} \cdot y\ [N] \qquad \text{(E. F-12)}$$

F.6 Inertial force from heave motion at maximum pitch angle

Fg. F-5 shows acceleration forces from a heave motion at maximum pitch angle.



Fg. F-5 Acceleration forces from heave motion at the maximum pitch angle

F_{HHP} is the horizontal heave acceleration force and F_{VHP} is the vertical heave acceleration force at the maximum pitch angle. They are expressed as in (E. F-13) and (E. F-14) respectively.

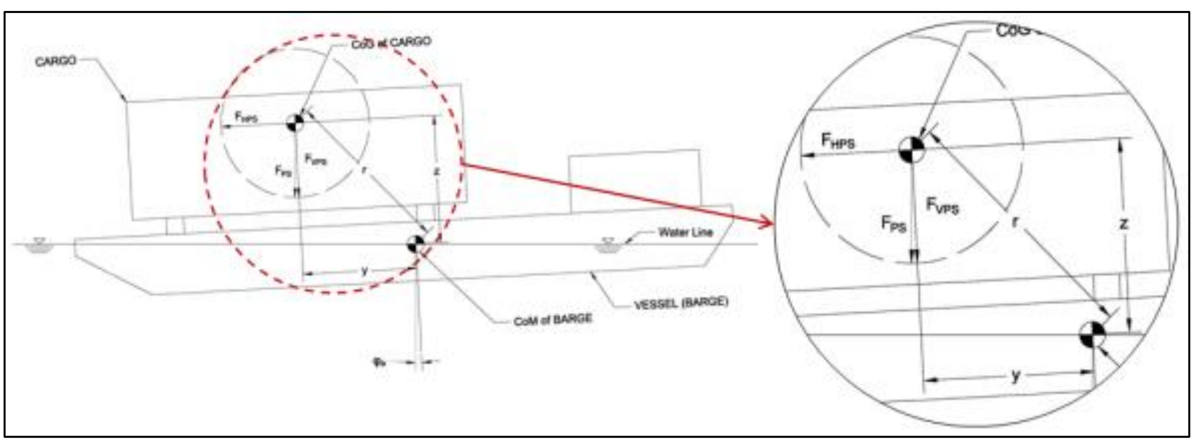
$$F_{HHP} = m \cdot \ddot{z} \cdot sin(\varphi_a)\ [N] \qquad \text{(E. F-13)}$$

$$F_{VHP} = m \cdot \ddot{z} \cdot cos(\varphi_a)\ [N] \qquad \text{(E. F-14)}$$

$$F_{VHP} = m \cdot \ddot{z} \cdot cos(\varphi_a)\ [N]$$

F.7 Gravitational (self-weight) force at maximum pitch angle

Fg. F-6 shows gravity force from a pitch motion.



Fg. F-6 Gravity (self-weight) forces from a pitch motion

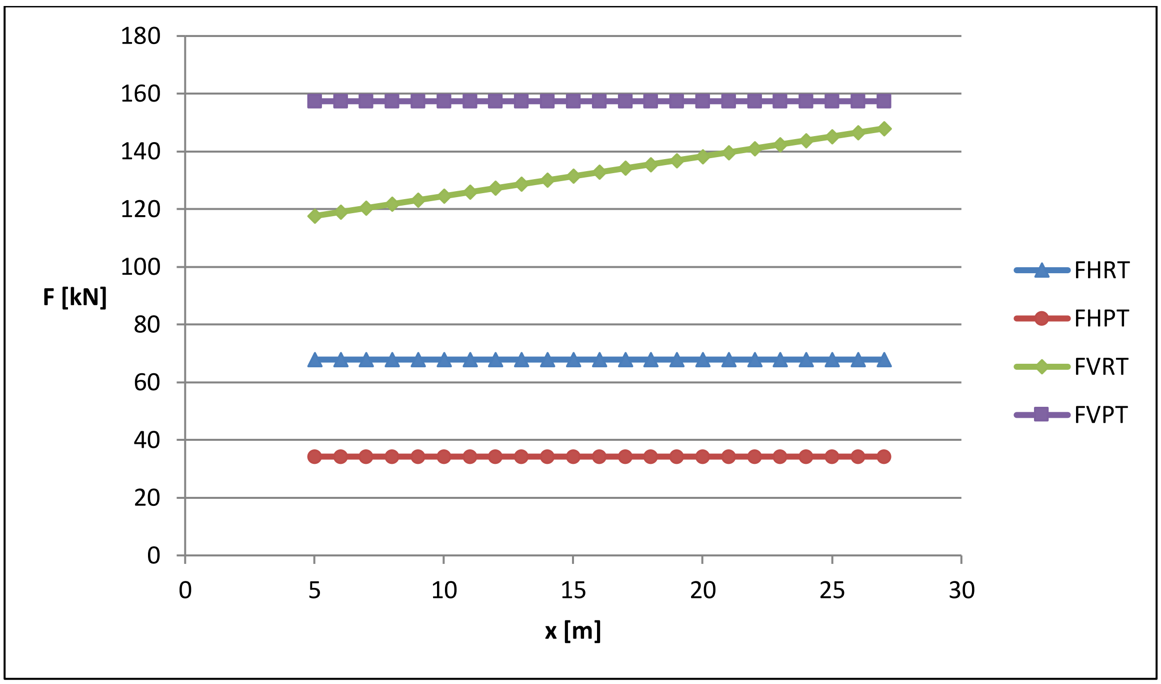
F_{HPG} is the horizontal pitch gravity force and F_{VPG} is the vertical pitch gravity force. They are expressed as in (E. F-15) and (E. F-16) respectively.

$$F_{HPG} = m \cdot g \cdot sin(\varphi_a) \; [N] \qquad \text{(E. F-15)}$$

$$F_{VPG} = m \cdot g \cdot cos(\varphi_a) \; [N] \qquad \text{(E. F-16)}$$

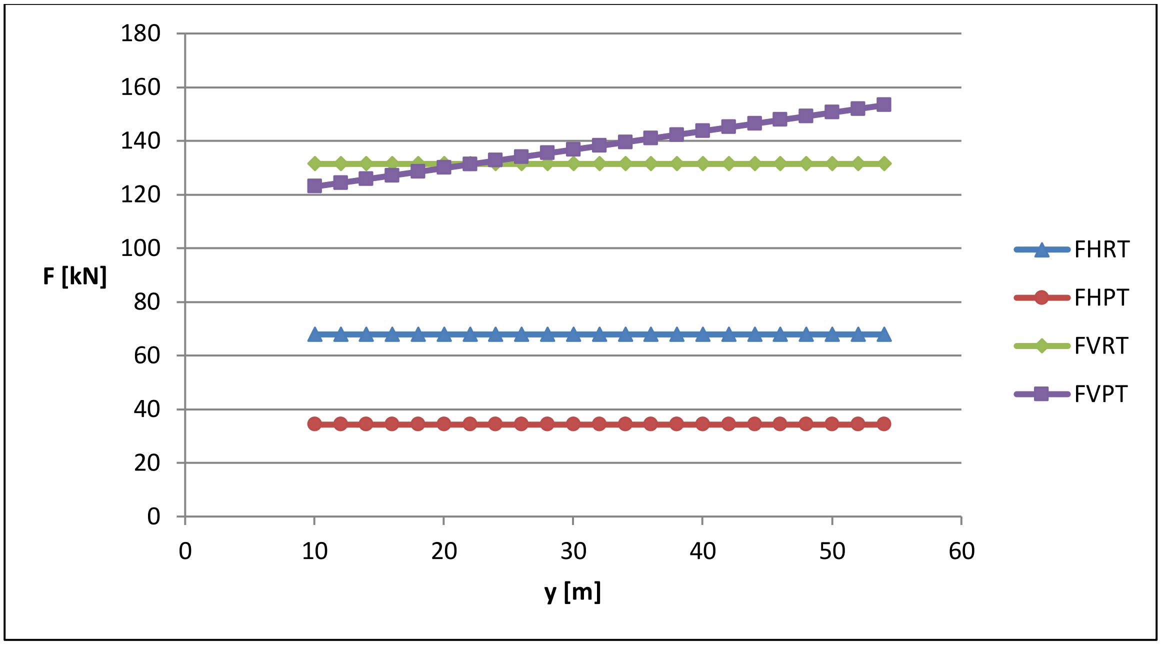
F.8 Sea transport loads respect to mass location

As a reference, it is checked that how total force was changing when its location is changing with respect to x, y and z direction. With this, it was found that what force is changing when the mass location changes. The mass was assumed 10 tons here.



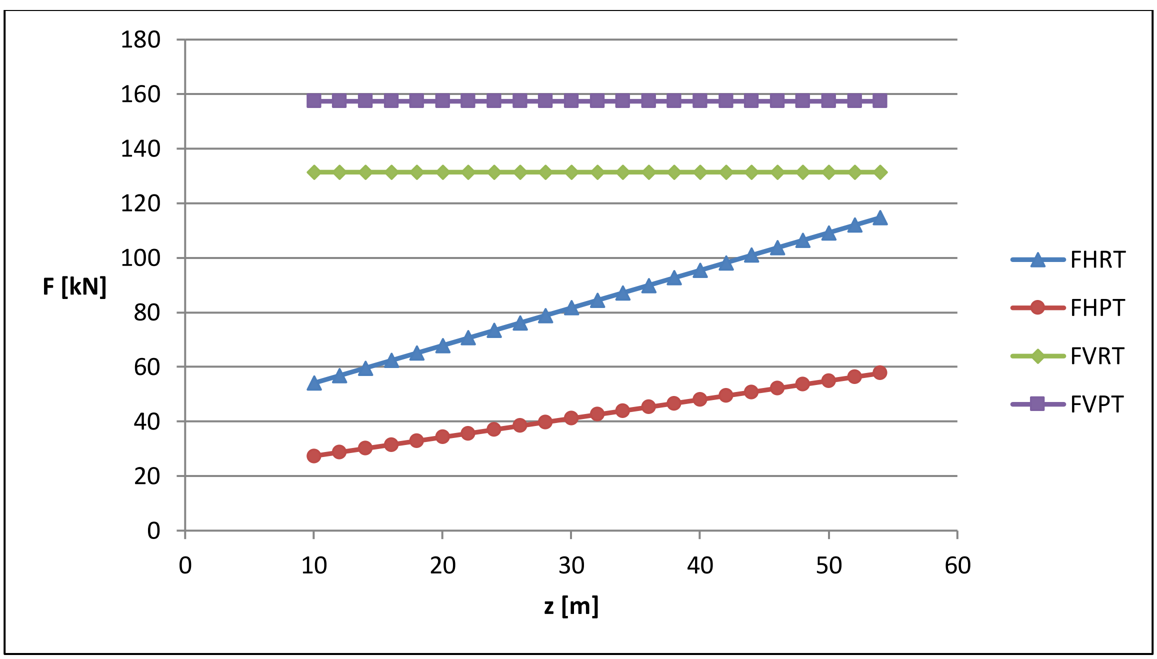
Fg. F-7 Force variation respect to x direction

Fg. F-7 shows that when location of mass changes along the x direction, only FVRT (total vertical force from roll motion) is changing.



Fg. F-8 Force variation respect to y direction

Fg. F-8 shows that when location of mass changes along the y direction, only FVPT (total vertical force due to pitching) is changing.



Fg. F-9 Force variation respect to the height

Fg. F-9 shows that when location of mass changes along the z direction, FHRT (total horizontal force due to rolling) and FHPT (total horizontal force from pitch motion) are changing. In the study, it is assumed that the pipe rack location on the vessel does not change, only the height of the pipe rack changes according to the choice of options which means only horizontal sea transport loads are changed.

F.9 Sea transport loads on the pipe rack

According to chapter 3.3.1 and Figure 3-5, sea transport loads on the pipe rack are calculated.

Option	Weight	Total ship motion Force				Acceleration Force				Heave				Gravity				2D			Coordinates		Acceleration	
			+		+		+		+		+		+		+		+							
			HF		VF		HF		VF		HF		VF		HF		VF	T	Roll	Heave	x	z	A_{ROLL}	A_{HEAVE}
	kN	X	kN	Z	kN/m	X	kN	Z	kN	X	kN	Z	kN	X	kN	Z	kN	sec	deg.	m	m	m	rad/sec²	m/sec²
1	70.0	X	34.8	Z	15.1	X	6.0	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	6.1	0.138	1.974
1	70.0	X	36.7	Z	15.1	X	8.0	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	8.1	0.138	1.974
1	70.0	X	38.7	Z	15.1	X	9.9	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	10.1	0.138	1.974
1	70.0	X	40.7	Z	15.1	X	11.9	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	12.1	0.138	1.974
1	70.0	X	42.6	Z	15.1	X	13.9	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	14.1	0.138	1.974
1	70.0	X	44.6	Z	15.1	X	15.8	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	16.1	0.138	1.974
1	70.0	X	46.6	Z	15.1	X	17.8	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	18.1	0.138	1.974
1	70.0	X	48.5	Z	15.1	X	19.8	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	20.1	0.138	1.974
1	70.0	X	50.5	Z	15.1	X	21.7	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	22.1	0.138	1.974
1	70.0	X	52.5	Z	15.1	X	23.7	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	24.1	0.138	1.974
2	70.0	X	40.4	Z	15.1	X	11.6	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	11.8	0.138	1.974
2	70.0	X	42.3	Z	15.1	X	13.6	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	13.8	0.138	1.974
2	70.0	X	44.3	Z	15.1	X	15.5	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	15.8	0.138	1.974
2	70.0	X	46.3	Z	15.1	X	17.5	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	17.8	0.138	1.974
2	70.0	X	48.2	Z	15.1	X	19.5	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	19.8	0.138	1.974
2	70.0	X	50.2	Z	15.1	X	21.4	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	21.8	0.138	1.974
2	70.0	X	52.2	Z	15.1	X	23.4	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	23.8	0.138	1.974
2	70.0	X	54.1	Z	15.1	X	25.4	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	25.8	0.138	1.974
2	70.0	X	56.1	Z	15.1	X	27.3	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	27.8	0.138	1.974
2	70.0	X	58.1	Z	15.1	X	29.3	Z	11.8	X	4.8	Z	13.2	X	23.9	Z	65.8	10.0	20.0	5.0	12.0	29.8	0.138	1.974

Tl. F-1 Roll motion loads

Option	Weight		Total ship motion Force				Acceleration Force				Heave				Gravity				2D			Coordinates		Acceleration	
			+			+		+		+		+		+		+		+							
			HF			VF		HF		VF		HF		VF		HF		VF	T	Pit.	Heave	y	z	A_{PITCH}	A_{HEAVE}
	kN	X	kN		Z	kN	X	kN	Z	kN	X	kN	Z	kN	X	kN	Z	kN	sec	deg.	m	m	m	rad/sec²	m/sec²
1	70.0	X	21.9		Z	100.5	X	3.7	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	6.1	0.086	1.974
1	70.0	X	23.2		Z	100.5	X	5.0	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	8.1	0.086	1.974
1	70.0	X	24.4		Z	100.5	X	6.2	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	10.1	0.086	1.974
1	70.0	X	25.6		Z	100.5	X	7.4	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	12.1	0.086	1.974
1	70.0	X	26.9		Z	100.5	X	8.7	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	14.1	0.086	1.974
1	70.0	X	28.1		Z	100.5	X	9.9	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	16.1	0.086	1.974
1	70.0	X	29.3		Z	100.5	X	11.1	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	18.1	0.086	1.974
1	70.0	X	30.6		Z	100.5	X	12.4	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	20.1	0.086	1.974
1	70.0	X	31.8		Z	100.5	X	13.6	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	22.1	0.086	1.974
1	70.0	X	33.0		Z	100.5	X	14.8	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	24.1	0.086	1.974
2	70.0	X	25.5		Z	100.5	X	7.3	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	11.8	0.086	1.974
2	70.0	X	26.7		Z	100.5	X	8.5	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	13.8	0.086	1.974
2	70.0	X	27.9		Z	100.5	X	9.7	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	15.8	0.086	1.974
2	70.0	X	29.1		Z	100.5	X	10.9	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	17.8	0.086	1.974
2	70.0	X	30.4		Z	100.5	X	12.2	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	19.8	0.086	1.974
2	70.0	X	31.6		Z	100.5	X	13.4	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	21.8	0.086	1.974
2	70.0	X	32.8		Z	100.5	X	14.6	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	23.8	0.086	1.974
2	70.0	X	34.1		Z	100.5	X	15.9	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	25.8	0.086	1.974
2	70.0	X	35.3		Z	100.5	X	17.1	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	27.8	0.086	1.974
2	70.0	X	36.5		Z	100.5	X	18.3	Z	18.4	X	3.0	Z	13.8	X	15.2	Z	68.3	10.0	12.5	5.0	30.0	29.8	0.086	1.974

Tl. F-2 Pitch motion loads

From Tl. F-1 and Tl. F-2, it is known that the ratio of the horizontal accelerations force becomes larger with the increase of the height which means the horizontal acceleration force becomes more important for higher structure.

Weight	**Distance from the center of the vessel motion**		**Horizontal Force**		**Vertical Force**	
	x	**z**				
70 kN	12 m	6.1 m (0.3m + 5.8m)	34.8 kN	-z	15.1 kN/m	-x
70 kN	12 m	8.1 m (2.3m + 5.8m)	36.7 kN	-z	15.1 kN/m	-x
70 kN	12 m	10.1 m (4.3m + 5.8m)	38.7 kN	-z	15.1 kN/m	-x
70 kN	12 m	12.1 m (6.3m + 5.8m)	40.7 kN	-z	15.1 kN/m	-x
70 kN	12 m	14.1 m (8.3m + 5.8m)	42.6 kN	-z	15.1 kN/m	-x
70 kN	12 m	16.1 m (10.3m + 5.8m)	44.6 kN	-z	15.1 kN/m	-x
70 kN	12 m	18.1 m (12.3m + 5.8m)	46.6 kN	-z	15.1 kN/m	-x
70 kN	12 m	20.1 m (14.3m + 5.8m)	48.5 kN	-z	15.1 kN/m	-x
70 kN	12 m	22.1 m (16.3m + 5.8m)	50.5 kN	-z	15.1 kN/m	-x
70 kN	12 m	24.1 m (18.3m + 5.8m)	52.5 kN	-z	15.1 kN/m	-x

Tl. F-3 Roll motion loads of option 1

Weight	**Distance from the center of the vessel motion**		**Horizontal Force**		**Vertical Force**	
	y	**z**				
70 kN	30 m	6.1 m (0.3m + 5.8m)	21.9 kN	-z	100.5 kN	-x
70 kN	30 m	8.1 m (2.3m + 5.8m)	23.2 kN	-z	100.5 kN	-x
70 kN	30 m	10.1 m (4.3m + 5.8m)	24.4 kN	-z	100.5 kN	-x

70 kN	30 m	12.1 m (6.3m + 5.8m)	25.6 kN	-z	100.5 kN	-x
70 kN	30 m	14.1 m (8.3m + 5.8m)	26.9 kN	-z	100.5 kN	-x
70 kN	30 m	16.1 m (10.3m + 5.8m)	28.1 kN	-z	100.5 kN	-x
70 kN	30 m	18.1 m (12.3m + 5.8m)	29.3 kN	-z	100.5 kN	-x
70 kN	30 m	20.1 m (14.3m + 5.8m)	30.6 kN	-z	100.5 kN	-x
70 kN	30 m	22.1 m (16.3m + 5.8m)	31.8 kN	-z	100.5 kN	-x
70 kN	30 m	24.1 m (18.3m + 5.8m)	33.0 kN	-z	100.5 kN	-x

Tl. F-4 Pitch motion loads of option 1

Weight	**Distance from the center of the vessel motion**		**Horizontal Force**		**Vertical Force**	
	x	**z**				
70 kN	12 m	11.8 m (6m + 5.8m)	40.4 kN	-z	15.1 kN/m	-x
70 kN	12 m	13.8 m (8m + 5.8m)	42.3 kN	-z	15.1 kN/m	-x
70 kN	12 m	15.8 m (10m + 5.8m)	44.3 kN	-z	15.1 kN/m	-x
70 kN	12 m	17.8 m (12m + 5.8m)	46.3 kN	-z	15.1 kN/m	-x
70 kN	12 m	19.8 m (14m + 5.8m)	48.2 kN	-z	15.1 kN/m	-x
70 kN	12 m	21.8 m (16m + 5.8m)	50.2 kN	-z	15.1 kN/m	-x
70 kN	12 m	23.8 m (18m + 5.8m)	52.2 kN	-z	15.1 kN/m	-x
70 kN	12 m	25.8 m (20m + 5.8m)	54.1 kN	-z	15.1 kN/m	-x
70 kN	12 m	27.8 m (22m + 5.8m)	56.1 kN	-z	15.1 kN/m	-x
70 kN	12 m	29.8 m (24m + 5.8m)	58.1 kN	-z	15.1 kN/m	-x

Tl. F-5 Roll motion loads of option 2 & 3

Weight	**Distance from the center of the vessel motion**		**Horizontal Force**		**Vertical Force**	
	y	**z**				
70 kN	30 m	11.8 m (6m + 5.8m)	25.5 kN	-z	100.5 kN	-x
70 kN	30 m	13.8 m (8m + 5.8m)	26.7 kN	-z	100.5 kN	-x
70 kN	30 m	15.8 m (10m + 5.8m)	27.9 kN	-z	100.5 kN	-x
70 kN	30 m	17.8 m (12m + 5.8m)	29.1 kN	-z	100.5 kN	-x
70 kN	30 m	19.8 m (14m + 5.8m)	30.4 kN	-z	100.5 kN	-x
70 kN	30 m	21.8 m (16m + 5.8m)	31.6 kN	-z	100.5 kN	-x
70 kN	30 m	23.8 m (18m + 5.8m)	32.8 kN	-z	100.5 kN	-x
70 kN	30 m	25.8 m (20m + 5.8m)	34.1 kN	-z	100.5 kN	-x
70 kN	30 m	27.8 m (22m + 5.8m)	35.3 kN	-z	100.5 kN	-x
70 kN	30 m	29.8 m (24m + 5.8m)	36.5 kN	-z	100.5 kN	-x

Tl. F-6 Pitch motion loads of option 2 & 3

APPENDIX G : STEEL QUANTITIES AND COST

G.1 Steel quantity of portal side frames

Dimensions	In-place		Option 1		Options 2		Options 3	
	Ton	%	Ton	%	Ton	%	Ton	%
W6xH12	3.1	100	3.1	100	3.6	116	4.0	129
W6xH18	7.3	100	7.3	100	8.1	111	8.5	116
W6xH24	13.5	100	13.5	100	15.4	114	16.9	125
W12xH12	4.6	100	4.7	102	5.7	124	7.1	154
W12xH18	10.6	100	10.6	100	12.9	122	14.2	134
W12xH24	16.3	100	17.3	106	20.4	125	23.6	145
W24xH12	8.0	100	8.1	101	10.2	128	13.4	168
W24xH18	18.0	100	19.0	106	22.9	127	26.8	149
W24xH24	27.8	100	32.3	116	38	137	44.1	159

Tl. G-1 Summary of portal side steel quantity (Pinned supports)

Dimensions	In-place		Option 1		Options 2		Options 3	
	Ton	%	Ton	%	Ton	%	Ton	%
W6xH12	3.1	100	3.1	100	3.6	116	3.5	113
W6xH18	7.3	100	7.3	100	8.1	111	8.2	112
W6xH24	13.5	100	13.5	100	15.4	114	15	111
W12xH12	4.6	100	4.7	102	5.7	124	5.8	126
W12xH18	10.6	100	10.6	100	12.9	122	12.4	117
W12xH24	16.3	100	17.3	106	20.4	125	20.8	128
W24xH12	8.0	100	8.1	101	10.2	128	10.4	130
W24xH18	18.0	100	19.0	106	22.9	127	22.3	124
W24xH24	27.8	100	32.3	116	38	137	38.3	138

Tl. G-2 Summary of portal side steel quantity (Clamped supports)

Dimensions	Options 2 – Option 1		Options 3 – Option 1	
	Ton	%	Ton	%
W6xH12	0.5	16	0.9	29
W6xH18	0.8	11	1.2	16
W6xH24	1.9	14	3.4	25
W12xH12	1	22	2.4	52
W12xH18	2.3	22	3.6	34
W12xH24	3.1	19	6.3	39
W24xH12	2.1	27	5.3	67
W24xH18	3.9	21	7.8	43
W24xH24	5.7	21	11.8	43

Tl. G-3 Comparison for portal side frames between option 1 and options 2 & 3 (Pinned supports)

Dimensions	Options 2 – Option 1		Options 3 – Option 1	
	Ton	**%**	**Ton**	**%**
W6xH12	0.5	16	0.4	13
W6xH18	0.8	11	0.9	12
W6xH24	2.0	14	1.5	11
W12xH12	1.0	21	1.1	23
W12xH18	2.3	22	1.8	17
W12xH24	3.1	18	3.5	20
W24xH12	2.1	26	2.3	28
W24xH18	3.9	21	3.3	17
W24xH24	5.7	18	6	19

Tl. G-4 Comparison for portal side frames between option 1 and options 2 & 3 (Clamped supports)

G.2 Steel quantity of bracing side frames

Dimensions	In-place		Option 1		Options 2 & 3	
	Ton	**%**	**Ton**	**%**	**Ton**	**%**
L24xH12	5.5	100	6.0	109	6.5	118
L24xH18	8.9	100	11.0	124	11.6	130
L24xH24	12.5	100	16.8	134	17.7	142
L36xH12	8.3	100	9.0	108	9.4	113
L36xH18	13.3	100	16.8	126	17.5	132
L36xH24	18.4	100	24.2	132	25.5	139
L60xH12	12.4	100	13.9	112	15.7	127
L60xH18	22.2	100	27.2	123	30.3	136
L60xH24	29.3	100	39.9	136	42.6	145

Tl. G-5 Summary of bracing side steel quantity (Pinned supports)

Dimensions	In-place		Option 1		Options 2 & 3	
	Ton	**%**	**Ton**	**%**	**Ton**	**%**
L24xH12	5.5	100	6.0	109	6.5	118
L24xH18	8.9	100	11.0	124	11.6	130
L24xH24	12.5	100	16.8	134	17.7	142
L36xH12	8.3	100	9.0	108	9.4	113
L36xH18	13.3	100	16.8	126	17.5	132
L36xH24	18.4	100	24.2	132	25.5	139
L60xH12	12.4	100	13.9	112	15.7	127
L60xH18	22.2	100	27.2	123	30.3	136
L60xH24	29.3	100	39.9	136	42.6	145

Tl. G-6 Summary of bracing side steel quantity (Clamped supports)

Dimensions	Option 2 & 3 – Option 1	
	Ton	%
L24xH12	0.5	8.3
L24xH18	0.6	5.5
L24xH24	0.9	5.4
L36xH12	0.4	4.4
L36xH18	0.3	4.2
L36xH24	1.3	5.4
L60xH12	1.8	13.0
L60xH18	3.1	11.4
L60xH24	2.2	6.8

Tl. G-7 Comparison for bracing side frames between options 1 and 2 & 3 (Pinned supports)

Dimensions	Option 2 & 3 – Option 1	
	Ton	%
L24xH12	0.5	8.3
L24xH18	0.6	5.5
L24xH24	0.9	5.4
L36xH12	0.4	4.4
L36xH18	0.3	4.2
L36xH24	1.3	5.4
L60xH12	1.8	13.0
L60xH18	3.1	11.4
L60xH24	2.2	6.8

Tl. G-8 Comparison for bracing side frames between options 1 and 2 & 3 (Clamped supports)

G.3 Steel quantity of each configuration

Dimensions			In-place [Ton]	Pinned supported			Clamped supported		
				Op. 1 [Ton]	Op. 2 [Ton]	Op. 3 [Ton]	Op. 1 [Ton]	Op. 2 [Ton]	Op. 3 [Ton]
W6	H12	L24	27	28	31	33	28	31	31
W6	H18	L24	54	59	63	66	55	63	66
W6	H24	L24	93	101	115	123	101	113	111
W12	H12	L24	40	42	47	55	42	48	48
W12	H18	L24	80	86	98	106	86	99	97
W12	H24	L24	119	137	157	175	137	155	157
W24	H12	L24	67	71	82	99	71	83	84
W24	H18	L24	135	150	171	192	150	173	170
W24	H24	L24	201	246	282	315	246	279	280
W6	H12	L36	38	40	44	47	40	44	44
W6	H18	L36	78	85	92	95	85	92	93
W6	H24	L36	132	143	159	169	143	159	156
W12	H12	L36	57	60	68	78	60	68	69
W12	H18	L36	114	125	142	152	125	143	140
W12	H24	L36	169	194	216	242	194	220	222
W24	H12	L36	97	102	117	141	102	118	120
W24	H18	L36	193	217	247	277	217	248	244
W24	H24	L36	287	348	389	436	348	393	396
W6	H12	L60	59	62	71	76	62	71	70
W6	H18	L60	125	135	149	154	135	149	151
W6	H24	L60	208	229	256	271	229	255	251
W12	H12	L60	88	93	109	125	93	110	111
W12	H18	L60	183	198	230	247	198	233	228
W12	H24	L60	267	310	347	388	310	353	357
W24	H12	L60	150	159	189	226	159	191	193
W24	H18	L60	309	345	400	447	345	404	397
W24	H24	L60	452	555	624	698	555	631	635

Tl. G-9 Summary of steel quantities

G.4 Steel work cost of each configuration

In Tl. G-10, it is possible to see steel costs for the 27 pipe racks of each option. For example, if the structure is pinned supported a structure which size is W24 x H24 x L60, costs 1,004,731 USD for option 1 while it's 1,129,983 for option 2 and 1,263,119 for option 3.

Dimensions			In-place [USD]	Pinned supported			Clamped supported		
				Op. 1 [USD]	Op. 2 [USD]	Op. 3 [USD]	Op. 1 [USD]	Op. 2 [USD]	Op. 3 [USD]
W6	H12	L24	48,146	49,956	55,205	59,549	49,956	56,291	55,567
W6	H18	L24	98,283	105,885	114,573	119,098	100,093	114,392	118,917
W6	H24	L24	167,787	183,172	208,874	221,725	183,172	203,625	200,186
W12	H12	L24	71,676	75,115	85,613	98,826	75,115	87,061	87,604
W12	H18	L24	144,257	155,841	177,561	191,136	155,841	179,552	175,751
W12	H24	L24	215,028	247,427	283,627	316,750	247,427	281,274	284,894
W24	H12	L24	121,994	128,148	148,239	179,190	128,148	150,773	152,764
W24	H18	L24	243,445	271,319	309,329	347,882	271,319	312,587	306,976
W24	H24	L24	364,534	444,355	510,058	570,512	444,355	504,085	507,162
W6	H12	L36	69,323	72,038	79,097	85,251	72,038	80,002	78,916
W6	H18	L36	140,818	153,488	165,615	172,312	153,488	165,615	167,787
W6	H24	L36	238,196	259,373	288,514	306,614	259,373	287,428	282,722
W12	H12	L36	103,351	108,419	122,537	141,180	108,419	123,623	124,528
W12	H18	L36	206,883	225,888	256,839	275,844	225,888	258,287	253,038
W12	H24	L36	306,252	350,597	391,141	437,477	350,597	397,476	402,544
W24	H12	L36	175,751	184,620	212,132	255,210	184,620	214,304	217,019
W24	H18	L36	348,968	392,951	446,346	500,465	392,951	449,423	441,459
W24	H24	L36	518,927	628,975	704,452	788,979	628,975	711,873	716,036
W6	H12	L60	106,971	112,220	127,605	137,198	112,220	129,053	127,243
W6	H18	L60	225,888	243,988	269,328	279,464	243,988	270,233	273,853
W6	H24	L60	375,575	413,947	462,636	491,053	413,947	461,007	453,586
W12	H12	L60	159,280	168,873	197,471	226,612	168,873	199,100	200,548
W12	H18	L60	331,954	359,104	416,843	446,708	359,104	420,825	412,499
W12	H24	L60	482,727	560,738	628,613	701,375	560,738	638,387	646,351
W24	H12	L60	271,138	287,609	341,547	409,241	287,609	344,805	349,149
W24	H18	L60	559,833	624,269	723,819	808,708	624,269	731,240	718,751
W24	H24	L60	818,120	1,004,731	1,129,983	1,263,199	1,004,731	1,141,748	1,148,445

Tl. G-10 Summary of steel work costs

Tl. G-11 shows the ratios between the options.

Dimensions			In-place [%]	Pinned supported			Clamped supported		
				Op. 1 [%]	Op. 2 [%]	Op. 3 [%]	Op. 1 [%]	Op. 2 [%]	Op. 3 [%]
W6	H12	L24	100	104	115	124	104	117	115
W6	H18	L24	100	108	116	121	108	117	119
W6	H24	L24	100	109	125	132	109	121	119
W12	H12	L24	100	105	119	138	105	121	122
W12	H18	L24	100	108	123	132	108	124	122
W12	H24	L24	100	115	132	147	115	131	132
W24	H12	L24	100	105	122	147	105	124	125
W24	H18	L24	100	111	127	143	111	128	126
W24	H24	L24	100	122	140	157	122	138	139
W6	H12	L36	100	104	114	123	104	115	114
W6	H18	L36	100	109	118	122	109	118	119
W6	H24	L36	100	109	121	129	109	121	119
W12	H12	L36	100	105	119	137	105	120	121
W12	H18	L36	100	109	124	133	109	125	122
W12	H24	L36	100	114	128	143	114	130	131
W24	H12	L36	100	105	121	145	105	122	123
W24	H18	L36	100	113	128	143	113	129	127
W24	H24	L36	100	121	136	152	121	137	138
W6	H12	L60	100	105	119	128	105	121	119
W6	H18	L60	100	108	119	124	108	120	121
W6	H24	L60	100	110	123	131	110	123	121
W12	H12	L60	100	106	124	142	106	125	126
W12	H18	L60	100	108	126	135	108	127	124
W12	H24	L60	100	116	130	145	116	132	134
W24	H12	L60	100	106	126	151	106	127	129
W24	H18	L60	100	111	129	144	111	131	128
W24	H24	L60	100	123	138	154	123	140	140

Tl. G-11 Summary of ratios

For the clamped supported pipe racks, the cost difference between option 1&2 and 1&3 are almost same whereas for the pinned supported pipe racks the differences between option 1&3 is almost two times bigger than options 1&2.