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SEISMIC RESPONSE OF 4-LEGGED FIXED OFFSHORE STRUCTURE IN MALAYSIA DUE TO SURROUNDING EARTHQUAKE

HONG JIA HI

Thesis submitted in fulfilment of the requirements
for the award of the degree of
B.Eng (Hons.) Civil Engineering

Faculty of Civil Engineering and Earth Resources
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JULY 2015

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I hereby declare that I have checked this thesis and in my opinion, this thesis is adequate in terms of scope and quality for the award of the degree of Bachelor of Civil Engineering (Hons.).

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Dedicated to my parents,

for their love and devotion

making me be who I am today

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ABSTRACT

Fixed offshore structure in Malaysia region are placing more emphasis on wind and wave effects analysis rather than seismic effect. However, Malaysia actually experienced tremors due to the earthquakes occurred in the neighbouring countries. Thus, the main objective of this study is tantamount to investigate the seismic vulnerability of existing fixed offshore structures in Malaysia region and determine the necessity of seismic design consideration for offshore structure in Malaysia. With this, a finite element seismic response simulation of a typical 4-legged fixed offshore structure using SAP2000 has been presented. Moreover, free vibration, time history and response spectrum analysis have been carried out and compared throughout this study. However, there is an assumption have been made when doing the 3D model of the structure. The fixed offshore platform structure is fixed to the ground instead of pilled and the soil interaction was neglected. Generally, fixed offshore structures in Malaysia region are capable of resisting this low seismic activity based on the study. This happens because the design of fixed offshore structures for environmental loading, can provide sufficient resistance against potential low seismic effects.

ABSTRAK

Struktur luar pantai di sekitar Malaysia direkabentuk dengan menekankan pada analisis angin dan kesan gelombang berbanding dengan kesan gempa bumi. Namun demikian, Malaysia sebenarnya mengalami gegaran akibat daripada gempa bumi yang berlaku di negara-negara jiran. Oleh itu, objektif utama kajian ini adalah untuk menyiasat ketahanlasakan dan kelemahan struktur luar pantai di sekitar Malaysia dan seterusnya menentukan keperluan seismik rekabentuk balasan bagi struktur sekitar Malaysia. Dengan ini, simulasi respons seismik tetap struktur luar pantai telah dibentangkan dengan menggunakan SAP2000. Selain itu, percuma getaran, masa sejarah dan analisis spektrum respons telah dijalankan dan berbanding sepanjang kajian ini. Walau bagaimanapun, terdapat suatu andaian yang telah dibuat apabila melakukan model 3D struktur. Struktur luar pantai tetap ke tanah bukannya menggunakan pilled dan interaksi tanah diabaikan. Secara umum, struktur luar pantai di rantau Malaysia berupaya melawan aktiviti seismik rendah berdasarkan kajian yang dijalankan. Ini adalah kerana rekabentuk struktur luar pantai mengambilkira beban alm sekitar yang agak berbeza dan besar daripada struktur biasa dan ini memberikan keupayaan lebih kepada struktur luar pantai ini untuk menanggung beban gempa bumi yang rendah.

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LIST OF SYMBOLS

mm	Millimeter
mm^2	Millimeter square
mm^3	Millimeter cube
mm^4	Bisquare Millimeter
s	second
kg	Kilogram
kg/m^3	Kilogram per meter cube
N	Newton
kN	Kilo newton
MN	Mega Newton
N/m	Newton per meter
N/mm^2	Newton per millimeter square
kN/m^2	Kilo newton per meter square
kN/m^3	Kilo newton per meter cube
kNm	Kilo newton meter
m/s	Meter per second
g	Gal
Hz	Hertz

LIST OF ABBREVIATIONS

EN	European Standards
ASTM	American Society for Testing and Materials
AISC	American Institute of Steel Construction Specification for design, fabrication and erection of structural steel building
M_L	Local Magnitude Scale
M_S	Surface Wave Magnitude Scale
M_w	Moment Magnitude Scale
API	American Petroleum Institute
V	Self-weight of the topside and structure
M_V	Moment with eccentric loading
L_B	Lateral loads on structure due to wind
L_C	Lateral loads on structure due to currents
L_W	Lateral loads on structure due to waves
M_B	Moments that is related to wind lateral loadings
M_C	Moments that is related to current lateral loadings
L_W	Cyclic loading caused by waves
M_W	Cyclic moment caused by waves
F	Wind force
ρ	Mass density of air
U	Wind speed
A	Area
F_w	Hydrodynamic force vector per unit length
F_D	Drag force per unit length

F_I	Inertial force per unit length
C_d	Drag coefficient
w	Density of water
C_m	Coefficient of inertia
k	Stiffness
T	Natural Period
f	Natural Frequency
MMD	Meteorological Malaysia Department
E	Young Modulus
G	Shear Modulus
FVA	Free Vibration Analysis
DL	Dead Load
LL	Live Load
EL	Environmental Load
TH	Time History
RS	Response Spectrum
I	Moment of Inertia
f_y	Yield Strength
f_u	Ultimate Strength
V_{Ed}	Maximum design shear force
$V_{c,Rd}$	Shear resistance
f_v	Shear stress
F_v	Allowable shear stress
M_{ed}	Maximum external design moment
M_{rd}	Moment resistance

f_b	Bending stress
F_b	Allowable bending stress
t	Thickness
d	Diameter

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND OF STUDY

Earthquakes are one of the world's most devastating and frightening natural disasters. Undoubtedly, we are deeply clear of the hazards, effect and damage caused by this unpredicted earthquake. Basically, earthquake does not kill people, but collapsed buildings and their content do. The greatest hazard in the earthquake is the collapse or fall of man-made and natural structures that caused the extensive loss of life and property. As a result, the seismic effects may not only consider in the countries that have a high risk of a strong earthquake, but also for those countries that are subject to low-to-moderate earthquake for instance like Malaysia since the power of the earthquake has shown us it is unpredicted (Ramli & Adnan, 2004).

Most of the Malaysians may feel that the country is generally free from any major active seismic activities, as a consequence of its strategic location. In fact, positioned at the periphery of the ring of fire and beside the Philippines and Indonesia, two neighbouring countries which have seen violent occurrences of seismological activities, the possibility of being jolted by moderate earthquake cannot be excluded. Moreover, Malaysian Meteorological Department (MetMalaysia) had detected the occurrence of eight earthquakes in East Malaysia which in the magnitude range of 2 to 4.5 Richter scale in the year 2012 (The Malaysian Insider, 2013). Even the Malaysia Peninsula has experienced earthquakes of local origin which associated with active fault that happens in Bukit Tinggi area from year 2007 to 2009 (New Straits Times, 2012).

The exploration and production activities in oil and gas industry remain vital for economy in Malaysia, where the fixed offshore platforms involve the most in the operation (Aulov & Liew, 2013). But then, the current Malaysian offshore structural design practice focused more on wind and wave effects analysis rather than seismic effect, even the parts of Sabah and Sarawak coastal waters are very close to the seismically active zone.

1.2 PROBLEM STATEMENT

Malaysia is located beyond the seismically active zones, but it is still questionable whether the numerous fixed offshore structures in the Malaysia region shall be designed to withstand an earthquake ground motion. In fact, parts of Sabah and Sarawak coastal waters are very close to the seismically active area and we experienced and felt the tremors truly owing to the earthquakes occurred in the neighbouring countries.

The current Malaysian offshore structural design practices focus more on wind and wave effects analysis rather than seismic effect. But, we cannot ensure the fixed offshore structure is safe at a specific level of earthquake acceleration. Therefore, the necessity of seismic design consideration for fixed offshore structure in Malaysia due to surrounding earthquake should be determined.

1.3 RESEARCH OBJECTIVE

There are many matters that require to be analysed in this research, but the main objectives of this research are:

i. To apply surrounding earthquake ground motion for analysis of an offshore structure in Malaysia

ii. To determine the seismic vulnerability of existing offshore structure in Malaysia

iii. To identify the necessity of the implementation of seismic design consideration for offshore structure in Malaysia due to surrounding earthquake

iv. To determine the seismic design criteria for fixed offshore structure located in Malaysia

1.4 SCOPE OF STUDY

The scopes of this study are:

i. The type of offshore structure used will be 4-legged fixed offshore structure

ii. The case study will be conducted in the surrounding earthquake region that affected the offshore platform in Malaysia

iii. The following seismic analyses have been carried out to determine the seismic response of a fixed offshore structure:

- Free vibration analysis has been carried out to obtain the natural period and the mode shape of the fixed offshore structure

- Time history seismic analysis has been carried out by referring to the time history seismic El Centro, 1940
- Response spectrum seismic analysis has been carried out by using response spectra curves of American Petroleum Institutes, API with earthquake ground motion intensity

iv. The computational analysis and structural modelling software used is SAP 2000

v. The considerations of design criteria are based on the approach of the American Petroleum Institutes, API 1993

1.5 SIGNIFICANCE OF STUDY

Throughout this research, we able to gain some general ideas about the earthquake ground motion and seismic effects of the offshore structure which located in Malaysia. Thus to achieve our main objective which to identify the necessity of the implementation of seismic design consideration for offshore structure in Malaysia due to surrounding earthquake through the determined seismic analyses and seismic vulnerability. The behaviours of fixed offshore structures determined from the study may be used to develop or determine some seismic design criteria for new fixed offshore structures positioned in Malaysia.

In addition, this study able to inform the public on earthquakes and their occurrences in Malaysia and the world so that they are aware of the hazards and risk poses by earthquakes. The importance of the seismic design will be noticeable, and thus the implementation of the seismic design can take note and carry out to reduce the seismic effect like loss of life, injuries and extensive property damage.

CHAPTER 2

LITERATURE REVIEW

2.1 EARTHQUAKE

Earthquakes are one of the world's most devastating and frightening natural hazards that result in great loss of life, injuries, extensive property damage and many of the terrible after effects. Basically, an earthquake is a sudden movement of the earth's crust part, followed and accompanied by a series of shakes or tremors which triggered by the sudden release of strain that has gathered over a lengthy period. For hundreds of millions of years, plate tectonics forces have shaped or formed the earth gradually as the huge plates under Earth's surface move under, over and past one another. The plates are locked or fastened together and unable to release the storing energy at other time. The plates will break free, once the accumulated energy raises strong enough. If an earthquake happens in a populated area, it may trigger numerous deaths and injuries and even extensive property damage.

In truth, earthquake does not kill people, but collapsed buildings and their content do. The greatest hazard in the earthquake is the collapse or fall of man-made and natural structures that caused the extensive loss of life and property. As a result, the seismic effects may not only consider in the countries that have a high risk of a strong earthquake, but also for those countries that are subject to low-to-moderate earthquake for instance like Malaysia since the power of the earthquake has shown us it is unpredicted (Ramli & Adnan, 2004).

2.2 CAUSES OF EARTHQUAKES

2.2.1 Tectonic earthquake

Earthquakes happen from the deformation of the brittle and outer portions of tectonic plates, which the earth's most outer layers of crust and the upper mantle. Heating and cooling of the rock below the tectonic plates, resulting in the convection and thus causing the adjacently overlying plates under the great stresses to move, and bring about the deformation. At the fault interface, the relative plate motion is limited by asperities and friction which are the interlocking areas caused by the protrusions in fault surfaces. Nevertheless, the strain energy builds up in the plates, eventually overcomes the resistance, and triggers slip between the both sides of the fault. This sudden slip, termed as elastic rebound which releases enormous amounts of energy, which comprises the earthquake.

2.2.2 Faults

The phrase fault is used to explain a discontinuity within rock mass, laterally which movement had occurred in the earlier. Plate boundary also represents a type of fault. Lineaments are capable linear surface features and may replicate subsurface phenomena. The lineament could be a joint, a fault or other linear geological phenomena. Generally, faults generate repeated displacements over the geologic time. The movement along the fault may be sometimes sudden or gradual, and thus causing an earthquake.

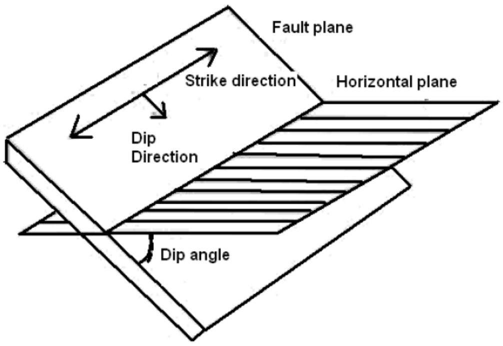


Figure 2.1: Several terminologies associated with rupture plane of fault.

Source: nptel.ac.in [Online image]. (2014). Retrieved September 16, 2014 from http://www.nptel.ac.in/courses/105101004/

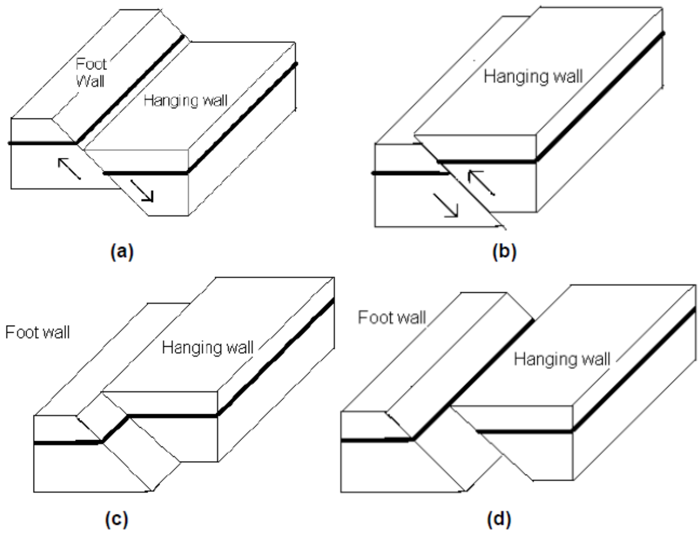


Figure 2.2: Types of faults (a) Normal fault; (b) Reverse fault; (c) Strike-slip fault; (d) Oblique fault.

Source: nptel.ac.in [Online image]. (2014). Retrieved September 16, 2014 from http://www.nptel.ac.in/courses/105101004/

Dip and strike are two of the important parameters associated to describe faults. The strike on the surface of the fault is in a horizontal line direction. The dip, deliberates in a vertical plane at right angles to the strike fault. The hanging wall of the fault indicates to the happen of upper rock surface along which displacement, while the foot wall is the phrase given to that below. Along a fault plane, vertical shift is termed as throw, whereas the horizontal displacement is called the heave.

Faults are categorized in to strike-slip faults, oblique-slip faults and dip-slip faults based on the direction of slippage all along the fault plane. In a strike-slip fault, the movement has occurred along the strike. The movement takes place diagonally along the fault plane in an oblique slip fault. In a case of dip-slip fault, the slippage happened along the dip of the fault. Based on the relative movement of foot walls faults and hanging are categorized into normal, wrench and reverse faults. In a case of a normal fault, the hanging wall has been moved downward relative to footwall. In wrench fault case, the hanging wall or the foot do not shift up or down in the relation to one another. In the reverse fault case, the hanging wall has been moved upward relative to the footwall. Thrust faults, which are the subdivision of reverse faults, tend to bring about the earthquakes.

Faults are nucleating surfaces for seismic activity. The stresses accumulated due to plate movement produces strain mostly along the boundary of the plates. This accumulated strain causes rupture of rocks along the fault plane.

2.3 SEISMIC WAVE

Seismic waves that caused by the faults rupture will result in the acceleration of the ground surface. There are essentially two types of seismic waves which are surface waves and body waves. Both P and S waves are under the category of body waves because they can pass through the interior of Earth. The surface waves are the wave that only detected when close to the earth surface, and they are categorized as Rayleigh waves and

Love waves. Surface waves are the result of the interaction between the earth surface materials and body waves. The types of seismic waves are as follows:

i. P wave (Body wave): The P wave also termed as primary wave, longitudinal wave and a compression wave. P wave is a seismic wave that triggers a series of compressions and dilations of the materials. The P wave is the fastest wave and also the first to reach a site. P waves can pass through both liquids and solids as a compression-dilation type of wave. P wave usually holds the least impact on ground surface movements since soil and rock is essentially resistant to the effects of compression-dilation.

ii. S wave (Body wave): The S wave also termed as the secondary wave, transverse wave and a shear wave. S wave triggers shearing deformations of materials. S waves only can travel and pass through solids because liquids have no shear resistance. The soil and rock shear resistance are normally less than the compression-dilation resistance, resulting S wave travels more slowly through the ground compare with the P wave. In terms of its shear resistance soil is weak and S waves have the greatest effect on the ground surface movements

iii. Rayleigh wave (Surface wave): Rayleigh waves have been defined as being similar to the produce of surface ripples by a rock thrown into the pond. This seismic waves generate both horizontal and vertical displacement of the ground as the outward of surface waves propagate.

iv. Love wave (Surface wave): Love waves are similar to S waves and they are transverse shear waves that travel adjacent to the ground surface.

2.4 EARTHQUAKE MEASUREMENT PARAMETERS

The size of an earthquake could be related to the damage triggered or measurement parameters like intensity and magnitude. These two useful earthquake measurement parameters or definitions of the earthquake’s size are sometimes confused.

2.4.1 Intensity

The intensity of seismic indicates the degree of destruction triggered by the earthquake. In other expressions, intensity of an earthquake is a compute or measure of shaking of ground severity and its consequent damage. The intensity is the empirical to some degree or extent because of the extent of damage or destruction that occurs in a construction in a given area that counts on many factors. The factors consist of: (i) the distance from epicenter, (ii) magnitude of earthquake (iii) type of the construction (iv) the compactness of underlying ground, (v) duration of the seismic activity and (vi) the depth of the focus. Intensity of an earthquake is the oldest measure of earthquake activity.

The earthquake intensity scale comprises of a series of specific key responses for example like people awakening, damage to chimneys, movement of furniture and the total destruction. Several earthquake intensity scales have been formed and developed over the last hundred years to evaluate and estimate the earthquakes effect. The most common and popular used is the scale of Modified Mercalli Intensity (MMI). Modified Mercalli Intensity (MMI) scale, composed of 12 rising intensity levels that range from unnoticeable shaking to catastrophic destruction, which defined by Roman numerals. In fact, it is the arbitrary ranking which based on the observed effects, rather providing the mathematical basis. The lower intensity scale numbers are usually dealing with the mode in which the seismic is felt by people. The greater scale numbers are based on observed structural damage.

2.4.2 Magnitude

The magnitude of a seismic is a measure of the absolute size of the seismic which related to the amount of energy released by the geological split or rupture, and without reference to the distance from the epicenter. While intensity of an earthquake is described in Roman numerals and it always a whole number, unlike the magnitude is described in Arabic numerals and need not be a whole number.

i. **Local Magnitude (M_L)**

This magnitude scale is also known as Richter Magnitude. The local magnitude M_L is the measures of the maximum earthquake wave amplitude recorded at the standard. This magnitude scale was formed and developed for local and shallow earthquakes. This magnitude scale is the most commonly used and best known magnitude scale. This local magnitude, M_L is calculated as follows:

$$M_L = \log A - \log A_0 = \log A/A_0$$

Where,
M_L = local magnitude
A_0 = the calibration factor that depends on distance

ii. **Surface Wave Magnitude (M_S)**

The scale of surface wave magnitude M_S is based on the amplitudes of LR-waves having a period of about 20s. This magnitude is defined as follows:

$$Ms = \log A' + 1.66 \log \Delta + 2.0$$

Where,

Ms = scale of surface wave magnitude

A’ = maximum ground displacement, μm

Δ = the ground-motion amplitude and period which measured in degrees

iii. Seismic Moment Magnitude (M_W)

The seismic moment magnitude scale M_W is one of the commonly used approaches for determining the large magnitude earthquakes. This magnitude is accounts for the mechanism of shear occur at seismic sources and not related to any wavelength. Thus, it can be used to obtain or measure the entire ground motions spectrum.

$$M_0 = \mu \, A_f \, D$$

Where,

M_0 = moment magnitude or the function of seismic moment

A_f = area of fault plane undergoing slip, m^2.

D = average displacement of ruptured segment of fault, m.

2.5 OFFSHORE PLATFORM STRUCTURE

There are more than 3500 offshore platform structures with numerous types now locating in the 35 countries offshore waters. (Moore, M. L., Ridge, M. W. Economy brings design challenges. Offshore, 42(13): 45 – 48 (1982)). Offshore platforms comprise broadly of two different components: (1) facilities for production and drilling operations, often named as topsides, and (2) the foundation and its supporting structure. The topsides to define the purpose of the offshore platform. Consist in the topside plant could be oil and gas processing equipment, drilling rigs and associated equipment, transportation pumps and compressors, utilities and even the living quarters. Moreover, most of the offshore platforms also possess a helideck for helicopters.

Offshore platform structure has to be formed to extract the oil from the oils wells which sitting deep at the bottom of sea. This structure can ensure the persisting the production of oil, thus offshore structure plays an important and major part in ensuring the supply of oil. The figure 2.2 showed the different types of offshore drilling platform.

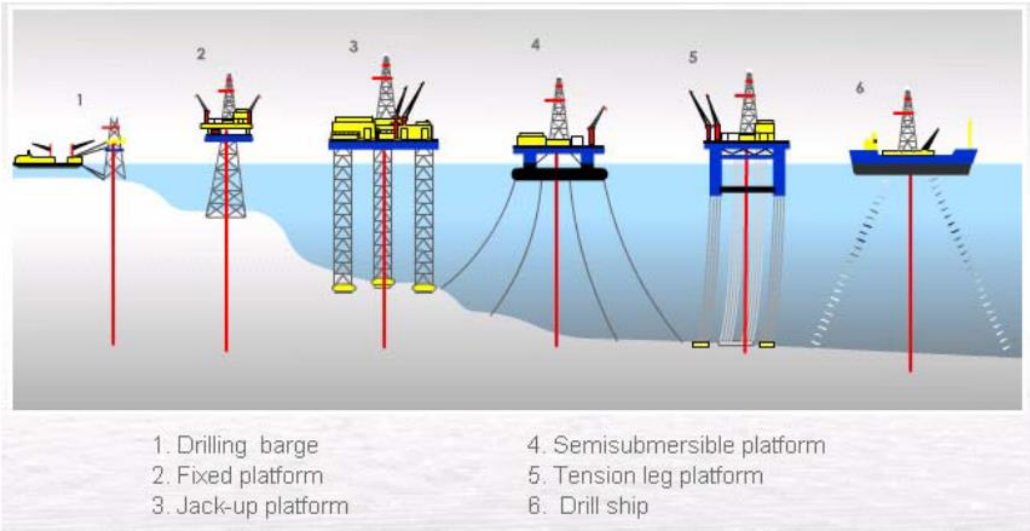


Figure 2.3: Types of offshore drilling platform

Source: Rozaina 2006

2.6 FIXED OFFSHORE STRCUTURE

The fixed offshore structure is described as a structure expanding above the water surface and sustained at the sea bed through spread footings, piling or others with the intention of persisting stationary over a prolonged time, which are extremely stable, but are limited to up to 500 meters water depths.

The fixed jacket type platform comprises of the following (API RP 2A-WSD, 2007):

i. Braced completely and redundant welded tubular space frame expanding from an elevation at or close to the sea bed to the water surface above, which is designed to assist the main element of the offshore structure, transferring vertical and lateral forces to the foundation.

ii. The other foundation elements or piles that permanently anchor the offshore structure to the ocean floor and support both vertical and lateral loads

iii. The superstructure is comprising of the necessary deck and trusses for supporting operational and the other loads.

The purposes of designing the offshore structure consist of materials handling, producing, drilling, storage, living quarters or some of these combination. Moreover, the consideration should be provided to equipment operational requirements, such as safety, clearances and access when sizing the offshore platform.

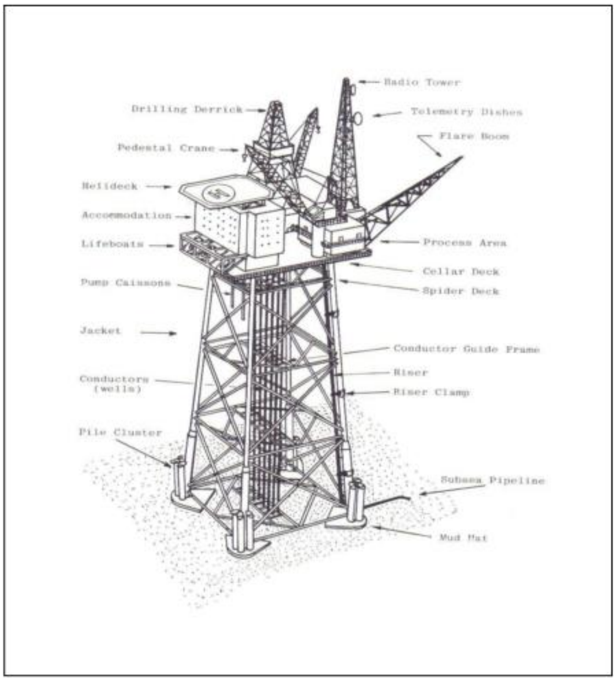


Figure 2.4: Fixed offshore platform

Source: Rozaina 2006

2.7 CURRENT MALAYSIA PRACTICE OF OFFSHORE STRCUTURE

Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms by American Petroleum Institute RP 2A – WSD (2000) provisions provide the design requirement and characterization of environmental load for fixed offshore structure for design used, define the analytical methods to establish the forces induced in the offshore structure system by ground motions, and also give guidance configuring steel elements and for sizing for the design forces. Moreover, the consideration of environmental loads is comprised earthquake ground motions, wave, current and wind loads.

2.8 DESIGN CRITERIA OF OFFSHORE STRCUTURE

The offshore structures are basically designed to subject various types of loading to ensure its structural safety and stability in the ocean. The offshore structures always has a larger magnitude and complexity of combinations of the various loads than onshore structures. Eventually, the main supporting structural element of the entire offshore structure are the foundations, and as such generally has to resist all the vertical loads from (Trevon Joseph, 2009):

i. The self-weight of the topside and structure, V

ii. Any associated moment with eccentric loading of the platform, M_V

iii. Lateral loads on structure due to wind, L_B, currents, L_C, and waves, L_W

iv. The moments that is related with these lateral loadings, M_B & M_C

v. Cyclic loading caused by waves, L_W

vi. Cyclic moment caused by waves, M_W

vii. Earthquake loads, E

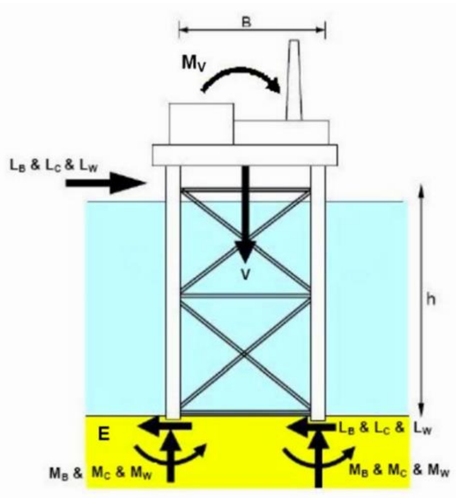


Figure 2.5: Loads on offshore structure foundations

Source: Dean, 2008

All of these loads are mostly withstood by this soil-pile-structure interface. Basically, the loads are transferred to offshore structure foundations through the load pathway for effective loading situations, even though it is known that in the design of limit state there will be the existence of redundant members to provide for emergencies, these loads in the end arise as compression, tension, bending moments, shear forces and torsion in the piles. The piles capacities are determined from equations that have been acquired by researchers and which are prescribed in the API RP 2A-WSD code.

The whole offshore structure is greatly impacted by the application of wave, wind and current loads. In accordance to previous practice, the complexity of these combinations of application is not directly applied to a single path of the structure. The metocean criteria would denote the angular range from True North. For the wind case, the wind speed at a given height is found and subjected at different angles. For the wave case, a specific height of weight is formulated and for the current loading case, the speeds along with the current profile are given. Some more, different operating cases have different loading cases, each wind speeds are vary. These, operating cases consist the normal operating case; the 100-year case of extreme storm; the 100-year case of design storm; and even the 10 year case of extreme storm (Trevon Joseph, 2009).

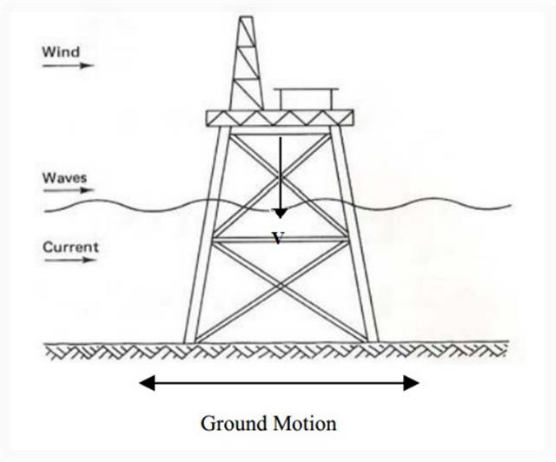


Figure 2.6: Environmental loads

Source: Rozaina 2006

2.8.1 Earthquake Loads

Earthquake Loads should be considered in offshore structure design for seismically active areas. Areas are seismically active on the basis of preceding earthquake activity record in magnitude and frequency of occurrence in terms of ground motion. The seismic activity of an area for intentions of offshore structures design is valued in terms of the probable severity of damage or collapse to these structures. Moreover, offshore structure in shallow water that may be exposed to tsunamis should be examined for the effects of the resulting forces.

To achieve the study of offshore structure under seismic load, the offshore structure was analyzed in a dynamic case and the methods being performed are as follows:

i. Free Vibration Analysis

ii. Time History Analysis

iii. Response Spectrum Analysis

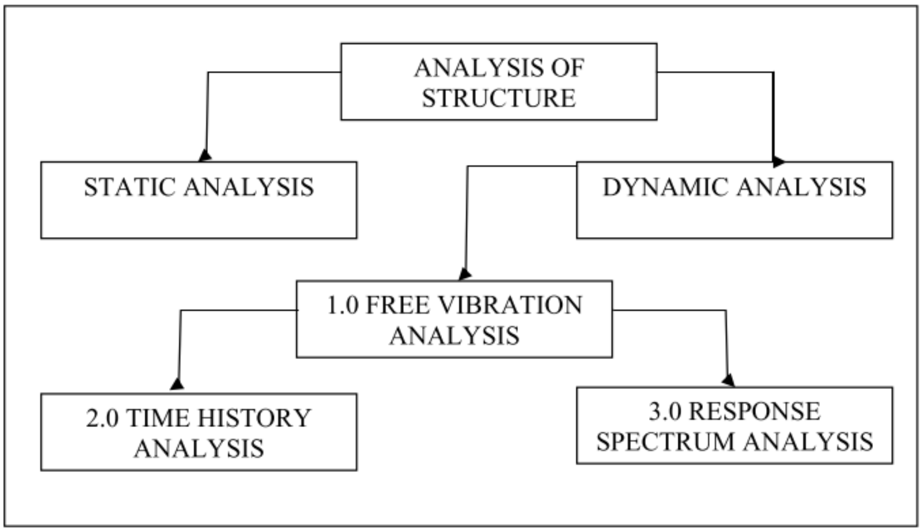


Figure 2.7: Structure design concept for earthquake

Source: Syahrul Izwan, 2008

2.8.2 Wind Loads

Wind load is subjecting on several structural components of the offshore structure, such as the members, the equipment, the facilities and so forth. These winds take account of gust forces and steady forces that applied rationally to the structure for example being made to act at certain height and certain duration such as one hour. The wind forces normally contributed to the conventional steel structures less than 10% of the total global load. But, it is discovered that the waves will contribute a much greater percentage, when deeper waters where compliant the offshore structures. This is significant where the wind frequency is close to the frequency of natural of the structures that resulted the resonance. As determined by API-RP2A, the wind force is estimated by following relationship (Trevon Joseph, 2009):

$$F = (\rho/2)u^2 C_s A$$

Where,

F = wind force [N],

ρ = mass density of air [1.2 kg/m^3],

U = wind speed [m/s],

C_s = Mass Coefficient,

A = area [m^2]

2.8.3 Wave Loads

Wave load plays a significant role in the design of offshore structure neither which is in non-seismic region or critical design load. It's actually quite difficult to examine the wave loading result exactly due to extreme randomness of the natural phenomenon. The waves can be incident on an offshore structure from any direction especially during the storms of design. Others than that, waves also influence the base soil of the offshore structure, this is significant if the soil is loose to medium dense material. The waves also created the cortices around the base of the structure leg where it cause to scouring that can deducted the pile capacity unless it was well designed against such (Trevon Joseph, 2009).

The strength of the wave also depends by its height, which is calculated from the crest to the trough. The waves imposed buoyant force and cyclic on the offshore structure and there were to be withstood by the foundation piles. The waves effect on the structures is examined by the application of Morrision's equation (Trevon Joseph, 2009):

$$F_w = F_D + F_I = C_D \frac{w}{2g} A\, U|U| + \mathrm{C}_m \frac{w}{g} V \frac{\delta U}{\delta t}$$

Where,

F_w = Hydrodynamic force vector per unit length acting normal to the axis of the member [N/m],

F_D = drag force per unit length of the member [N/m],

F_I = inertial force per unit length [N/m]

C_d = drag coefficient,

w = density of water,

U_w = component of velocity vector caused by wave [m/s],

$|U|$ = absolute value of U [m/sec],

C_m = coefficient of inertia,

$\frac{\delta U}{\delta t}$ = component of local acceleration vector of water.

2.8.4 Current loads

The vector summation of circulatory currents, tidal currents and storm generated currents is the current loading. The location of the offshore structures influences the magnitude of currents, the most of the strengths of vectors risen with the proximity to the shoreline. The strength, profile and direction of the current need to be considered for deposits of inland and the oceanic material, as well as placement of berthing and boats docking equipment in the offshore structure design. In order to calculate the current force on the structural members, no wave conditions need to be applied, by substituting the $\frac{\delta U}{\delta t} = 0$ (Trevon Joseph, 2009).

$$F = F_D + F_I = C_D \frac{w}{2g} A\, U|U| + C_m \frac{w}{g} V \frac{\delta U}{\delta t}$$

$$F_c = F_D = C_D \frac{w}{2g} A\, U|U|$$

CHAPTER 3

RESEARCH METHODOLOGY

3.1 PLANNING OF THE STUDY

In this chapter, we will be discussed about the process to conduct the whole study. Scheduling and planning have been done in the early stage to make the project work effectively. The offshore structure used in this project is 4-legged fixed offshore structure. This offshore structure is the existing structure which located at Terengganu, Malaysia. Free vibration analysis, time history and response spectrum seismic analysis have been carried out. The computational analysis and structural modelling software used is SAP 2000.

Response spectrum analysis has been performed using response spectra curves of Eurocode 8, where the time history analysis has been performed by referring to the earthquake data obtained from the Malaysian Meteorological Department. The results of the response spectrum and time history analysis have been compared. Result of the mode shape and the natural period of the offshore structure were obtained and discussed in detail. Moreover, manual calculation has been performed for comparison purpose.

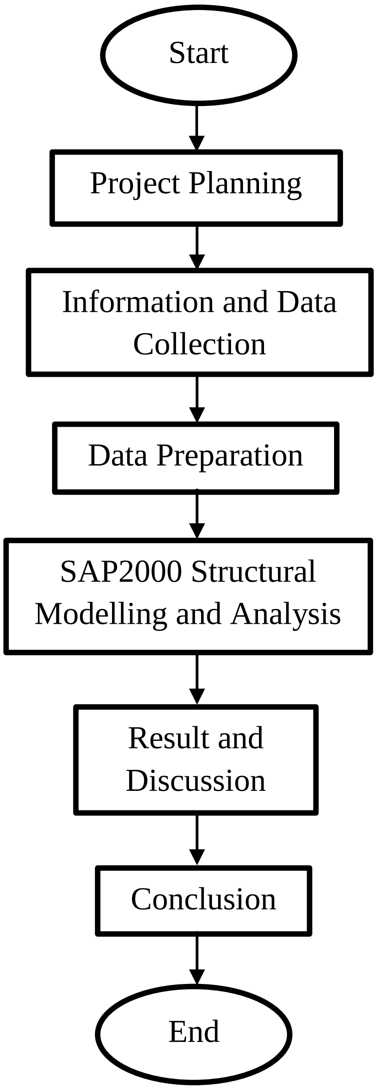


Figure 3.1: Flow Chart of Methodology

3.2 INFORMATION AND DATA COLLECTION

To obtain important information and data for the modelling and analysis work, further data collection works have been performed during this stage. The information and data needed including:

i) Architecture and structural drawing of the typical four-legged fixed offshore structure

ii) Location, background and the material used of the offshore structure.

iii) Environmental data from the site location.

iv) Earthquake data – From Malaysian Meteorological Department (MMD)

3.2.1 Offshore Structure Description

The type of offshore structure used in this analysis is 4-legged fixed offshore structure. Figure 3.2 shows two of the side views of this fixed offshore structure. This offshore structure is the existing structure which located at Terengganu, Malaysia. The height of this offshore structure is 53.2 m and has the maximum area at bottom 24.6m x 24.6m. This structure is using rolled tubular member and primary plate and the material is ASTM A572 GR50 that has yield strength 345MPa. This structure is the design followed the design code American Institute of Steel Construction Specification for design, fabrication and erection of structural steel building (AISC) and American Petroleum Institute RP2A-WSD (API). Due to beyond to the bachelor degree standard, we have implement another design code in our manual calculation and computation analysis parts for example Eurocode 3: Design of steel structures, Eurocode 8: Design of structures for earthquake resistance and American Petroleum Institute RP2A-WSD (API).

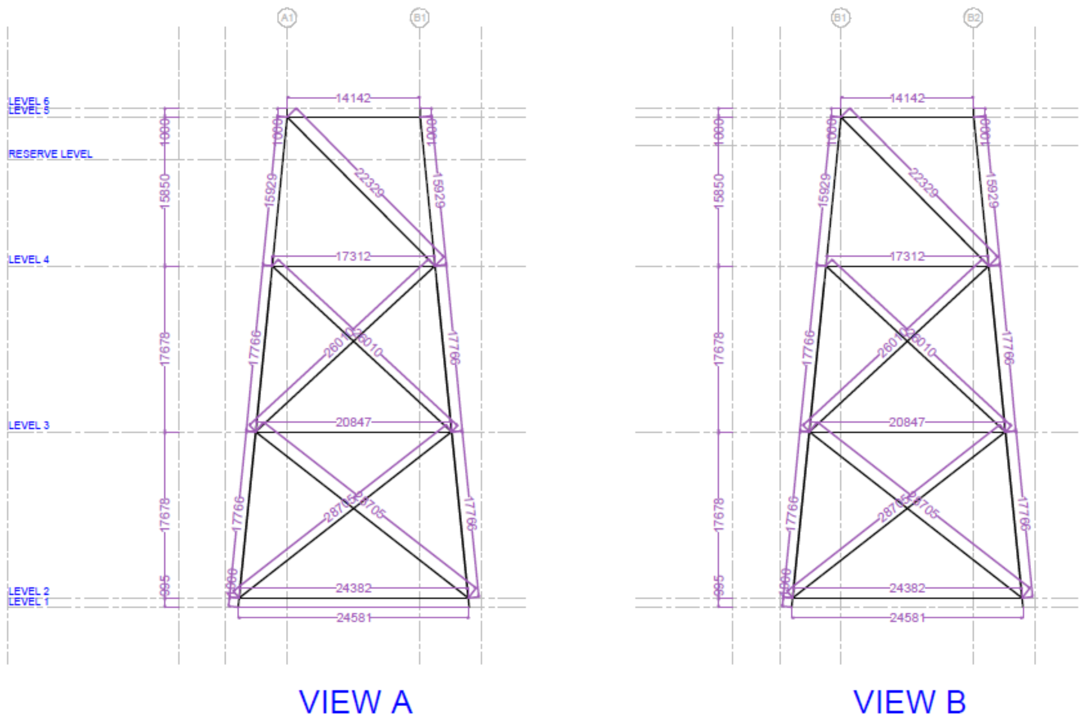


Figure 3.2: The side views of offshore structure

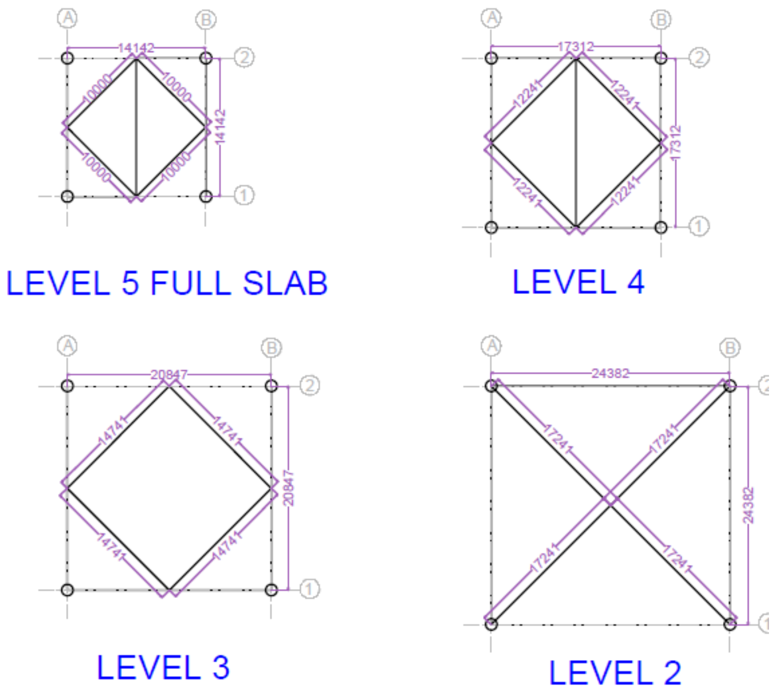


Figure 3.3: The plan views of offshore structure

3.2.2 Material Properties

Moreover, we also have to implement another type of material use in this design. Figure 3.4 shows the material properties for the fixed offshore structure that we have implemented and used in SAP2000. The modulus of elasticity of this material used is 210 000 N/mm^2, where shear modulus is 81 000 N/mm^2, and the minimum yield stress is 355 N/mm^2.

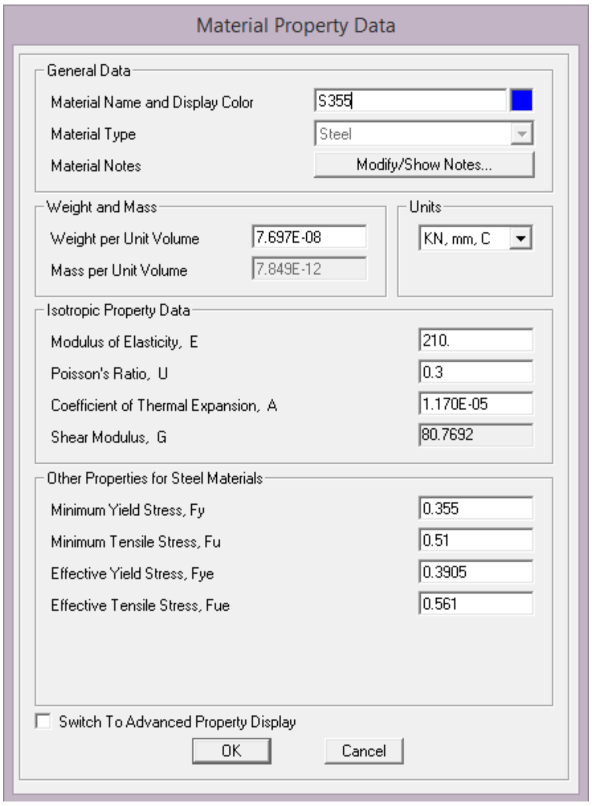


Figure 3.4: Material properties for structural steel

The stiffness and total mass of the fixed offshore structure will be presented in table form which is located in **APPENDICES B** and **C.** The total stiffness and mass of the offshore structure is 8.30 x 10^8 and 323161.35 kg respectively.

3.3 LOAD DESCRIPTION

The load combinations of dead load, deck loads, wind, wave, current and earthquake loads have been defined.

3.3.1 Dead and Live loads

The self-weight of the fixed offshore structure will be auto-generated by the SAP2000 computer software. The dead load and live load of this structure are listed in the table below:

Table 3.1: Dead load and live load description

No.	Load descriptions	Weight [MN]
1	Jacket appurtenances weight	0.339
2	Topside dead loads	0.393
3	Topside live loads	1.150
4	Piping & equipment weights	0.400
	TOTAL	2.282

3.3.2 Environmental Loads

The environmental loads are based on the metocean data. Table 3.2 shows the environmental criteria for the offshore structure which located in Terengganu. Therefore all the environmental loads which including wind, wave and current action were calculated by using the provided environmental criteria and formula from the American Petroleum Institute (API).

Table 3.2: Environmental criteria for offshore structure at Terengganu

MSL		Design Condition
47.629 m	Wave height (m)	10.79
	Wave period (s)	10.9
	Current velocity (m/s)	0.750
	Wind speed (m/s)	21.8
	Max. tide (m)	2.0
	Storm surge (m)	0.4

3.3.3 Wind Load

The wind profile, gusts and wind drag force are calculated using following equation that is taken directly from the API RP 2A-WSD practice.

3.3.3.1 Wind profile and Gusts

For strong wind conditions the design wind speed $u(z,t)$ (ft/s) at height z (ft/s) above sea level and corresponding to an average time period t (s) [where t $\leq$ $t_o; t_o = 3600\ sec$] is given by:

$$u(z,t) = U(t)x\ [1 - 0.41]x\ I_u(z)x\ ln\left(\frac{t}{t_o}\right)$$

Where the 1 hour mean wind speed $U(z)$ (ft/s) at level z (ft) is given by:

$$U(z) = U_o\, x\ [1 + C\ x\ ln\left(\frac{z}{32.8}\right)]$$

$$C = 5.73\ x\ 10^{-2} \text{x}\ (1 + 0.0457 \text{ x } U_o\ )^{1/2}$$

And where the turbulence $I_u(z)$ at level z is given by:

$$I_u(z) = 0.06x\ [1 + 0.0131x\ U_O]\ x \left(\frac{z}{32.8}\right)^{-0.22}$$

Where $U_o\ (z)\ (ft/s)$ is the 1 hour mean wind speed at 32.8 ft.

3.3.3.2 Wind Speed and Force Relationship

The wind drag force on object should be calculated as:

$$\mathrm{F} = (\rho/2)\mathrm{u}^2\mathrm{C_s A}$$

Where,

F = wind force [N],

ρ = mass density of air [1.2 kg/m^3],

U = wind speed [m/s],

C_s = Mass Coefficient,

A = area [m^2]

3.3.4 Wave Load

The wave force can then be computed as the sum of a drag force and an inertia force, as the Morison Equation below:

$$F_w = F_D + F_I = C_D \frac{w}{2g} A\, U|U| + C_m \frac{w}{g} V \frac{\delta U}{\delta t}$$

Where

F_w = Hydrodynamic force vector per unit length acting normal to the axis of the member [N/m],

F_D = drag force per unit length of the member [N/m],

F_I = inertial force per unit length [N/m]

C_d = drag coefficient,

w = density of water,

U_w = component of velocity vector due to wave [m/s],

$|U|$ = absolute value of U [m/sec],

C_m = inertia coefficient,

$\frac{\delta U}{\delta t}$ = component of local acceleration vector of the water.

3.3.5 Current Load

In order to calculate the current force on the structural members, no wave conditions need to be applied, by substituting the $\frac{\delta U}{\delta t} = 0$.

$$F = F_D + F_I = C_D \frac{w}{2g} A\, U|U| + C_m \frac{w}{g} V \frac{\delta U}{\delta t}$$

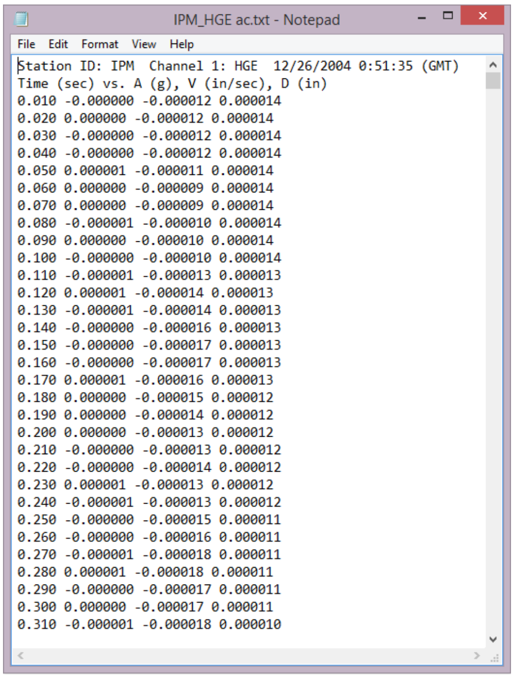
$$F_c = F_D = C_D \frac{w}{2g} A\, U|U|$$

3.3.6 Earthquake Load

For the earthquake load, it will be performed by carried out by the response spectrum analysis by using responses spectra curves of EuroCode8 2004 through SAP2000 computation software. Same goes to the time history analysis, it will be performed by using the collected earthquake data that obtained from MMD through SAP2000 computation software.

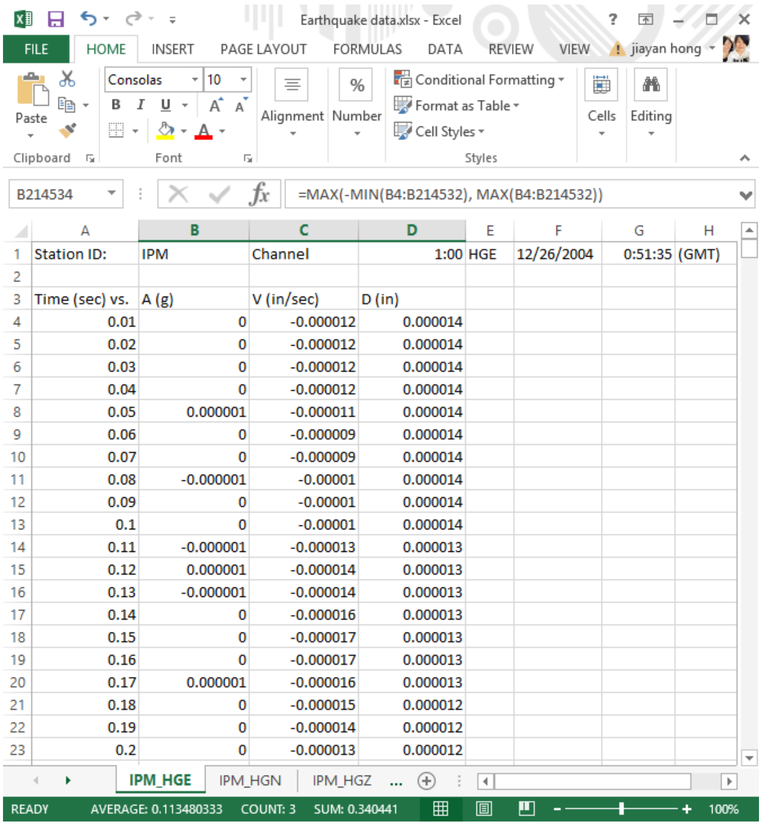
3.4 DESCRIPTION OF EARTHQUAKE DATA

After collected the required earthquake data from the Malaysian Meteorological Department, which in three different directions east (E), north (N) and Z direction. We need to convert all the earthquake data from Notepad Document into MS Excel File. Figure 3.5.1 shows the earthquake data which in Notepad Document form, where figure 3.5.2 shows the earthquake data in MS Excel File form. The main purpose of the data conversion is to obtain the most critical or maximum data through the MS Excel application. Moreover, the graph, bar chart, table or any comparison also can easily obtain by this MS Excel File.


IPM_HGE ac.txt - Notepad
File Edit Format View Help

```
Station ID: IPM  Channel 1: HGE  12/26/2004 0:51:35 (GMT)
Time (sec) vs. A (g), V (in/sec), D (in)
0.010 -0.000000 -0.000012 0.000014
0.020 0.000000 -0.000012 0.000014
0.030 -0.000000 -0.000012 0.000014
0.040 -0.000000 -0.000012 0.000014
0.050 0.000001 -0.000011 0.000014
0.060 0.000000 -0.000009 0.000014
0.070 0.000000 -0.000009 0.000014
0.080 -0.000001 -0.000010 0.000014
0.090 0.000000 -0.000010 0.000014
0.100 -0.000000 -0.000010 0.000014
0.110 -0.000001 -0.000013 0.000013
0.120 0.000001 -0.000014 0.000013
0.130 -0.000001 -0.000014 0.000013
0.140 -0.000000 -0.000016 0.000013
0.150 -0.000000 -0.000017 0.000013
0.160 -0.000000 -0.000017 0.000013
0.170 0.000001 -0.000016 0.000013
0.180 0.000000 -0.000015 0.000012
0.190 0.000000 -0.000014 0.000012
0.200 0.000000 -0.000013 0.000012
0.210 -0.000000 -0.000013 0.000012
0.220 -0.000000 -0.000014 0.000012
0.230 0.000001 -0.000013 0.000012
0.240 -0.000001 -0.000013 0.000012
0.250 -0.000000 -0.000015 0.000011
0.260 -0.000000 -0.000016 0.000011
0.270 -0.000001 -0.000018 0.000011
0.280 0.000001 -0.000018 0.000011
0.290 -0.000000 -0.000017 0.000011
0.300 0.000000 -0.000017 0.000011
0.310 -0.000001 -0.000018 0.000010
```

Figure 3.5: Earthquake data in Notepad Document form



	A	B	C	D	E	F	G	H
1	Station ID:	IPM	Channel	1:00	HGE	12/26/2004	0:51:35	(GMT)
2								
3	Time (sec) vs.	A (g)	V (in/sec)	D (in)				
4	0.01	0	-0.000012	0.000014				
5	0.02	0	-0.000012	0.000014				
6	0.03	0	-0.000012	0.000014				
7	0.04	0	-0.000012	0.000014				
8	0.05	0.000001	-0.000011	0.000014				
9	0.06	0	-0.000009	0.000014				
10	0.07	0	-0.000009	0.000014				
11	0.08	-0.000001	-0.00001	0.000014				
12	0.09	0	-0.00001	0.000014				
13	0.1	0	-0.00001	0.000014				
14	0.11	-0.000001	-0.000013	0.000013				
15	0.12	0.000001	-0.000014	0.000013				
16	0.13	-0.000001	-0.000014	0.000013				
17	0.14	0	-0.000016	0.000013				
18	0.15	0	-0.000017	0.000013				
19	0.16	0	-0.000017	0.000013				
20	0.17	0.000001	-0.000016	0.000013				
21	0.18	0	-0.000015	0.000012				
22	0.19	0	-0.000014	0.000012				
23	0.2	0	-0.000013	0.000012				

Figure 3.6: Earthquake data in MS Excel File

3.4.1 Data Conversion

The following steps are shown the procedure for converting all the required earthquake data from Notepad Document into MS Excel File. First of all, start the MS Excel application by clicking the data command from the main menu, follow by selecting the from text command since the data obtained is in text file form. Import the text file required to MS Excel. There are three steps in the text import wizard. Firstly, we have to choose the file type that best describe the data, delimited are selected in the case. And pick the delimiters such as tab, semicolon, comma and space to delimit the data contains. Lastly, select the column data format and the location for inserting the data. Once the earthquake data had been converted, we can obtain the maximum or the most critical data through MS Excel application. Formula of =MAX(-MIN(Range), MAX(Range)) have been used in determined the maximum data form both positive and negative values. The maximum data obtained for the three different E, N and Z direction are 0.001222g, 0.00108g and 0.00096g respectively. Time (sec) versus Acceleration graph in three different directions has been obtained and the result is shown as below.

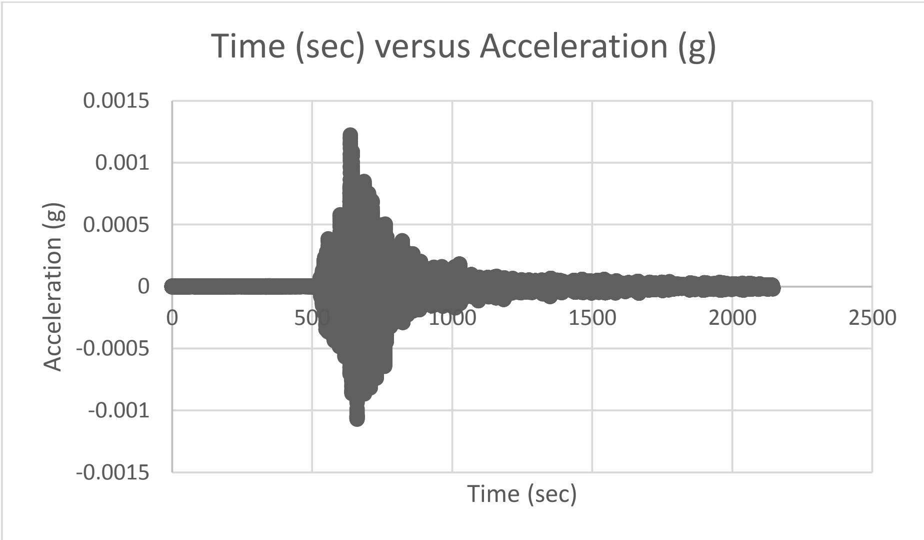


Figure 3.7: Time (sec) vs Acceleration (g) in E-direction

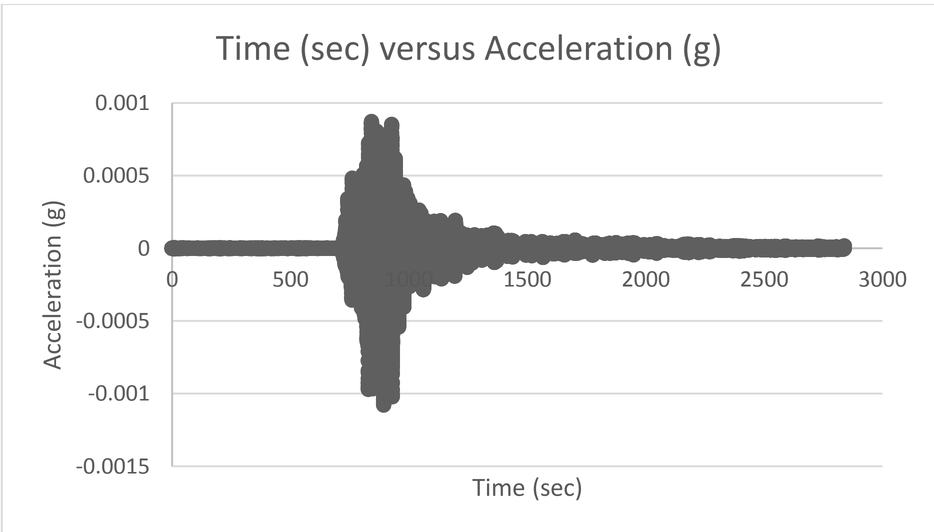


Figure 3.8: Time (sec) vs Acceleration (g) in N-direction

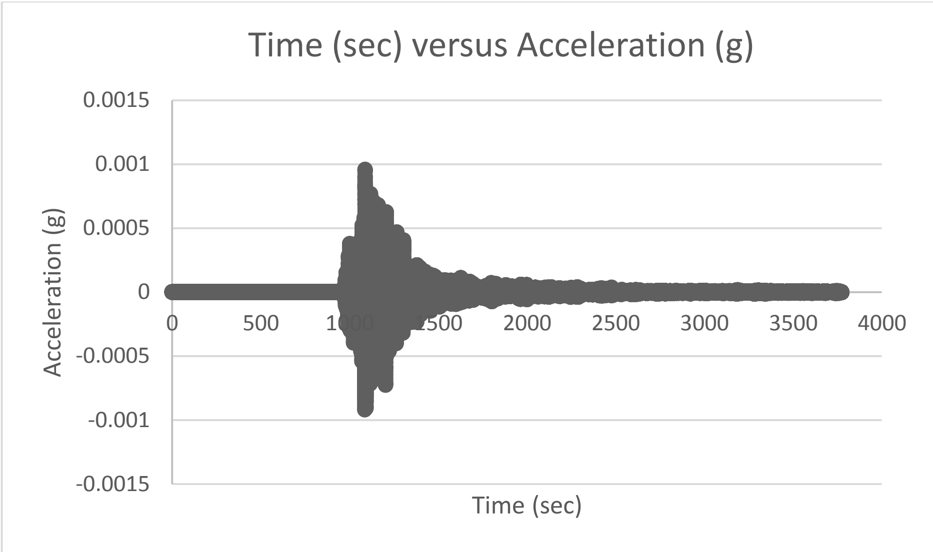


Figure 3.9: Time (sec) vs Acceleration (g) in Z-direction

3.5 ANALYSES

The fixed offshore structures have been modelled and analysed using SAP2000, an integrated software for structural analysis and design. The free vibration analysis, time history earthquake analysis and response spectrum earthquake analysis have been performed in this study. The loads that considered in this analysis consist of dead load, live load, modal load, environmental load, time history load and also response spectrum load, while the environmental load comprises of wind, wave and current loads. There is several combination of load cases that were applied in this analysis, these consist of:

i. Free vibration analysis (FVA)
ii. Dead load (DL) + live load (LL)
iii. Environmental load (EL)
iv. Dead load (DL) + live load (LL) + environmental load (EL) + time history load (TH)
v. Response spectrum load (RS)

Moreover, the results obtained from this study are as follows:

i. Mode shape of the offshore structures
ii. Natural period and natural frequency of the offshore structures
iii. Maximum unity check of all the structures member
iv. Maximum shear stress and bending stress in the member
v. Shear force and bending moment under various load cases
vi. Joint displacement, velocity and acceleration under different combination of load cases

3.6 SAP2000 COMPUTATIONAL PROGRAM

Analysis software used is SAP 2000 which an integrated software for structural analysis and design. Generally, SAP2000 is a stand-alone finite-element-based structural program for the analysis and design of civil structures. It offers an intuitive, yet powerful user interface with many tools to aid in the quick and accurate construction of models, along with the sophisticated analytical techniques needed to do the most complex projects.

SAP2000 is object based, meaning that the models are created using members that represent the physical reality. A beam with multiple members framing into it is created as a single object, just as it exists in the real world, and the meshing needed to ensure that connectivity exists with the other members is handled internally by the program. Results for analysis and design are reported for the overall object, and not for each sub-element that makes up the object, providing information that is both easier to interpret and more consistent with the physical structure.

3.6.1 Checklist of SAP2000 Modeling and Analysis

Modelling and analysis of the 4-legged jacket of fixed offshore platform are done by using SAP 2000 program. Checklist and step in modelling and analyses the offshore structure are as below:

- Define the type of the model
- Create, define and coordinate the grid systems for the model
- Define material and structural section properties
- Draw the frame geometry and assigning member section properties
- Determine area section of the model
- Determine the restrains at base condition
- Define all load cases
- Assign load cases at specific frame element/ joint
- Define functions of Time History and Response Spectrum
- Analyze the Model
- Display result and output table
- Check for the structure design

3.6.2 Steps in SAP2000 Modeling and Analysis

Step 1: Define the type of the model

First, the units of this project “KN, m, C” will be selected and follow by the template of “Grid Only” to set up the coordinates of the grid line for the fixed offshore structure.

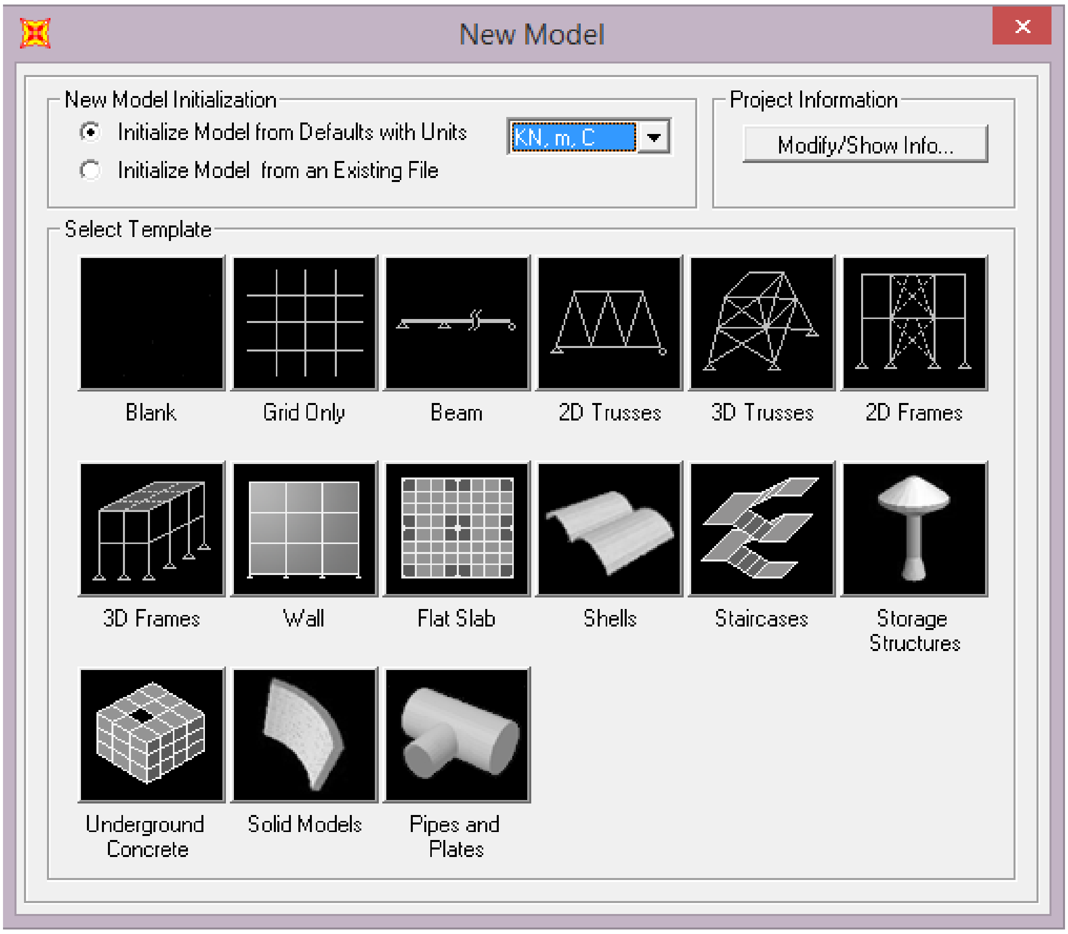


Figure 3.10: Select the structure model type

Step 2: Setting up the coordinates of grid line

Insert all the grid data in X, Y and Z direction accordingly in order to assign the frame element and draw the model of the structure easily in the next stage.

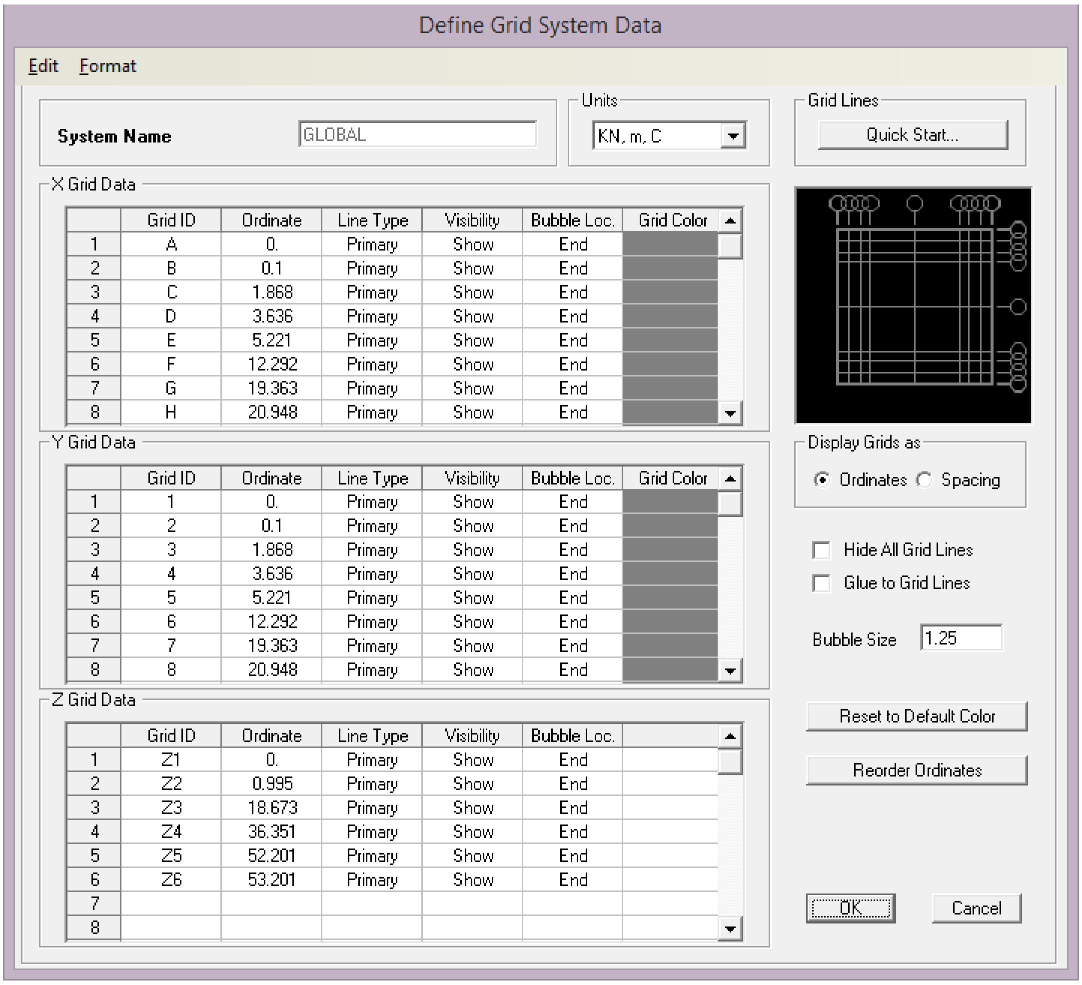


Figure 3.11: Define grid system data

Step 3: Define material and structural section properties

Define all the material types and section properties which are presented in this offshore structure. Material type of structural steel S355 has defined and used all along the study and together with its standard and material property data. Select Pipe which also known as Circular Hollow Sections (CHS) from the list of steel frame section property type and define the pipe section accordingly by inserting its material type and dimension.

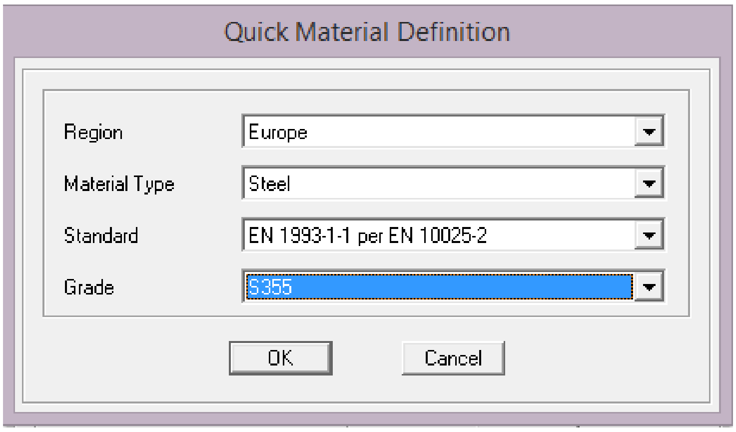


Figure 3.12: Define material types

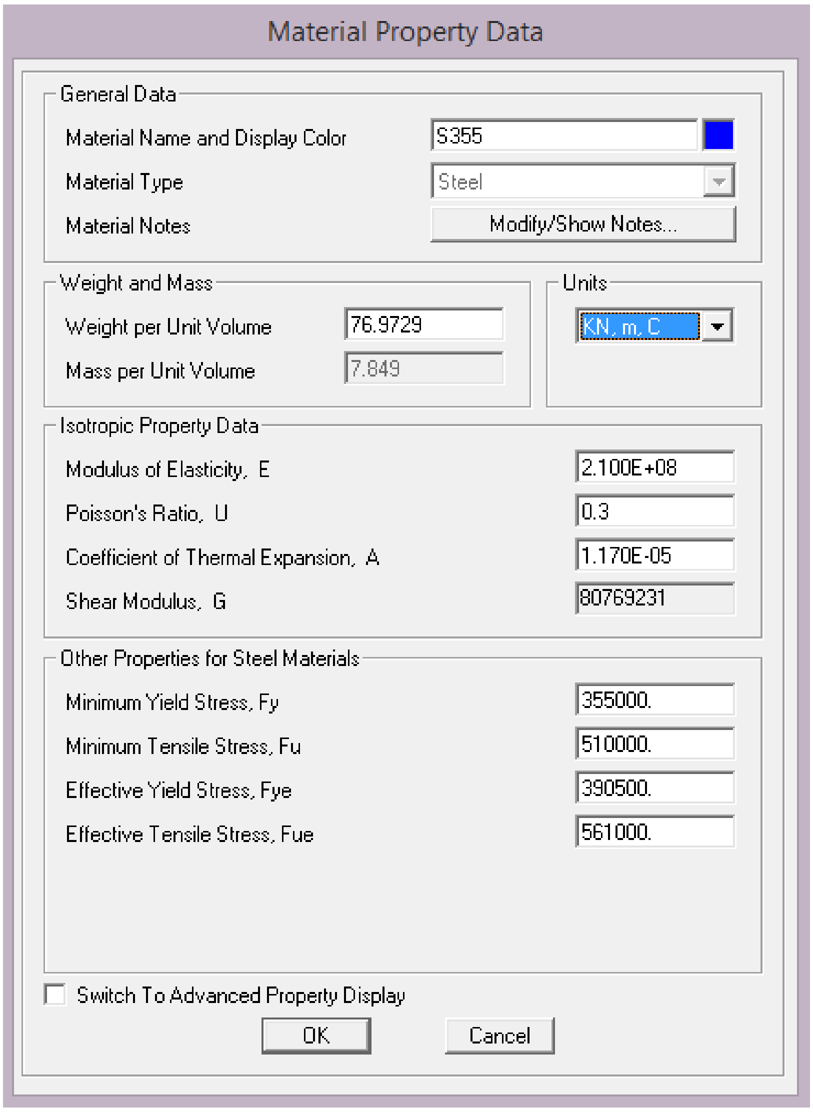


Figure 3.13: Material property data

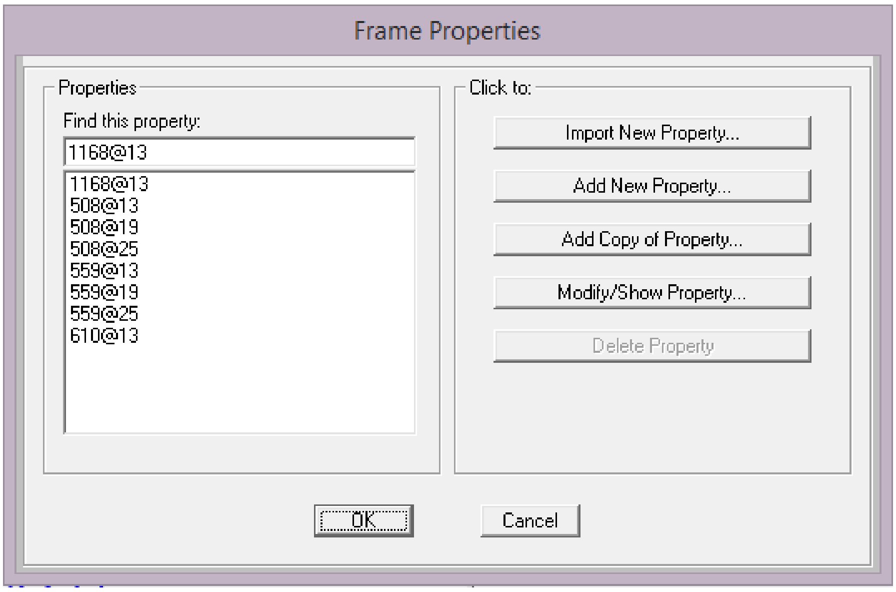


Figure 3.14: Define frame properties

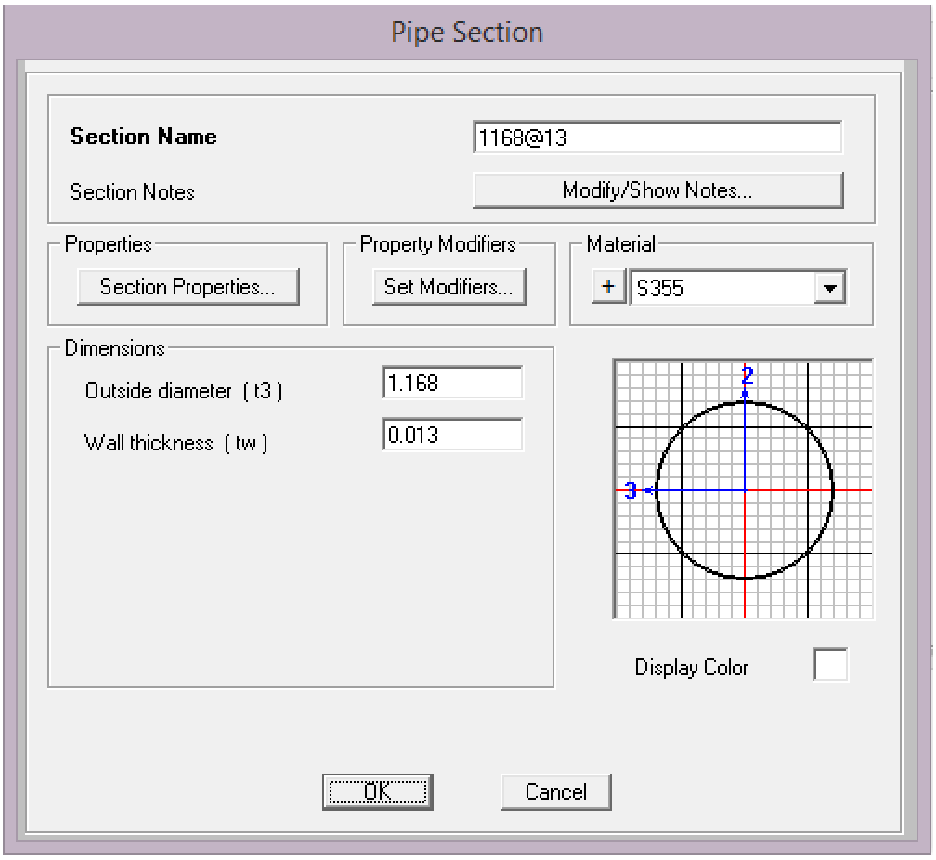


Figure 3.15: Define pipe section

Step 4: Draw the frame geometry and assigning member section properties

Modeled the fixed offshore structure by assigning the frame elements according to the sizing of the member based on the architectural drawing. After assigning all the frame geometry according, added the slab on the top of the structure and make the slab and the frame connected with each other by using "Auto-mesh" function. Lastly, assigning and add the restraints of the structure as fixed support at the base condition.

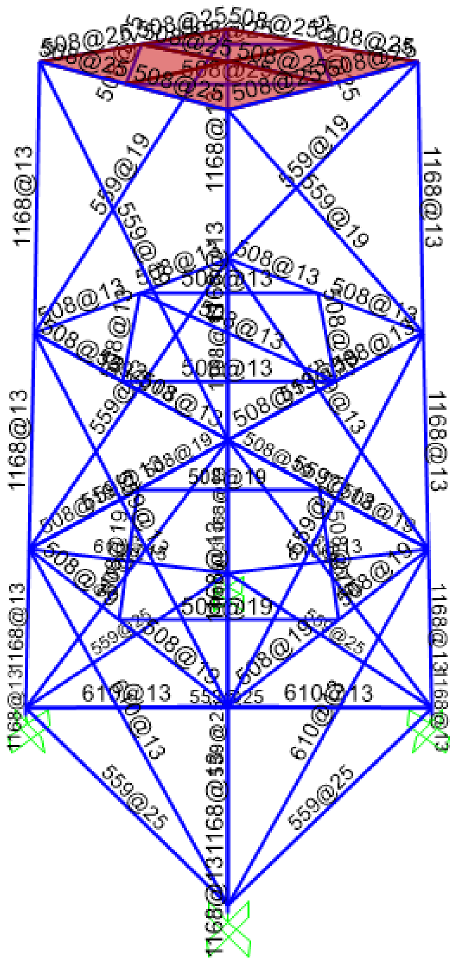


Figure 3.16: Structure Layout in SAP2000 (3D)

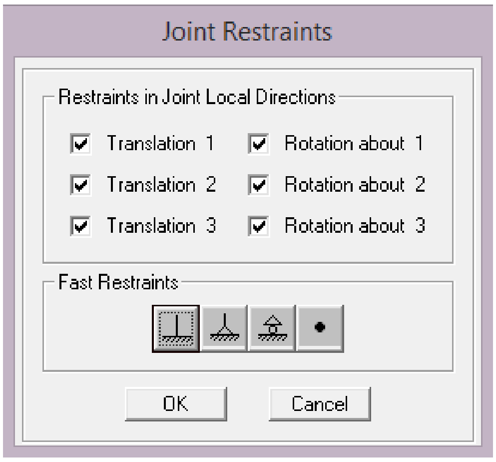


Figure 3.17: Add restraints at the base condition

Step 5: Define load cases

Define all the load cases for the structure accordingly, the load cases consist of dead load, live load and environmental load. After defined the load cases, load assigning are carried out.

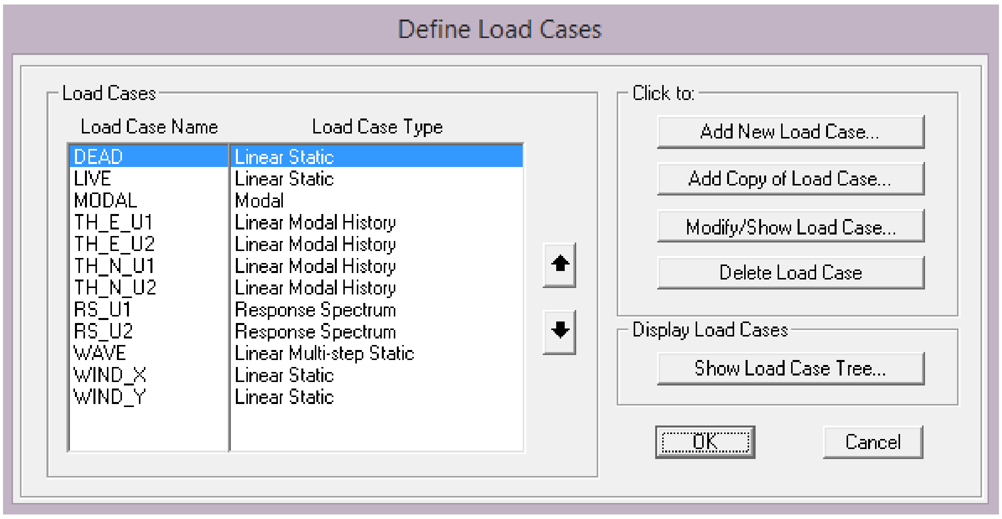


Figure 3.18: Define all load cases

After calculated the total dead load and live load per unit area, the load has been inserted as gravity area load which subjected on the slab area. Set the load cases as static and analysis type as linear for both dead load and live load.

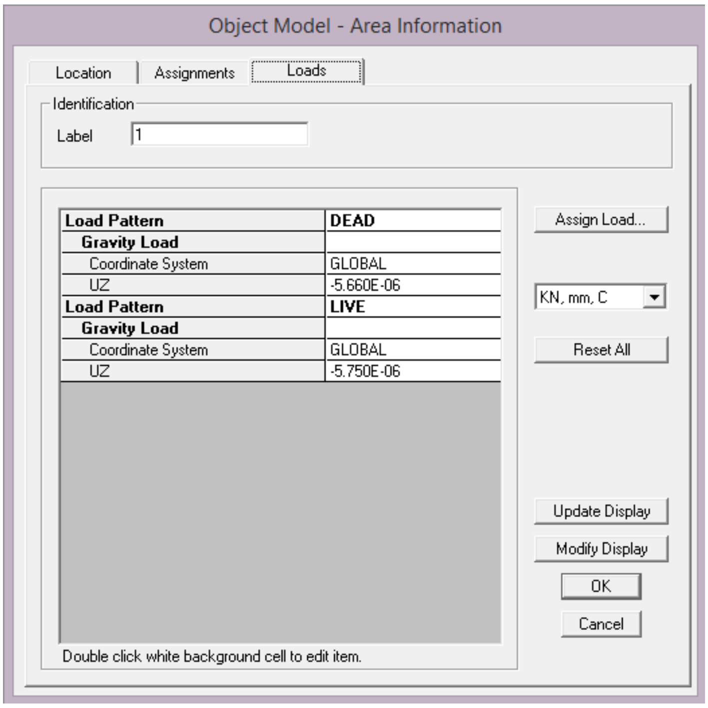


Figure 3.19: Dead & live loads

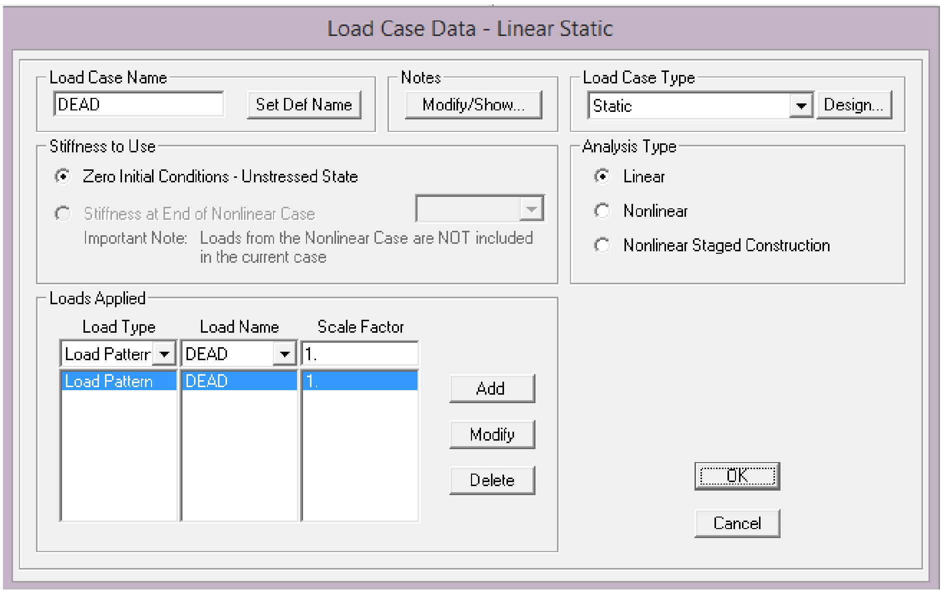


Figure 3.20: Dead load case data

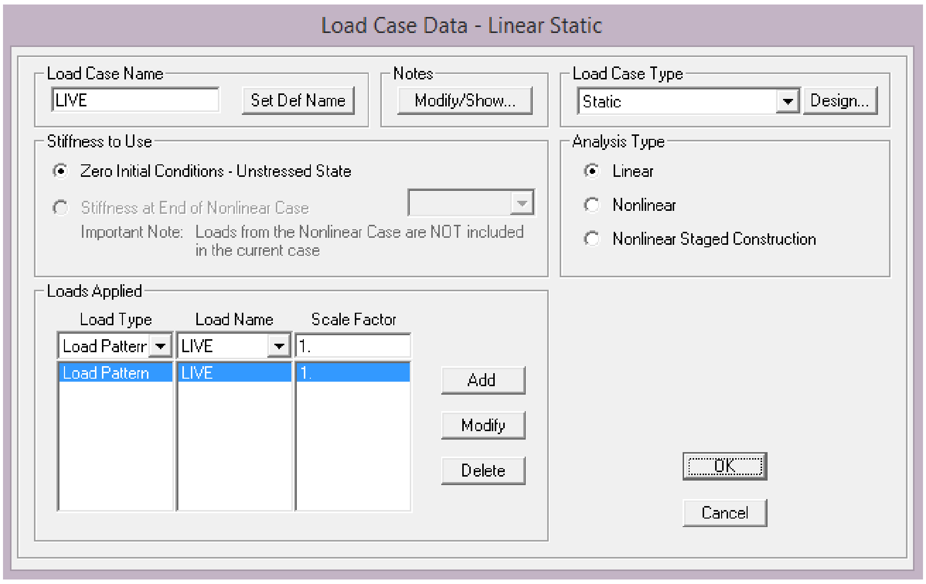


Figure 3.21: Live load case data

The environmental load in this study consists of wind load, wave load and current load. The load patterns of each of the environmental load have been defined. API 4F 2008 design codes have been implemented in the wind load case for both X and Y direction. While, wave and current load has been implemented API WSD 2000 as the design code. The collected details and data of the environmental load have been inserted in the SAP2000 program in the next stage.

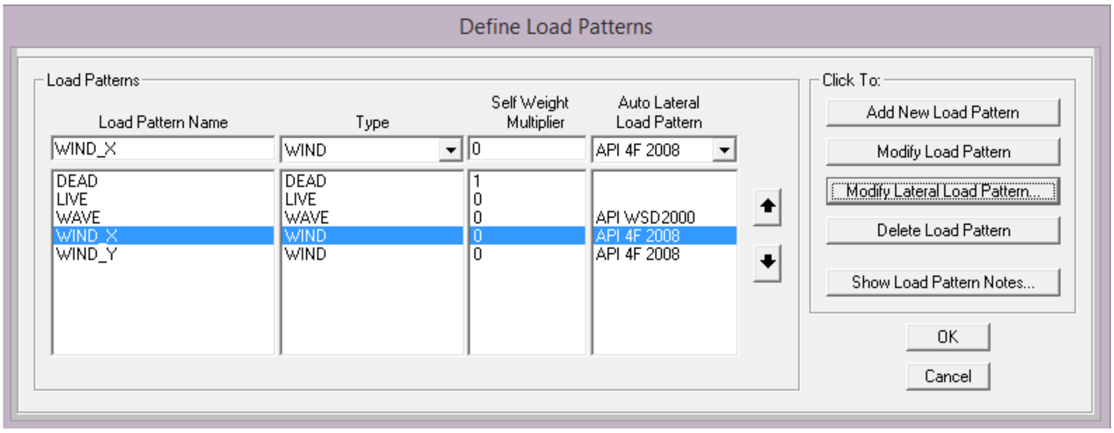


Figure 3.22: Define load patterns

For wind load case, the data on wind coefficients, exposure height and wind exposure parameters were inserted as showed at below. The wind load acted on the frame span is also showed as below.

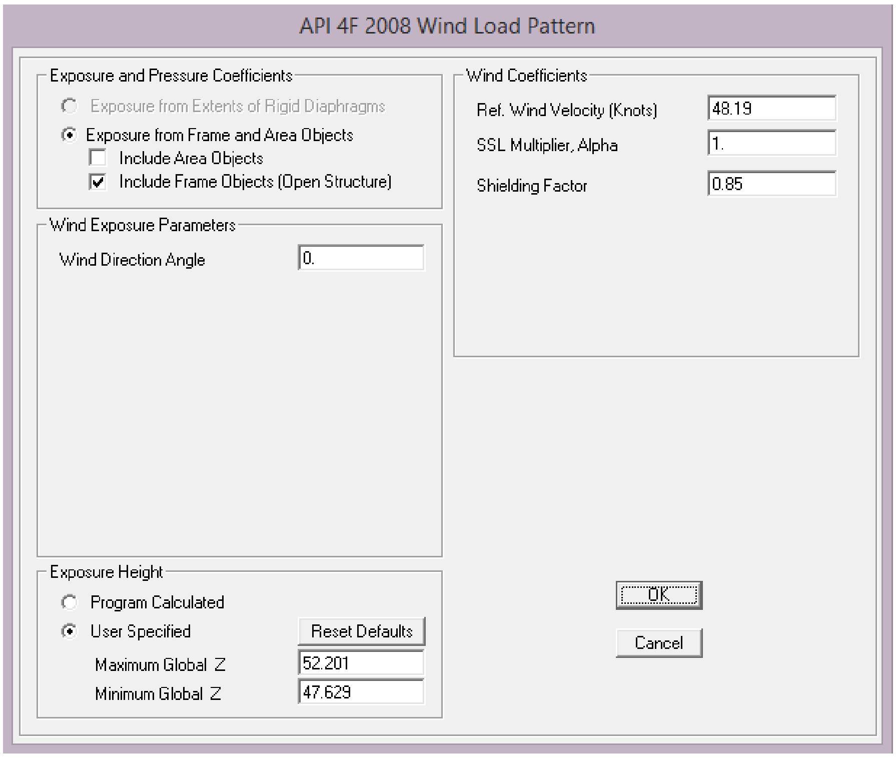


Figure 3.23: Wind load properties

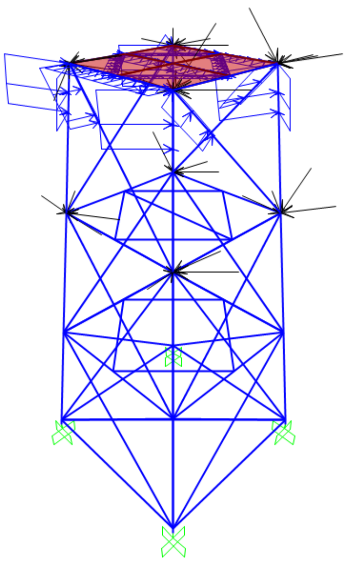


Figure 3.24: Frame open structural wind load

The wave and current load part, the wave and current data have been inserted accordingly based on the environment data after selected the design code.

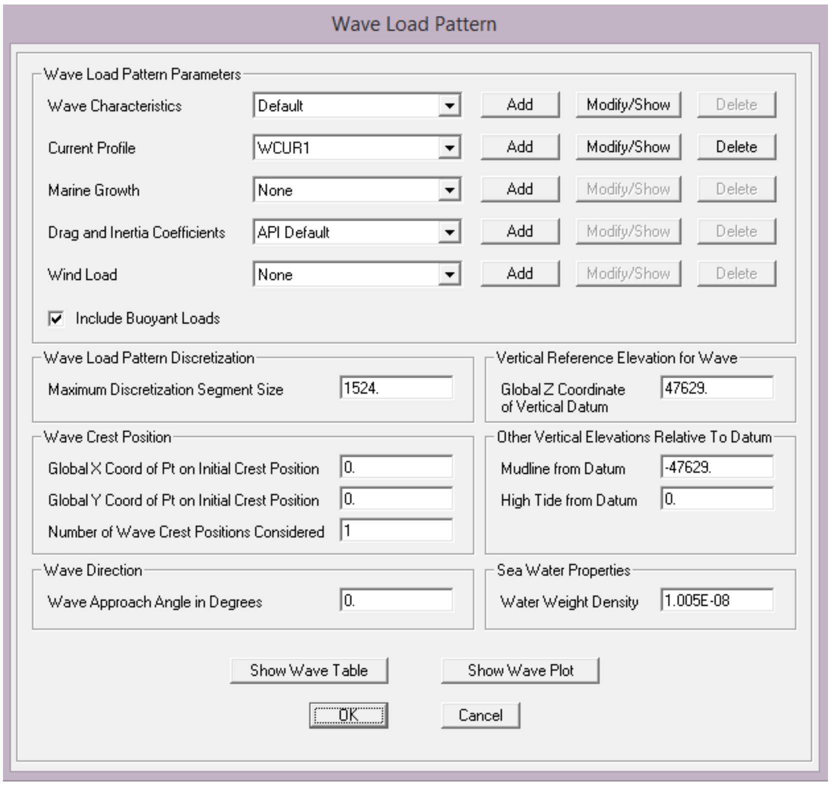


Figure 3.25: Wave load pattern

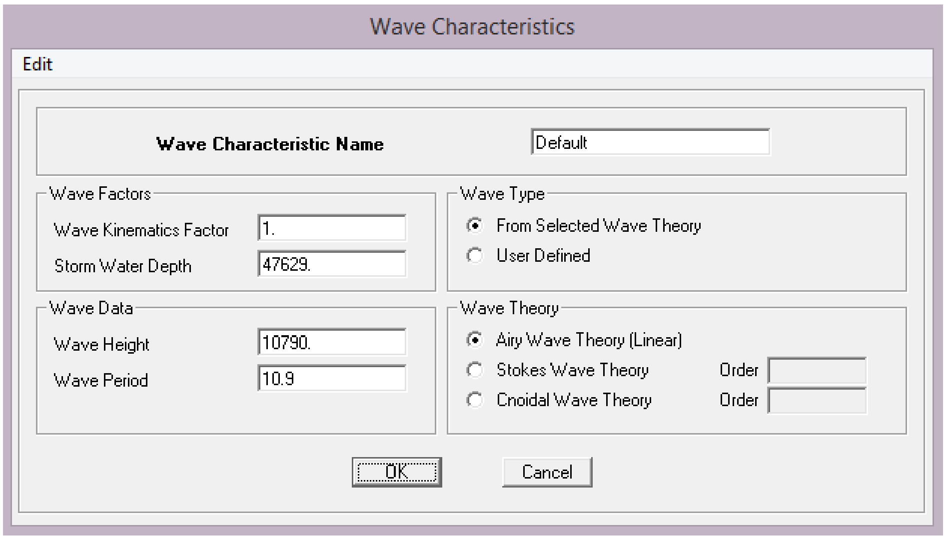


Figure 3.26: Wave characteristics

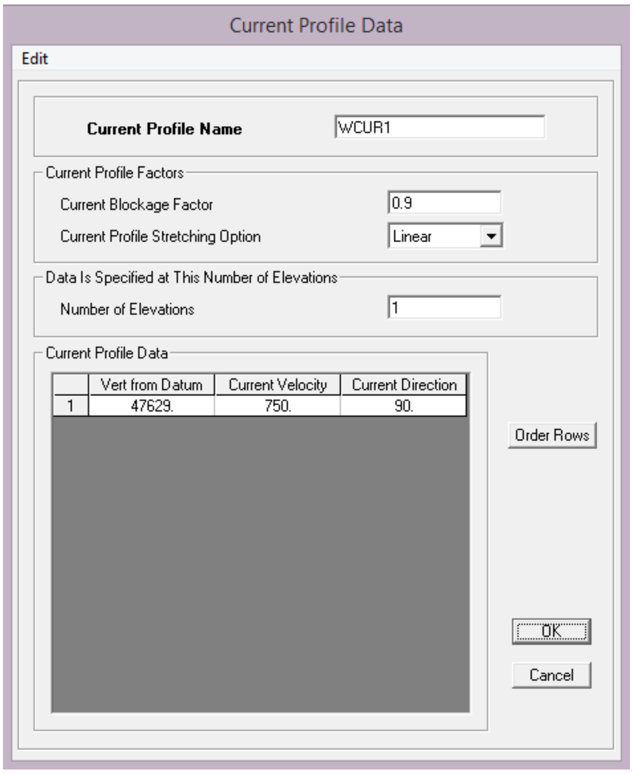


Figure 3.27: Current profile data

The wave plot display has been checked to ensure that the data is correctly inserted. The wave and current load acted on the frame span is also shown as below.

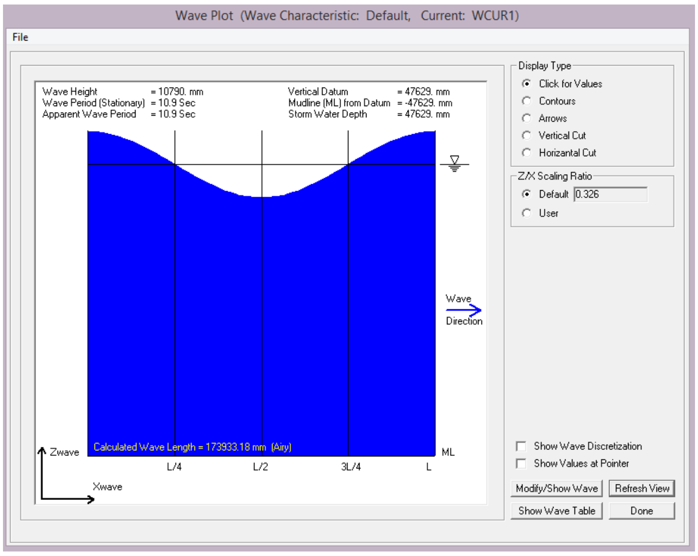


Figure 3.28: Wave Plot

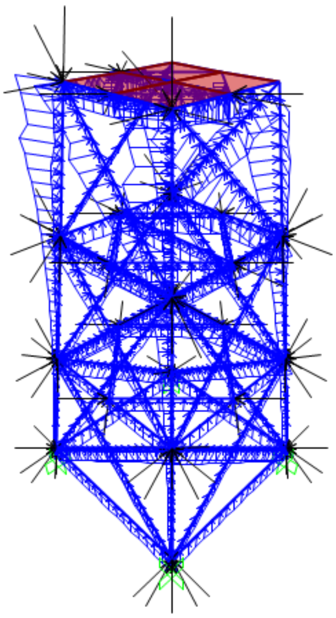


Figure 3.29: Frame Span Wave & Current Loads

Step 6: Define functions of Time History and Response Spectrum

Define the functions of Time History by attached the seismic data which in the notepad file and adjusting time interval based on the seismic data. Both E and N direction of Time History functions are determined by applying seismic data in a different direction. Moreover, load applied at U1 and U2 in each direction are determined. Eventually, four different Time History load cases will be performed in the analysis.

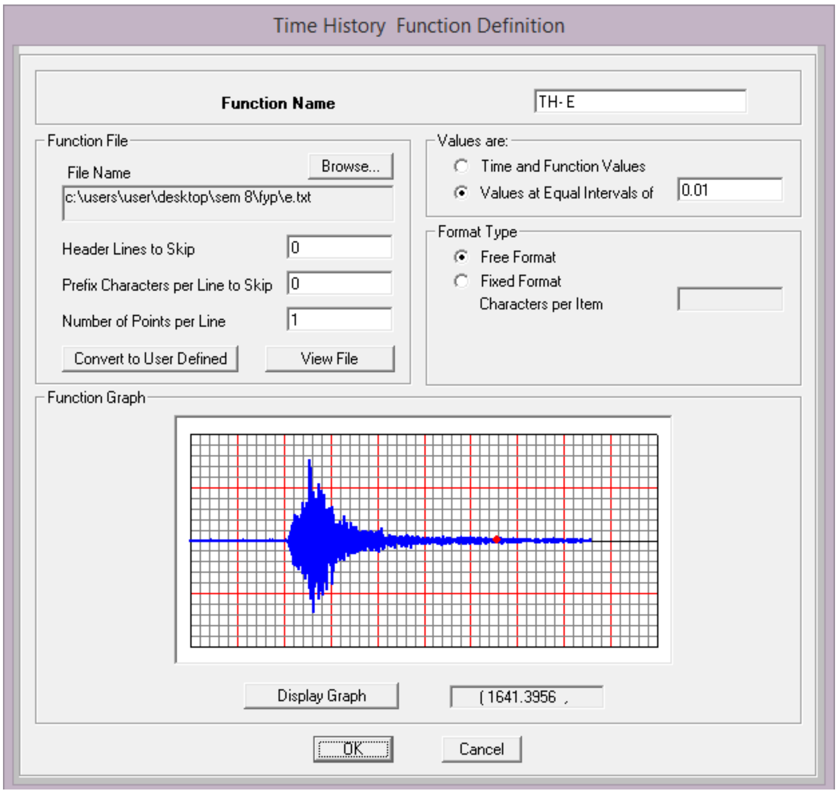
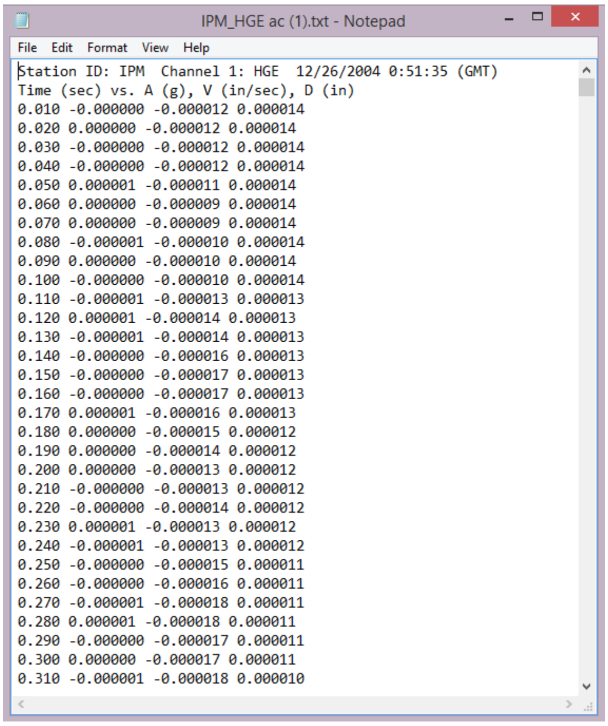


Figure 3.30: Define Time history function


IPM_HGE ac (1).txt - Notepad

File Edit Format View Help

```
Station ID: IPM  Channel 1: HGE  12/26/2004 0:51:35 (GMT)
Time (sec) vs. A (g), V (in/sec), D (in)
0.010 -0.000000 -0.000012 0.000014
0.020 0.000000 -0.000012 0.000014
0.030 -0.000000 -0.000012 0.000014
0.040 -0.000000 -0.000012 0.000014
0.050 0.000001 -0.000011 0.000014
0.060 0.000000 -0.000009 0.000014
0.070 0.000000 -0.000009 0.000014
0.080 -0.000001 -0.000010 0.000014
0.090 0.000000 -0.000010 0.000014
0.100 -0.000000 -0.000010 0.000014
0.110 -0.000001 -0.000013 0.000013
0.120 0.000001 -0.000014 0.000013
0.130 -0.000001 -0.000014 0.000013
0.140 -0.000000 -0.000016 0.000013
0.150 -0.000000 -0.000017 0.000013
0.160 -0.000000 -0.000017 0.000013
0.170 0.000001 -0.000016 0.000013
0.180 0.000000 -0.000015 0.000012
0.190 0.000000 -0.000014 0.000012
0.200 0.000000 -0.000013 0.000012
0.210 -0.000000 -0.000013 0.000012
0.220 -0.000000 -0.000014 0.000012
0.230 0.000001 -0.000013 0.000012
0.240 -0.000001 -0.000013 0.000012
0.250 -0.000000 -0.000015 0.000011
0.260 -0.000000 -0.000016 0.000011
0.270 -0.000001 -0.000018 0.000011
0.280 0.000001 -0.000018 0.000011
0.290 -0.000000 -0.000017 0.000011
0.300 0.000000 -0.000017 0.000011
0.310 -0.000001 -0.000018 0.000010
```

Figure 3.31: Seismic data in notepad

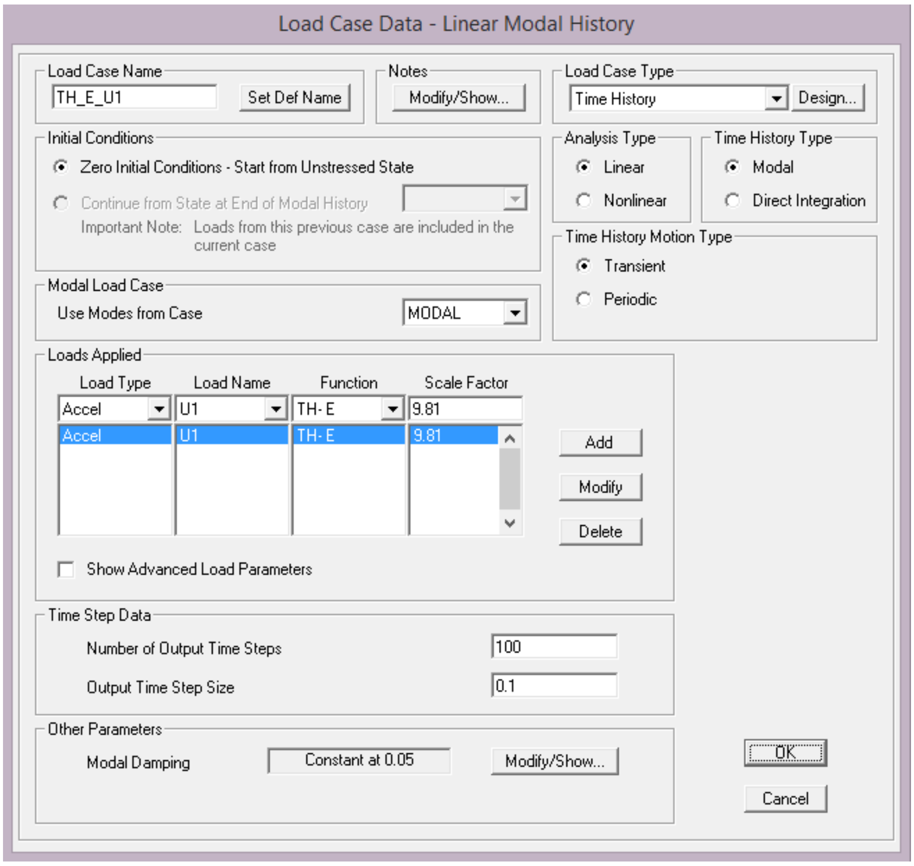


Figure 3.32: Linear modal history

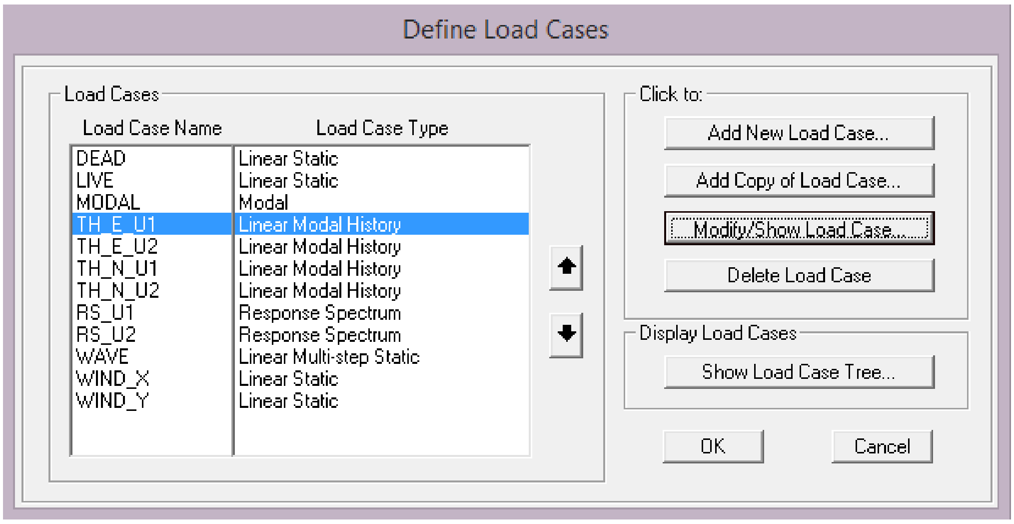


Figure 3.33: Time History load cases

While for the function of Response Spectrum, Eurocode8 2004 has selected as the function type for response spectrum. The details and data in the Response spectrum Eurocode8 2004 function definition are remain the default.

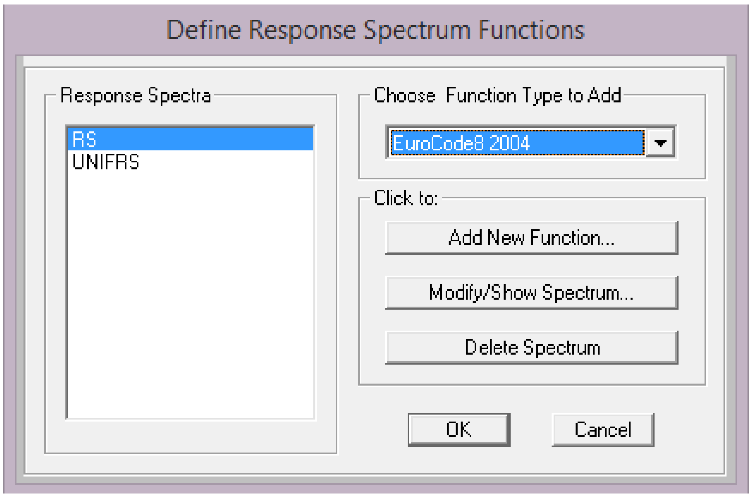


Figure 3.34: Response spectrum function

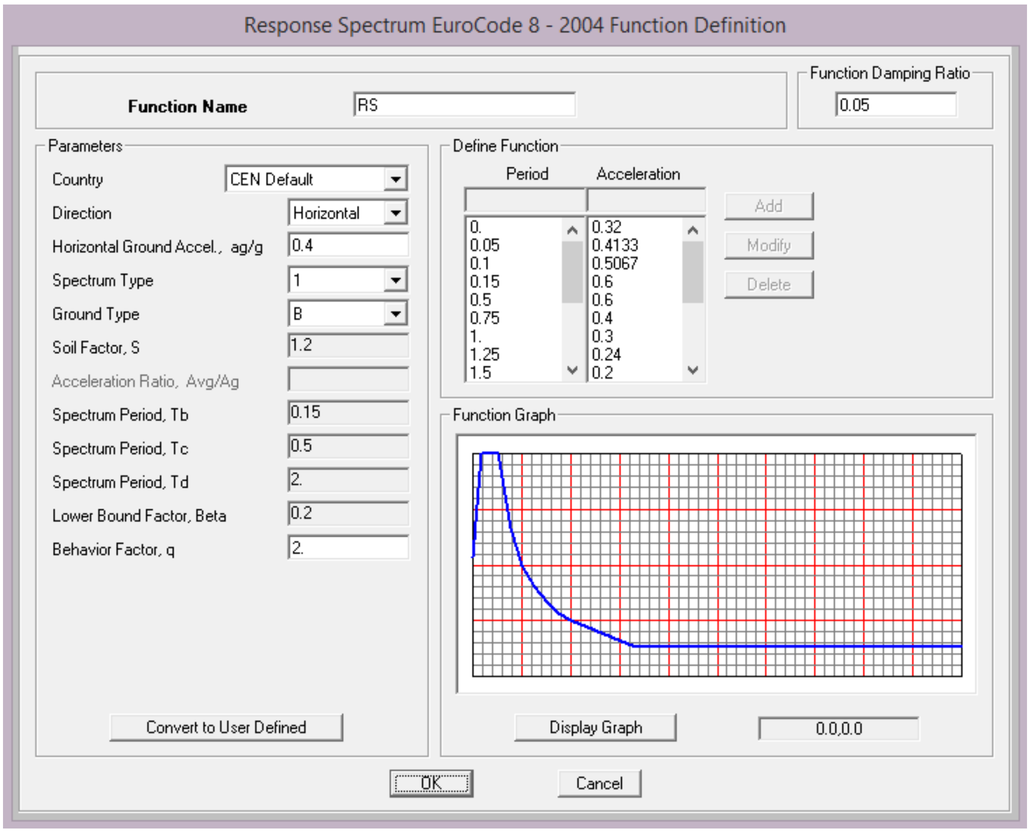


Figure 3.35: Response spectrum EC8 2004 function definition

Load applied for both U1 and U2 are considered in the analysis. Therefore, total of two response spectrum load cases is performed.

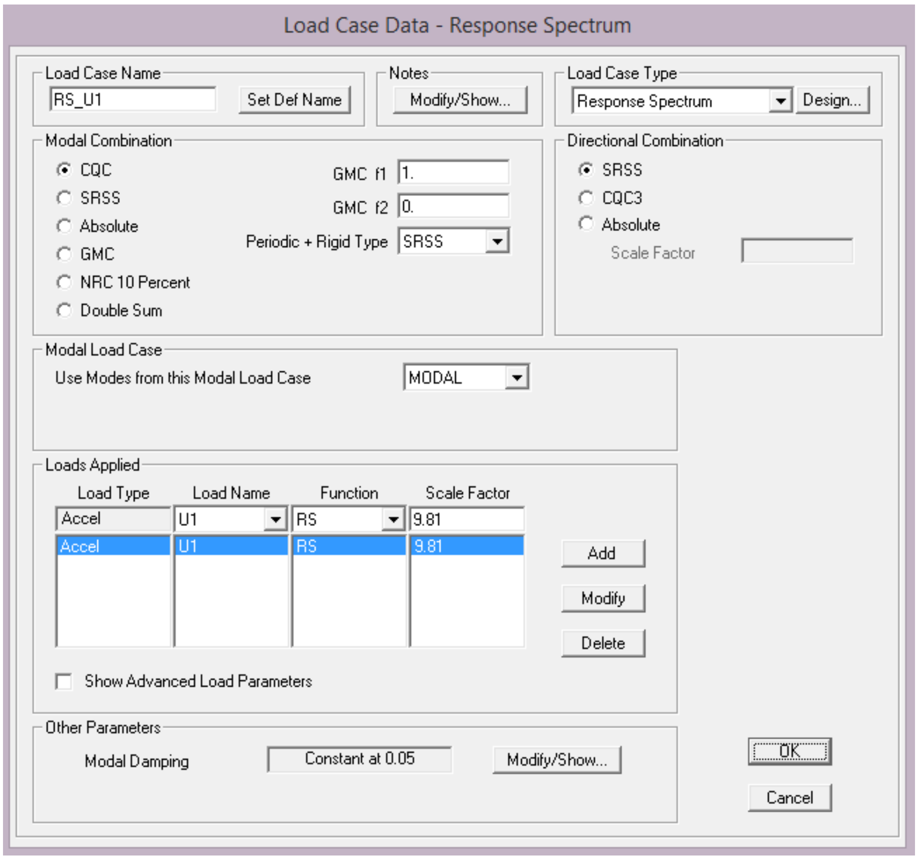


Figure 3.36: Response spectrum load case data

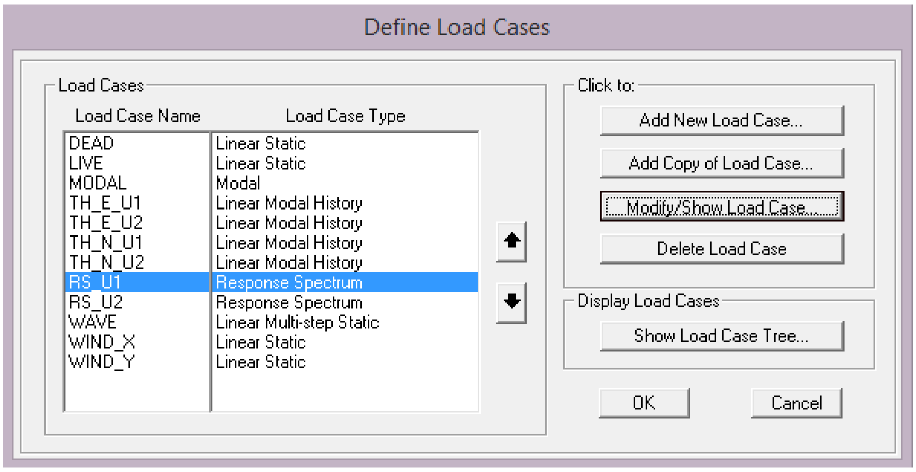


Figure 3.37: Response Spectrum load cases

Moreover, the modal load case is set as the figure below, maximum 12 numbers of modes shape and minimum 1 number of mode shape are having this study.

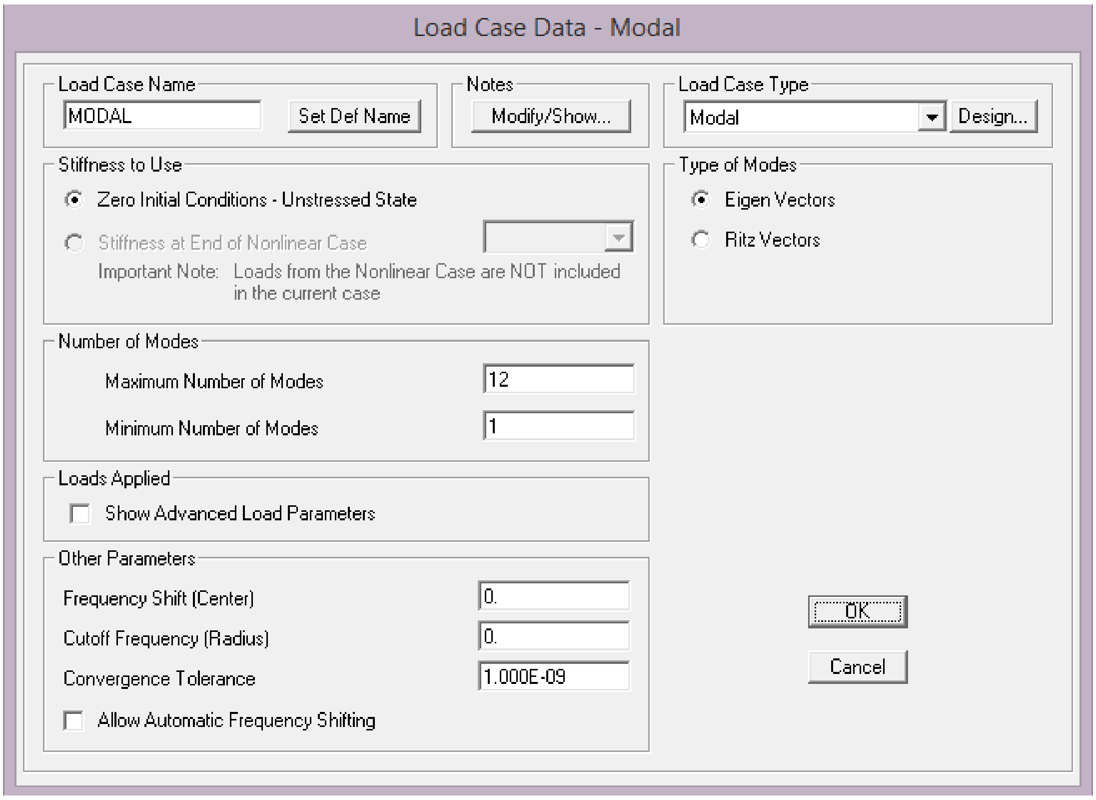


Figure 3.38: Modal load cases

Step 6: Analysis the Model

There is several combination of load cases that was applied in this analysis, these consist of:

i. Free vibration analysis (FVA)

Select the modal load case and run the analysis.

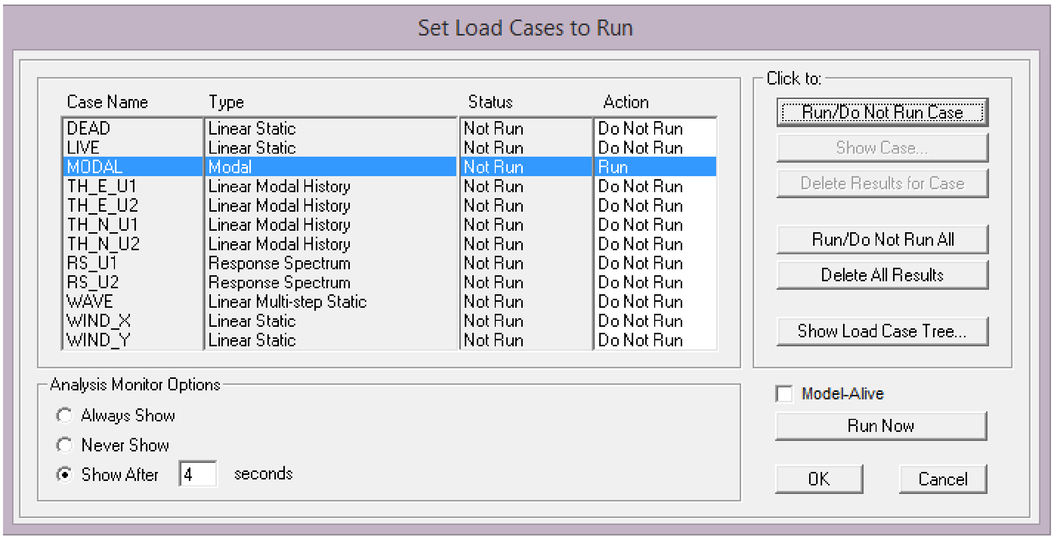


Figure 3.39: Modal load case

12 mode shapes, natural frequency and the natural period of the offshore structures were determined through this analysis.

ii. Dead load (DL) + live load (LL)

Select the dead load and live load cases and run the analysis.

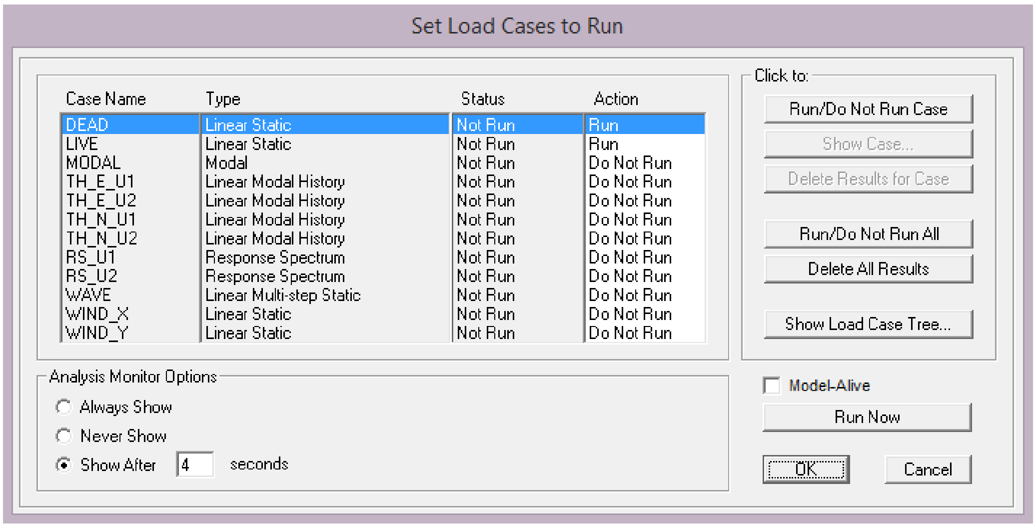


Figure 3.40: Dead load + Live load cases

After running the load cases analysis, check the steel structure accordance to EuroCode3, the critical member and maximum unity check of all the structures member are determined.

iii. Environmental load (EL)

Select the wind load, wave load and current load cases and run the analysis.

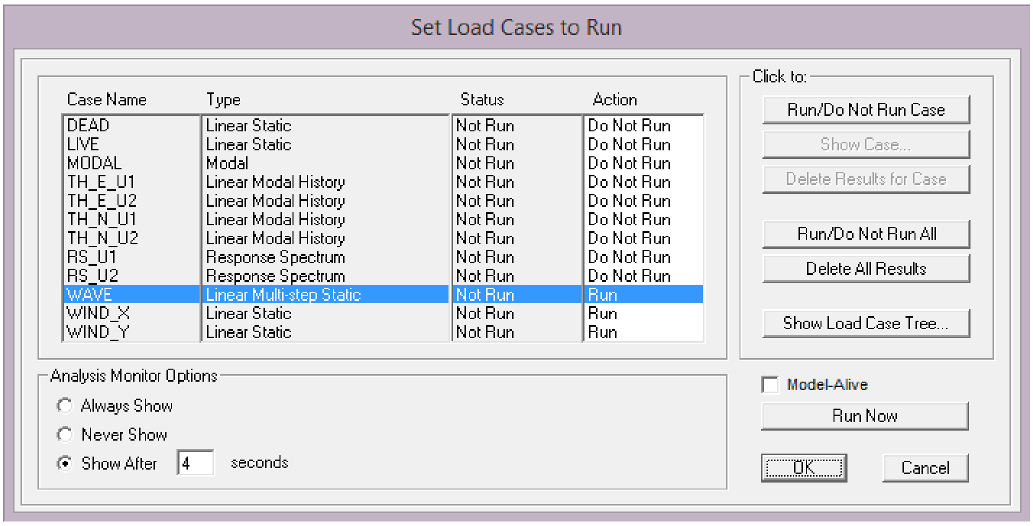


Figure 3.41: Environmental load cases

After running the load cases analysis, check the steel structure accordance to EuroCode3, the critical member and maximum unity check of all the structures member are determined.

iv. Dead load (DL) + live load (LL) + environmental load (EL) + time history load (TH)

Select the dead load, live load, environmental load and time history load cases and run the analysis.

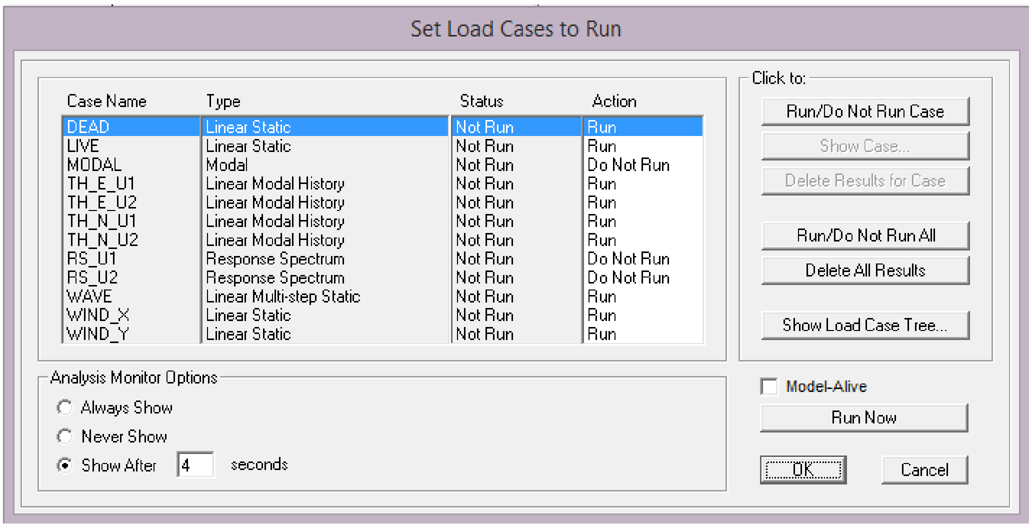


Figure 3.42: Dead Load + Live Load + Environmental Load + Time History load cases

After running the load cases analysis, check the steel structure accordance to EuroCode3, the critical member and maximum unity check of all the structures member are determined.

v. Response spectrum load (RS)

Select the both direction of response spectrum load cases and run the analysis.

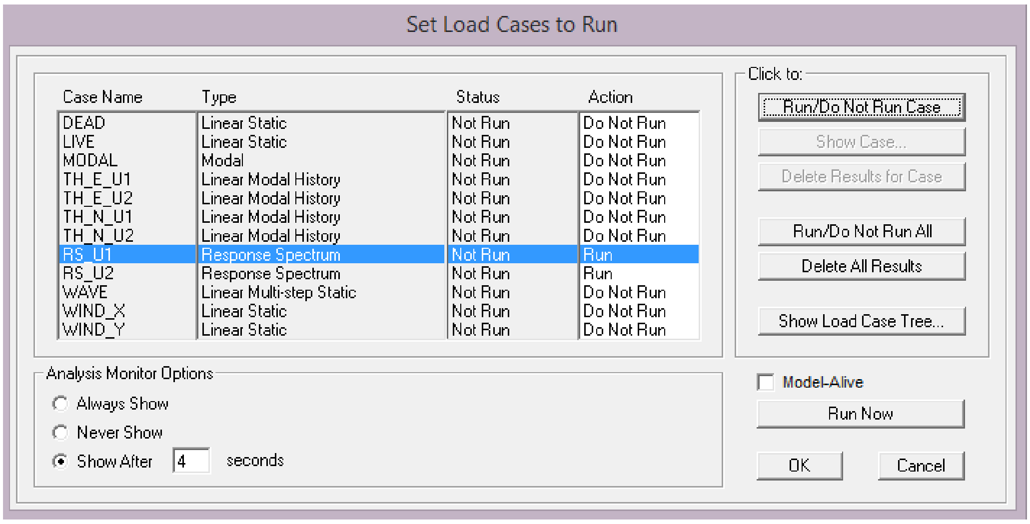


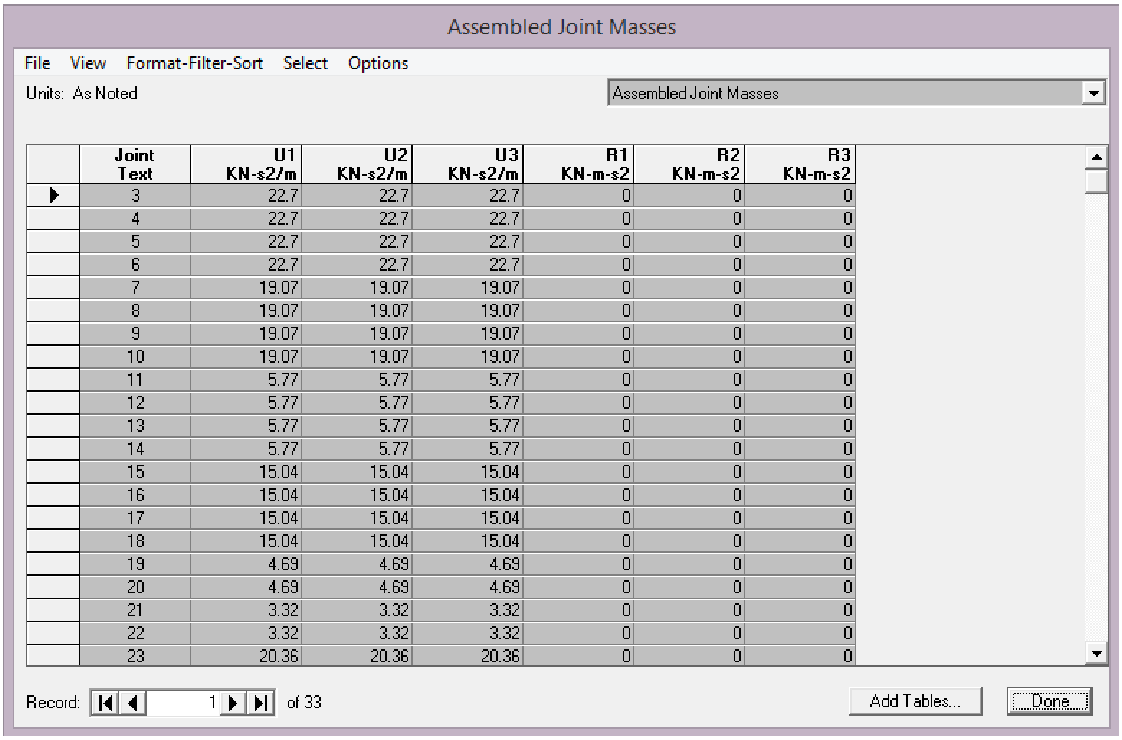
Figure 3.43: Response spectrum load cases

After running the load cases analysis, check the steel structure accordance to EuroCode3, the critical member and maximum unity check of all the structures member are determined.

Step 7: Display result and output table

After performed all the analysis, the results obtained from this study are as follows:

(i) Mode shape of the offshore structures
(ii) Natural period and natural frequency of the offshore structures
(iii) Maximum unity check of all the structures member
(iv) Maximum shear stress and bending stress in the member
(v) Shear force and bending moment under various load cases
(vi) Joint displacement, velocity and acceleration under different combination of load cases



Joint Text	U1 KN-s2/m	U2 KN-s2/m	U3 KN-s2/m	R1 KN-m-s2	R2 KN-m-s2	R3 KN-m-s2
3	22.7	22.7	22.7	0	0	0
4	22.7	22.7	22.7	0	0	0
5	22.7	22.7	22.7	0	0	0
6	22.7	22.7	22.7	0	0	0
7	19.07	19.07	19.07	0	0	0
8	19.07	19.07	19.07	0	0	0
9	19.07	19.07	19.07	0	0	0
10	19.07	19.07	19.07	0	0	0
11	5.77	5.77	5.77	0	0	0
12	5.77	5.77	5.77	0	0	0
13	5.77	5.77	5.77	0	0	0
14	5.77	5.77	5.77	0	0	0
15	15.04	15.04	15.04	0	0	0
16	15.04	15.04	15.04	0	0	0
17	15.04	15.04	15.04	0	0	0
18	15.04	15.04	15.04	0	0	0
19	4.69	4.69	4.69	0	0	0
20	4.69	4.69	4.69	0	0	0
21	3.32	3.32	3.32	0	0	0
22	3.32	3.32	3.32	0	0	0
23	20.36	20.36	20.36	0	0	0

Figure 3.44: Result output table

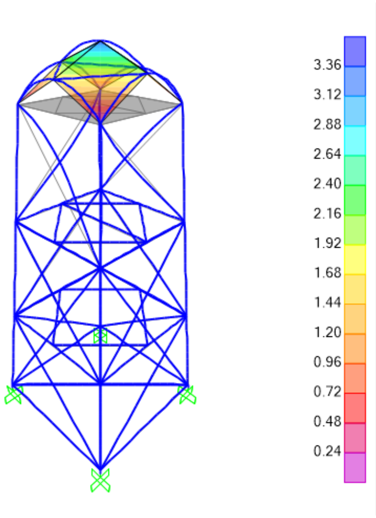
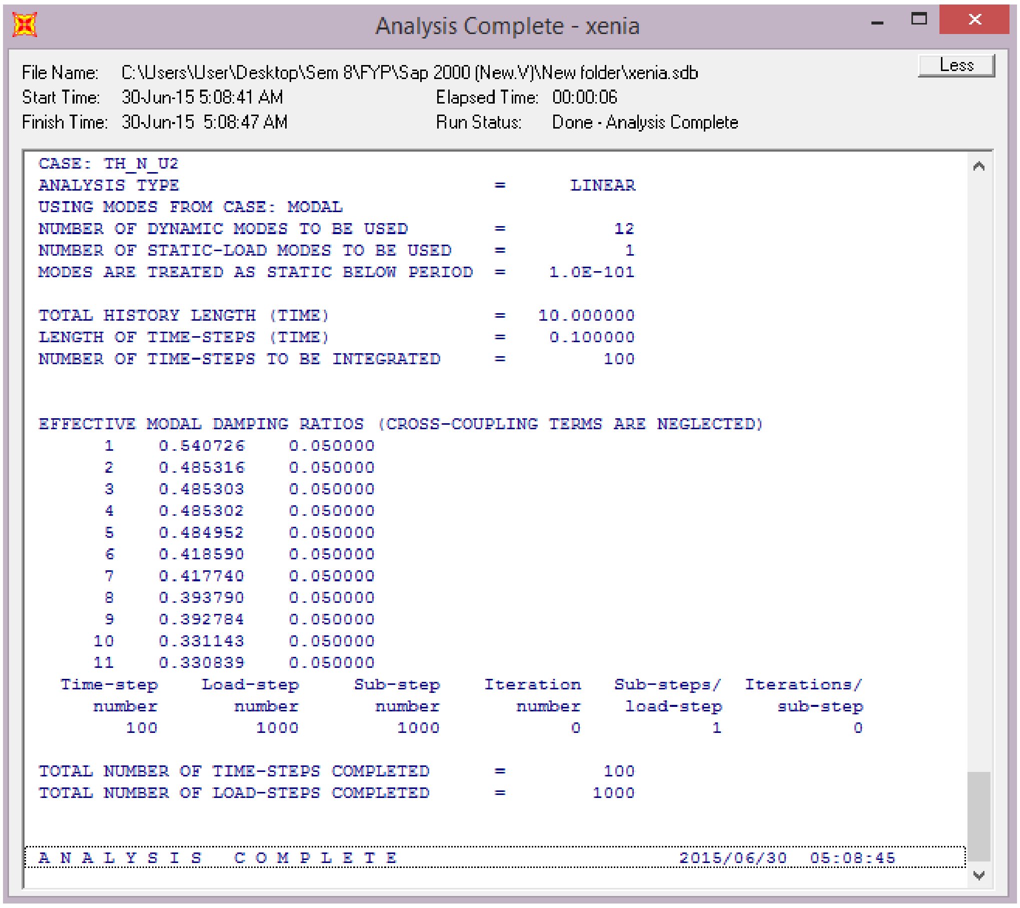


Figure 3.45: Mode shape result



Analysis Complete - xenia

Less

File Name: C:\Users\User\Desktop\Sem 8\FYP\Sap 2000 (New.V)\New folder\xenia.sdb
Start Time: 30-Jun-15 5:08:41 AM   Elapsed Time: 00:00:06
Finish Time: 30-Jun-15 5:08:47 AM   Run Status: Done - Analysis Complete

```
CASE: TH_N_U2
ANALYSIS TYPE                               =      LINEAR
USING MODES FROM CASE: MODAL
NUMBER OF DYNAMIC MODES TO BE USED          =          12
NUMBER OF STATIC-LOAD MODES TO BE USED      =           1
MODES ARE TREATED AS STATIC BELOW PERIOD    =    1.0E-101

TOTAL HISTORY LENGTH (TIME)                 =   10.000000
LENGTH OF TIME-STEPS (TIME)                 =    0.100000
NUMBER OF TIME-STEPS TO BE INTEGRATED       =         100


EFFECTIVE MODAL DAMPING RATIOS (CROSS-COUPLING TERMS ARE NEGLECTED)
      1     0.540726     0.050000
      2     0.485316     0.050000
      3     0.485303     0.050000
      4     0.485302     0.050000
      5     0.484952     0.050000
      6     0.418590     0.050000
      7     0.417740     0.050000
      8     0.393790     0.050000
      9     0.392784     0.050000
     10     0.331143     0.050000
     11     0.330839     0.050000
  Time-step     Load-step     Sub-step     Iteration    Sub-steps/   Iterations/
     number        number       number        number     load-step      sub-step
        100          1000         1000             0             1             0

TOTAL NUMBER OF TIME-STEPS COMPLETED        =         100
TOTAL NUMBER OF LOAD-STEPS COMPLETED        =        1000


A N A L Y S I S   C O M P L E T E                      2015/06/30   05:08:45
```

Figure 3.46: Summary of result

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 OFFSHORE STRUCTURE ANALYSIS

The fixed offshore structures have been modelled and analyzed using SAP2000, an integrated software for structural analysis and design. The free vibration analysis, time history earthquake analysis and response spectrum earthquake analysis have been performed in this study. The loads that considered in this analysis consist of dead load, live load, modal load, environmental load, time history load and also response spectrum load, while the environmental load comprises of wind, wave and current loads. As mentioned in chapter 3, there are several combination of load cases that was applied in this analysis, these consist of:

i. Free vibration analysis (FVA)
ii. Dead load (DL) + live load (LL)
iii. Environmental load (EL)
iv. Dead load (DL) + live load (LL) + environmental load (EL) + time history load (TH)
v. Response spectrum load (RS)

Moreover, the results obtained from this study are as follows:

i. Mode shape of the offshore structures
ii. Natural period and natural frequency of the offshore structures
iii. Maximum unity check of all the structures member
iv. Maximum shear stress and bending stress in the member
v. Shear force and bending moment under various load cases
vi. Joint displacement, velocity and acceleration under different combination of load cases

4.2 OFFSHORE STRUCTURE MODELLING

The fixed offshore structure modelling has been done using SAP2000, an integrated software for structural analysis and design. Figure 4.1 shows the 3d model of this fixed offshore structure. This offshore structure was modelled using the linear properties and several assumptions have been made throughout the modelling stage. There are:

i. The geometry, section properties and material properties of the structure is represented almost the same from the actual structure.
ii. The structure was assumed to fix on the ground instead of pilled.

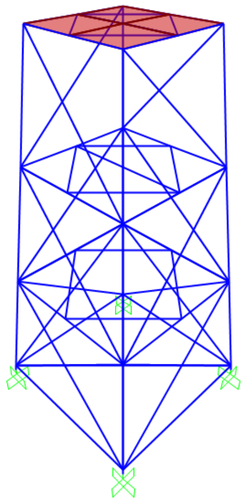


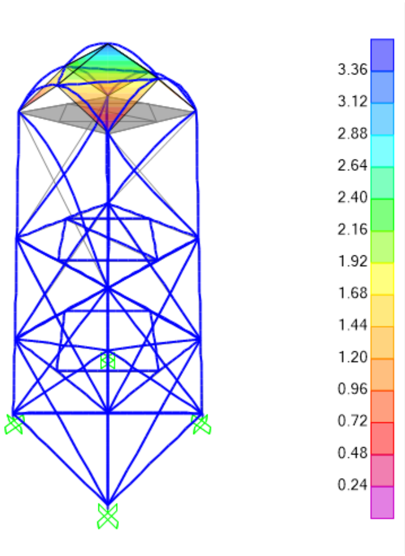
Figure 4.1: 3D model of the fixed offshore structure

4.3 FREE VIBRATION ANALYSIS

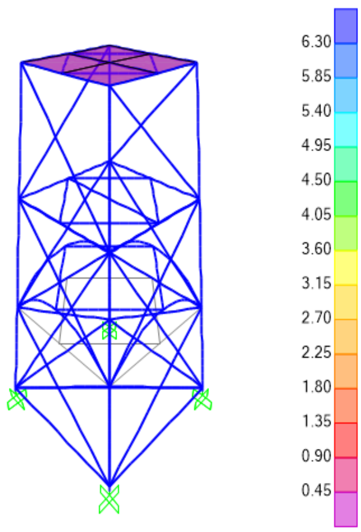
Free vibration analysis has been carried out by using SAP2000 software. The natural frequency, natural period and twelve (12) mode shapes of offshore structure have been obtained based on this analysis. The summary of the results has been tabulated in Table 4.1. Generally, the mode shape represents the shape that the structure will vibrate in free motion and the same shape tends to dominate the motion of the structure during an earthquake. By understanding the modes of vibration, it will help to easier and better design structures to withstand earthquake. The first mode of vibration is the one of primary interest because the first mode usually has the largest contribution to the structure's motion. The natural period of the first mode is the longest and follows by the second mode. The mode shape 1 and 2 are shown in Figure 4.2 (a) and (b) respectively. The grey colour lines show the original undeflected shape of the offshore structure, while the blue colour lines show the deflected shape of the offshore structure.

Table 4.1: The result from the free vibration analysis

Mode	Natural Period, T (sec)	Natural Frequency, f (Hz)
1	0.5407	1.8494
2	0.4853	2.0605
3	0.4853	2.0606
4	0.4853	2.0606
5	0.4850	2.0621
6	0.4186	2.3890
7	0.4177	2.3938
8	0.3938	3.5394
9	0.3928	2.5459
10	0.3311	3.0198
11	0.3308	3.0226
12	0.3172	3.1527



(a)



(b)

Figure 4.2: Two of the vibration modes (a) mode shape 1 (0.5407 sec) (b) mode shape 2 (0.4853 sec)

4.4 TIME HISTORY EARTHQUAKE ANALYSIS

Time history earthquake analysis has been performed on the fixed offshore structure by referring to the earthquake data obtained from the Malaysian Meteorological Department. Figure 4.3 and 4.4 show the graph of time versus acceleration in both E and N direction. The result obtained from this analysis is time history responses at joints and stresses for frame elements of the fixed offshore structure.

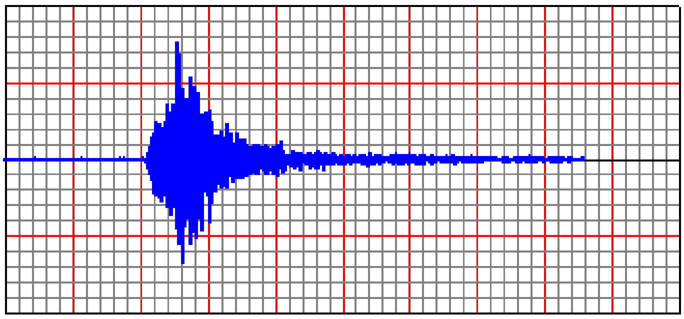


Figure 4.3: Time (sec) vs Acceleration (g) in E-direction

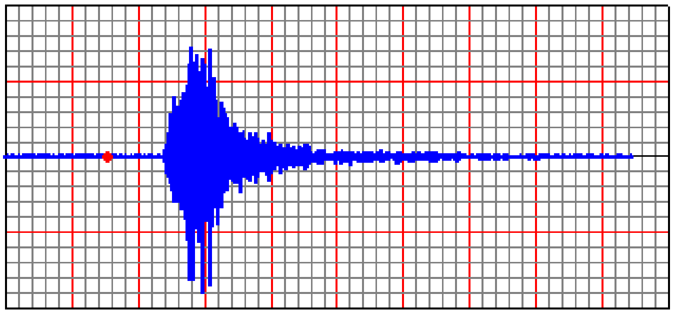


Figure 4.4: Time (sec) vs Acceleration (g) in N-direction

4.5 LINEAR ANALYSIS

There are several combination of load cases that been applied in this linear analysis, these consist of combination of dead load (DL) + live load (LL), environmental load (EL), dead load (DL) + live load (LL) + environmental load (EL) + time history load (TH) and response spectrum load (RS). The most critical frame element in this offshore structure has been determined which is frame element 27 with 0.518 ratio. Moreover, the results of the various combination of load cases of this frame element have been determined and presented in the table and diagram form. The result and output obtained consist of the shear force diagram and bending moment diagram

The table 4.2 showed the shear force diagrams of several combination of load cases that was applied in this linear analysis. The result of the shear force diagram for the combination of dead load (DL) + live load (LL) linear analysis is shown as below. From the shear force diagram, the maximum shear force is 165.431 KN which happen at the frame element 27. From the maximum shear stress result, shear stress and allowable capacity check have been calculated and tabulated in the table 4.3, which is 4,364.93 kN/m^2 and 102,750 kN/m^2 respectively.

Moreover, the result of the shear force diagram for the combination of environmental load (EL) also showed as below. Wind load, wave load and current load are the load that comprises in this environmental load analysis. From the shear force diagram, the maximum shear force for environmental load analysis is 11.195 KN which happen at the frame element 27. From the maximum shear stress result, shear stress and allowable capacity check has been calculated and tabulated in the table 4.3, which is 295.38 kN/m^2 and 102,750 kN/m^2 respectively.

The response spectrum (RS) analysis have been performed, the shear force diagram have been obtained as showed in the table 4.2, the maximum shear force of this analysis which happens at the frame element 27 is 14.702 KN. From the maximum shear stress result, shear stress and allowable capacity check have been calculated and tabulated in the table 4.3, which is 387.92 kN/m^2 and 102,750 kN/m^2 respectively.

For the result of combination load cases of dead load (DL) + live load (LL) + environmental load (EL) + time history load (TH) are determined, the shear force diagram have obtained as showed in the table 4.2. The maximum shear force of this analysis which happens at the frame element 27 is 14.702 KN. From the maximum shear stress result, shear stress and allowable capacity check has been calculated and tabulated in the table 4.3, which is 4,159.74 kN/m^2 and 102,750 kN/m^2 respectively.

Table 4.2: Shear force diagrams of several combination of load cases happen at frame element 27

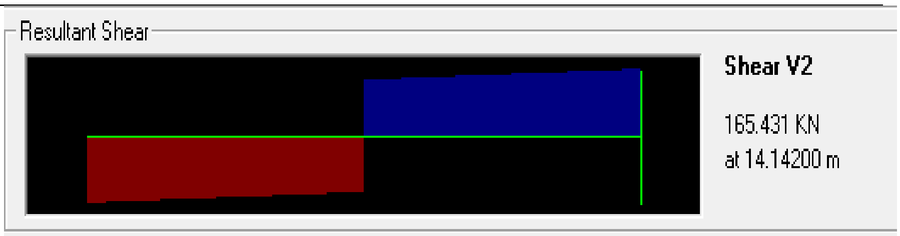
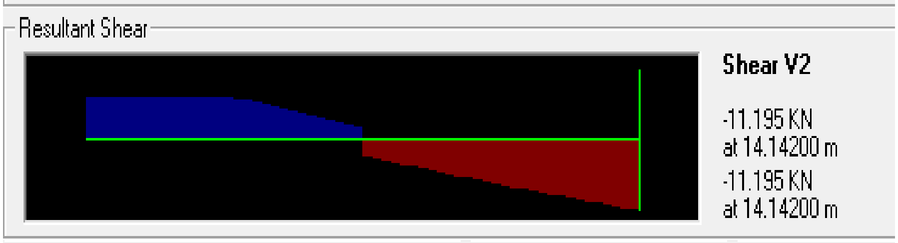
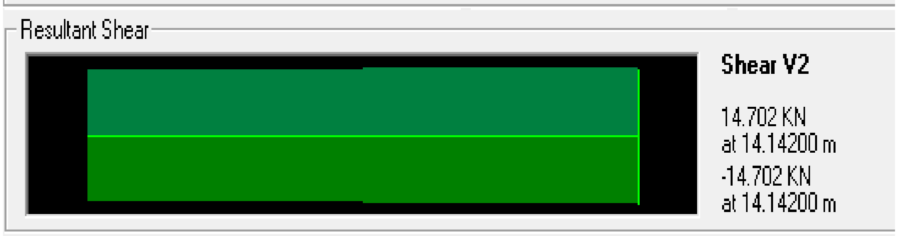
Load combination	**Shear Force Diagram**
DL + LL	
EL	
RS	
DL + LL + EL + TH	

Table 4.3: Shear stress and allowable capacity check of several combination of load cases

Load combination	Design shear force, V_{Ed} (kN)	Shear stress, f_v (kN/m^2)	Allowable shear stress, F_v (kN/m^2)
DL + LL	165.431	4,364.93	102,750
EL	-11.195	-295.38	102,750
RS	14.702	387.92	102,750
DL + LL + EL + TH	-157.654	-4,159.74	102,750

The table 4.4 showed the bending moment diagrams of several combination of load cases that was applied in this linear analysis. The result of bending moment diagram for the combination of dead load (DL) + live load (LL) linear analysis are shown as below. From the bending moment diagram, the maximum bending moment is 1,071.21 KNm which happen at the frame element 27. From the maximum bending moment result, bending stress and allowable capacity check has been calculated and tabulated in the table 4.5, which are 245,297.57 kN/m^2 and 474,498.51 kN/m^2 respectively.

Moreover, the result of bending moment diagram for the combination of environmental load (EL) also showed as below. Wind load, wave load and current load are the load that comprises in this environmental load analysis. From the bending moment diagram, the maximum bending moment for environmental load analysis is 49.76 KNm which happen at the frame element 27. From the maximum bending moment result, bending stress and allowable capacity check has been calculated and tabulated in the table 4.5, which are 11,394.78 kN/m^2 and 474,498.51 kN/m^2 respectively.

The response spectrum (RS) analysis have been performed, the bending moment diagram have been obtained as showed in the table 4.4, the maximum bending moment of this analysis which happens at the frame element 27 is 103.96 KNm. From the maximum bending moment result, bending stress and allowable capacity check has been calculated and tabulated in the table 4.5, which are 23,804.88 kN/m^2 and 474,498.51 kN/m^2 respectively.

For the result of combination load cases of dead load (DL) + live load (LL) + environmental load (EL) + time history load (TH) are determined, the bending moment diagram has obtained as showed in the table 4.4. The maximum bending moment of this analysis which happens at the frame element 27 is 1,023.68 KNm. From the maximum bending moment result, bending stress and allowable capacity check has been calculated and tabulated in the table 4.5, which are 234,412.07 kN/m^2 and 474,498.51 kN/m^2 respectively.

Table 4.4: Bending moment diagrams of several combination of load cases happen at frame element 27

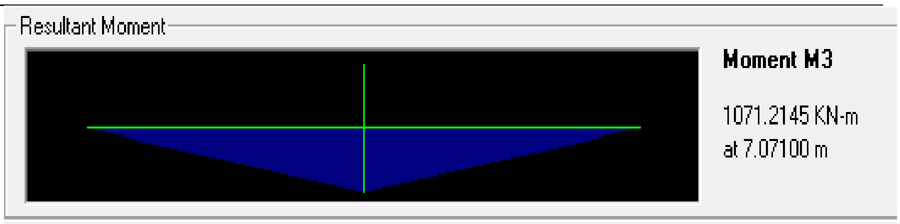
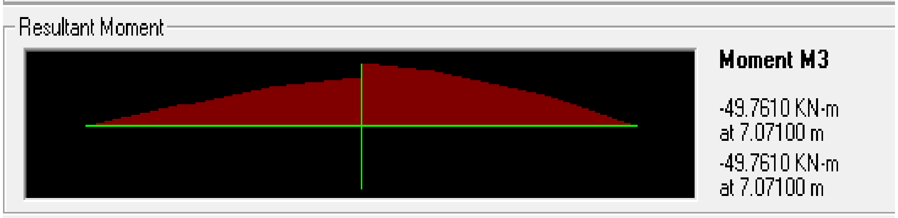
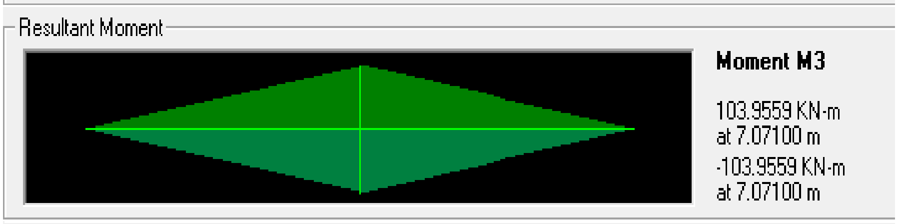
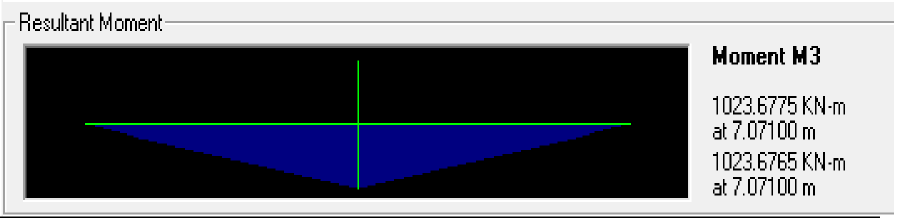
Load combination	**Bending Moment Diagram**
DL + LL	
EL	
RS	
DL + LL + EL + TH	

Table 4.5: Bending moment and allowable capacity check of several combination of load cases

Load combination	Design moment, M_{ed} (kNm)	Bending stress, f_b (kN/m^2)	Allowable bending stress, F_b (kN/m^2)
DL + LL	1,071.21	245,297.57	474,498.51
EL	49.76	11,394.78	474,498.51
RS	103.96	23,804.88	474,498.51
DL + LL + EL + TH	1,023.68	234,412.07	474,498.51

4.6 COMPARISON OF RESULTS MANUAL CALCULATION AND SAP2000

In order to obtain a safe design for our structure, we had tried a variety of design options including design manually or design using a computer. Then, we compared our manual design with our computer design. The results for both manual calculation and computer design indicate that our structural design is safe and sound.

Below shows in the comparison of our manual design and design using SAP2000. We checked and compared the final steel P-M interaction ratios obtained from both the manual calculation with the SAP2000 calculation. All steel frames passed the stress/ capacity check as the ratio is less than 1 as shown in the table below. For the SAP2000, the most critical frame member in this offshore model is a frame member 27 of 0.518 ratio. While, for manual calculation the most critical frame member in this offshore model is a frame member 27 with 0.75 ratio. We are fortunate to obtain a safe design for our structure and the results are shown below.

Table 4.6: P-M interaction ratios of manual calculation and SAP2000

Element Design	Frame Number	P-M interaction ratios	
		Manual Calculation	SAP2000
Beam Design	27	0.75	0.518
Column Design	66	0.09	0.199
Truss Design (Tension)	81	0.01	0.417
Truss Design (Compression)	99	0.001	0.406

4.7 SUMMARY OF THE ANALYSIS

In the end of the analysis, element unity check has been performed, the final steel P-M interaction ratios are determined and displayed. All steel frames passed the stress/ capacity check as the ratio is less than 1 as showed in the table below. The most critical frame member in this offshore model is a frame member 27 of 0.518 ratio.

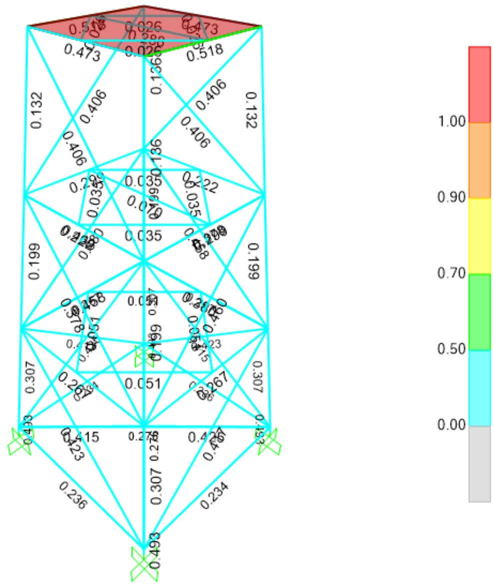


Figure 4.5: The final steel P-M interaction ratios are displayed.

Table 4.7: Unity ratio of the Frame Member

Frame Member	Design Section	Design Type	Maximum Unity Check
2	559@25	Beam	0.234
3	559@25	Beam	0.236
4	559@25	Beam	0.234
5	559@25	Beam	0.236
6	559@25	Beam	0.276
7	559@25	Beam	0.276
8	508@19	Beam	0.267
9	508@19	Beam	0.267

10	508@19	Beam	0.267
11	508@19	Beam	0.267
12	508@19	Beam	0.051
13	508@19	Beam	0.051
14	508@19	Beam	0.051
15	508@19	Beam	0.051
16	508@13	Beam	0.299
17	508@13	Beam	0.222
18	508@13	Beam	0.299
19	508@13	Beam	0.222
20	508@13	Beam	0.070
21	508@13	Beam	0.035
22	508@13	Beam	0.035
23	508@13	Beam	0.035
24	508@13	Beam	0.035
25	508@25	Beam	0.518
26	508@25	Beam	0.473
27	508@25	Beam	0.518
28	508@25	Beam	0.473
29	508@25	Beam	0.288
30	508@25	Beam	0.026
31	508@25	Beam	0.026
32	508@25	Beam	0.026
33	508@25	Beam	0.026
59	1168@13	Column	0.493
60	1168@13	Column	0.307
61	1168@13	Column	0.199
62	1168@13	Column	0.136
64	1168@13	Column	0.493
65	1168@13	Column	0.307
66	1168@13	Column	0.199
67	1168@13	Column	0.132
69	1168@13	Column	0.493
70	1168@13	Column	0.307
71	1168@13	Column	0.199
72	1168@13	Column	0.136
74	1168@13	Column	0.493
75	1168@13	Column	0.307
76	1168@13	Column	0.199
77	1168@13	Column	0.132
79	610@13	Brace	0.423
80	610@13	Brace	0.415
81	610@13	Brace	0.417
82	610@13	Brace	0.425
83	610@13	Brace	0.415

84	610@13	Brace	0.423
85	610@13	Brace	0.417
86	610@13	Brace	0.425
87	559@13	Brace	0.458
88	559@13	Brace	0.378
89	559@13	Brace	0.460
90	559@13	Brace	0.380
91	559@13	Brace	0.458
92	559@13	Brace	0.378
93	559@13	Brace	0.460
94	559@13	Brace	0.380
96	559@19	Brace	0.406
98	559@19	Brace	0.406
99	559@19	Brace	0.406
100	559@19	Brace	0.406

Figures 4.6 and 4.7, show the bending stress and shear stress based on load cases. In figure 4.6, it shows that the highest bending stress occurs at the load cases of dead and live loads at 245,300kN/m^2, while the allowable bending stress is 474,500kN/m^2. It means that, although the value of bending stress is the high, but still the structure can resist. This is because the intensity of the earthquake is not strong enough to make the structure collapse. The reason for the result of load cases of dead load, live load, environmental load and time history load is much greater than the load cases of dead and live load may be due to the counteract of the stress. Counteract may occur and act against stress and bringing the result of reducing or neutralize by its force.

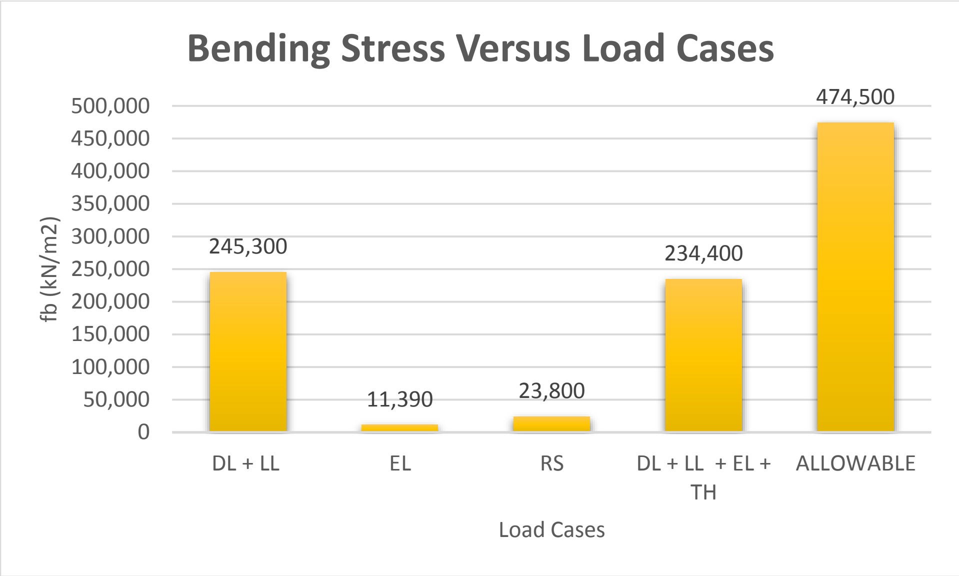


Figure 4.6: Graph of Bending Stress versus Load Cases

Figure 4.7 shows the graph for the shear stress in each case. From the graph, we can justify that the highest shear stress is 4,364.93kN/m^2, while the allowable shear stress is 120,750kN/m^2. Same with the bending stress, all values of shear stress do not exceed the allowable shear stress.

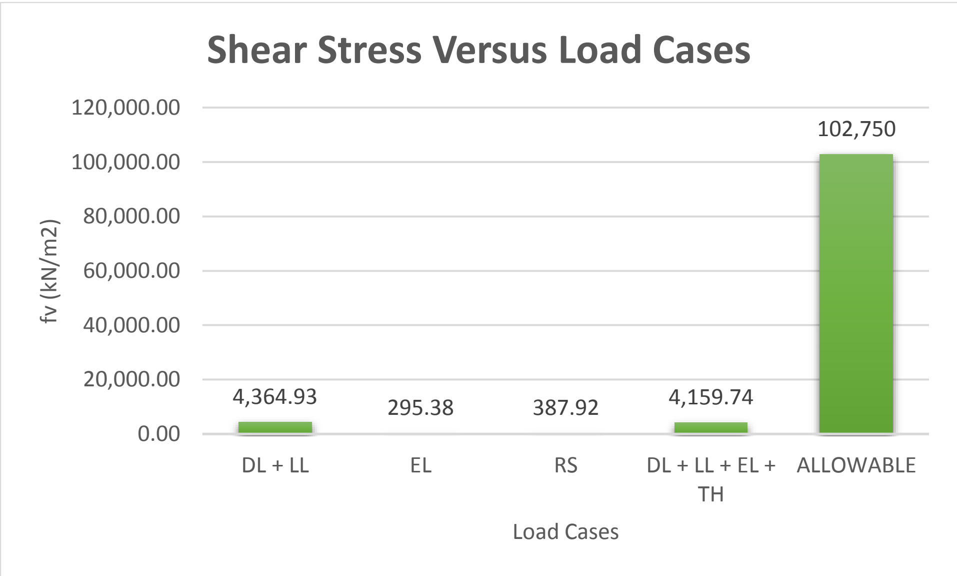


Figure 4.7: Graph of Shear Stress versus Load Cases

Usually the most critical part of the structure is the joint connection. Hence, the joint displacement under varies load cases has been determined in this study. The result of joint displacements for joint 25 with difference of load cases has been tabulated and illustrated in table 2 and figures 8 respectively. The highest value of displacement is 0.0289m in U1 direction which occurred in the response spectrum load case.

Table 4.8: Joint Displacements in Different Load Combination

Load Cases	U1 (m)	U2 (m)	U3 (m)
DL + LL	-0.0009	-0.0009	-0.0054
EL	0.0077	0.0019	0.0005
RS	0.0289	0.0286	0.0060
DL + LL + EL + TH	0.0068	0.0010	-0.0049

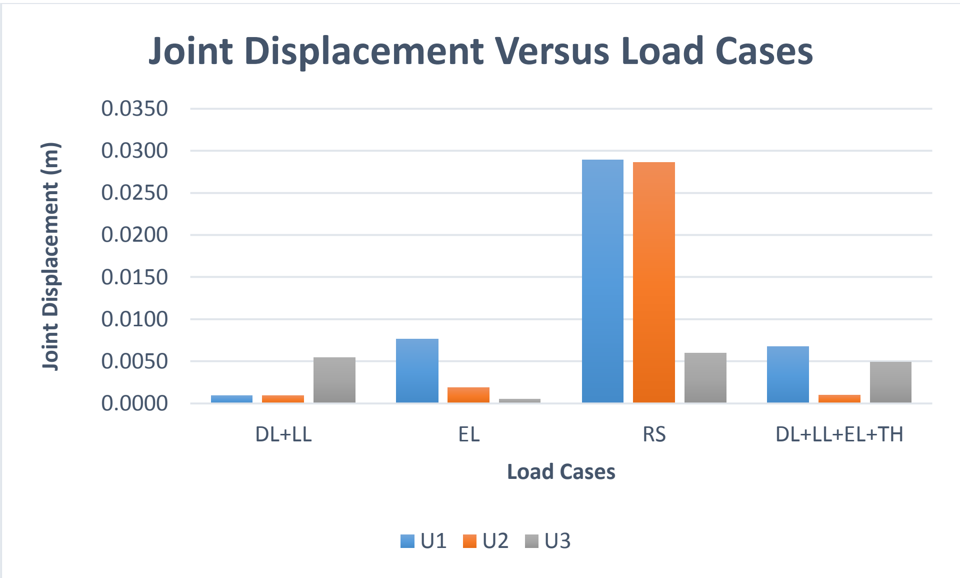


Figure 4.8: Graph of Joint Displacement versus Load Cases in Three Different Directions

CHAPTER 5

CONCLUSION & RECOMMENDATIONS

5.1 CONCLUSION

Based on the finding of the research, the conclusion that can be made consist of:

i. The simulation of the fixed offshore structure model is not 100% represent the actual structure. This is due to the assumptions made on the restrains at the base condition and the joint connection of the offshore structure. The restraint at base condition of the fixed offshore structure is assumed fixed to the ground as a replacement for pilled and soil interaction is neglected. Moreover, the connection of the offshore structure was not designed according to the EuroCode3 design specification.

ii. Fixed offshore structures in Malaysia region are capable of resisting this low seismic activity based on the study, since the maximum shear stress and bending stress is below the allowable capacity checks after several combination load cases have been applied.

iii. Element unity check has been performed, the final steel P-M interaction ratios are determined and displayed. All steel frames passed the stress/ capacity check as the ratio is less than 1 as showed in the table. The most critical frame member in this offshore model is a frame member 27 with 0.518 ratio.

iv. From the free vibration analysis, the highest value of the natural period is 0.5407 sec of mode shape 1.

v. The maximum shear force that happens at the most critical frame element 27 is 165.431 kN when load combination dead load + live load being applied.

vi. The maximum bending moment that happens at the most critical frame element 27 is 1071.21 kNm when load combination dead load + live load being applied.

vii. The highest value of displacement is 0.0289m in U1 direction which occurred in the response spectrum load case.

5.2 RECOMMENDATIONS

For the future study, the soil interaction and restraint of the offshore structure both should be considered in the analysis. This is because the earthquake load transfer from the ground surface to the upper surface. Thus, it might have slightly different results from what happen on site if neglecting the soil interaction in the analysis.

In fact, the failure of the steel structure usually happens at the joint connection. The main reason causing the failure is because of the stress at the connection is too high. Thus, the welded joint should be modelled in SAP 2000 computational software according to EuroCode3.

In addition, in order to increase the accuracy on the seismic response study, the earthquake data should be always updated, so that the higher intensity of the earthquake should be considered for the analysis. For example, the earthquake with magnitude 5.9

occurred at the Ranau, Sabah lately. At least 18 people killed on Mount Kinabalu, some climbers injured, some building damaged seriously, water supply disrupted in the area around Kundasang and Ranau, water color turned black near Ranau, and rockfalls trigged in the mountain area. (Earthquake.usgs.gov, 2015)

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APPENDICES A

Grid System Data

X Grid Data		Y Grid Data		Z Grid Data	
Grid ID	Ordinate	Grid ID	Ordinate	Grid ID	Ordinate
A	0	1	0	Z1	0
B	100	2	100	Z2	995
C	1868	3	1868	Z3	18673
D	3636	4	3636	Z4	36351
E	5221	5	5221	Z5	52201
F	12292	6	12292	Z6	53201
G	19363	7	19363		
H	20948	8	20948		
I	22716	9	22716		
J	24484	10	24484		
K	24584	11	24584		

APPENDICES B

Table of Stiffness

LEVEL	FRAME NUMBER	ELEMENT MEMBER	MEMBER LENGTH (mm)	MODULUS OF ELASTICITY, E (N/mm2)	MOMENT OF INERTIA, I (mm4)	STIFFNESS, k	MEMBER OF SAME CONDITION	TOTAL STIFFNESS, k
1~2	59, 64, 69, 74	COLUMN	1005.00	210000	7.87E+09	1.64E+12	4	6.58E+12
2~3	60, 65, 70, 75	COLUMN	17853.94	210000	7.87E+09	9.25E+10	4	3.70E+11
3~4	61, 66, 71, 76	COLUMN	17853.94	210000	7.87E+09	9.25E+10	4	3.70E+11
4~5	62, 67, 72, 77	COLUMN	16007.72	210000	7.87E+09	1.03E+11	4	4.13E+11
2	2, 3, 4, 5	M.BEAM	24384.00	210000	1.50E+09	1.29E+10	4	5.16E+10
2	6, 7	S.BEAM	34484.18	210000	1.50E+09	9.12E+09	2	1.82E+10
3	8, 9, 10, 11	M.BEAM	20848.00	210000	8.74E+08	8.80E+09	4	3.52E+10
3	12, 13, 14, 15	S.BEAM	14741.76	210000	8.74E+08	1.24E+10	4	4.98E+10
4	16, 17, 18, 19	M.BEAM	17312.00	210000	6.20E+08	7.52E+09	4	3.01E+10
4	21, 22, 23, 24	S.BEAM	12241.43	210000	6.20E+08	1.06E+10	4	4.25E+10
4	20	S.BEAM	17312.00	210000	6.20E+08	7.52E+09	1	7.52E+09
5	25, 26, 27, 28	M.BEAM	14142.00	210000	1.11E+09	1.65E+10	4	6.59E+10
5	30, 31, 32, 33	S.BEAM	9999.90	210000	1.11E+09	2.33E+10	4	9.32E+10
5	29	S.BEAM	14142.00	210000	1.11E+09	1.65E+10	1	1.65E+10

LEVEL	FRAME NUMBER	ELEMENT MEMBER	MEMBER LENGTH (mm)	MODULUS OF ELASTICITY, E (N/mm2)	MOMENT OF INERTIA, I (mm4)	STIFFNESS, k	MEMBER OF SAME CONDITION	TOTAL STIFFNESS, k
2~3	79, 80, 81, 82, 83, 84, 85, 86	TRUSS	28759.71	210000	1.09E+09	7.94E+09	8	6.35E+10
3~4	87, 88, 89, 90, 91, 92, 93, 94	TRUSS	26070.75	210000	8.31E+08	6.70E+09	8	5.36E+10
4~5	96, 98, 99, 100	TRUSS	22384.67	210000	1.18E+09	1.10E+10	4	4.41E+10
						TOTAL	**68**	**8.30E+12**

APPENDICES C

Table of Mass

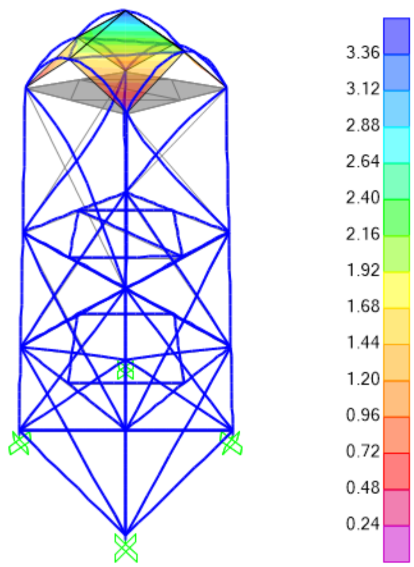
LEVEL	FRAME NUMBER	ELEMENT MEMBER	MEMBER LENGTH (mm)	OUTSIDE DIAMETER (mm)	WALL THICKNESS (mm)	VOLUME (mm3)	MASS (kg)	MEMBER OF SAME CONDITION	TOTAL MASS (kg)
1~2	59, 64, 69, 74	COLUMN	1005.00	1168.00	13	4.74E+07	372.10	4	1.49E+03
2~3	60, 65, 70, 75	COLUMN	17853.94	1168.00	13	8.42E+08	6610.35	4	2.64E+04
3~4	61, 66, 71, 76	COLUMN	17853.94	1168.00	13	8.42E+08	6610.35	4	2.64E+04
4~5	62, 67, 72, 77	COLUMN	16007.72	1168.00	13	7.55E+08	5926.79	4	2.37E+04
2	2, 3, 4, 5	M.BEAM	24384.00	559.00	25	1.02E+09	8026.97	4	3.21E+04
2	6, 7	S.BEAM	34484.18	559.00	25	1.45E+09	11351.84	2	2.27E+04
3	8, 9, 10, 11	M.BEAM	20848.00	508.00	19	6.09E+08	4776.31	4	1.91E+04
3	12, 13, 14, 15	S.BEAM	14741.76	508.00	19	4.30E+08	3377.36	4	1.35E+04
4	16, 17, 18, 19	M.BEAM	17312.00	508.00	13	3.50E+08	2747.01	4	1.10E+04

LEVEL	FRAME NUMBER	ELEMENT MEMBER	MEMBER LENGTH (mm)	OUTSIDE DIAMETER (mm)	WALL THICKNESS (mm)	VOLUME (mm3)	MASS (kg)	MEMBER OF SAME CONDITION	TOTAL MASS (kg)
4	21, 22, 23, 24	S.BEAM	12241.43	508.00	13	2.47E+08	1942.43	4	7.77E+03
4	20	S.BEAM	17312.00	508.00	13	3.50E+08	2747.01	1	2.75E+03
5	25, 26, 27, 28	M.BEAM	14142.00	508.00	25	5.36E+08	4210.79	4	1.68E+04
5	30, 31, 32, 33	S.BEAM	9999.90	508.00	25	3.79E+08	2977.48	4	1.19E+04
5	29	S.BEAM	14142.00	508.00	25	5.36E+08	4210.79	1	4.21E+03
2~3	79, 80, 81, 82, 83, 84, 85, 86	TRUSS	28759.71	610.00	13	7.01E+08	5503.86	8	4.40E+04
3~4	87, 88, 89, 90, 91, 92, 93, 94	TRUSS	26070.75	559.00	13	5.81E+08	4563.04	8	3.65E+04
4~5	96, 98, 99, 100	TRUSS	22384.67	559.00	19	7.22E+08	5663.22	4	2.27E+04
							TOTAL	**68**	**3.23E+05**

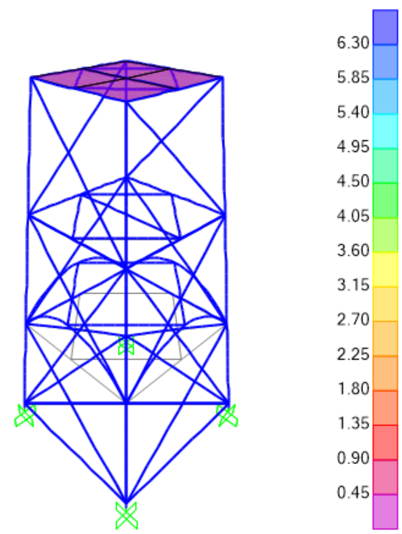
APPENDICES D

Offshore Structural Mode Shape

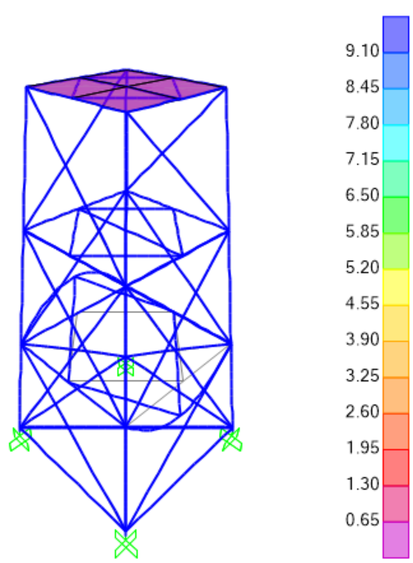
Mode 1 (T= 0.5407 sec; f= 1.8494 Hz)



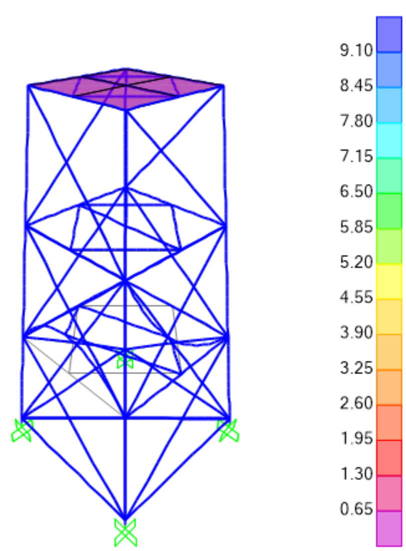
Mode 2 (T= 0.4853 sec; f= 2.0605 Hz)



Mode 3 (T= 0.4853 sec; f= 2.0606 Hz)

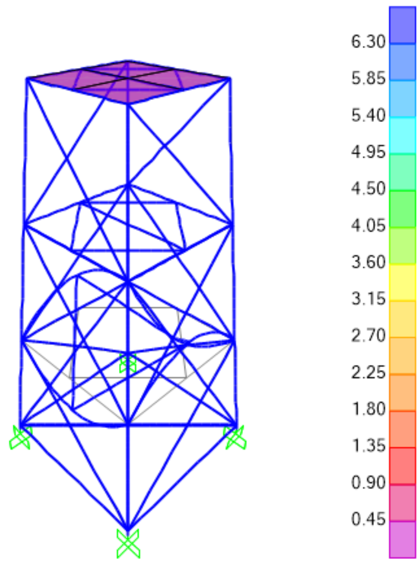


Mode 4 (T= 0.4853 sec; f= 2.0606 Hz)

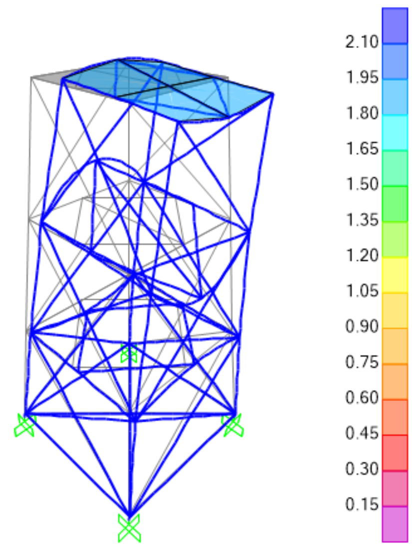


APPENDICES D: Continued

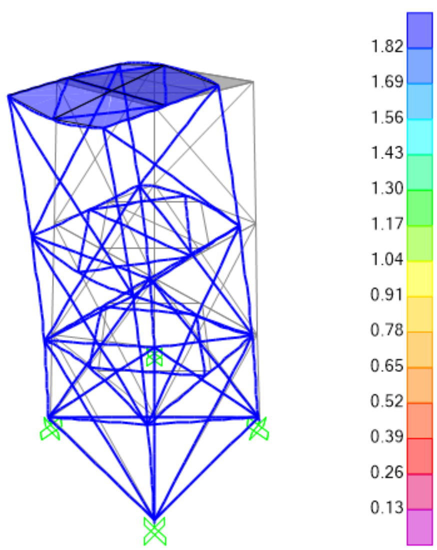
Mode 5 (T= 0.4850 sec; f= 2.0621Hz)



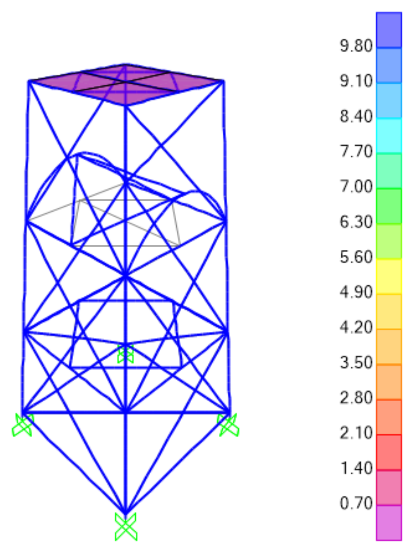
Mode 6 (T= 0.4186 sec; f= 2.3890 Hz)



Mode 7 (T= 0.4177 sec; f= 2.3938 Hz)

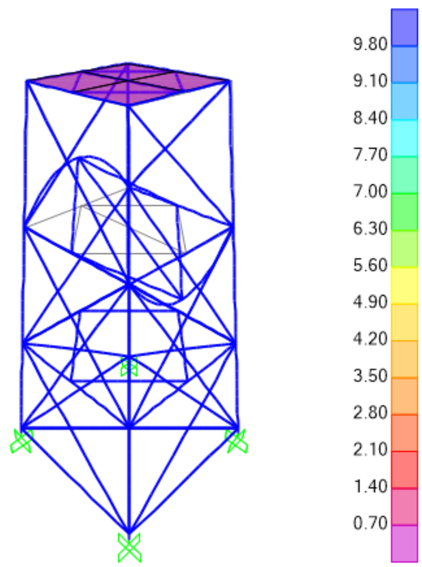


Mode 8 (T= 0.3938 sec; f= 3.5394 Hz)

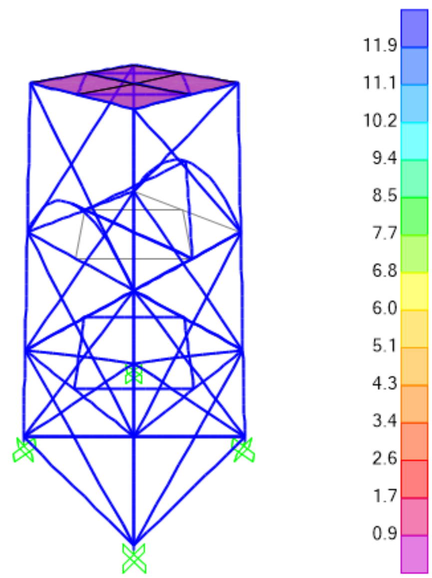


APPENDICES D: Continued

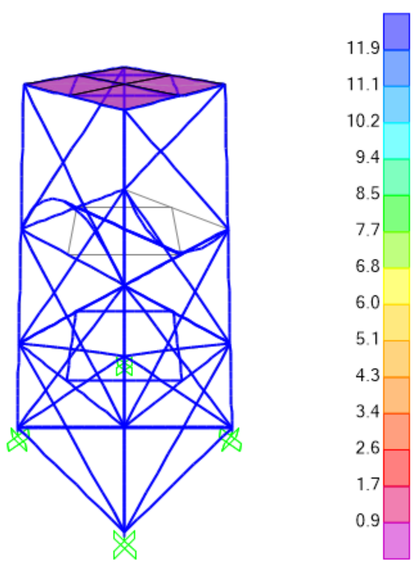
Mode 9 (T= 0.3928 sec; f= 2.5459 Hz)



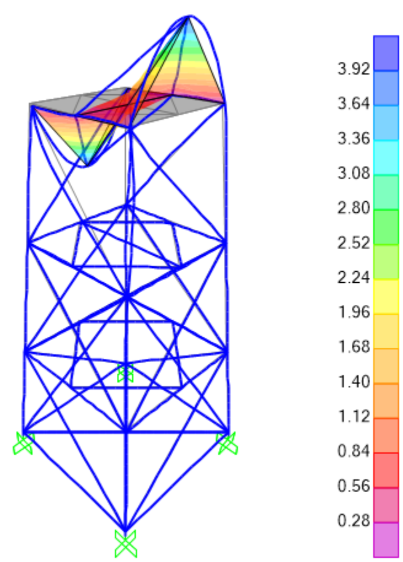
Mode 10 (T= 0.3311 sec; f= 3.0198 Hz)



Mode 11 (T= 0.3308 sec; f= 3.0226 Hz)



Mode 12 (T= 0.3172 sec; f= 3.1527 Hz)



APPENDICES E

Manual Calculation

<u>Load Description</u>

The load combinations of dead load, deck loads, wind, wave, current and earthquake loads have been defined.

Self-weight and functional loads

The self-weight and functional loads of this offshore structure is listed in Table 3.1.

Table 3.1: Dead load and live load description

No.	Load descriptions	Weight [MN]
1	Jacket appurtenances weight	0.339
2	Topside dead loads	0.393
3	Topside live loads	1.150
4	Piping & equipment weights	0.400
	TOTAL	2.282

Dead and Live Loads

$\text{Dead load} = 0.339 + 0.393 + 0.400 = 1.132\ \text{MN}$

$\text{Uniform dead load} = (1.132 \times 1000)\text{kN} \div (14.143 \times 14.143)\text{m}^2$

$= 5.659\text{kN}/\text{m}^2$

$\text{Live load} = 1.150\ \text{MN}$

APPENDICES E: Continued

$$\text{Uniform live load} = (1.150 \times 1000)\text{kN} \div (14.143 \times 14.143)\text{m}^2$$

$$= 5.749 \text{ kN}/\text{m}^2$$

Environmental loads

The environmental loads are based on the metocean data. Table 3.6.2 shows the environmental criteria for the offshore structure which located in Terengganu.

Table 3.2: Environmental criteria for offshore structure at Terengganu

MSL		Design Condition
47.629 m	Wave height (m)	10.79
	Wave period (s)	10.9
	Current velocity (m/s)	0.750
	Wind speed (m/s)	21.8
	Max. tide (m)	2.0
	Storm surge (m)	0.4

Therefore all the environmental loads which including wind, wave and current action were calculated by using the provided environmental criteria and formula from American Petroleum Institute (API).

Wind Load

The wind profile, gusts and wind drag force are calculated using following equation that are taken directly from the API RP 2A-WSD practice.

APPENDICES E: Continued

Wind profile and Gusts

$U(z) = U_o\, x\, [1 + C\ x\ ln\left(\frac{z}{32.8}\right)]$

$z = 4572\ mm + 1000mm$

$= 5572\ mm$

= 18.281 ft

$U_o\ = 21.8\ m/s$

$= 71.522\ ft/s$

$C\ = 5.73\ x\ 10^{-2}$x $(1 +\ 0.0457$ x $U_o\ )^{\ 1/2}$

$= 5.73\ x\ 10^{-2}$x $(1 +\ 0.0457$ x $71.522\ ft/s)^{\ 1/2}$

$= 0.118$

$U(z) = 71.522\ m/s\ x\ [1 + 0.118\ x\ ln\left(\frac{18.281\ ft}{32.8}\right)]$

$= 66.588\ ft/s$

$I_u(z) = 0.06x\ [1 + 0.0131x\ U_O]\ x \left(\frac{Z}{32.8}\right)^{-0.22}$

$= 0.06x\ [1 + 0.0131x\ 71.522\ ft/s\ ]\ x \left(\frac{18.281\ ft}{32.8}\right)^{-0.22}$

$=$ 0.132 ft/s

$u(z,t) =\ 66.588\ ft/s\ x\ [1 - 0.41]x\ 0.132 ft/s\ x\ ln\left(\frac{60\ s}{3600\ s}\right)$

= 81.343 ft/s

= 24.79 m/s

APPENDICES E: Continued

Wind Speed and Force Relationship

The wind drag force on object should be calculated as:

$$\begin{aligned} F &= \left(\frac{\rho}{2}\right) u^2 C_S A \\ &= \left(\frac{1.22\ \mathrm{kg/m3}}{2}\right)(24.79\ \mathrm{m/s})^2(1.0)(24.10) \\ &= \mathbf{7.7\ kN} \end{aligned}$$

$$C_S = Overall\ project\ area\ platform = 1$$

$$A = 2\,\pi\,\mathrm{rh}$$

$$= 2\,\pi\ x\ 0.584\ m\ x\ 5.595\ m$$

$$= 20.53\ m^2$$

Wave Load

Based on American Petroleum Institute (API), the drag coefficient, C_D and inertia coefficient, C_m are according to the following:

Table 3.3: Drag coefficient and inertia coefficient

	Drag Coefficient, C_D	**Inertia Coefficient, C_m**
Smooth	0.65	1.6
Rough	1.05	1.2

The wave load **above** sea water level was considered as rough:

$$F_w = C_d \frac{W}{2g} AU|U| + C_m \frac{W}{g} V \frac{dU}{dt}$$

APPENDICES E: Continued

$$= 1.05\left(\frac{10100}{2\times 9.81}\right)(1.168)(0.75)|0.75| +$$
$$1.2\left(\frac{10100}{9.81}\right)\left[\frac{\pi(1.168)^2}{4}\right]\left[\frac{0.75}{100\ X\ 365\ X\ 24\ X\ 60\ X\ 60}\right]$$
$$= \mathbf{355.12\ N/m}$$

While the wave load **below** sea water level was considered as smooth:

$$F_w = C_d \frac{W}{2g} AU|U| + C_m \frac{W}{g} V \frac{dU}{dt}$$
$$= 0.65\left(\frac{10100}{2\times 9.81}\right)(1.168)(0.75)|0.75| +$$
$$1.6\left(\frac{10100}{9.81}\right)\left[\frac{\pi(1.168)^2}{4}\right]\left[\frac{0.75}{100\ X\ 365\ X\ 24\ X\ 60\ X\ 60}\right]$$
$$= \mathbf{219.84\ N/m}$$

Current Load

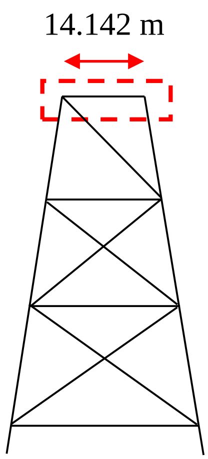
In order to calculate the current force on the structural members, no wave conditions need to be applied, by substitute the $\frac{\delta U}{\delta t} = 0$.

$$F_c = C_d \frac{w}{2g} AU|U|$$
$$= 0.65\left(\frac{10100}{2\times 9.81}\right)(1.168)(0.75)|0.75|$$
$$= \mathbf{219.84\ N/m}$$

APPENDICES E: Continued

Design for Restrained Beam

Beam Load:



Level 5

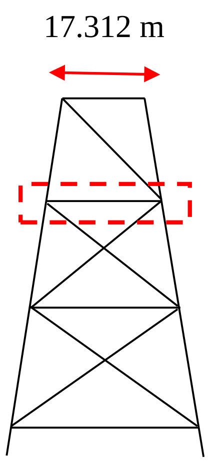
Self-weight $= \left(7.697 \text{ x } 10^{-8} \frac{\text{kN}}{mm^3}\right) \times$

$(5.36 \text{ x } 10^{8} mm^3)$

$= 41.29$ kN

Load, q $= 1.35$ (41.29 kN)

$= 55.74$ kN



Level 4

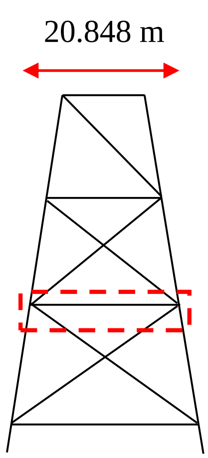
Self-weight $= \left(7.697 \text{ x } 10^{-8} \frac{\text{kN}}{mm^3}\right) \times$

$(3.50 \text{ x } 10^{8} mm^3)$

$= 26.94$ kN

Load, q $= 1.35$ (26.94 kN)

$= 36.37$ kN



Level 3

Self-weight $= \left(7.697 \text{ x } 10^{-8} \frac{\text{kN}}{mm^3}\right) \times$

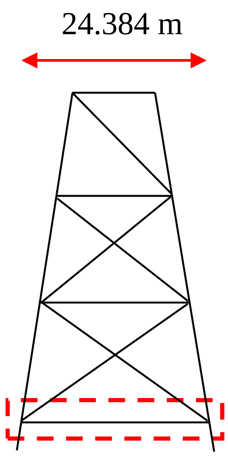
$(6.09 \text{ x } 10^{8} mm^3)$

$= 46.84$ kN

Load, q $= 1.35$ (46.84 kN)

$= 63.23$ kN

APPENDICES E: Continued



Level 2

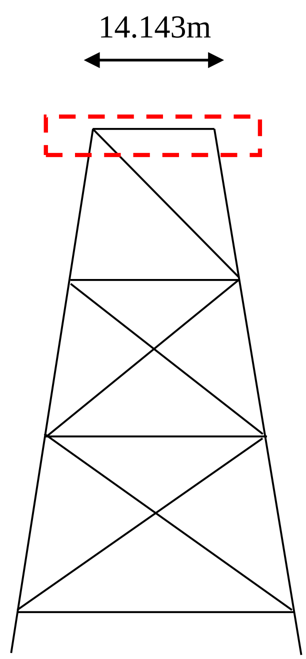
Self-weight $= \left(7.697 \text{ x } 10^{-8}\,\frac{\text{kN}}{mm^3}\right) \times (1.02 \text{ x } 10^9 mm^3)$

$= 78.72$ kN

Load, q $= 1.35\ (78.72 \text{ kN})$

$= 106.27$ kN

APPENDICES E: Continued



Given:

Dead loads = 1.132 MN ÷ 4 (because of four sides)

= 283 kN

Live loads = 1.150 MN ÷ 4

= 287.5 kN

Thus, try,

Steel size with **d = 508 mm**

Self-weight $= \left(7.697 \text{ x } 10^{-8} \frac{\text{kN}}{mm^3}\right) \times$

$(5.36 \text{ x } 10^{8} mm^3)$

$= 41.29$ kN

1) Design

Design load, q = $1.35 \text{ Gk} + 1.5 \text{ Qk} + 1.2 \text{ EL}$

$= 1.35\ (283 + 41.29) + 1.5\ (287.5) + 1.2\ (9.8)$

$= 880.80$ kN

Shear force, V_{Ed} $= \frac{q}{2}$

$= \frac{880.80}{2}$

$= 440.40$ kN

Bending Moment, M_{Ed} $= \frac{qL}{8}$

$= \frac{880.80\ (14.142)}{8}$

$= 1557.03$ kNm

APPENDICES E: Continued

2) Try Section

d = 508 mm	I = 1.11 x 10^9 mm^4
t = 25 mm	W_{el} = 4366863 mm^3
M = 297.75 kg/m	W_{pl} = 5837433 mm^3
A = 37934.73 mm^2	G = 81000 N/mm^2
i = 171 mm	E = 210000 N/mm^2

3) Design Strength

Referring to Table 3.1, Eurocode 3 (Part 1-1)

For steel grade S355,

t = 25 mm < 40 mm

f_y = 355 N/mm^2

f_u =510 N/mm^2

4) Section Classification

Referring to Table 5.2, Eurocode 3 (Part 1-1)

$\varepsilon = \sqrt{235/f_y} = 0.81$

$\varepsilon^2 = (0.92)^2 = 0.66$

$\frac{d}{t} = \frac{508}{25} = 20.32 \quad < \quad 50\varepsilon^2 = 50(0.66) = 33$ $\qquad \frac{d}{t} < 50\varepsilon^2$ Class 1

The section is classified as Class 1 since d/t meets Class 1 standard.

5) Shear Resistance of Section

i. Maximum external design shear force, V_{Ed} = 440.40 kN

APPENDICES E: Continued

ii. Shear resistance of the section, $V_{C,Rd}$

$V_{C,Rd}$ $= V_{Pl,Rd}$

$$V_{Pl,Rd} = \frac{A_v\left(f_y/\sqrt{3}\right)}{\gamma M_0} = \frac{24150\left(355/\sqrt{3}\right)}{1.00}$$

$= 4949768.20$ N

$= 4949.77$ kN

- $\gamma M_0 = 1.00$ (Based on EC3: Part 1)
- $f_y = 355\ N/mm^2$
- $A_v = \frac{2A}{\pi} = \frac{2(37934.73)}{\pi} = 24150\ mm^2$

~ $A = \pi r_{outer}{}^2 - \pi r_{inner}{}^2$

$= \pi\left(\frac{508}{2}\right)^2 - \pi\left(\frac{508}{2} - 25\right)^2$

$= 37934.73\ mm^2$

iii. Design Check

$$\frac{V_{Ed}}{V_{C,Rd}} = \frac{440.40}{4949.77}$$

$= 0.09 \leq 1.0$

The section is satisfactory.

APPENDICES E: Continued

6) Bending Moment Resistance of Section

i. Maximum external design moment, $M_{Ed} = 3114.07$ kNm

ii. Moment resistance for Class 1 cross section, $M_{C,Rd}$

$$M_{C,Rd} = M_{Pl,Rd}$$

$$M_{Pl,Rd} = \frac{W_{Pl}(f_y)}{\gamma M_0} = \frac{5837433\ (355)}{1.00}$$

$$M_{Pl,Rd} = 2072288715 \text{ Nmm}$$

$$= 2072.29 \text{ kNm}$$

- $W_{Pl} = 5837433 \text{ mm}^3$
- $f_y = 355 \text{ N/mm}^2$
- $\gamma M_0 = 1.00$ (Based on EC3: Part 1)

iii. Design Check

$$\frac{M_{Ed}}{M_{C,Rd}} = \frac{1557.03}{2072.29}$$

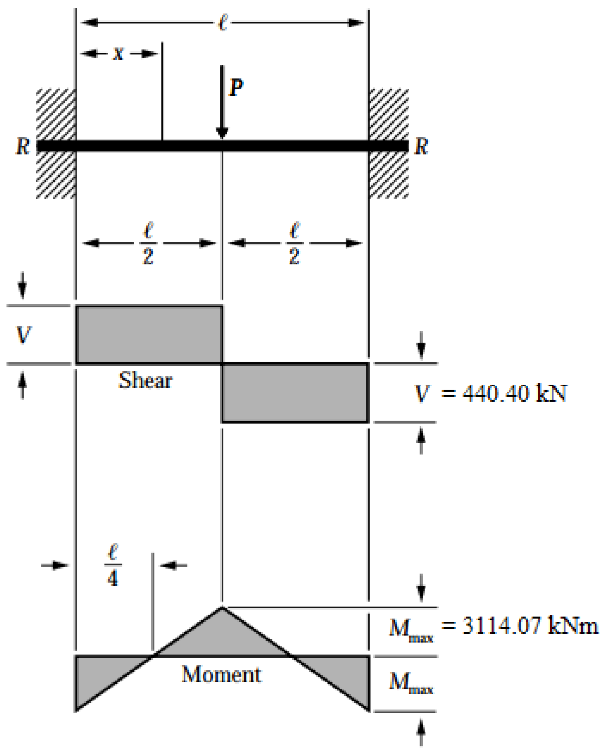
$$= 0.75 \leq 1.0$$

The section is satisfactory.

APPENDICES E: Continued

7) Combined Bending and Shear Resistance

Consider a section where moment is maximum, referring to Shear Force and Moment diagrams, the mid-span section where M_{Ed} = 3114.07 kNm and V_{Ed} = 440.40 kN is considered.



a) Shear force at maximum moment, V_{Ed} = 440.40 kN

b) 0.5 $V_{C,Rd}$ = 0.5 (4949.77)

= 2474.89 kN

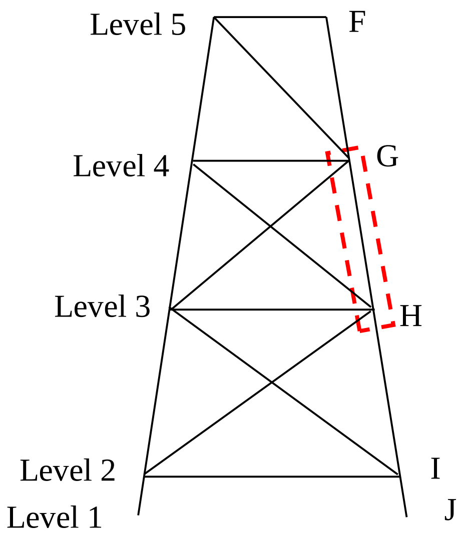
c) Since $V_{Ed} < 0.5\ V_{C,Rd}$

The shear, V_{Ed} is small and it does not affect the moment resistance, $M_{C,Rd}$. The beam section is able to carry the most critical combination of bending and shear. No reduction in the design strength of the steel, f_y and the design moment resistance remains, $M_{C,Rd} = 3114.07 \text{ kNm}$.

APPENDICES E: Continued

Design for Column

Axial load in column GH, N_{Ed} = $\mathbf{1003.84\ kN}$



Design Bending Moments due to eccentricities

Nominal Moment at Level 4

$M_{2,Ed}$ = $F_{1,d}(\frac{h}{2} + 100)$ = 109.75 kN x $\left(\frac{1168}{2} + 100\right)$ mm = 75.07 kNm

Distributed Moment at Level 4

Column bending stiffness k = EI/L

Based on table 5, Hence k_1 =9.25 x 10^{10}, and k_2 = 1.03 x 10^{11}

M_{Ed} = 75.07 x $\frac{9.25 \text{ x } 10^{10}}{(9.25 \text{ x } 10^{10})+(1.03 \text{ x } 10^{11})}$ = 35.52 kNm

Section Properties

Try section d = 1168 mm, t =13 mm CHS in grade S355

APPENDICES E: Continued

$d = 1168$ mm

$t = 13$ mm

$M = 370.25$ kg/m

$A = 47171.01\ mm^2$

$i = 408.38$ mm

$G = 81000\ N/mm^2$

$I = 7.867 \times 10^9\ mm^4$

$W_{el} = 13470737\ mm^3$

$W_{pl} = 17343057\ mm^3$

$I_t = 1.573 \times 10^{10}\ mm^4$

$E = 210000\ N/mm^2$

By referring to Table 3.1, Eurocode 3 (Part 1-1)

For steel grade S355,

$t = 13\ mm < 40\ mm$

$f_y = 355\ N/mm^2$

$f_u = 510\ N/mm^2$

Partial factors for resistance

$\gamma M_0 = 1.0$, $\gamma M_1 = 1.0$

Classification of Cross Section

By referring to Table 5.2 (Sheet 3 of 3), Eurocode 3 (Part 1-1)

$$\varepsilon = \sqrt{235/f_y} = 0.81$$

Class I: $d/t \leq 50\varepsilon^2$, $89.85 > 50(0.81)^2 = 32.81$

Class II: $d/t \leq 70\varepsilon^2$, $89.85 > 70(0.81)^2 = 45.93$

Class III: $d/t \leq 90\varepsilon^2$, $89.85 > 90(0.81)^2 = 59.05$

APPENDICES E: Continued

Therefore, this section is classified as **Class IV** as $^{d}/_{t}$ doesn't meets the Class III standard.

Minimum Buckling Resistance, $N_{min,b,Rd}$

$$N_{min,,b,Rd} = \frac{\chi A f_y}{\gamma M_1}$$

Column Slenderness

λ_1 = 93.9ε = 93.9 x 0.81 = 76.06

Buckling Length, L_{cr} = 0.7L = 0.7(17853.94)mm = 12497.76 mm

$$\bar{\lambda} = \left(\frac{L_{cr}}{i_y}\right)\left(\frac{1}{\lambda_1}\right) = \left(\frac{12497.76}{408.38}\right)\left(\frac{1}{76.06}\right) = 0.40$$

Column Imperfection

By referring to Table 6.2 & 6.1, Eurocode 3 (Part 1-1):

Table 6.2: Curve 'a'

Table 6.1: Buckling curve 'a'} α_y = 0.21

Reduction Factor

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

$$= 0.5\,[1 + 0.21\,(0.40 - 0.2) + 0.40^2] = 0.60$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 + \bar{\lambda}^2}} \leq 1.0$$

$$= \frac{1}{0.60 + \sqrt{0.60^2 - 0.40^2}}$$

$$= 0.76$$

APPENDICES E: Continued

Minimum Buckling Resistance

$$N_{min,,b,Rd} = \frac{\chi A f_y}{\gamma M_1} = \frac{0.76 \text{ x } 47171.01 \text{ x } 355}{1.00} = 12726.74 \text{ kN}$$

Section Resistance, $M_{z,b,Rd}$

$$M_{z,b,Rd} = \frac{W_{pl,z} f_y}{\gamma M_0} = \frac{17343057 \text{ x } 355}{1.0} = 6156.79 \text{ kNm}$$

Interaction Equation

$$\frac{N_{Ed}}{N_{min,b,Rd}} + \cancel{\frac{M_{Ed}}{M_{y,b,Rd}}} + \frac{1.5 M_{Ed}}{M_{z,b,Rd}} < 1.0$$

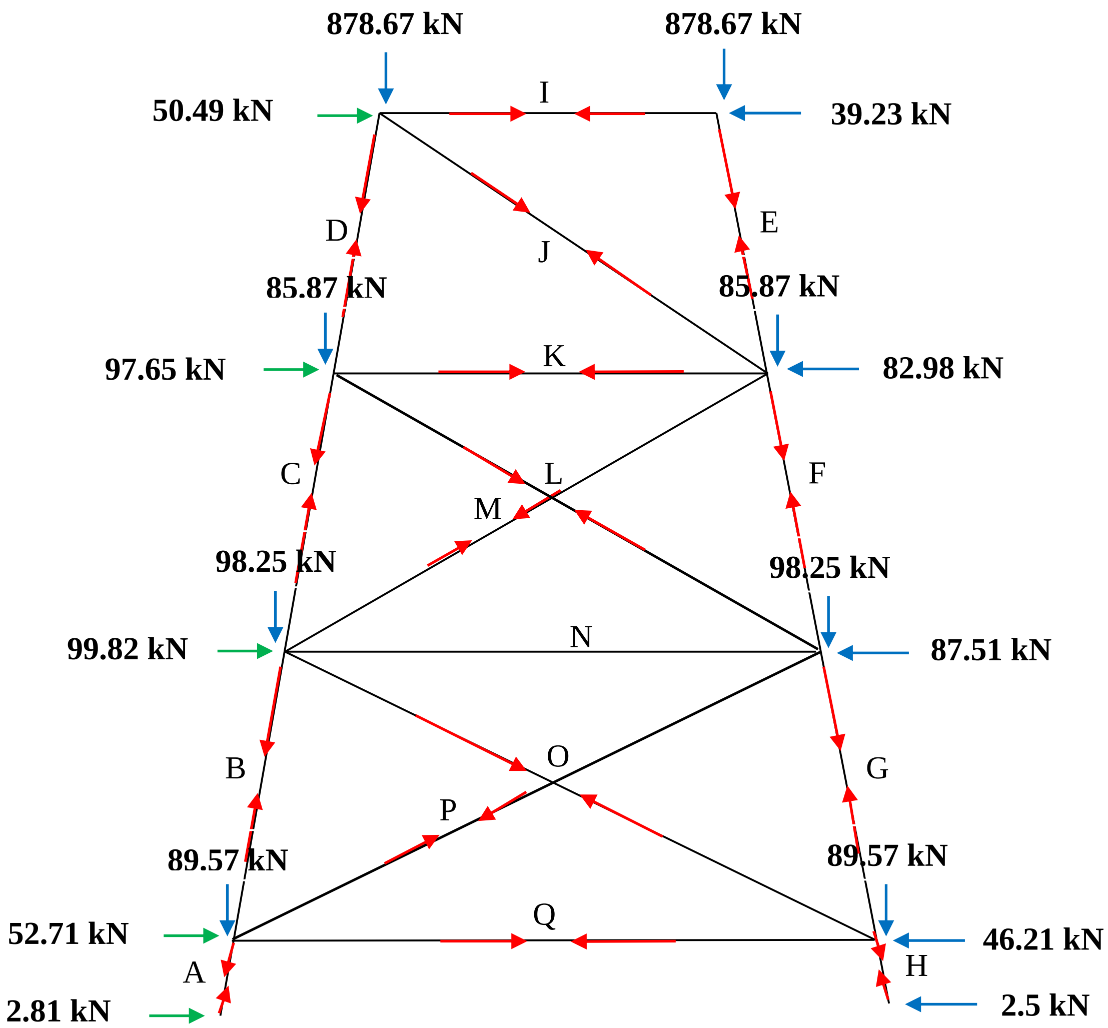
∴ For the **Lateral Torsional Buckling Resistance**, $\mathbf{M_{y,b,Rd}}$ of the circular hollow section, it doesn't required to determine in accordance to **EC3-Part 1, Clause 6.3.2.1**.

$$\frac{1003.84}{12726.74} + \frac{1.5(35.52\ )}{6156.79} = 0.09 < 1.0$$

∴ This section size with diameter of 1168 mm and thickness of 13 mm CHS is satisfactory for the design, while select a smaller section will result in more economical and compact design.

APPENDICES E: Continued

<u>Design for Truss Member</u>



- Green colour arrow indicated the environmental load (wind loads, wave load and current load)
- Blue colour arrow indicated the reaction forces

APPENDICES E: Continued

Truss Member	Internal Force (kN)	Type of Force
A	26.88	Compression
B	11.33	Tension
C	960.61	Compression
D	872.81	Compression
E	883.51	Compression
F	1003.84	Compression
G	8.72	Tension
H	23.92	Compression
I	131.58	Compression
J	15.05	Compression
K	109.75	Compression
L	3.37	Tension
M	33.96	Tension
N	96.11	Compression
O	92.70	Tension
P	83.71	Tension
Q	122.67	Compression

Table: Internal Forces

*Yellow rows indicated the critical truss members in tension and compression

Design of tension member with single angle section

The maximum design tension force among the truss members is truss member O, $\mathrm{N_{ed(o)}} = 92.70$ kN. Thus, Tension force, $\mathbf{N_{ed} = 92.70\ kN}$

APPENDICES E: Continued

Tension resistance, $N_{t,Rd}$

Try CHS d=610 mm, t= 13 mm of steel grade S355,

A= 24381.90 mm^2, i= 211.12 mm

Design strength, f_y (Table 3.1):

t= 13 mm < 40 mm, steel grade S355, f_y = 355 N/mm^2, f_u = 510 N/mm^2

In this case, there is no hole for the bolts, hence no reduction in the cross sectional of the angle section.

The design tension resistance $N_{t,Rd}$ should be taken as smaller of:

i. The design plastic resistance of the gross cross-section

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} = \frac{(24381.90\ mm^2)\left(355\,\frac{N}{mm^2}\right) x\ 10^{-3}}{1.0} = 8655.57\ kN$$

ii. The design ultimate resistance of the net cross-section

Since there was reduction in the cross section of angle, then

$A_{net} = A = \ 24381.90\ mm^2$

$$N_{u,Rd} = \frac{0.9A_{net}f_u}{\gamma_{M2}} = \frac{0.9(24381.90\ mm^2)\left(510\ \frac{N}{mm^2}\right) x\ 10^{-3}}{1.25} = 8953.03\ kN$$

Therefore, $\mathbf{N_{t,Rd} = 8655.57\ kN}$

APPENDICES E: Continued

Check for equilibrium:

$$\frac{\mathbf{N_{ed}}}{\mathbf{N_{t,Rd}}} = \frac{92.70 \text{ kN}}{8655.57 \text{ kN}} = \mathbf{0.01} < 1.0$$

The section is **suitable** to carry the force.

Design of the critical compression member with single angle section

The maximum design compression force in among the members is $N_{ed(F)} = 1003.84$ kN. But we assumed member F as column member, thus we used member P to carry out truss design. The maximum design compression force in truss member J, $N_{ed(J)} = 15.05$ kN. Thus, axial compression force, $\mathbf{N_{ed} = 15.05\ kN}$

Try CHS d=559 mm, t= 19 mm of steel grade S355, length of member P= 22384.67 mm

A= 32232.74mm^2, i= 191.04 mm

Material properties, Clause 3.2.1

Table 3.1: Steel grade S355, t= 19 mm < 40 mm, f_y = 355 N/mm^2, f_u = 510 N/mm^2, $\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81$

Classification of Cross section

$$\frac{d}{t} = \frac{559 \text{ mm}}{19 \text{ mm}} = 29.42$$

50 ε^2 = 50 $(0.81)^2$ = 32.81 > d/t = 29.42, class 1

APPENDICES E: Continued

Thus, section is **class 1**.

Partial factors of resistance, Clause 6.1

$\gamma_{M0} = 1.0 \qquad \gamma_{M1} = 1.0$

Buckling resistance about the axis, $N_{b,Rd}$(Clause 6.3.1 EC3)

$$N_{b,Rd} = \frac{XAf_y}{\gamma_{M1}}$$

Where

$$L_{cr} = 0.7L = 0.7\ (22384.67\ \text{mm}) = 15669.27\ \text{mm}$$

$$\bar{\lambda} = \frac{L_{cr}}{93.9\ \varepsilon\ x\ i} = \frac{15669.27}{93.9\ (0.81) x\ 191.04} = 1.08$$

From Table 6.2, for hollow sections, hot finished, Steel grade S355, buckling curve = a

From Table 6.1, for buckling curve ‘a’, imperfection factor, $\alpha = 0.21$

$$\emptyset = 0.5[1 + \propto (\bar{\lambda} - 0.2) + \bar{\lambda}^2]$$

$$= 0.5[1 + 0.21(1.08 - 0.2) + 1.08^2]$$

$$= 1.18$$

$$X = \frac{1}{\emptyset + \sqrt{\emptyset^2 - \lambda^2}} = \frac{1}{1.18 + \sqrt{1.18^2 - 1.08^2}} = 0.60 < 1.0$$

$$\mathbf{N_{b,Rd}} = \frac{XAf_y}{\gamma_{M1}} = \frac{(0.60)(32232.74)(355) x\ 10^{-3}}{1.0} = \mathbf{6865.57\ kN}$$

APPENDICES E: Continued

Check for equilibrium

$$\frac{\mathbf{N_{ed}}}{\mathbf{N_{b,Rd}}} = \frac{15.05\ \text{kN}}{6865.57\ \text{kN}} = \mathbf{0.002} < 1.0$$

The section is **suitable** to carry the force.

Full section resistance of Compression Member, N_{cRd} (Clause 6.2.4 EC3)

$$N_{c,Rd} = \frac{Af_y}{\gamma M_0} = \frac{(32232.74)(355)\text{x } 10^{-3}}{1.0} = \mathbf{11442.62\ kN}$$

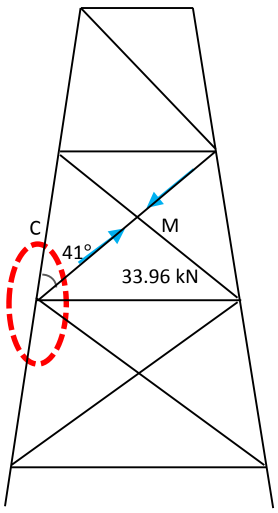
Check for equilibrium

$$\frac{\mathbf{N_{ed}}}{\mathbf{N_{c,Rd}}} = \frac{15.05\ \text{kN}}{11442.62\ \text{kN}} = \mathbf{0.001} < 1.0$$

The section was **suitable** to carry the force.

APPENDICES E: Continued

Design for Joint



Try column section (Member C): CHS d_0=1168 mm, t_0= 13 mm of steel grade S355;

Truss section (Member M): CHS d_1=559 mm, t_1= 13 mm of steel grade S355,

Design strength, f_y (Table 3.1):

t < 40 mm, steel grade S355, f_y = 355 N/mm^2, f_u = 510 N/mm^2

Design force of welded joints between CHS braces members and CHS chords

$N_{Ed} = 33.96$ kN

Design axial resistances of welded joints between CHS brace members and CHS chords

APPENDICES E: Continued

$$N_{i,Rd} = \frac{f_{y0}}{\sqrt{3}} t_0 \pi d_1 \frac{1 + \sin\theta_1}{2\sin^2\theta_1} / \gamma_{M5}$$

$$= \frac{355}{\sqrt{3}} (13)(\pi)(559)(\frac{1+\sin 41°}{2\sin^2 41°})/1.0$$

= 9001.87 kN

$N_{Ed} < N_{i,Rd}$, Ok.

Design resistances moments of welded joints between CHS brace members and CHS chords

$$M_{1p,1,Rd} = \frac{f_{y0} t_0 {d_1}^2}{\sqrt{3}} \frac{1 + 3\sin\theta_1}{4\sin^2\theta_1} / \gamma_{M5}$$

$$= \frac{(355)(13)(559)^2}{\sqrt{3}} \frac{1 + 3\sin 41°}{4\sin^2 41°} / 1.0$$

= 1435.42 kN