TARPON MONOPOD MARGINAL FIELD STRUCTURE SUBJECTED TO SEISMIC GROUND ACCELERATION

by

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CERTIFICATION OF APPROVAL

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Approved by:

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CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible to the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources and person.

(Najwa Amirah Hussim)

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ABBREVIATIONS

BOE	Barrel of Oil Equivalent
DAF	Dynamic Amplification Factor
GLND	GL Noble Denton
GNI	Gross National Income
GSHAP	Global Seismic Hazard Assessment Program
GTS	Global Technical Solution
OECU	Offshore Engineering Centre Unit
PCSB	PETRONAS Carigali Sdn Bhd
PGA	Peak Ground Acceleration
PMO	Peninsular Malaysia Operation
PMT	PETRONAS Management Team
PSHA	Probabilistic Seismic Hazard Assessment
RSR	Reserve Strength Ratio
SACS	Structural Analysis Computer System
SBO	Sabah Borneo Operation
UC	Unity Check
USGS	United States Geological Survey

ABSTRACT

For seismically active areas it is preferred that the intensity and characteristics of seismic ground motion used for design be determined by a site specific study. Since Malaysia is not located within seismically sensitive zone, seismic ground acceleration tends to be neglected from dynamic load design of offshore structures within the region. However, it is reported that tremors have been occurring and felt by platform operators in Malaysian Water. As Tarpon Offshore Platform is relatively contemporary within PETRONAS assets in Malaysia, there are no available specific tarpon inspection requirements or maintenance guidelines. Platform robustness and integrity cannot be ascertained. In regards to recent study of PSHA which is carried out by site-specific study, it is obtained that the mean hazard predicted is somewhat higher than of seismic model published by other studies, as well as API benchmark for evaluation of seismic activity to an offshore structure for a particular region. In this paper, the platform response towards seismic ground acceleration is investigated. By taking extreme condition of environmental loads and suggested ground acceleration values, the author will define the threshold unit at which ground acceleration is possibly controls the overall performance of a marginal field platform. In conjunction with that, platform natural behaviour and ultimate resisting force is identified in order to evaluate the platform integrity hence verify the latter findings.

CHAPTER 1

INTRODUCTION

1.1 Background: Overview of Tarpon Monopod Platform

Tarpon Monopod Platform is a type of offshore structure which mainly considers optimization in design which purport in minimizing cost of installation for marginal field development. The first cable-guyed caisson platform, known as “Tarpon” was first used in 1987 and patents of the system are owned by Stolt Comex Seaway. Nowadays, Tarpon Platform is still considered as a covert option in the oil and gas industry and there is very little documentation found in the open literature.

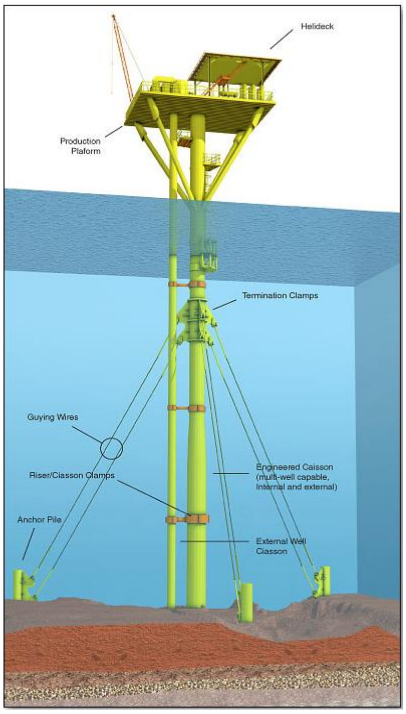


FIGURE 1.1 Conceptual design of a Tarpon Platform [Source: Tarpon Systems]

Commonly installed in a small field and low reservoir capacity, Tarpon Platform has short design life, in which it depends to the respective estimated field life. The structure usually consists of central caisson which safeguards the conductor inside it and held by 3 pairs of guyed wire cables attached to the central caisson and anchor pile on the seabed. One end of the guyed-wire cable is pinned to an anchor pile at or below the mudline and another end is pinned to the central caisson body below the water. They are 120 degrees apart from each pair to another and the horizontal distance of anchor piles is set to be approximately 170% of the water depth from caisson body. However, despite of these specification and dimension, different platform possesses different design. There is no any definite design value set within an existing Tarpon Platform but certainly, they hold similar concept.

1.2 Problem Statement

As stated by GL Noble Denton (2011), the Tarpon structures in both Peninsular Malaysia Operations (PMO) and Sabah Borneo Operation (SBO) waters were labelled as 'red' (Very High Risk) under PMT / PCSB Structural Health Cockpit Traffic Light System due to the following reasons:

1. No availability of structural models
2. Inspections performed to date for these platforms appeared to be based on typical conventional jacket underwater requirements and specific tarpon inspection requirements incorporating any safety critical elements (SCEs) which could have significant impact on the robustness of the tarpon had not been addressed or covered in any detail.

GLND is the engaged party to undertake the appropriate scope of work of PETRONAS assets so that PMT/PCSB could provide Management with the Structural Integrity Technical assurance for the continuing operating of these facilities. As such the compliance and long term integrity of these structures cannot be ascertained and effectively managed during its operating life. PMT/PCSB instructed GTS to engage GLND to undertake the appropriate scope of work so that PMT/PCSB could provide Management with the Structural Integrity Technical assurance for the continuing operating of these facilities. In addition as result of the above deliverables GTS would also be able to undertake any future tarpon structural detail assessments on behalf of PCSB if required.

On the other hand, recent occurrence of earthquake events from far field imparts tremors which are felt by platform operators in Malaysian waters. Two portions of the border of the Sunda Plate are seismically active interplate boundaries, namely between the Indo-Australian and Eurasian plates in the west and between the

Eurasian and Philippines Sea plates in the east. The Malaysian peninsula is located within the stable interior of the Sunda Plate in an area comprised between the Java trench in the west and south, and the Philippine plate and trench in the east. Thus, as stated in the distribution of earthquakes events with magnitude greater than 5 **(APPENDIX 1)**, this area is located in a seismically stable zone characterized by low anelastic strain rates as indicated by both geodetic data (Rangin et al., 1999; Becker et al., 2000; Bock et al., 2003) and the very low rate of shallow earthquakes (Bird, 2003). Only a few weak, generally deep, earthquakes have occurred in the past. Nevertheless, this does not exclude certain portions of the development fields in particular Malacca Strait and West Malaysia from being affected by ground motion from strong earthquakes generated by the Sumatra Fault system and the Sumatra subduction zone, situated about 300-600 km away.

Besides, it has been a concern that according to PETRONAS Technical Standard (PTS); the *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress*, which has been implemented and revised by the American Petroleum Institute (API), offshore project for zones with horizontal ground acceleration lower than 0.05g requires no earthquake analysis. This is supported by the prediction that the design for environmental loading other than earthquake will provide sufficient resistance against potential effects from neighbouring seismically active zones **(APPENDIX 2)**. Wave forces are assumed to be controlling the overall response of platform structure. However, One research states that ocean waves do not always act as a damping medium for seismic loads as was assumed so far. According to Yamada, Iemura, Kawano, & Venkataramana (1989), the response due to earthquake loadings is larger when the soil-structure interaction effects are considered. The hydrodynamic damping forces are higher in random seas than in still water and sea waves reduce the seismic response of offshore structures. Studies on the first passage probabilities of response indicate that small sea waves enhance the reliability of offshore structures against earthquake forces. Seismic and ocean waves acting simultaneously in different direction might even increase each other’s impacts. In addition, the *Seismic Hazard Study for Offshore Sabah, Sarawak and West Malaysia* carried out by the Italian Consultancy D’Appolonia found values to describe the seismic activity and return period for seismic activities. These values update and exceed the so far utilized values from ISO or GSHAP (Global Seismic Hazard Assessment Program).

1.3 Objectives

The primary aim of this assessment is to perform a computer-based simulation assessment on the structural response of a Tarpon Monopod platform in its inplace and intact conditions when subjected to a combination of metocean and seismic loads by using SACS suits of programmes. By stating extreme intact conditions, the environmental load is taking to 100 years return period of PETRONAS Technical Standards (PTS) 34.19.10.30, Offshore Engineering Center and UTP (OECU) Joint Density Parameters and also the As-Designed metocean criteria. Apart from that, soil profile data of BH-ANOA-L1, Ledang Anoa seabed is also modelled of which to characterize soil group.

The second objective is thus to identify a particular seismic loading as a threshold which controls the overall response over same loading on a Tarpon platform. By conducting incremental computer driven dynamic earthquake analysis according to suggested value of several published seismic models, a threshold ground acceleration unit will be ascertained at any rational magnitude which causes similar or perhaps greater responses as the extreme condition wave forces.

To complement the latter, the third objective is to evaluate the natural behaviour of the platform; in this case, a cable guyed monotower applicable for Malaysian waters, by considering its natural frequency, material stiffness and effective mass. It is then serves to be the reference or baseline to the platform response towards external actions so initial engineering intuitive can be made within. The aforementioned natural behaviour is set to be dealt with platform mode shapes, natural period, natural frequency and ultimate resisting stress.

1.4 Scope of Study: LDPA as an Ideal Option for Seismic Assessment

PETRONAS Carigali Sdn. Bhd. (PCSB) operates a handful of marginal platforms in the offshore of Peninsular Malaysia as well as Sabah and Sarawak. Narrowing the scope to Peninsular Malaysia Operation (PMO), there are 3 Tarpon Platforms in the oil and gas field in East Coast of Peninsular Malaysia – Ledang Drilling Platform (LDPA), Penara Drilling Platform (PeDPA) and North Lukut Drilling Platform (NLDPA). LDPA is chosen to be assessed for this project due to its completeness in available data. It will act as the sample representing group of Tarpon Monopod Platforms operated by PCSB. Since the Tarpon design is very repeatable and standardized in nature, LDPA will be a perfect exemplary to any other platform of similar type.

LDPA

FIGURE 1.2 Topside of LDPA Platform

(Source: LDPA Major Platform Inspection Report 2008)

Field	PM9
Structure Function	Drilling Platform
Year Installed	2006
Water Depth	77.1 metres (as per As-Built Drawing)
Manned	Unmanned
Design Life	20 years

1.5 Project Relevancy & Feasibility

Throughout the course of approximately 8 months, the project requires the author to conduct intense study regarding platform response towards seismic activities within Malaysian water region, hence execute thorough and comprehensive static and dynamic simulation by using SACS Suite of Programs. As the platform selected is relatively new to PCSB assets, there is extensive available data in fulfilling requirement for software simulation. This is important to ensure the simulation activity will be closely collateral to real condition. Since seismic design is usually neglected to be one of the basic loads of an offshore structure, within the region of South Asian Sea, and Malaysia is comprehended within, the author then deduce the project as industrially relevant.

Of the first half of total project duration, the author keeps much diligence to critical study on any relevant source of information regarding the subject matter as well as proficiency in handling the software. Whereas, the second half of total project duration is filled with simulation activities of combination loads generated by the author. Any result obtained, in the form of joint displacement and unity check, is to be analysed and validated by its relevancy and feasibility to be affecting tarpon safety critical element. Within the time frame provided, the objectives are considered highly achievable.

CHAPTER 2

LITERATURE REVIEW

2.1 Marginal Offshore Field Development

According to Meek & Sliggers (2001)

> Offshore reservoirs containing hydrocarbons will only be exploited if the estimated revenues of the recoverable reserves exceed the costs of the exploitation investment and operating expenditure to such an extent that an acceptable return on investment can be achieved. The minimum required return on investment would be set by the corporate philosophy of the oil company involved. Reservoirs that hardly can meet such a requirement are referred to as marginal fields. (p. 142)

The term marginal field incurred to oil and gas field with reservoir condition where higher investments are necessary to exploit the field. For shallow water condition, it is usually a small field with short estimated field life and low reservoir capacity. These marginal offshore fields can be hardly economic or rather unattractive for conventional development and it needs alternate development schemes which considerably reduce the costs required. In conceiving internal and external variables which were to be considered such as water depth, reservoir size, environmental conditions, soil conditions, equipments required and local market conditions, the best development plan for each specific field must be done to ensure it is economically viable.

A significant proportion of Malaysia's remaining resources lay in fields with less than 30 million barrels of recoverable oil. Developing these fields in an economically attractive manner is often challenging, as they need the same expensive infrastructure as large fields, while the expected revenue streams are smaller due to the smaller reserve sizes. Adjusting the development framework for small fields will increase Malaysia's oil production by approximately 55,500 barrels per day in 2020 (Economic Transformation Programme, 2012). The total investment needed to achieve this is approximately RM13.3 billion and the contribution to GNI is RM5.5

billion, which makes up for the GNI that would have been lost due to declining production if small field development were not deployed (Worldvest, 2014).

2.2 Tarpon Monopod Marginal Field Structure

Uniquely designed for water depth that ranges between 75 ft and 350 ft, Tarpon Monopod is one type of offshore platforms which mainly considers optimization in design. The low cost solution for marginal field development and matured assets allows it to be applied at a series of potential development locations (Tanjung Offshore, 2006). This type of platform is considered a low cost solution for marginal field development due to its simpler construction and design capabilities. Like many other minimal platform concepts, the Tarpon's design is highly standardized; this is especially true for its substructure. Such standardizations come with cost and time benefits which further enhances the Tarpon Monopod as an attractive alternative to conventional methods when developing a marginal field.

Tarpon structure is best utilized for topside loading <400 tonnes. Depending on the field requirement, the tarpon platform can handle heavier topside but it will lose its competitiveness. The installation process is relatively easy and fast. The Tarpon Monopod can be installed by means of a combination of a jack up drilling rig, and a couple of work vessels, where the drill rig will install the caisson, after which the guying system will be placed by the work vessels. With proper planning, the fact that the drill rig need not be removed in the installation phase, will lead to savings in expenditures and early cash flows, hence further justifying the economics of the marginal field (Lee Hsiu Eik, 2013).

There are currently more than 56 Tarpon platforms in use worldwide (Tarpon Systems, 2012). The platform, which consists of a minimum superstructure supported on a single main caisson guyed to three symmetrical pre tensioned cables, which is attached to the central caisson body at one end and anchor pile at another end. To date in Malaysian waters, tarpon platform concept is considered relatively new. The concept was first used in Semarang Kecil oil field in 2000 and was later applied in North Lukut and Penara oil fields in 2002 and Ledang Anoa in 2006. **TABLE 2.1** shows tarpon's track record in Malaysia prior to year 2006:

TABLE 2.1: Tarpon's track record in Malaysia [Source: Tanjung Offshore]

Asset	Semarang Kecil	North Lukut	Penara	Ledang Anoa
Date	2000	2002	2002	2006
Water Depth (m)	53 m (180 ft)	61 m (200 ft)	61 m (200 ft)	79 m (260 ft)
Conductors	2+1	5+1	5+1	2+1
Topside	250 tonnes	280 tonnes	280 tonnes	250 tonnes
Application	Tarpon + CPP	Tarpon + FPSO	Tarpon + CPP	Tarpon + FPSO

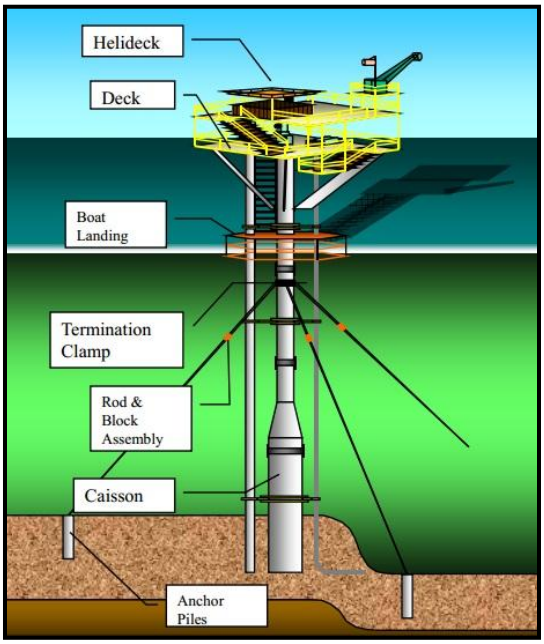


FIGURE 2.1: Major Components of a Tarpon structure [Source: Tarpon Systems]

FIGURE 2.2: Photograph of topside of PEDPA Tarpon Platform [Source: PCSB Collection]

The functions of some of the major component as shown in **FIGURE 2.1** are briefly summarized below (Syamsul, 2012):

TABLE 2.2: Major component of Tarpon structure and corresponding functions

Anchor Piles	To anchor / fix the guy wires to the mudline /seabed.
Caisson	A steel caisson with a diameter typically larger than the conductors which acts as the platform's leg, bracing points for the conductors via clamps, and in some cases, can be used to house several internal wells
Conductor	A steel caisson or riser used to protect the well and production tubing
Conductor Clamp	To vertically fix the conductor casings to the caisson
Guy Cables	To provide lateral resistance and stability for the platform
Topside	The superstructure located above the reach of waves, equipped with facilities such as production equipment, jib crane, boat landing, helideck and a flare boom

2.3 Earthquake and Seismic Waves

An earthquake is an occurrence resulted from sudden slip of the earth blocks past one another. The surface where they slip is called the fault or fault plane. The location below the earth's surface where the earthquake starts is called the hypocenter while the location directly above it on the surface of the earth is called the epicentre. Lisa Wald (2012) in her article from USGS Website, the earth is made of four major layers which are the inner core, outer core, mantle and crust **(FIGURE 2.3)**. The crust and the top of the mantle make up a thin skin on the surface of the planet. But this skin is not all in one piece – it is made up of many pieces like a puzzle covering the surface of the earth. Not only that, these puzzle pieces keep slowly moving around, sliding past one another and bumping into each other. These puzzle pieces are called tectonic plates and the edges of the plates are called the plate boundaries, as shown in **FIGURE 2.4**. The plate boundaries are made up of many faults, and most of the earthquakes around the world occur on these faults. Since the edges of the plates are rough, they get stuck while the rest of the plate keeps moving. Finally, when the plate has moved far enough, the edges detached on one of the faults and there is an earthquake.

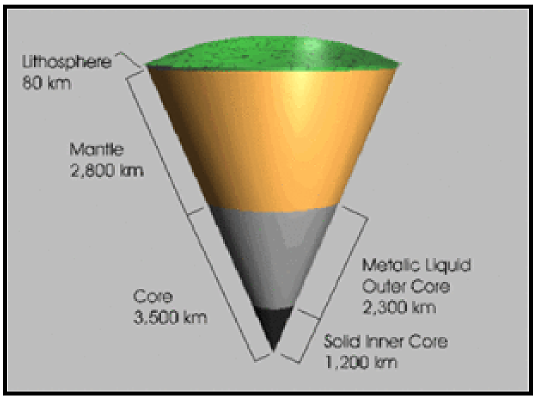


FIGURE 2.3: Four major layers of the earth [Source: USGS]

FIGURE 2.4: Tectonic plates and plate boundaries [Source: USGS]

While the edges of faults are attached together, and the rest of the block is moving, the energy that would normally cause the blocks to slide past one another is being stored up. When the force of the moving blocks finally overcomes the friction of the jagged edges of the fault and it detaches, all that stored up energy is released. The energy radiates outward from the fault in all directions in the form of seismic waves like ripples on a pond. The seismic waves shake the earth as they move through it, and when the waves reach the earth's surface, they shake the ground and anything on it.

Earthquakes are recorded by instruments called seismograph which translates the results into a recording called seismogram. Seismogram comes in handy for locating earthquakes and being able to see the P waves and the S waves. P waves are faster than S waves and within this facts, it is possible to detect the origin of an earthquake. By looking at the amount of time between the P and S waves on a seismogram recorded on a seismograph, scientists can tell how far away the earthquake was from that location. However, they cannot predict in what direction from the seismograph the earthquake was, only how far away it was. If scientists draw a circle on a map around the station where the radius of the circle is the determined distance to the earthquake, it can be predicted that the earthquake lies somewhere on the circle. Scientists then use a method called triangulation to determine exactly where the earthquake was **(FIGURE 2.5)**.

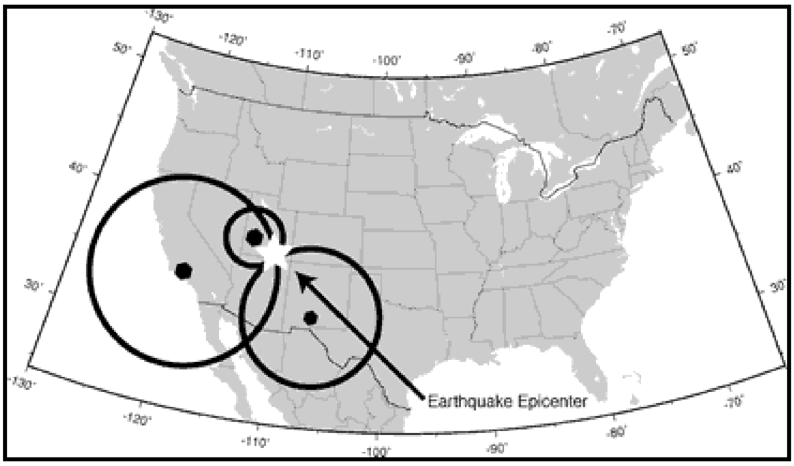


FIGURE 2.5: Identification of epicentre by 'triangulation' method [Source: USGS]

2.4 Seismicity in Peninsular Malaysia Region

Seismic waves are generated by an impulse such as sudden breaking of rock within the earth or explosion which termed as earthquake. It may travel either along or near the earth's surface or through the earth's interior (USGS, 2012). Generally, Malaysia is situated close to two seismically active plate boundaries which are:

1. The inter-plate boundary between the Eurasian and Philippines Sea Plates on the East of Malaysia, shown in **FIGURE 2.6**
2. The inter-plate boundary between the Indo-Australian and Eurasian Plates on the West of Malaysia, shown in **FIGURE 2.7**

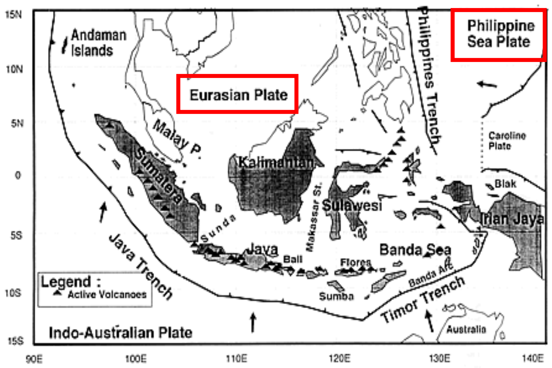


FIGURE 2.6: Location of East Malaysia from Eurasian Plate and Philippines Sea Plate [Source: Google Image]

FIGURE 2.7: Location of West Malaysia from Indo-Australian and Eurasian Plates [Source: Google Image]

These subduction plate boundaries has been responsible for several earthquake events in the past, the best known being the December 26, 2004 magnitude 9.3. However, the Malaysian Peninsula and South China Sea are located within the stable interior of the Sunda Plate in an area constituted between the Java Trench (Indo-Australian and Eurasian Plates) in the west and south, and the Eurasian and Philippine Sea Plate in the east. This area is located in a seismically stable zone where only a few weak earthquakes have been took place in the past (D'Appolonia, 2008). As LDPA is located in the East Coast of Peninsular Malaysia, seismic hazard is mainly controlled by the earthquakes associated with Eurasian Continental Plate compared to the subduction beneath Indonesia.

Nevertheless, this does not exclude certain portions particularly in Malacca Straits and West Malaysia from being affected by ground motion from strong earthquakes generated by the Sumatra Fault system and the Sumatra subduction zone situated about 300-600 km away. It is described in one of the USGS publication (2013) that earthquake intensity is expressed based on the observed effects of ground shaking on people, buildings, and natural features. It varies from place to place within the disturbed region depending on the location of the observer with respect to the earthquake epicenter. This in fact explains the effects from earthquake occurrences in neighbouring Indonesia with magnitudes ranging between 6.0 and 8.0 which were responsible for the largest ground motions that are felt in buildings in Singapore and Kuala Lumpur (D'Appolonia, 2008). Within a region as large as covered by the five development locations within Malaysia **(APPENDIX 3)**, the seismotectonic settings are varied where they covered several countries. This has resulted in the publication of several seismic models that cover different portions of the area of interest.

Acceleration is the most relevant measures to be used as the structural codes prescribe how much horizontal force should a structure be able to withstand during an earthquake occassion. This force is related to the ground acceleration. The peak acceleration is the maximum acceleration experienced by the particle during any course of earthquake motion. A small particle attached to the earth during an earthquake will be moved back and forth rather irregularly. This movement can be described by its changing position as a function of time, or by its changing velocity as a function of time, or by its changing acceleration as a function of time (USGS, 2007). Since any one of these descriptions can be obtained from any other, whichever most convenient may be chosen.

2.5 Structural Dynamics

2.5.1 Natural Frequency and Mode Shape

The first usual step in performing a dynamic analysis is determining the natural frequencies and mode shapes of the structure. The equation of motion consists of restoring force, damping force and inertia force which are all resisting the external force.

Equation of motion: $\boxed{\mathrm{F} = \mathrm{Kx} + \mathrm{C}\dot{\mathrm{x}} + \mathrm{m}\ddot{\mathrm{x}}}$

Where:

F: Sum of applied force (total horizontal force)

K: Stiffness of structure

c: constant damping ratio

m: Effective deck mass

In the absence of the external excitation, the structure is actually in a free vibration mode as the equation becomes $Kx + C\dot{x} + m\ddot{x} = 0$. By considering parameters of the three (3) terms, it defines the natural frequency and mode shapes of the structure. These results describe the basic dynamic behaviour of the structure and are an indication of how the structure will respond to dynamic loading. The deformed shape of a structure at a specified natural frequency of vibration is termed as its normal mode of vibration. Each mode shape is associated with a specific natural frequency. Natural frequencies and mode shapes are functions of the structural properties (i.e. elastic modulus) and boundary conditions (i.e. welded joints). If the structural properties change, the natural frequencies change, but the mode shapes may not necessarily change; but if the boundary conditions change, then the natural frequencies and mode shapes both change.

Derivation of natural frequency:

$$\boxed{\omega_n = \left(\frac{K}{m}\right)^{1/2}}$$

Where:

ω_n : Natural Frequency

K: Stiffness of structure

m: Mass of structure

An accurate analysis of the eigenvalue and mode shapes of an offshore platform is a fundamental matter to the solution of its dynamic responses due to seismic and environmental loads. There are many reasons to compute the natural frequencies and mode shapes of a structure. All of these reasons are based on the fact that real eigenvalue analysis is the basis for many types of dynamic response analyses.

2.5.2 Dynamic Amplification Factor (DAF)

Quantification of the dynamic response, relative to the static response, can be represented in a dimensionless form by defining Dynamic Amplification Factor (DAF) expressed as (Barltrop and Adams, 1991):

$$\boxed{\mathrm{DAF} = \frac{Dynamic\ Response\ Amplitude}{Static\ Response\ Amplitude}}$$

DAF will be very high when the natural frequency is close to the wave frequency. If the DAF is less than 1.1, it is enough that the design is based on a regular design wave and static methods of analysis. But if it exceeds 1.1, dynamic analysis is then appropriate to be executed for that particular design loads.

Use of DAFs is widespread for linear structural systems as a simplification of structural dynamic analyses. By knowing the DAF value and the static response amplitude of a system, the dynamic response amplitude of the system can easily be evaluated. In such an approach, there is obviously no need for complicated and time consuming dynamic analyses.

CHAPTER 3

METHODOLOGY

3.1 Project Methodology

Literature Review

Marginal Offshore Field Development, Tarpon Monopod Marginal Field Structure, Earthquake and Seismic Waves, Seismicity in Malaysia, Structural Dynamic Models , SACS

Data Gathering

Platform Generic Details, Weight Data, Material Properties, Environmental Loads, Peak Ground Acceleration, Soil Properties

Structural Response Evaluation by using SACS Dynamic Analysis:

1) Platform Topline Characteristics and Behaviour

Identifying LDPA platform natural behaviour by considering restoring force (stiffness of structure), damping force (constant damping ratio) and inertia force (effective deck mass). Displacement of caisson members will be assessed for mode shape and its natural frequency.

Structural Response Evaluation by using SACS Static Analysis:

2) Platform Response when subjected to Environmental Loads

Conducting sensitivity and parametric study by exerting three relevant metocean criteria of different magnitudes (wave and current only) in order to prompt for its Unity Check results of every each caisson members. Distinctive high UC values are investigated.

Structural Response Evaluation by using SACS Dynamic Analysis:

3) Platform Response when subjected to Seismic Ground Acceleration

Conducting sensitivity and parametric study by exerting incremental ground acceleration values in reference to various seismic models. DIsplacement of caisson members (critical members) will be appraised for the deflection of platform leg

Structural Response Evaluation by using SACS Static Analysis:

4) Identification of platform ultimate strength

Exerting incremental single horizontal action to one critical member along caisson leg in order to identify maximum load at which platform can withstand. Point load at failure indicates platform ultimate strength

Results Interpretation

Identification of a particular seismic load where the design seismic threshold unit controls the overall respond of wave load

Comparison between resulted force acted upon platform when subjected to maximum ground acceleration and platform ultimate strength

Platform integrity

3.2 Research Tools

- ✓ Internet resources
- ✓ Codes and Standards
- ✓ Research papers
- ✓ PCSB reports
- ✓ Computer Aided Design - SACS
- ✓ Verbal delivery from supervisor and seniors
- ✓ Reading materials from Information Resource Centre

3.3 Data Required

3.3.1 Platform Generic Details (**TABLE 3.1)**

No	Platform Details	LDPA Data
1	Field	PM9
2	Platform Type	Monopod Platform
3	Manned/Unmanned/Quarters	Unmanned, no quarters
4	Operator, Year Installed	PETRONAS, 2006
5	Operational Status	Active
6	Water Depth	76.3 m
7	Jacket Height	82.2 m
8	Air Gap	1.5 m
9	Deck Elevation	9.8 m
10	Number of Decks	3
11	Number of Legs	1
12	Maximum Leg Diameter	1981.2 mm
13	Number of Conductors	3
14	Maximum Conductor Diameter	0.762 m
15	Number of Slots	3
16	Helipad	0
17	Number of Cranes	1
18	Maximum Crane Size	3 MT

19	Number of Risers	1
20	Number of Caissons	1
21	Boat Landing	1
22	Number of Piles	3
23	Design Marine Growth	0.153 m
24	Design Scour	0.9 m
25	Shore Distance	200 km
26	Design Code	API RP 2A 21st
27	Design Life	20 years
28	Design Return Period	100 years

3.3.2 Weight Data **(TABLE 3.2)**

No	Element	Weight
1	Topside	200.00 T
2	Substructure	800.00 T
3	Conductors	244.18 T
4	Caisson	290.19 T
5	Boat Landing	35.00 T
6	Guyed Wire + Piles	150.34 T

3.3.3 Material Properties **(TABLE 3.3)**

Material	Property	Value
Steel	Density	7,850 kg/m^3
	Modulus of Elasticity	210,000 MPa
	Shear Modulus	77,000 MPa
	Poisson's Ratio	0.3
	Coefficient of Thermal Exp.	1.175E-5/°C

3.3.4 Environmental Loads

Environmental load is the main key to designing the coastal structure which consists of wind speed, wave height and wave current. These environmental loads significantly affect all kinds of maritime activities especially the platform stability, and their worst effect is typically caused by the maximum wave criteria (Idzwan Selamat, 2013).

Platforms are usually designed based on the parameter of 100-year return period. The 100-year return period is for the wind speed design, wave height design and also for the current design. The data is collected either by in-situ measurement or by Hindcast analysis which has been practiced by the operation for better research and findings. In-situ measurement is the measured live data taken at any particular area by using instruments such as wave radar rex and wind observer. Within 10 minutes interval, the data is taken by its mean or average value.

Three different metocean criteria are considered within this study which are PETRONAS Technical Standard, As-Designed and Joint Density. The PTS values are taken for PMO condition at 100 years storm condition suggested by PETRONAS, the As-Designed values are the maximum out of the storm event while the joint density values are the metocean criteria suggested for cost & time optimization with lighter platform design. **TABLE 3.1** describes the values aforementioned.

TABLE 3.4: General environmental loads for three different metocean criteria

Parameters		**OECU Joint Density**	**PTS (100 Year Storm Event)**	**As-Designed**
Wave	Max Height	5.70 m	5.77 m	11.30 m
	Period	8.00 s	8.06 s	9.30 s
Current	Surface	0.15 m/s	1.67 m/s	1.30 m/s
	Mid-Depth (0.5D)	-	1.33 m/s	-
	Near Seabed (0.01D)	0.69 m/s	0.36 m/s	0.70 m/s

3.3.5 Peak Ground Acceleration (PGA) in West Malaysia Region

Probabilistic Study Hazard Assessment (PSHA) has been carried out by an Italian Consultancy, D'Appolonia within Sabah, Sarawak and West Malaysia concessions in the South China Sea with the aim to update seismic design criteria for the particular area. Based on the comparison to previous studies, the mean hazard predicted by the current PSHA is somewhat higher than published by other studies. The following table shows PSHA results of the most recent study carried out by the Italian Consultancy D'Appolonia and other seismic models within West Malaysia region.

TABLE 3.5: Summary of PGA values of several seismic models

[Source: Poggi et. al.; D'Appolonia PSHA Report]

Reference	Site Class	Return Period (Years)	PGA (g)
Mc Cue (1999)	Rock	475	<0.08
Petersen et. al (2004)	Rock	475	<0.03
		2475	<0.08
Adnan et. al (2005)	Rock	475	<0.01
		2475	<0.015
D'Appolonia PSHA (2008)	Rock	475	0.04
		1000	0.065
		2475	0.114

3.3.6 Load Cases Involved **(TABLE 3.6)**

Load Case	Description
1	Sacs Calculated Model Self-weight
2	Topsides Structural Appurtenances
3	Open Area Live Load
4	Equipment Dry Weight
5	Equipment Operating Weight

6	Piping/Electrical/Instrumentation Dry Weight
7	Piping/Electrical/Instrumentation Operating Weight
8	Substructure Appurtenances Dead Loads
9	Substructure Appurtenances Buoyancy Loads
10	PTS 100 Year Storm Condition
11	As Designed for 100 Year Storm with Maximum Wave Height
12	OECU Joint Density

3.4 Structural Analysis Computer System (SACS)

SACS is an integrated suite of software that supports the analysis, design, fabrication, and installation of offshore structures, including oil, gas, and wind farm platforms and topsides (Bentley Systems, 2014). For the purpose of this project, SACS 5.3 Suite of Programs will be used extensively for both modelling and simulation. Below are the corresponding programs which have been using throughout the assessment. [2]

3.4.1 PRECEDE – Interactive Full Screen Graphics Modeller

This program provides special handling of structures that are jacket oriented, but is also adept at handling non-jacket structures. It is to be used as the graphical user modeller. PRECEDE can automatically generate 5 different structure types, such as jackets, decks, dolphin/wharves, towers or space frames. Structures generated using the automatic generation facility have elevation, plan and face views created that may be displayed easily. As for this assessment, PRECEDE program is used during platform model refining stage where the member properties and basic loads are fixed in its intact condition throughout the assessment.

3.4.2 SEASTATE – Environmental Loads Generator

SEASTATE generates and calculates the environmental effects on an offshore structure which implements the API 20th edition and supports five wave theories. This module processes, through the computer, user-supplied environmental and design data and calculates the static and dynamic forces within and upon each component of the structure. Within this assessment, static analysis is advanced by basic loads and extreme condition environmental loads while dynamic analysis takes part when ground acceleration is considered together with Pile/Structure Interaction.

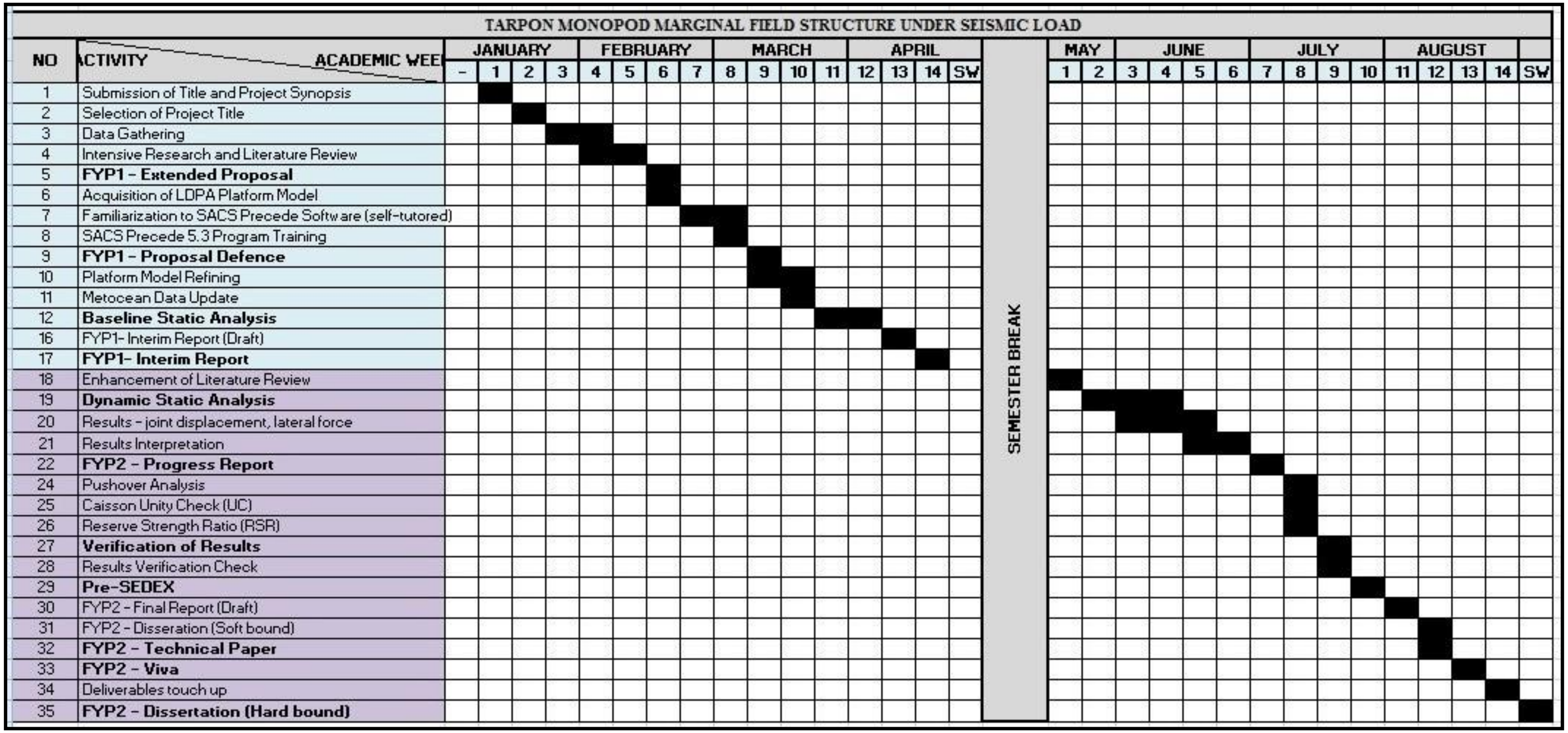
3.4.3 POSTVUE – Interactive Graphics Post Processor

POSTVUE enables the author to interpret the results interactively and graphically. It processes a large quantity of output data generated from input modules which is then organized and printed in a systematic distribution that facilitates further engineering measures.

3.4.4 DYNPAC – Dynamic Characteristics

DYNPAC provides the function of Guyan reduction of non-essential degrees of freedom. It can be either lumped or consistent structural mass generation. The program is able to prompt automatic virtual mass generation and complete seastate hydrodynamic modelling by taking user input distributed or concentrated mass. It is a non-structural weight modelling with full 6 DOF modes available for forced response analysis.

3.5 Gantt Chart & Key Milestones



CHAPTER 4

RESULTS & DICUSSIONS

With regards to the three main objectives aforementioned in the earlier part, this chapter is to emphasize primarily on marginal structure response towards external forces and loads; as for this study, environmental loads and seismic ground acceleration. Platform natural behaviour complements the latter results by providing basic or topline characteristics & responses of the platform for its own integrity. Last but not least, threshold at which ground acceleration controls the dominant wave forces is evaluated by considering every parametric increment of seismic ground acceleration vs. environmental loads.

4.1 Topline Platform Characteristics and Responses

The damping values for offshore structures typically range from about 5% to 10% of critical damping (API RP 2A 21st, 2007). However, damping ratio is taken as 3% for this study as its natural frequency is less than three seconds, in which only small value of damping is considered within the system. The damping in waves is usually higher than the damping in the free oscillation of the system while the dynamic mass system is selected as ‘consistent/continuous’ mass in contrast to the lumped mass model. Because there remains only one movable mass (the effective mass at the top of the structure) and only one direction of sensible motion (the horizontal direction), this case is considered as single degree of freedom cantilever with a uniformly distributed mass and a lumped mass at the top.

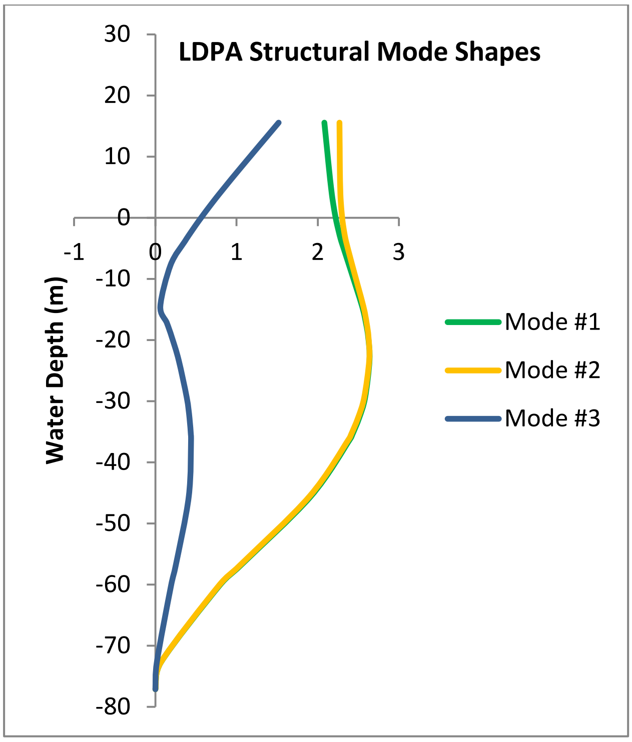


FIGURE 4.1 LDPA Structural Mode Shapes based on Joint Displacement

By considering a total of 10 number of mode shapes, the first three is selected. From the equation of motion $F = Kx + C\dot{x} + m\ddot{x}$, it can be justified that the dynamic behaviour of the platform is contributed by its restoring force, damping force and inertial force. According to PTS, the natural period of a fixed steel jacket platform is 2.5s. No literature is found to state the natural period or frequency for a tarpon platform but the value is presumed to be at close approximation to the one of fixed steel jacket. Below are the results obtained from extract mode shapes dynamic analysis:

TABLE 4.1 Corresponding natural period and frequency of the platform

Mode	Tn (s)	Ωn (Hz)
1	1.907	0.524
2	1.898	0.527
3	1.639	0.610

4.2 Platform Response Subjected to Environmental Loads

Based on API Recommended Practice 2A-WSD (clause 2.3), the wave loads on a platform are dynamic in nature. However, it can be adequately represented by their static equivalents to quasi-static loads. Quasi-static loads are actually due to dynamic phenomena but they remain constant for a relatively long period. For most template, tower, gravity and caisson types of platforms, the design fluid dynamic load is predominantly due to waves while currents and winds playing a secondary role (API Recommended Practice 2A-WSD, Clause 1.5). This can be supported by the fact that waves made up of approximately 70% of the total stress at a given point of a structure compared to wind and current (Kurian V. John, 2014).

Morison et al. (1950), as of Morison Equation proposed that the force exerted by unbroken surface waves on a vertical cylindrical member is composed of two components, inertia and drag. Due to these forces (a portion of total loads imposed), the members experienced time-varying stresses hence contributes to cantilever effect as deflection on the central caisson members.

Unity Check has been used to be the chosen parameter to evaluate platform response toward environmental loads because it can be able to portray the most critical member by its strength and capacity. In the approach of using working Stress Design, unity check is the safety factors which are mostly covered in the components part reside in the formulas for members, joints and foundation calculation (M. Shahir Liew, 2011).

Free standing caissons, guyed and braced caissons, as well as single leg deck units and other single member structural systems have less redundancy and may not necessarily exhibit the same characteristics as the conventional fixed jacket platform. By considering metocean criteria and general actions acted upon the structure, the allowable stress interaction ratio (or unity check) must be limited to 0.85 for free standing caissons or single element structural systems during storm conditions (API RP 2A 21st, 2007).

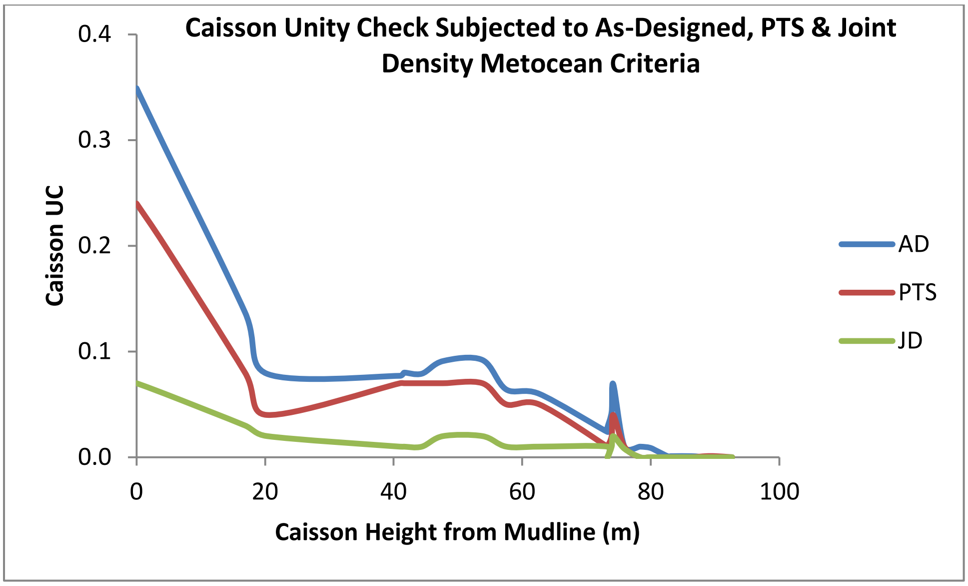


FIGURE 4.2 Caisson Unity Check Subjected to As-Designed, PTS & Joint Density Metocean Criteria

As been accentuated from the graph, it can be observed that there are two significant peaks from the whole plots. The most critical joint is at the lowest elevation of caisson leg where its fixed end resulted in the highest bending moment. The UC values at the members where caisson leg and mooring lines intersects (cable terminators) portray distinct increase compared to neighbouring members due to high shearing forces caused by the tension of mooring lines.

4.3 Platform Response Subjected to Seismic Ground Acceleration

Under seismic motion, the excitation is transmitted to the structure through the ground therefore the dynamic interaction effects between soil, piles and structure attains particular importance. The difference in response is due to the spectral characteristics of waves and earthquake ground acceleration (A. Gurpinar et al.). In this context, the PSI assessment seems necessary but it is out of reach within this limited period of time. By preparing spectral earthquake input files, the soil condition is characterized as C [2] due to the soil condition which consisted mainly clay and silt deep down to approximately 150m.

[2]Type C is defined as deep strong alluvium soil (API, 2005)

PGA values are chosen to be used for analysis numerical input because there is very limited information regarding ground acceleration in the open literature. As PGA is the maximum suggested value for ground acceleration, it is always good to put high benchmark for design loads/forces so the assumption always goes for worst case scenario. With preliminary value of PGA 0.01g, it is obtained that the maximum XY deflection along 92.69m length of central caisson is 1.43m, whereas for PGA 0.114g, the maximum caisson joint displacement is 16.62m. Defining the pattern of the deflection plot, every incremental of 0.01g resulted in caisson joint displacement of 1.1 to 1.5 of the previous one (considering maximum displacement at every seismic ground acceleration value). For example, the maximum caisson displacement for PGA 0.09g is 12.80 cm while PGA 0.08g is 11.38 cm.

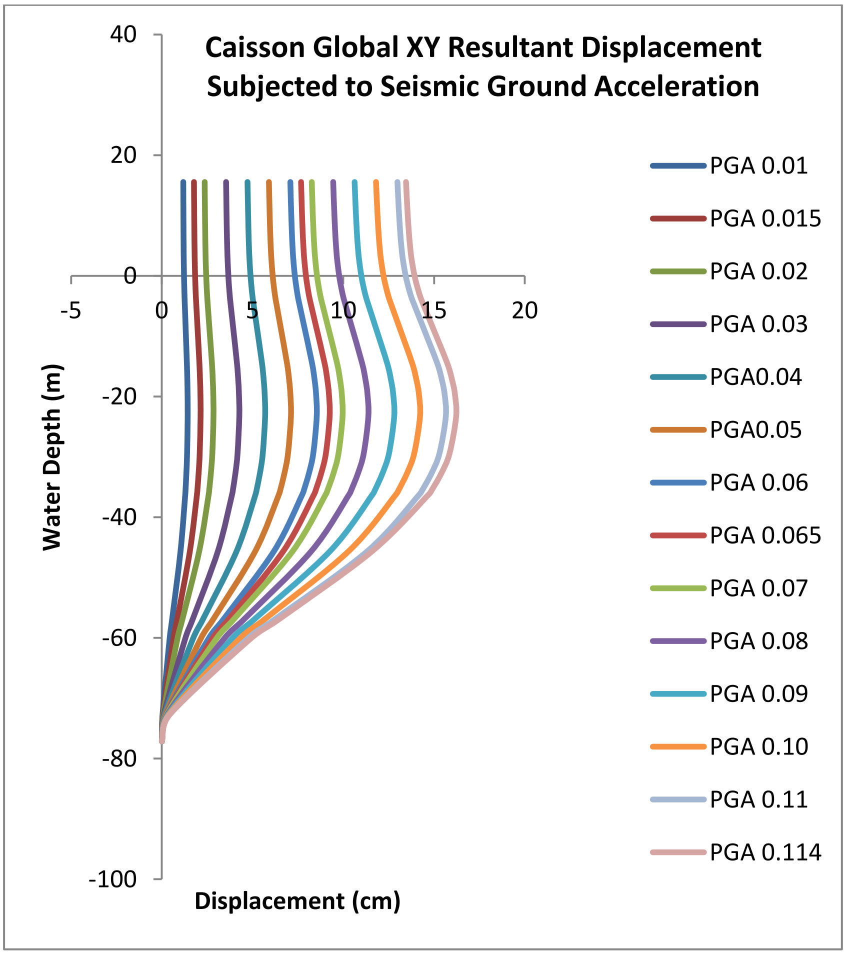


FIGURE 4.3 Caisson Global XY Resultant Displacement Subjected to Seismic Ground Acceleration

4.4 Determining Threshold on Controlling Seismic Ground Acceleration vs. Wave Forces

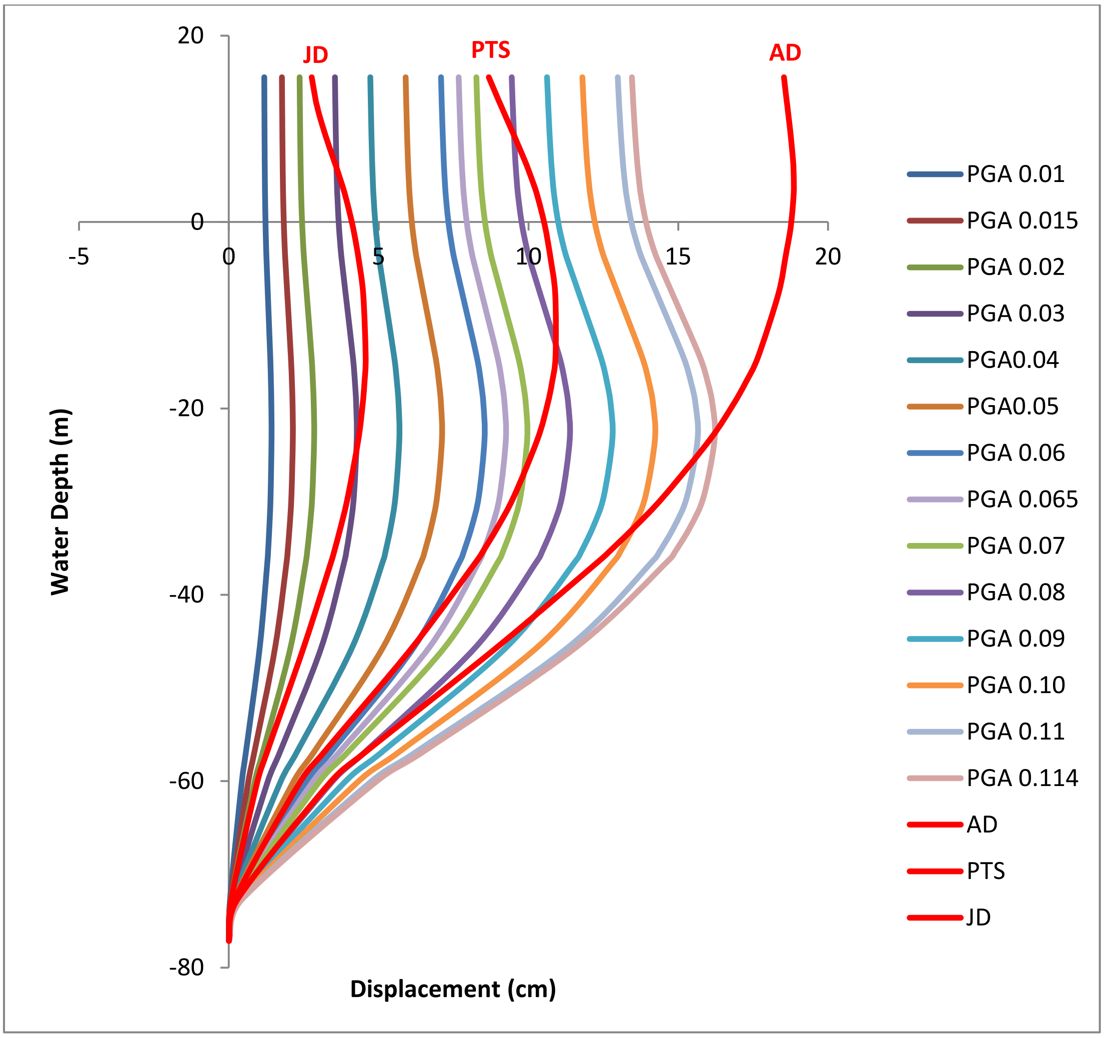


FIGURE 4.4 Displacement of caisson leg members with respect to various ground acceleration and metocean criteria

It is decided to be using displacement as the parameter to evaluate platform responses to both types of action (seismic motion and environmental loads) because the displacement induced into a structure is caused by its internal forces and stresses. Lateral forces can be another option for results analysis but there were certain drawbacks which make it to be irrelevant as the stresses induced by seismic motion into the structural elements are not as concentrated as those induced by wave (Floeck, 2013). By considering joint displacement along caisson members, Unity Check can be defined by the capacity of internal stresses of a member which has been used. No joint displacement values are taken within topside members as the topside is designed to be able to experience some damage, without leading to collapse, loss of life or major environmental hazards.

By means of incremental iterative ground seismic acceleration values and different load designs of different metocean criteria, the author identifies the threshold at which seismic ground acceleration starts to control the overall structural responses in terms of displacement, as in the table below:

TABLE 4.2 Threshold at which seismic motion overtake the overall structural response by the dominant wave forces

	Wave Height (m)	**PGA (g)**
OECU Joint Density	5.70	0.03 g
PTS 100 Year Storm Condition	5.77	0.065 g
As-Designed	11.30	0.09 g

Next, the question possibly yields from this finding is predictive to be, 'Is it possible for the suggested seismic ground acceleration values to be causing structural failure?' In order to verify this, the author has been setting up another separate static analysis where one of the members along caisson leg is picked (highest displacement) to be the joint where a horizontal point load is exerted. The idea of this final simulation is to identify the maximum load the member can withstand until failure by evaluating its UC results. By inputting random magnitude of loads, the author continues the simulation subjected to load increment till failure. From this analysis, a final value of at 6700kN has been identified to be the turning point for structural failure (UC>1). From the previous spectral earthquake dynamic analysis, the platform response when subjected to seismic ground acceleration of 0.114g yields lateral force of 1861.5kN. From here, it can be proven that seismic ground acceleration is not possible to cause structural failure to the platform, yet it is able to control the overall response of platform at a particular ground acceleration magnitude.

4.5 Dynamic Amplification Factor (DAF)

From the extract mode shape dynamic analysis, it is obtained that the natural frequency for this platform is 1.906 s (The first mode value is taken) while damping ratio is taken as 3%. By using the DAF equation, the values are obtained as follows:

	OECU Joint Density	**PTS (100 Year Storm Event)**	**As-Designed**
Wave Period, T (s)	8.00	8.06	9.3
Wave Frequency, ω (Hz)	0.7854	0.7796	0.6756
DAF	1.204	1.200	1.143
Notes	Static Analysis is not appropriate and **dynamic analysis is required**		

TABLE 4.3 DAF result of different metocean criteria

It is obtained that all three metocean criteria resulted in DAF values greater than 1.1. However, only static analysis has been done to the structure when subjected to environmental loads as the DAF values are only calculated after the analysis has been done. Nevertheless, this will surely be a good element for any further study to be carried out by replacing the static analysis to extreme wave dynamic analysis or other options of wave and wind driven dynamic analysis.

CHAPTER 5

CONCLUSION & RECOMMENDATIONS

5.1 Conclusion Overview

This report prepared within the scope of Final Year Project, has been taking the author to study the structural response of a marginal offshore structure when subjected to external actions, especially the seismic ground motion. By using the SACS 5.3 Executive Software, the author requires several data in order to execute 4 sets of analysis inclusive static and dynamic in order to study the platform characteristics and responses. Throughout the assessment, the author steps into agreement that considering seismic wave criteria into platform design is certainly an apt option due to the probability of any significant seismic event to be occurring to offshore oil and gas fields in Malaysia. It should not be an impossible occurrence that seismic ground motion may eventually occurred as recent studies has suggested values exceeded the so far utilized ones from ISO or GSHAP (Global Seismic Hazard Assessment Program) within the West Malaysia region, as well as other parts of the country. The author foresees that this assessment can serve as a reference for PETRONAS to instill additional reasoning for imparting earthquake/seismic into design considerations within Malaysian region.

5.2 Results Summary

- LDPA platform conveys natural period of close approximation to 2s, in which the simulation resulted 1.907s, 1.898s and 11.639s for the first three modes.
- The highest UC (most critical joint) values are obtained at the lowest elevation of caisson leg where its fixed end resulted in the highest bending moment.
- The UC values at the members where caisson leg and mooring lines intersects (cable terminators) portray distinct increase compared to neighbouring members due to high shearing forces caused by the tension of mooring lines.
- The maximum XY deflection along 92.69m length of central caisson is 1.43m when PGA 0.01g is exerted, whereas for PGA 0.114g, the maximum caisson joint displacement is 16.62m.

- Defining the pattern of the deflection plot, every incremental of 0.01g resulted in caisson joint displacement of 1.1 to 1.5 of the previous one (considering maximum displacement at every seismic ground acceleration value).
- As per existing design load of LDPA platform with wave height of 11.30m, the seismic action is capable of controlling the overall response of the platform at ground acceleration of 0.09g.
- Structural failure point is reached when a horizontal point load of 6700kN is applied. It is obtained that the maximum lateral force yielded from seismic ground action 0.114g is only 1861.5kN. This proves that the suggested magnitude of earthquake/seismic action is not possible to cause failure to the platform, yet it is able to control the overall response of the platform at a certain ground acceleration value.

5.3 Future Considerations

- This study will not be able to represent other single leg tarpon monopod platform, especially of which outside the region of East Coast of Peninsular Malaysia because they do not exhibits similar data; soil type, ground acceleration, return period, etc., and located in the location exposed to different tectonic plates. Hence, it is suggested that Separate study needs to be conducted on Tarpon structure in other region especially in the East Malaysia as the updated ground acceleration is somewhat higher than of in Peninsular Malaysia.
- Seismic loads are now being considered as a result of its frequency and how it can be affecting any particular region with offshore platform of absence in seismic design incorporation. With the DAF results obtained (stated in the Chapter 4 of this report), and the factual knowledge that loads arising from wind, wave, current and seismic are dynamic in nature, the correct way forward is to execute dynamic analysis to every measures it takes within the assessment.

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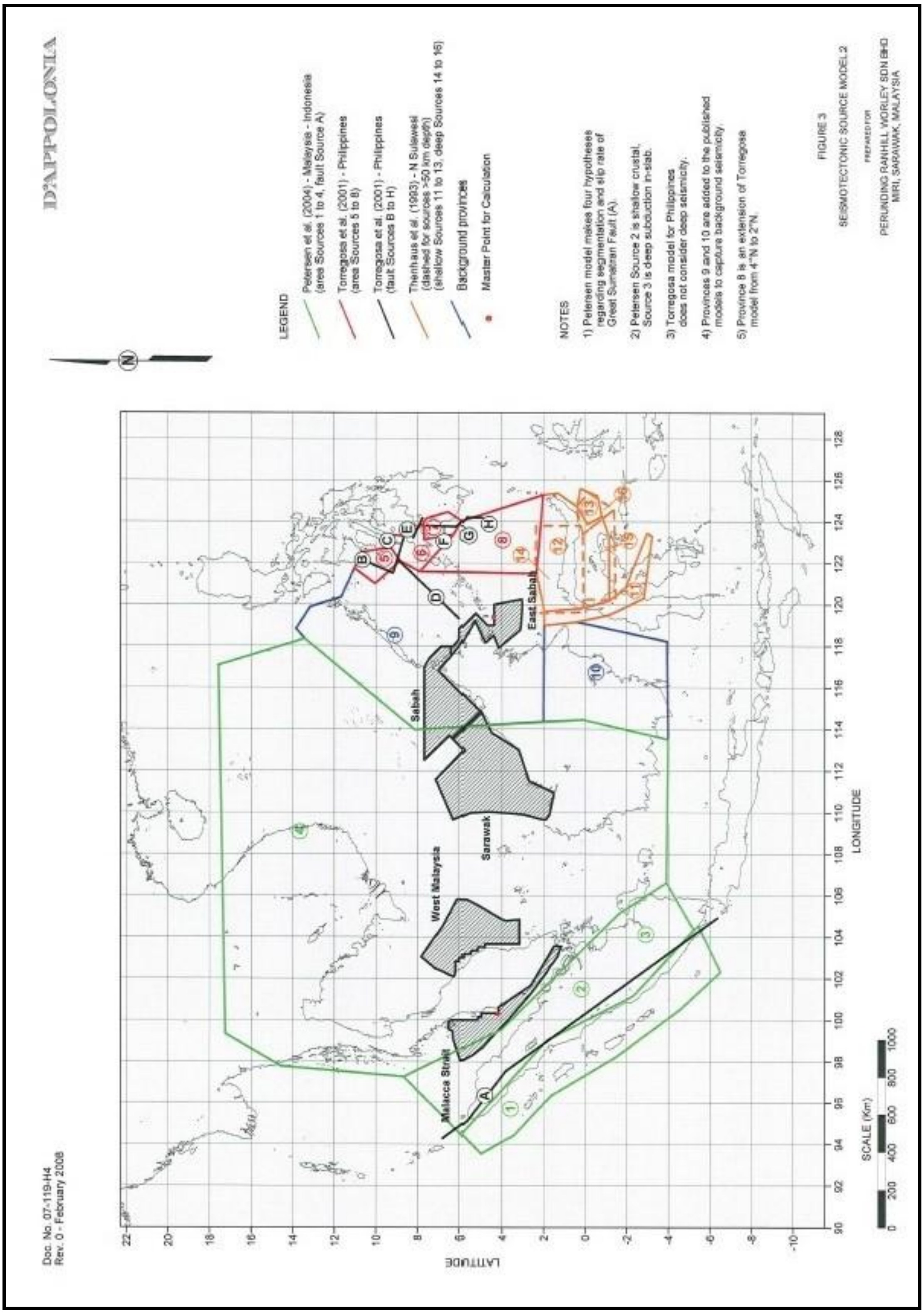
APPENDIX A.1


D'APPOLONIA
Doc. No. 07-119-H4
Rev. 0 - February 2008
LEGEND
Petersen et al. (2004) - Malaysia - Indonesia (area Sources 1 to 4, fault Source A)
Torregosa et al. (2001) - Philippines (area Sources 5 to 8)
Torregosa et al. (2001) - Philippines (fault Sources B to H)
Thenhaus et al. (1993) - N Sulawesi (dashed for sources >50 km depth) (shallow Sources 11 to 13, deep Sources 14 to 16)
Background provinces
Master Point for Calculation
NOTES
1) Petersen model makes four hypotheses regarding segmentation and slip rate of Great Sumatran Fault (A).
2) Petersen Source 2 is shallow crustal, Source 3 is deep subduction in-slab.
3) Torregosa model for Philippines does not consider deep seismicity.
4) Provinces 9 and 10 are added to the published models to capture background seismicity.
5) Province 8 is an extension of Torregosa model from 4°N to 2°N.
FIGURE 3
SEISMOTECTONIC SOURCE MODEL 2
PREPARED FOR
PERUNDING RANHILL WORLEY SDN BHD
MIRI, SARAWAK, MALAYSIA
LATITUDE
LONGITUDE
SCALE (Km)
Malacca Strait
West Malaysia
Sarawak
Sabah
East Sabah

APPENDIX A.2 (1)

4.0 DESIGN CRITERIA FOR SUBSTRUCTURES (IN-PLACE CONDITIONS)

4.1. Basic Loads

The basic loads, as detailed below, shall be used in the design loading conditions described in Section 4.4. Adequate allowances shall be considered when establishing these loads in order to allow any weight growth during final design and/or fabrication. The effect of topside stiffness on the substructure shall be considered in any computer analyses. The loads to be considered in the in-place condition include the following:

a) **Deck Self Weight.** The structural self weight of the deck consists of the total net weight of the decks including all beams, flooring, stairways, walkways, boat landing, equipment sheds and buildings and all fitments to these parts.

b) **Jacket Self Weight.** The structural self weight of the jacket consists of the installed weight of the jacket (with appurtenances) and piles. A proportion of jacket braces (usually 10% by volume) shall be considered flooded.

c) **Weight of Other Topside Structures (Tower, Helideck, etc.).** The self weight of these structures consisting of the total weight including all beams, floorings, stairways and all fitments to these parts.

d) **Derrick Equipment Set Loads.** Where applicable, the loads resulting from the derrick equipment set shall be considered.

e) **Equipment Operating Load.** The equipment operating load is the load under normal operating conditions with associated fluid load included.

f) **Equipment Under Hydrostatic Test Conditions.** This load is to be considered if the hydrostatic test condition loads are greater than those considered for the normal operating conditions. All equipment shall be considered to be simultaneously filled with water during hydrostatic testing, unless it can be clearly demonstrated that this will not occur. In such a situation, the most onerous combination of hydrostatic test loads in combination with other simultaneous loads should be considered. Any restrictions on hydrotesting of equipment simultaneously, should be clearly documented and included in both the hook-up and commissioning as well as the operating procedure manuals.

g) **Uniformly Distributed Loads (UDL).** All main and lower deck areas not permanently occupied by equipment shall be loaded by a UDL as determined in Appendix A.

h) **Wind Loads.** The wind loads are the loads due to wind acting on the platform (topsides and superstructures) in the same direction as, and simultaneously with, the wave and current. For the analysis of the substructure a one minute mean wind speed relating to design or operating conditions should be used. Basic wind speeds are to be referenced to +10 metres MSL.

i) **Wave/Current Loads.** The wave/current loads are the total loads due to the wave and current acting on the platform simultaneously in the same direction.

j) **Soft Mooring Loads.** A tender assisted drilling platform shall be designed to accept loads from soft mooring of the drilling tender. A load of 100 s.tons per leg shall be considered acting simultaneously just above the top of jacket bracing on the two legs nearest to the drilling tender (or on the next two legs) in combination with the mooring operating condition.

For other jackets (except vent and bridge support jackets), a load of 50 s.tons per leg shall be considered acting just above the top jacket bracing on any two legs

APPENDIX A.2 (2)

(to be defined at the start of design) in combination with the mooring operating condition.

For minimum tripod structures as defined in Section 2.3.2, a load of 25 s.tons per leg shall be considered acting just above the top of jacket bracing on any two legs in combination with the mooring operating condition.

For monopod structures, a single load of maximum 25 s.tons shall be considered acting on the main substructure element strong point in combination with mooring operating condition.

This limited mooring capacity shall also be highlighted at the platform with adequate signboards.

A specific attachment needs to be provided for tender assisted drilling platforms. On other platforms soft mooring will be wrapped round the jacket legs.

k) **Earthquake Loads.** Earthquake loading shall be considered for seismic active zone as per API latest edition.

l) **Bridge Reactions.** Loads, imposed by the bridge onto the structure including self weight, equipment, piping, UDL loads and wind loads on the bridge, as well as any bridge friction forces.

4.2. Environmental Criteria

General environmental criteria for several regions specified in Appendix L shall be used unless specified otherwise by the Company in the Scope of Work. Throughout this specification the following definitions are used:

a) Normal Operating Case: 1 year return period

b) Mooring Operation Case: annual 90% non-exceedence threshold

c) Extreme / Survival Case: 100 year return period

4.3. Air Gap

The minimum air gap between the underside of the lowest part of the cellar deck and the maximum extreme storm case crest elevation shall be 1.5 m. Provision shall be made for additional air gap due to seabed subsidence. As an example, 0.5m seabed subsidence should result a minimum air gap of 2.0m.

APPENDIX A.2 (3)

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"Alaska Beaufort Sea Gravel Island Design," Exxon Company, U.S.A., 1979.

"Beaufort Sea Summer Oceanographic Measurement Programs," Oceanographic Services, Inc., 1979–1983.

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"Extreme Wave Heights Along the Atlantic Coast of the United States," E. G Ward, D. J. Evans, and J. A. Pompa, Offshore Technology Conference, OTC paper 2846, 1977.

"Characterization of Currents over Chevron Tract #510 off Cape Hatteras, North Carolina," Science Applications, Inc., 1982.

"An Interpretation of Measured Gulf Stream Current Velocities off Cape Hatteras, North Carolina," Evans-Hamilton, Inc., 1982.

"Final Report—Manteo Block 510 Hurricane Hindcast Study," Oceanweather, Inc., 1983.

2.3.5 Ice

In areas where ice is expected to be a consideration in the planning, designing or constructing of fixed offshore platforms, users are referred to API Bulletin 2N: "Planning, Designing, and Constructing Fixed Offshore Platforms in Ice Environments," latest edition.

2.3.6 Earthquake

2.3.6.a General

This section presents guidelines for the design of a platform for earthquake ground motion. Strength requirements are intended to provide a platform which is adequately sized for strength and stiffness to ensure no significant structural damage for the level of earthquake shaking which has a reasonable likelihood of not being exceeded during the life of the structure. The ductility requirements are intended to ensure that the platform has sufficient reserve capacity to prevent its collapse during rare intense earthquake motions, although structural damage may occur.

It should be recognized that these provisions represent the state-of-the-art, and that a structure adequately sized and proportioned for overall stiffness, ductility, and adequate strength at the joints, and which incorporates good detailing and welding practices, is the best assurance of good performance during earthquake shaking.

The guidelines in the following paragraphs of this section are intended to apply to the design of major steel framed structures. Only vibratory ground motion is addressed in this section. Other major concerns such as those identified in Sections 1.3.7 and 1.3.8 (e.g., large soil deformations or instability) should be resolved by special studies.

2.3.6.b Preliminary Considerations

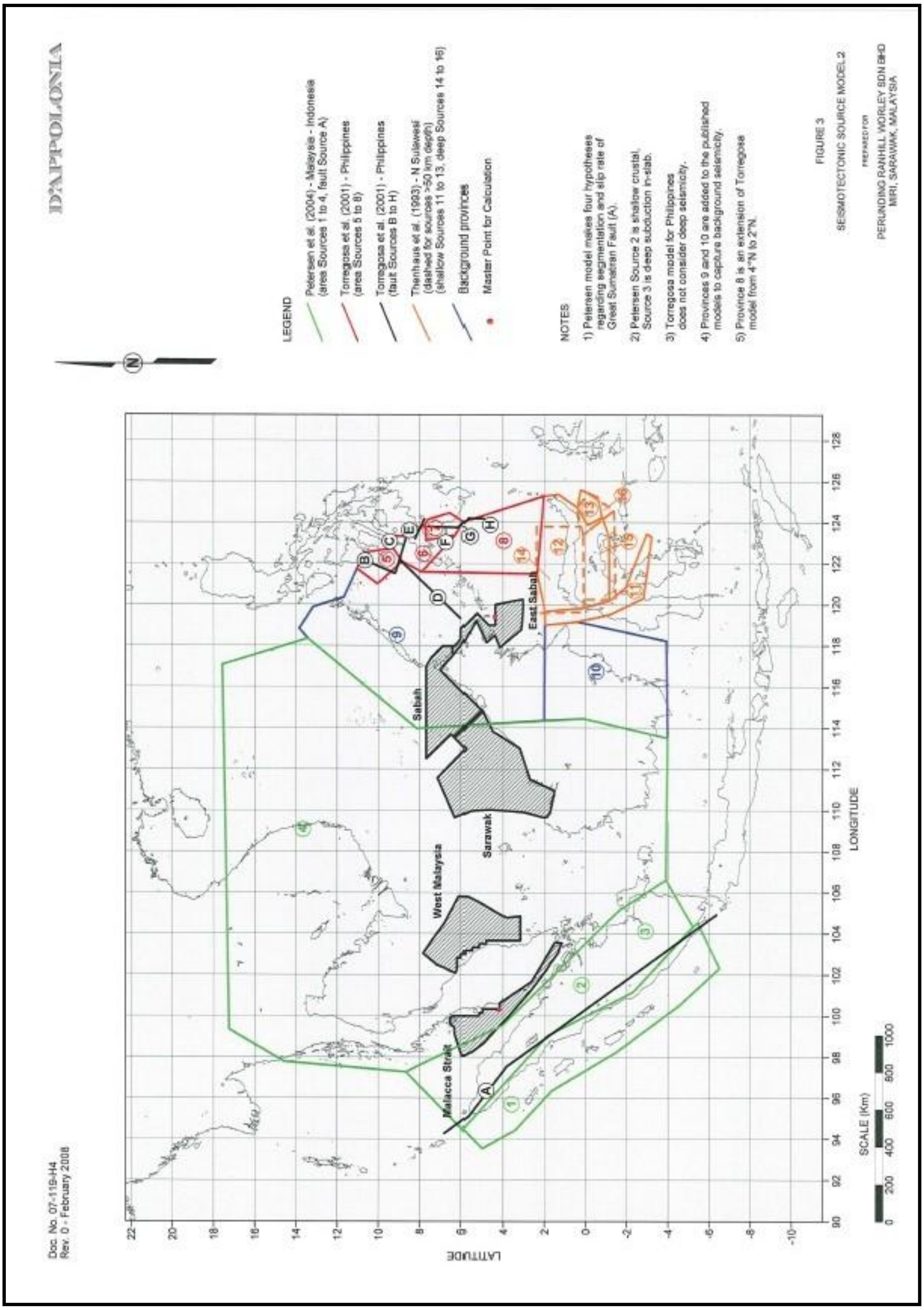
1. **Evaluation of Seismic Activity**. For seismically active areas it is intended that the intensity and characteristics of seismic ground motion used for design be determined by a site specific study. Evaluation of the intensity and characteristics of ground motion should consider the active faults within the region, the type of faulting, the maximum magnitude of earthquake which can be generated by each fault, the regional seismic activity rate, the proximity of the site to the potential source faults, the attenuation of the ground motion between these faults and the platform site, and the soil conditions at the site.

To satisfy the strength requirements a platform should be designed for ground motions having an average recurrence interval determined in accordance with Section 1.5.

The intensity of ground motion which may occur during a rare intense earthquake should be determined in order to decide whether a special analysis is required to meet the ductility requirements. If required, the characteristics of such motion should be determined to provide the criteria for such an analysis.

2. **Evaluation for Zones of Low Seismic Activity**. In areas of low seismic activity, platform design would normally be controlled by storm or other environmental loading rather than earthquake. For areas where the strength level design horizontal ground acceleration is less than 0.05g, e.g., the Gulf of Mexico, no earthquake analysis is required, since the design for environmental loading other than earthquake will provide sufficient resistance against potential effects from seismically active zones. For areas where the strength level design horizontal ground acceleration is in the range of 0.05g to 0.10g, inclusive, all of the earthquake requirements, except those for deck appurtenances, may be considered satisfied if the strength requirements (Section 2.3.6c) are met using the ground motion intensity and characteristics of the rare, intense earthquake in lieu of the strength level earthquake. In this event, the deck appurtenances should be designed for the strength level earthquake in accordance with 2.3.6e2, but the ductility requirements (Section 2.3.6d) are waived, and tubular joints need be designed for allowable stresses specified in

APPENDIX A.3


Doc. No. 07-119-H4
Rev. 0 - February 2008
D'APPOLONIA
N
LEGEND
Petersen et al. (2004) - Malaysia - Indonesia (area Sources 1 to 4, fault Source A)
Torregosa et al. (2001) - Philippines (area Sources 5 to 8)
Torregosa et al. (2001) - Philippines (fault Sources B to H)
Thenhaus et al. (1993) - N Sulawesi (dashed for sources >50 km depth) (shallow Sources 11 to 13, deep Sources 14 to 16)
Background provinces
Master Point for Calculation
NOTES
1) Petersen model makes four hypotheses regarding segmentation and slip rate of Great Sumatran Fault (A).
2) Petersen Source 2 is shallow crustal, Source 3 is deep subduction in-slab.
3) Torregosa model for Philippines does not consider deep seismicity.
4) Provinces 9 and 10 are added to the published models to capture background seismicity.
5) Province 8 is an extension of Torregosa model from 4°N to 2°N.
Malacca Strait
West Malaysia
Sarawak
Sabah
East Sabah
LATITUDE
LONGITUDE
SCALE (Km)
0 200 400 600 800 1000
FIGURE 3
SEISMOTECTONIC SOURCE MODEL 2
PREPARED FOR
PERUNDING RANHILL WORLEY SDN BHD
MIRI, SARAWAK, MALAYSIA

APPENDIX A.4: Tabulation of joint displacement when subjected to As-Designed metocean criteria.

No	Joint	EL (+) m	As-Designed			
			x	y	z	xy
1	CD71	15.545	18.4386	1.8546	-0.1438	18.53164
2	C001	13.000	18.5556	1.583	-0.582	18.623
3	CS04	12.500	18.5812	1.5286	-0.5682	18.64397
4	C004	12.250	18.5917	1.5023	-0.581	18.6523
5	CD62	11.430	18.6282	1.4154	-0.1435	18.68189
6	C005	9.250	18.7219	1.1855	-0.5767	18.7594
7	CS13	6.858	18.8059	0.9412	-0.5676	18.82944
8	C006	6.350	18.8199	0.891	-0.5723	18.84098
9	C002	6.000	18.8295	0.8565	-0.5718	18.84897
10	CS12	3.810	18.8521	0.6556	-0.556	18.8635
11	CS30	2.727	18.8433	0.5648	-0.5497	18.85176
12	CS32	1.219	18.8098	0.4478	-0.5411	18.81513
13	CS33	0.000	18.773	0.3595	-0.5343	18.77644
14	CS34	-1.219	18.7249	0.2774	-0.5273	18.72695
15	CS26	-2.950	18.6272	0.172	-0.5177	18.62799
16	CS25	-3.200	18.6134	0.1579	-0.5163	18.61407
17	CS27	-3.864	18.5763	0.1214	-0.5133	18.5767
18	CS24	-7.737	18.3436	-0.0632	-0.496	18.34371
19	CS11	-14.478	17.6644	-0.2794	-0.4642	17.66661
20	CS10	-17.221	17.2497	-0.3354	-0.4519	17.25296
21	CS23	-19.585	16.8322	-0.3788	-0.4419	16.83646
22	CS09	-23.317	16.062	-0.4196	-0.4257	16.06748
23	CS08	-29.413	14.5205	-0.457	-0.3948	14.52769
24	CS22	-32.737	13.5369	-0.459	-0.3751	13.54468
25	CS07	-35.509	12.6454	-0.4515	-0.3583	12.65346
26	CD61	-36.119	12.4411	-0.4488	-0.0979	12.44919
27	CS21	-45.585	8.9608	-0.3638	-0.2848	8.968182
28	CS06	-56.845	4.5664	-0.2018	-0.1984	4.570857
29	CS20	-60.238	3.3498	-0.151	-0.1663	3.353202
30	CD60	-72.695	0.2847	-0.0136	-0.0131	0.285025
31	CD02	-77.140	0	0	0	0

APPENDIX A.5: Tabulation of Unity Check when subjected to As-Designed, PTS and Joint Density metocean criteria.

Joint	Elevation (+) m	Height from Mudline (m)	AD	PTS	JD
CD71	15.545	92.659	0.000	0.000	0.000
C001	13.000	90.114	0.000	0.001	0.000
CS46	11.430	88.544	0.000	0.001	0.000
C005	9.25	86.364	0.001	0.000	0.000
C006	6.350	83.464	0.001	0.000	0.000
C002	6.000	83.114	0.001	0.000	0.000
C003	5.500	82.614	0.001	0.000	0.000
CS12	3.810	80.924	0.006	0.000	0.000
CS30	2.727	79.841	0.009	0.000	0.000
CS32	1.219	78.333	0.010	0.000	0.000
CS34	-1.219	75.895	0.010	0.010	0.008
CS26	-2.950	74.164	0.069	0.040	0.020
CS25	-3.200	73.914	0.043	0.020	0.010
CS27	-3.864	73.250	0.028	0.010	0.000
CS27	-3.864	73.250	0.024	0.010	0.010
CS11	-14.478	62.636	0.060	0.050	0.010
CS23	-19.585	57.529	0.064	0.050	0.010
CS09	-23.317	53.797	0.092	0.070	0.020
CS08	-29.413	47.701	0.091	0.070	0.020
CS22	-32.737	44.377	0.079	0.070	0.010
CS07	-35.509	41.605	0.080	0.070	0.010
CS45	-36.119	40.995	0.077	0.070	0.010
CS06	-56.845	20.269	0.079	0.040	0.020
CS20	-60.238	16.876	0.137	0.080	0.030
CS44	-72.695	4.419	0.293	0.200	0.060
CS05	-77.114	0.000	0.349	0.240	0.070

APPENDIX A.6 Tabulation of joint displacement when subjected seismic ground acceleration PGA 0.114g

No	Joint	EL (+) cm	PGA 0.114			
			Joint Displacement (cm)			
			x	y	z	xy
1	CD71	15.545	9.905	9.107	0.000	13.455
2	C001	13.000	9.891	9.182	0.004	13.496
3	CS04	12.500	9.889	9.197	0.004	13.505
4	C004	12.250	9.888	9.204	0.004	13.509
5	CD62	11.430	9.885	9.230	0.000	13.524
6	C005	9.250	9.877	9.298	0.004	13.565
7	CS13	6.858	9.876	9.380	0.004	13.621
8	C006	6.350	9.877	9.398	0.004	13.634
9	C002	6.000	9.878	9.411	0.004	13.643
10	CS12	3.810	9.893	9.500	0.004	13.716
11	CS30	2.727	9.909	9.552	0.004	13.763
12	CS32	1.219	9.944	9.636	0.004	13.847
13	CS33	0.000	9.980	9.713	0.004	13.926
14	CS34	-1.219	10.026	9.800	0.004	14.020
15	CS26	-2.950	10.116	9.942	0.004	14.184
16	CS25	-3.200	10.133	9.966	0.004	14.213
17	CS27	-3.864	10.177	10.029	0.004	14.288
18	CS24	-7.737	10.500	10.451	0.003	14.815
19	CS11	-14.478	11.080	11.159	0.003	15.725
20	CS10	-17.221	11.248	11.365	0.003	15.990
21	CS23	-19.585	11.345	11.490	0.003	16.147
22	CS09	-23.317	11.376	11.557	0.003	16.217
23	CS08	-29.413	11.101	11.321	0.003	15.856
24	CS22	-32.737	10.773	11.004	0.002	15.400
25	CS07	-35.509	10.399	10.633	0.002	14.873
26	CD61	-36.119	10.304	10.538	0.000	14.738
27	CS21	-45.585	8.209	8.209	0.002	11.609
28	CS06	-56.845	4.524	4.646	0.001	6.485
29	CS20	-60.238	3.380	3.473	0.001	4.846
30	CD60	-72.695	0.303	0.311	0.000	0.434
31	CD02	-77.140	0.000	0.000	0.000	0.000