REDEFINING THE DESIGN OF TARPON MONOPODS FOR MARGINAL FIELDS

by

Lee Hsiu Eik
Civil Engineering
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Supervised by, Associate Professor Ir. Dr. Mohd Shahir Liew
Internal Examiner: Professor Dr. Kurian Velluruzhathil John
External Examiner: Mr. Salleh Mohd Hanapiah

UNIVERSITI TEKNOLOGI PETRONAS

Bandar Seri Iskandar

31750 Tronoh

Perak Darul Ridzuan

CERTIFICATION OF APPROVAL

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A project dissertation submitted to the

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Approved by,

____

(Assoc. Prof. Ir. Dr. Mohd Shahir Liew)

Universiti Teknologi PETRONAS

Bandar Seri Iskandar

31750 Tronoh

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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.

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(Lee Hsiu Eik)

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ABSTRACT

In the race to produce from a marginal field with a greater return on investment, technological innovations such as the minimal platform concept were introduced like that of the Tarpon monopod. PETRONAS currently owns six Tarpons, all which are installed in Malaysian waters. There is, hence, a need to assess the characteristics of the Tarpons' structural system. A single platform is chosen to represent the fleet of Tarpon Monopods owned by PETRONAS. This study envelops a simulation approach that will effectively evaluate four sets of different environmental criteria; PETRONAS Technical Standards (PTS) 34.19.10.30, Offshore Engineering Center UTP (OECU) Joint Density (T = 8 sec , T = 6 sec) and metocean criteria for the As Designed Worst Condition. The platform is modelled in SACS 5.3 suit of programs for its intact and damaged conditions by varying its guying system and soil foundation characteristics. For each scenario, a static in-place analysis with pile soil interaction is conducted to plot the caisson's deflection and unity checks alongside their respective interpretation and take aways. The static analysis is complemented by Dynamic Amplification Factors obtained from the analysis of SACS Dynpac and Wave Response. A comparison is made against the platform's ultimate strength obtained via the SACS Collapse module. The results show that the Tarpon is relatively insensitive to the soil beneath it in its intact condition. As expected, the as designed metocean induces the largest deflection of the caisson. The Tarpon's integrity is highly sensitive to its guying condition – even failure of one of the three sets of guy cables may induce failure in unfortunate environmental conditions. The Tarpon monopod (in water depth 70m-80m) is not a very robust structure with its initiating mode of failure coming from its anchor piles.

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CHAPTER 1

INTRODUCTION

1.1 Background : Overview of Tarpon Monopod

More than often, smaller oil and gas fields would be deemed marginally economic, should it be developed with conventional offshore technologies like that of multi leg space frame platforms or floating systems. Such discoveries are usually left untapped until a good mix of high oil prices, innovative technologies and revamped company policies eventually justify their economic viability. The Tarpon Monopod, also known as the cable guyed caisson, is one of the many innovative minimal platform designs used in developing marginal fields. Generically, the platform consists of a main caisson guyed with three sets of cables to anchor piles secured at the sea bed. There are currently more than 56 Tarpon platforms in use worldwide, with the bulk growing from a meagre 37 back in the late 90s **(Oil and Gas Journal, 1999).** The platform, which consists of a minimum superstructure supported on a single main caisson guyed to three symmetrical pre tensioned cables, has been installed worldwide in water depths ranging from 60ft up to 350ft **(Tarpon Systems, 2012)**. An example is as depicted in Figure 1.

Figure 1 A Monopod platform in Cook Inlet (Source: Google Images http://www.oilprice.com/uploads/AC863.png)

To date in Malaysian waters, PETRONAS Carigali (PCSB) operates six Tarpon structures both in Peninsular Malaysia and Sabah. 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.

1.2 Problem Statement

In June 2011, the Tarpon structures in both Peninsular Malaysia Operations (PMO) and Sabah Borneo Operation (SBO) waters were labelled ‘red’ (very high risk) under PETRONAS Management Team (PMT) / PETRONAS Carigali Sdn. Bhd’s Structural Health Cockpit Traffic Light System. The cause of the alarm is that of the unavailability of structural models and lack of proper Tarpon-specific inspection guidelines. As a result, the robustness of the structure and its response to the degradation of any Safety Critical Elements (SCE) cannot be ascertained. (GLND,2010).

Very recently, a routine underwater inspection conducted for one of the Tarpon Platforms operated by PETRONAS Carigali revealed that one of the three sets of guy cables was completely severed, leaving the single caisson supporting the superstructure to be only guyed to the two remaining sets of cables. The platform, however, despite the failure of one of its guy cable set, was observed to be functioning normally. In fact, it was only when alerted by the inspection team, did the issue became known – otherwise it would have probably gone unnoticed under normal operating conditions by personnel on the decks. (via interview with A.P Dr. Ir. Shahir Liew).

This suggests that the structure might have some degree of redundancy or alternate load pathways or perhaps, the environmental loading has somewhat been in the lower bounds. Nonetheless, the author infers here that there is simply too much uncertainty on the sensitivity and response of the Tarpon to the loadings imposed on it. As such, this study is themed on the assessment of the structural response/performance of the

Tarpon Monopod under defined simulated conditions that which will be discussed further, later in this report, in order to shed some light on its robustness.

1.3 Objectives

The primary aim of this study is to perform a computer-based simulation assessment on the structural response of the Tarpon Monopod, using a single Tarpon platform to represent the entire fleet under PCSB, in intact or damaged conditions when subjected to four different metocean criteria which are extracted respectively from PETRONAS Technical Standards (PTS) 34.19.10.30, Offshore Engineering Center UTP (OECU) Joint Density Parameters (T = 8 sec , T = 6 sec) and Metocean Criteria for FEED at Pulai-Anoa-Ledang. The seabed soil foundation conditions are also modelled to be intact or degraded and are added to the mix of scenarios described later in this report.

To complement the latter, the second objective is thus to assess the structural responses evoked by each of the different scenario models with a compare and contrast approach mainly reliant on the main structural caisson deflection with its unity checks and the corresponding reserve strengths. Herein, the Tarpon platform can be evaluated for various conditions to determine its structural sensitivity and robustness.

A third objective is to structure the report in a way that makes it a general approach/guide that can be used to perform similar assessments on Tarpons in similar sea states, hence justifying the notion of using a single platform to represent the entire family of Tarpons under PCSB. In this context, it is the hope of the author that this study may be of use for the management team for better informed decision making.

1.4 Scope Of Study: Platform Selection and Software

For obvious reasons, like that of complete data availability, a single Tarpon - the Ledang Platform (LDP-A) – selected from PCSB's fleet of guyed caissons, will be used as the model for this project, in effect, acting as the sample representing the group of Tarpon Monopods operated by PCSB. Since the Tarpon design is very repeatable and standardized in nature, the latter assumption is seemingly justified.

The entire range of computer simulations is performed using SACS 5.3 suite of software. The uncompleted model of the platform is made available to the author by which certain modifications, redefinitions and additions of primary and secondary members were performed to reflect as accurately as possible, the correct global stiffness of the structure. The scope of analysis as of this report covers from the Linear Static in Place (with pile soil interaction) analysis and the Non Linear Collapse analysis to a 1^{st} level dynamic approximation analysis study.

The dynamic analyses will be conducted via SACS Dynpac and Wave Response Programs with a goal to compute the Dynamic Amplification Factors which will then be factored into the seastate models. The goal here is not to provide the reader with a detailed insight into the dynamic sensitivity of the Tarpon; the author recommends this to be done as an exclusive study to itself. Instead, the DAF approximations function to ensure a conservative result, rather than ignoring the dynamic effects entirely.

1.5 Feasibility & Relevancy

This project addresses the pressing issue of a need for structural sensitivity and response studies for the Tarpon platforms owned by PCSB in both PMO and SBO waters by providing an insight into the robustness of the Tarpon design with regards to differing metocean criteria, intact/damaged conditions and soil foundation characteristics (intact/degraded). The author then appropriately infers this to deem the project as industrially relevant.

As for the time basis, the author reports that the project is progressing as planned and although there were several hiccups along the way thus far, the project will be able to be completed as scheduled or at best, earlier.

CHAPTER 2

LITERATURE REVIEW

This chapter encompasses a succinctly comprehensive review of the key concepts and terms which are crucial to gain a sound grasp on the jest of this project. These terms can be readily abstracted from the project theme – Minimal platforms, Tarpon Monopods (generic), marginal field, design of Tarpon Monopods and assessment of Tarpon Monopods. The platform data pertaining to this study alongside relevant literatures are also briefly reviewed towards the end of this section.

2.1 The Minimal Platform Concept – A worldwide perspective

Subrata K. Chakrabarti (2005) , in the publication- *Handbook of Offshore Engineering Vol. 1*- defined minimal platforms as fixed production platforms with a small deck used for the development of marginal fields in shallow water. The minimum configurations for such platforms include typically less than ten wells, a small deck where it is possible to accommodate a coil tubing or wire line unit, a test separator and well header, a small crane, a boat landing and in some cases a minimum helideck.

Dunn et. al (2009) published a study on the use of minimal platforms in the hostile waters of the Nova Scotian Offshore (NSO), eastern Canada. The paper took into consideration, three minimum platform designs namely caisson type, tripod type and jack up structure type. All three designs would not require the use of a heavy lift vessel for installation. The conclusion of this study revealed that the design of the single caisson and tripod type can be done in a way that would meet the minimal structural definitions whilst providing excellent production and structural capacity, all delivered with potential cost savings as compared to past conventional developments in the NSO region. The self-elevating jack up concept was also shown to be suitable for NSO's harsh environment. In short, all three concepts under scrutiny in the case study prove to be worthwhile of serious considerations for developers that are eyeing the marginal fields in the NSO region (as paraphrased from **Buacharoen , 2010**). Figure 2 depicts two of the platforms that were studied.

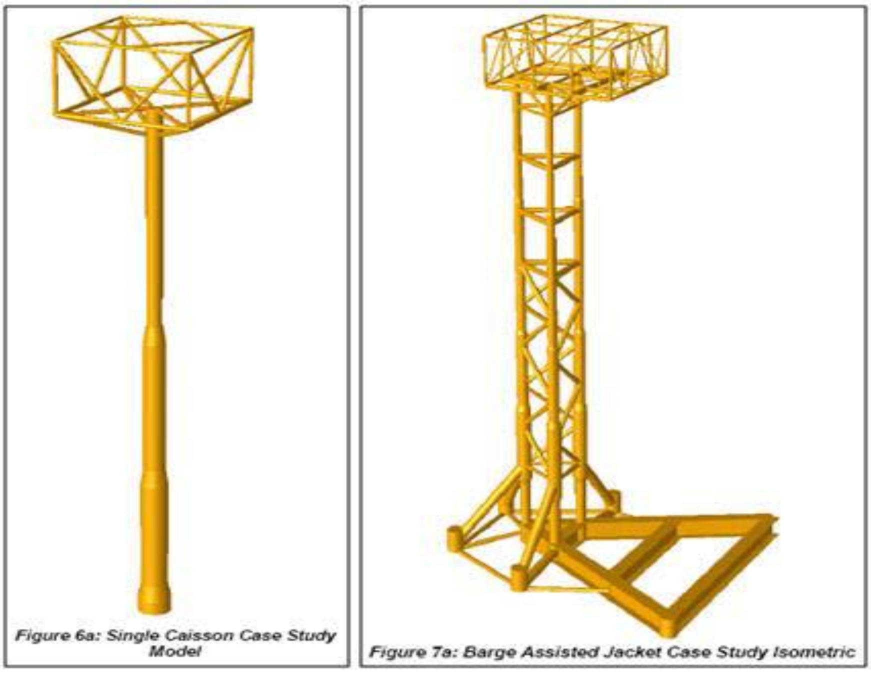

Figure 6a: Single Caisson Case Study Model

Figure 7a: Barge Assisted Jacket Case Study Isometric

Figure 2 Minimal platform concepts: Single Caisson and Tripod type
Source: Dunn et. all (as cited from Buacharoen , 2010)

2.2 Introduction to Tarpon Monopods

The tarpon monopod is actually, in its physical sense, a cable-guyed caisson minimal production platform. As of the year 1999, there were 37 of such platforms operating in the Gulf of Mexico, West Aftica and Indonesia. It was first used back in 1987 with Stolt Comex Seaway as the owner of the patents for the system.

Fast forward to more recent times, there are now more than 56 installations worldwide and they can be designed for water depths of 60ft to 350ft (**Tarpon Systems, 2012**).

The major substructure of the Tarpon concept is made up of a central caisson, capable of housing multiple wells internally or even externally via conductor clamps. This caisson is stabilized by three cable guys at 120 degrees apart. Each set of guy cables consist of two wire ropes with one end pinned to the anchor pile at or below the mud line and the other, pinned to the caisson below the water line. Generically, the anchor cables would be engineered to form a 35 degree angle from the mudline hence, giving the subsequent approximate horizontal distance of the anchor piles from the caisson to be 170 % of the water depth **(Oil and Gas Journal, 1999).**

Tarpon Systems (2012) lists the life cycle cost advantages of a Tarpon system to be; Low capital expenditure, simple construction, ease of installation, early production capability, low abandonment cost, recoverable and reusable components.

2.3 Tarpon Monopod Design (Oil and Gas Journal, 1999)

By its design, the Tarpon Monopod is a quasi-compliant structure. The response and deflections of the structure to loadings are highly dependent on the cable tension of the guy system and the deck mass. As the pretension load in the guys is increased, the cable spring system would have the tendency to exhibit more linear properties which would effectively lower the natural period of the platform. This in turn will incur benefits like that of smaller deflection and hence, better fatigue life. This increased functionality comes with a price, however; it would mean larger cables, larger diameter and longer anchor piles. The amount of pretension in the cables would be decided on the grounds of an optimum balance between fatigue life and human response to motion.

Existing guyed caisson platforms have documented natural periods in the range of 2 seconds to 3.5 seconds with an extreme outlier where a period of 4.2 seconds was measured for an installation in 218ft of water with a deck load of 350 Short Tons. As compared to braced systems, the guyed caisson is capable of handling larger lateral loads, credited to the relatively wide spread design nature of the anchor piles. This will prove advantageous for the Tarpon as it would have greater reserve strength than that of the braced caisson which subsequently reduces the cost for water depths greater than 120ft. Whilst in water depths less than 120ft, the Tarpon geometry enables a full 360 degree boat access; this, however, is not the case for the braced caisson or the tripod alternate designs.

Cables are usually designed to approximately 50 % of the nominal breaking strength, using only one of the two cables in the pair. The repetitive design parameters inherent in the concept of the guyed caisson minimum platform has enabled a certain standardization to be achieved, leading to shorter structural design times and lower end cost. By the recommended practices of API RP 2A –WSD, the combined stress unity ratios for the caisson are limited to 0.9 – 0.1 short to unity – or less, it being in the scope of minimal structures

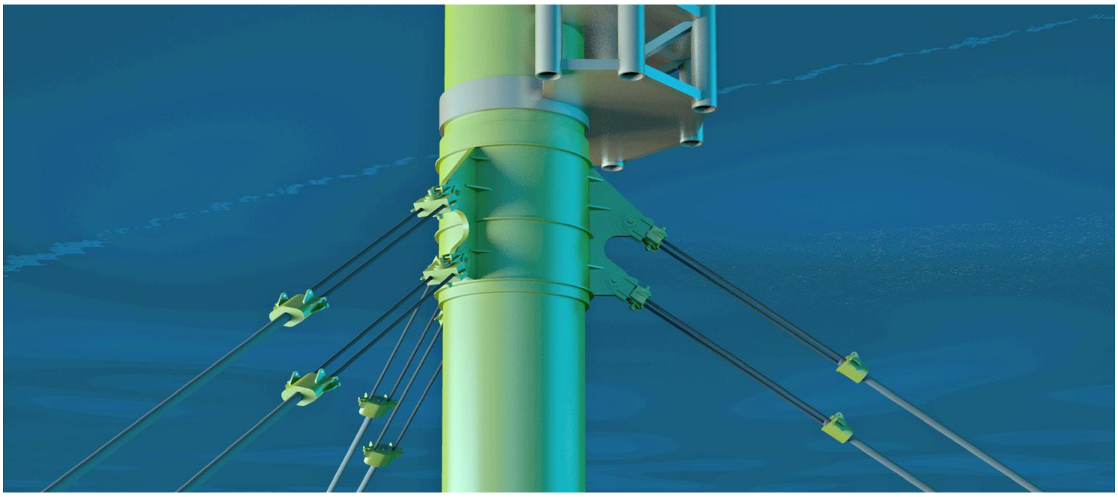

Figure 3 The Termination Clamp/Sleeve with pad eyes located below the water level
(Source: Tarpon Systems, 2012)

The guy cables are connected to the termination clamp on the caisson by means of a pin connection to a pad eye on the clamp or sleeve. 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.

2.4 The Basic Components of a Tarpon Monopod

The functions of the structural elements as shown in Fig. 2.4 on the next page are briefly summarized below **(Syamsul, 2012).**

- 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.

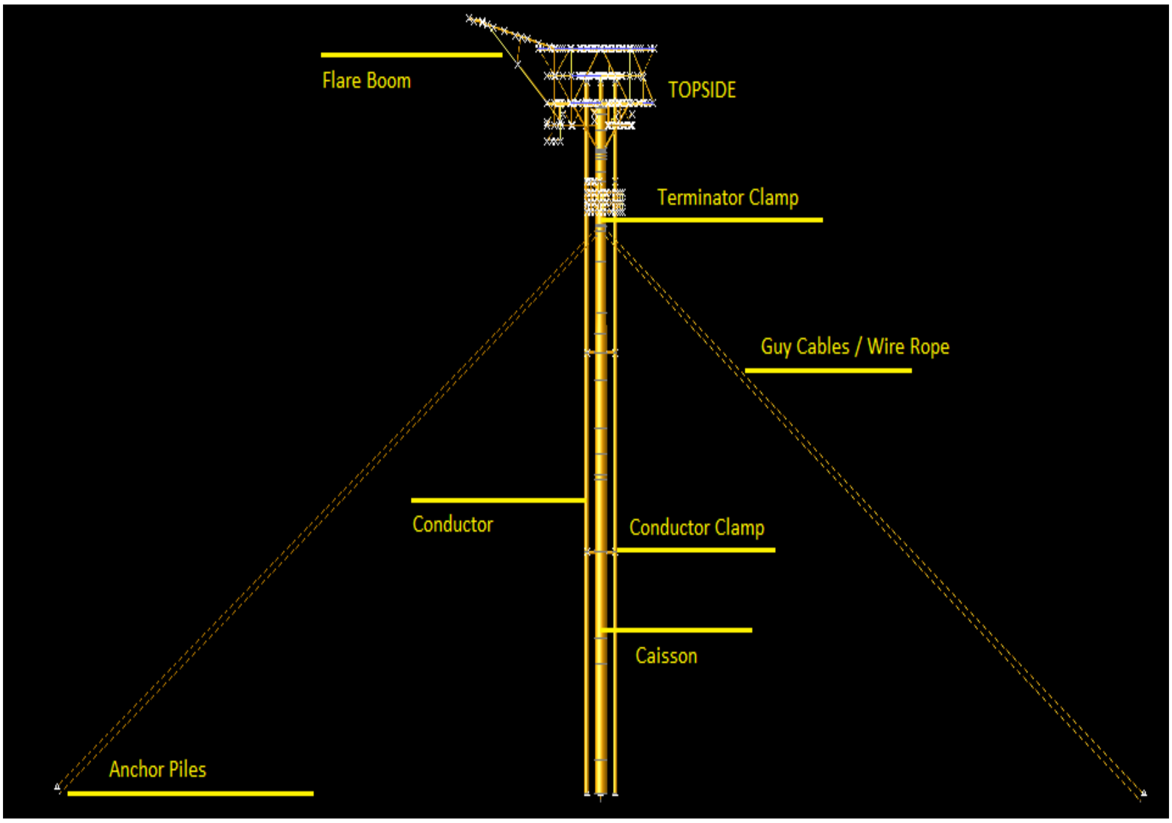

Figure 4 Basic structural components of the Tarpon Monopod
(Source: as modelled in SACS 5.3)

2.5 An Overview of Marginal Fields

Abdelazim's (2008) study listed several definitions of marginal fields, with the most relevant definition as shown below(direct citation):-

- "A marginal field is a limited reserve that may not produce enough net income or a minimum required return on investment, to make it worth developing at a given time; should technical and economic conditions change, such a field may become commercial. Marginal offshore fields may contain small recoverable reserves in shallow water (i.e. up to maximum 100 meter water depth) or relatively large reserves in deep water (i.e. more than 500 meter water depth), where higher investments are necessary to exploit the field." **(p. 3)**

2.6 Ultimate and Reserve Strength

In more recent times, the reserve strength of a platform is defined as the ability of a structure to take loads that are greater than its design value. Should a particular member fail, the event would not limit the overall structural capacity to take more loads. This is because, at a global structural level, given sufficient ductility and redundancy, loads can be redistributed in the event of a local failure. In structures with high redundancy, several components may fail in sequence before the ultimate strength is achieved. Among the limitations of elastic design is that the capacities are defined by the calculated occurrence of first component failure. **(Bolt H M , C J Bilington & J K Ward, 1996)**

The Reserve Strength Ratio (RSR) as defined by **Titus and Banon (1988)**, **(cited from Bolt et. al, 1996)** is as below;

$$RSR = \frac{Ultimate\ platform\ resistance}{Design\ Load}$$

In their publication, **Bolt et. al (1996)** also interestingly defined the terminology of the Residual Resistance Factor (RIF) to be;

$$RIF = \frac{Damaged\ structural\ capacity}{Ultimate\ capacity}$$

2.7 Dynamic Amplification Factors for Fixed Platforms

Shehab Mourad, Mohamed Fayed, Mostafa Zidan and Mohamed Harb (2005), noted in their publication that a direct dynamic analysis on offshore structures would be difficult especially due to the non-linearity of waves. A method herein is to perform the static analysis with Dynamic Amplification Factors (DAF) applied to the static wave forces which makes account for the dynamic interactions. The normal in place static analysis allows the use of nonlinear wave theories and nonlinear foundation effects. Two methods were employed in calculating the DAFs ; one by taking the ratio of dynamic and static overturning moments and the other by the approximation formula typically used in practice when the jacket's first natural period is less than 2.5s. The results suggest that for jackets with relatively simple configurations, the approximate equation underestimates the DAF values by up to 15% and for more complex jacket configurations, the underestimation percentile reached 35%.

2.8 Ledang Platform (LDP-A) Characteristic & Design Data

Syamsul (2012), noted in his dissertation that the guy cables used for the LDP-A model have an effective area of 4894 mm^2 / cable and an effective diameter of 4.395" with an elastic modulus of 14 000 ksi. In the study, it is further stated that three pairs of post tensioned cables are used to guy the central caisson (2133.6 mm & 1828.8 mm in diameter) to 1828.8 mm diameter anchor piles on the sea bed, located symmetrically around the caisson at 120 degrees apart. The key platform characteristics, selected in relevance to this study are extracted from PCSB's Structural Information Computer System and summarized in Table 2.1.

Table 1 Relevant key data for LDP-A
(source : Syamsul (2012))

Platform Details	**LDP-A data**
Field	PM9
Platform Type	Monopod Platform
Manned/Unmanned/Quarters	Unmanned , No quarters
Operator , year installed	PETRONAS , 2006
Operational Status	Active
Water Depth	76.2 m
Jacket Height	82.2 m
Air Gap	1.5 m
Deck Elevation	9.8 m
Number of legs	1
Number of Piles	3
Maximum Leg Diameter	1981.2 mm
Deck Weight	184.8 MT
Jacket Weight	800 MT
Pile Weight	150.34 MT
Shore Distance	200km
Number of slots	3
Number of Caissons	1
Number of Conductors	3
Number of Risers	1

Number of Decks	3
Number of Cranes	1
Maximum Conductor Diameter	0.762m
Maximum Crane size	3 MT
Boat landing	1
Helipad	0
Design Code	API RP 2A 21st
Design Service	D
Design Life	20 years
Design Return Period	100 years
Design Marine Growth	0.153 m
Design Scour	0.9 m

2.9 Ledang Platform Substructure Design Basis (ECL, 2008)

The LDP-A guyed caisson substructure design was performed by ECL and documented in their report. The topside design, done by Perunding Ranhill Worley will not be covered in this brief literature review. Located in a depth of approximately 76m, the tapered caisson has diameters (external / internal) of 84" x 72". Three pairs of EIPS-IWRC 6 × 61 class - 4" diameter, post tensioned wire ropes are used symmetrically around the caisson to guy the it to three 72" anchor piles on the sea bed, placed in a radius of approximately 357 feet from the caisson.

The SACS software package was used to perform the analysis on the guyed caisson. Several codes were used in the design namely- API RP 2A, AISC-ASD, and PTS 20.073 whereby, under consent from Petronas Carigali, PTS standards will take precedence over the other two codes. The analysis performed covered the in place, dynamic, spectral fatigue, caisson transport, caisson and pile lift analyses. The engineering design data used in the design was provided by Petronas Carigali.

The in place analysis was performed to extreme 100 year and 1 year return period environmental event conditions respectively , besides modelling the structure to nominal operating conditions. The worst storm approach direction was chosen to simulate the maximum load in a single cable, with dynamic amplification factors and cable pre tension taken into consideration with a one third increase in allowables. No increase in the allowables was used for the 1 year storm case. The worst case of boat impact/mooring conditions was also simulated and analysed.

The dynamic analysis of the platform revealed the natural frequency of the structure to be 3.2 seconds. Since the latter value is larger than 3 seconds, SACS wave response program was used to obtain the dynamic amplification factors for the wave loadings. These revised values were then applied to the final analysis for conservative results. The model used in the dynamic analysis is then subsequently analysed for fatigue. The calculated minimum fatigue life for the caisson substructure is 583 years, providing ample of safety factor over the design life of 20 years.

3.0 Previous Platform Re-assessment

In order to address the issue of their Tarpon platforms, GL Noble Denton was engaged to undertake in the structural re-assessment works. For the first part, both in place and dynamic analysis were carried out for the LEDP-A platform in accordance to API RP-2A 21st Edition and AISC ASD via SACS suit of programs. The analysis is performed under static loading conditions with a linear elastic response in a mean sea level of 77.11m. Also included is the calculation of the Dynamic Amplification factors and their inclusion into the analysis based on the appropriate dynamic SACS modules and user input. It can be deduced from the results that the Caisson and Conductor substructure are within the 0.8 Unity Check limit with a maximum Caisson UC value of 0.72 and that both the Caisson and the anchor piles are well above the minimum requirements of factor of safety. **(GLND – in place, 2011).**

GLND also performed ultimate strength analyses for the LEDP-A platform by using the USFOS suite of programs. The platform was simulated for in-place ultimate strength analysis in its intact and damaged conditions to determine its RSR against the 100-year storm metocean event. The probability of failure is then calculated based on the calculated RSR and hazard curves as provided by PCSB. By using information from PCSB, the tarpon structure is then risk categorized based on the risk matrix and consequence category provided. **(GLND – ultimate, 2011)**.

CHAPTER 3

METHODOLOGY & PROGRESS

This section houses an elaborate discussion on the means used in performing the study, from how information was sourced to how the project was structured and planned.

3.1 Research Tools

Internet resources**.** The beginning research phase was aimed at conducting a sound study on several key components in the project, such as in place/dynamic analysis whilst sourcing for literature prevalent to Tarpon Monopods. Access to UTP's online subscribed resources via OpenAthens other than materials from Google Scholar played a significant role in allowing the author to perform a concise study.

Conversing with lecturers and seniors**.** To make up for the short comings of the small number of relevant documented materials made available, some parts of the research would be performed by word of mouth, via consultation with lecturers, email threads with past Seniors and chatter with post graduate students/researchers.

Computer Aided Design (CAD), plays a crucial role in the modelling and results generating phase, done with *SACS Executive 5.3* and *Solidworks SP0 2012***.** SACS is primarily used for the modelling and simulation of the platform as a whole, while Solidworks can aid in sketching detailed 3-D engineering drawings where required.

3.2 Project Methodology

This project is broken down into three major sections. The first part is planned as a preparatory stage which gives great emphasis on data collection and familiarization, alongside extensive literature reviews and CAD SACS software training. The second segment would cover completing the existing structural model and the subsequent re-modelling of its in place sea state, foundation, guy cable conditions, followed by the revised model's analysis, all performed through SACS suite of programmes. The third part focusses on the interpretation of results from the second segment, and presenting them in a useful and organized way. This is illustrated in Figure 5 below.

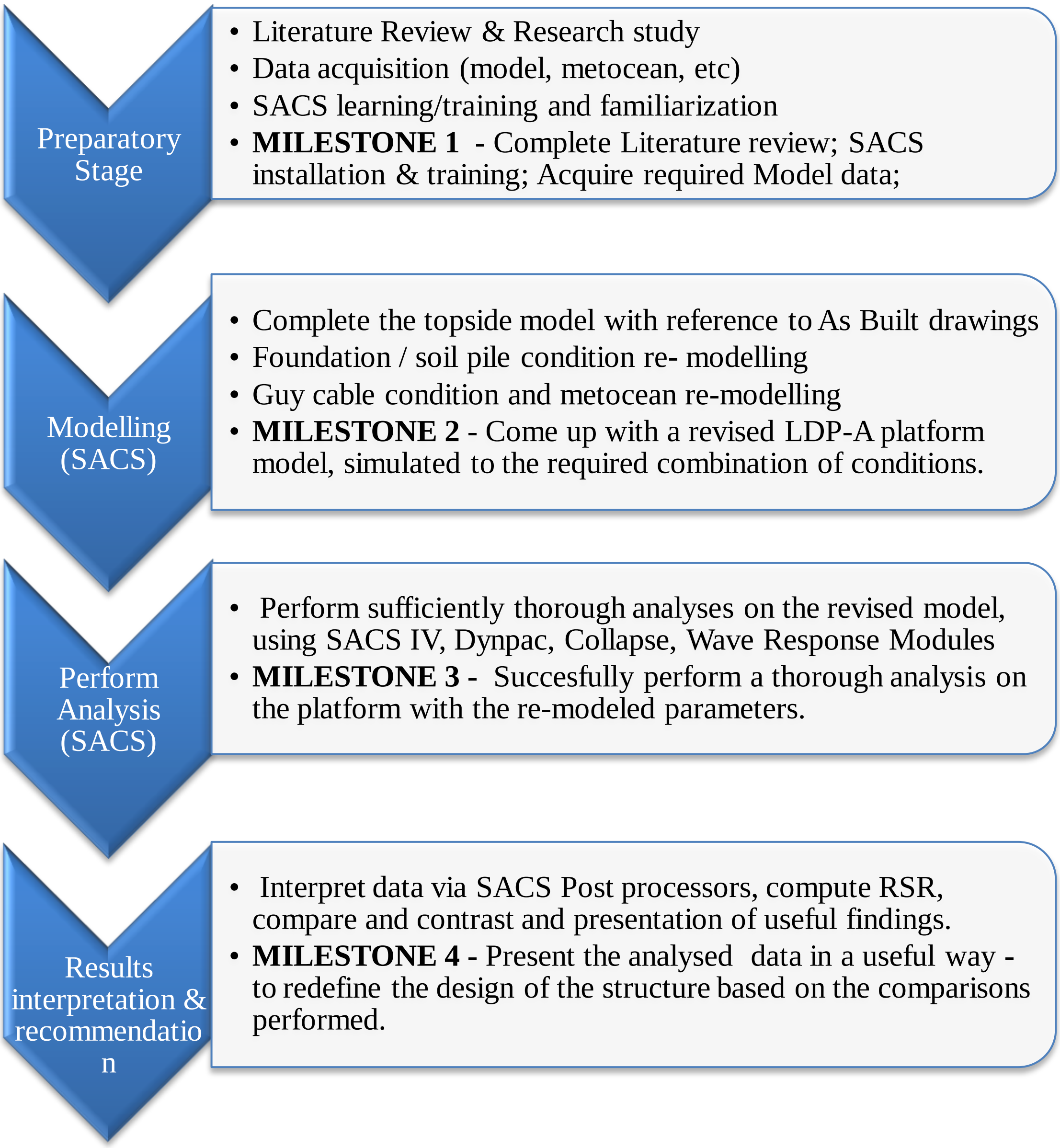

Figure 5 Generic Project Methodology/ Flow with key Milestones

3.2.1 Modelling & Simulation Approach

For the purpose of this project, SACS 5.3 Suite of Programs will be used extensively for both modelling and simulation. Several SACS modules will be used herein. The first is the PRECEDE program, to be used as the graphical user modeller. The actual metocean data acquired from Offshore Engineering Centre UTP, PTS and As Designed will be generated in the SEASTATE program. The PSI module would be used to model the soil-pile interaction. The SACS IV module would be used to process and perform the Linear static analysis coupled with non linear pile soil effects. The COLLAPSE module will be used to perform the Pushover Anlaysis. The results can then be viewed in SACS post processors such as POSTVUE which enables the author to interpret the results interactively and graphically. DYNPAC and Wave Response will be employed to obtain the Dynamic Amplification Factors.

In the scope of this project, the author drafted and adhered to the following steps to obtain a comprehensive model representing the LDP-A as it is built:-

- Compile and review all data pertaining to the Ledang Platform.
- Perform critical in-depth checks on the validity of the available SACS Input data based on the relevant documents.
- Re-develop the linear elastic model where it is incomplete with reference to the as built drawings.
- Model the soil foundation properties for bad soil condition with reference to the original (good) soil condition.
- Model the Seastate in SACS Precede based on the four different Metocean criteria.
- Model the linear elastic model in its damaged and intact condition by varying the number of wire ropes/ cables.
- Perform In Place Static Analysis with Pile Soil Interaction and Collapse Analysis on SACS to determine RSR for each scenario, using assumed or computed (via DYNPAC and Wave Response) Dynamic Amplification factors for the amplification of wave forces in the Static analysis.
- Extract results from SACS and make Excel plots – this will include the structural caisson deflections coupled with unity checks, maximum topside displacement, and useful plots of bending moments for the caisson from the mudline up.

3.2.2 In Place Scenario Definitions (SACS)

One of the key drivers in this project is in the proper definition and combinations of the appropriate scenarios which are to be used in the simulations. The author has summarized them in Table 2 as shown below.

Table 2 Summarized Simulation Scenarios

No.	Metocean Data	Guyed by;	Soil Data	Analysis type
1	As Designed	3 cables	BH-ANOA L1 Intact Soil (Original site Soil Investigation/Borehole results as provided by PCSB)	SACS 5.3 Linear Static In Place with Non Linear Pile Soil Interaction. *(with DAF for seastate)*
	PTS			
	Joint Density(T=8s)			
	Joint Density(T=6s)			
2	As Designed	2 cables		
	PTS			
	Joint Density(T=8s)			
	Joint Density(T=6s)			
3	As Designed	1 cable		
	PTS			
	Joint Density(T=8s)			
	Joint Density(T=6s)			
4	As Designed	Free Standing		
	PTS			
	Joint Density(T=8s)			
	Joint Density(T=6s)			
5	As Designed	3 cables	Modified BH-ANOA L1 (to simulate degraded soil condition)	SACS 5.3 Linear Static In Place with Non Linear Pile Soil Interaction. *(with approx.. DAF for Seastate)*
	PTS			
	Joint Density(T=8s)			
	Joint Density(T=6s)			
6	As Designed	2 cables		
	PTS			
	Joint Density(T=8s)			
	Joint Density(T=6s)			
7	As Designed	1 cable		
	PTS			
	Joint Density(T=8s)			
	Joint Density(T=6s)			
8	As Designed	Free Standing		
	PTS			
	Joint Density(T=8s)			
	Joint Density(T=6s)			
9	Incremental loading from the lowest of the four criteria.	3 cables	BH-ANOA L1	SACS 5.3 Non Linear Static Collapse Analysis
		2 cables		
		1 cable		
		Free Standing		

3.2.3 Dynamic Analysis (SACS)

The natural periods of the Tarpon structure were simulated via SACS Dynpac module. Here, two sets of analyses are performed – one with a fixed base assumption and the other includes structure-soil-pile interaction effects by means of pile foundation super element creation. The tarpon model was simulated via SACS generated mass and the author's choice of load to mass conversion for dynamic eigenvalue analysis. The Caisson and Conductors below the mean sea level are designated as flooded members to account for the added mass from the displaced water column. The dynamic mass system was selected as 'consistent/continuous mass' in contrast to the lumped mass model. A total of 10 modes of vibration were obtained for the fixed assumption while the author opted for 20 modes in the case of the structure-pile-soil interaction. The modes of vibration were then used as input files into the wave response module where the ratio of dynamic mudline moments to the static mudline moments where taken as the approximated DAFs. The DAF values showed several outliers , of which was omitted from the result data. Note that the main goal of performing the DAF approximation is to provide a more realistic / conservative platform response, instead of underestimating the scenarios.

3.2.4 Results Interpretation Approach

The analysis results obtained after successful simulations as stipulated in Table 2 will then be extracted, organized and interpreted as shown in Table 3.

Table 3 The planned result representation outline

Scenario No. (Refer table 2)	Key results description	Remarks
1 to 4	Reacted Base Shear, Maximum topside deflection, Caisson deflection, Critical Members Unity Check,	Compare and contrast the response of the Tarpon platform.
5 to 8		

9	Load at failure, deflection at failure, Ultimate Base Shear, , Ultimate Topside Deflection	Used to determine the Reserve Strength of the structure and its primary mode of failure

3.2.5 Codes and Standards

PETRONAS Technical Standards (PTS) 34.19.10.30 – Design of Fixed Offshore Structures (January 2010, Revision No.6).

API RP 2A 21st Edition – Recommended practice for planning, designing and constructing fixed offshore platforms , Working Stress Design.

ANSI/AISC 360-10 – Specification of Structural Steel Buildings.

3.2.6 Assumptions

The author has defined several important assumptions pertaining to this project, that which will be listed in the proceeding points.

- Simulated for only TWO(2) predominant wave directions. Modified North East – This, by a simplistic force analysis is determined to cause the maximum tension in a set of guy cables. The other approach is from the true South West direction. As the guying system comprise a major part of the Tarpon's Safety Critical Elements, the model for maximising the load in one cable is seemingly justified.

- Wind loading on the platform was not performed via the SACS Seastate program, as no equipments and topside appurtenances were modelled and that this would give a false value for the automatically generated wind area. Hence, wind calculations would be performed manually and modelled as joint loadings.

- For a ‘degraded’ soil model, a 30% reduction in Design Shear Strength for both Clay and Sand soil is used in the modification of the BH-ANOA L1 Soil Investigation Data. Also, a 30% reduction in Unit Skin Friction for Clay and 50% reduction for Soil alongside 30% and 50% reduction in densities of clay and sand respectively. The reduction fractions were chosen primarily without any mathematical formulation, but rather with speculative $1/3^{rd}$ and 1/2 reduction of the intact soil’s key properties.

- No code specific load factors were used (all unity) in the load combinations for wind, live or dead loads, equipment and operational loads EXCEPT where the dynamic effects were taken into account by assuming DAFs computed from the ratio of Dynamic Moment to the Static Moment as generated in SACS Wave Response Module. The DAFs were applied to the static wave models.

- A full Dynamic analysis will be omitted. The author, however, does not dismiss the notion of including Dynamic effects as part of this project, and would include it for a better representation of the actual response of the structure, by the use of a linear static analysis whose dynamically categorized loads are factored with Dynamic Amplification Factors. Therein, the author assumes that the factored static analysis (in place with nonlinear PSI) provides sufficient accuracy in redefining the design of the Tarpon Monopod.

- Cable pretension is modeled in SACS via temperature loading and this has been proven to work as calculated in the Appendix. Structural integrity is generally defined from the mudline up, specifically on the Tarpon’s main structural member – its main caisson. Detailed studies on the piles will not be included.

3.2.7 Project Activities

A Gantt Chart detailing the major activities expected throughout the life cycle of this Final Year Project is as illustrated in Figure 6.

The author wishes to highlight that all the Key Milestones as seen in Figure 5, have been successfully achieved, and the project has matured and is now comple. The main deliverables is to prepare useful interpretations for the results (graphs, plots, tables, etc..) and present them in a meaningful manner that which is succinct accomplished in the results chapter.

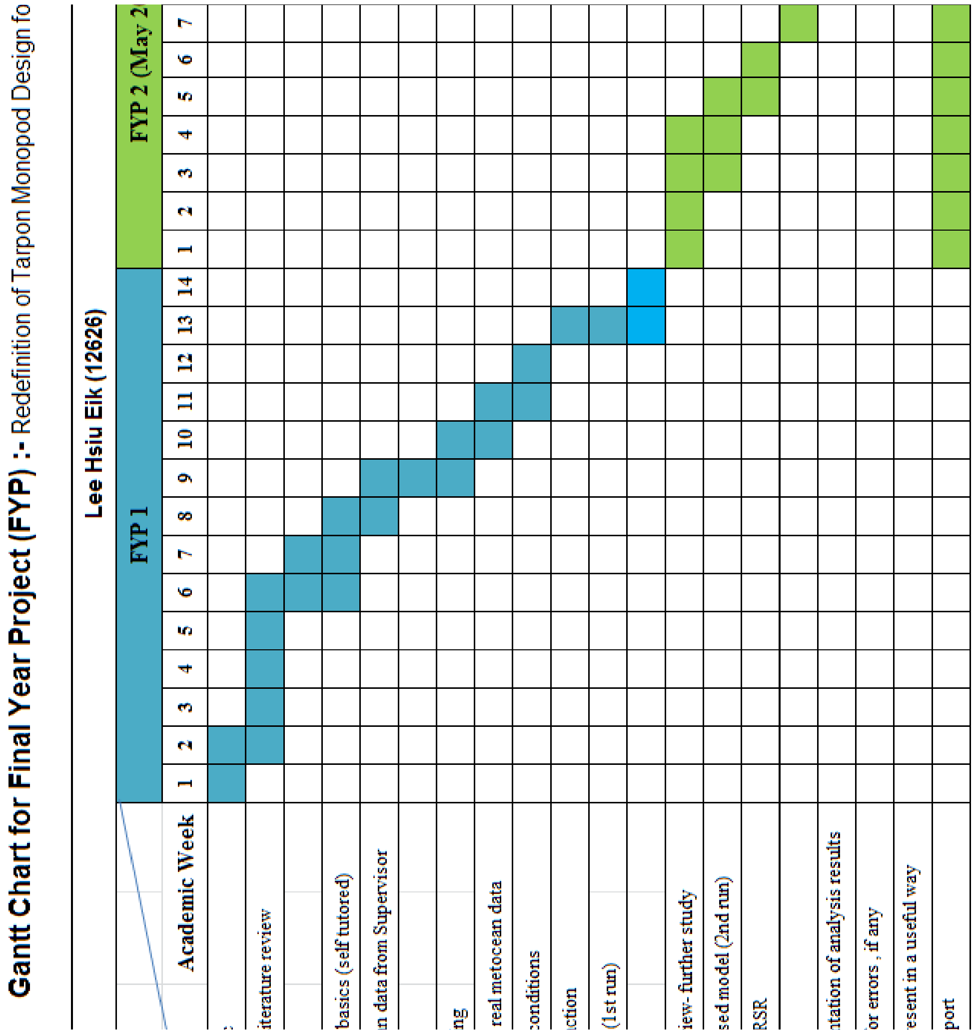

Figure 6 FYP Gantt Char\

3.3 In-Place Data

3.3.1 General

The design water depth of LDP-A platform will be as seen in the Soil Investigation Report (BH-ANOA L1) with additional tide and storm surge data from the FEED at Pulai-Anoa-Ledang (DCE/MET/ANOA/2005). Table 4 compiles this.

Table 4 Design water depth

Description	Min	Max
Mean Sea Level, MSL(m)	76.3	76.3

Highest Astronomical Tide (m)	Not applicable	1.06
Lowest Astronomical Tide (m)	-1.13	Not applicable
Storm Surge (100 year) (m)	-	0.6
Design Water Level (m)	75.12	77.96
	Use 78m for metocean loading water level	

Marine growth thickness used is as per recommended by PTS; shown in Table 5.

Table 5 Marine growth for Offshore East Peninsular Malaysia (PTS)

Depth (m)	Layer Thickness (mm)	Density (tonne/m^3)
MSL	51	1.02
-4.6	153	1.02
-48.8	102	1.02
Mudline	25	1.02

3.3.2 Wave & Current

The wave heights and periods used in this project are from three notably different sources namely PTS, Joint Density and As Designed, as compiled in Table 6.

Table 6 Wave & Current Data

Analysis data	100 year return period		
	PTS	Joint Density	As Designed
Wave Height (m)	5.77 (H_s)	5.7 (H_s)	11.3 ($H_{max)}$
Wave Period (s)	8.06	Assume 6 and 8 seconds	9.3
Current (m/s)	1.67 @ surface 1.33 @ mid depth 0.36 @ seabed	0.69 @ -3m	1.3 @ surface 0.7 @ seabed

A global wave and current drag and mass coefficient shall be employed in the SACS model in conjunction with the Morrison’s Equation as per the API code requirements. The values are displayed here in Table 7.

Table 7 API Cm and Cd values

For tubular members	Clean Member	Fouled Members
Drag Coefficient, C_d	0.65	1.05
Mass Coefficient, C_m	1.6	1.20

The Wave Kinematics Factor as recommended by API RP 2A , is adopted as 0.9 while the Current Blockage factor for the Caisson is effectively 1.0 (unity).

Purposeful calculations for determining the apparent wave period due to Doppler effects of currents on the wave are not performed within the scope of this report.

All the metocean criteria (wave, current, wind) are simulated in only two predominant directions determined to induce the largest practical loading on a set of guy cable. This will be elaborated further in Section 3.5.

3.3.3 Wind

API RP 2A- WSD 21st Edition recommends the aspect of Spatial Coherence for wind loading as excerpted below **(API-21st, 2000).**

"……..For structures with negligible dynamic response to winds, the one-hour sustained wind is appropriate for total static superstructure wind forces associated with maximum wave forces……**"** **(pg. 19).**

In the scope of this project, in line with the author assumes that dynamic responses of the structure to wind is ignored, the 1 hour mean (100 year return period) wind speed at 10m above MSL with their respective metocean criteria set will be used for the in place analysis as documented in Table 8. To convert the different wind averaging intervals into the uniform 1 hour mean wind speed, the author employs factors from the Durst Curve.

Note that the calculations works are included in the Appendix.

Table 8 One hour wind speed

Criteria	1 hour Mean Wind Speed (m/s) for 100 year return period with an **assumed** direction of () degrees from true north.		
	PTS	Joint Density	As designed
Given/available data	29 (1 min mean)	19.9 (3 second gust)	39 (10 min)
After conversion to 1 hr mean	23.2	13.2	36.4

Wind forces will be modelled as joint loads calculated as shown in the Appendix. The logic of not using SACS's own built in wind force generator interface is that the model does not include topside equipment, their wind area and wind sheltering effects. Hence, the author will assume an overall projected area of the platform with an assumed enclosure of 70% for computing wind loads. The wind load will then be divided into 16 joint loads for the single wind direction and modelled at different joints symmetrically on the topside.(Illustrated in the Appendix).

3.3.4 Foundation Model

There are four piles and three 30” diameter conductors (2 external and 1 internal) penetrated below the mudline. The piles consist of three anchor piles of 72” diameter and one caisson leg of 84” diameter. The caisson leg penetrates 34.595m into the seabed, while the anchor piles all have a penetration of 24.384m.

The soil foundation input for the SACS model used in this study comprise of two variations – ‘Actual’ and ‘Degraded’, compiled and integrated into the analysis via SACS Pile Soil Interaction (PSI) module. The ‘Actual’ model reflects the site soil characteristics as seen from the Soil Investigation BH-ANOA L1 document (see reference). The ‘Degraded’ model is defined by the author to be a reduced capacity version its ‘Actual’ counterpart to simulate a user defined bad soil condition as part of the analysis scenario combinations. Based on the Soil Investigation Results, the author defines the Degraded soil as follows;

- A 30% reduction (approximately 1/3rd) of the Design Shear Strength of the ‘Real’ soil for all Clay, Sand and Silt. And a 30% reduction for Unit Skin Friction in Clay and 50% in Silt and Sand.
- A 30% reduction in the density of clay and 50% reduction for silt and sand.

The piles and soil modelling are done by SACS Data Gen for Pile-Soil Interaction (PSI). The PSI input file will then be used in tandem with the SACS Model Input file to produce a linear static structural analysis with a non linear pile soil interaction. Note that the geotechnical terms and properties used here are in tandem with the actual Soil Investigation Report (BH-ANOA L1). The modelled soil properties can be seen in the Appendix.

3.3.5 Dynamic Amplification Factors (DAF)

Previous literatures have documented that the Ledang Platform has a natural period of 3seconds, since greater than 2.5 seconds, hence would require dynamic effects to be accounted for. As of the scope of this project, the author considers only the wave forces' dynamic amplification factors. The specialized SACS inertial load condition generations will be ignored for the time being due to their sheer complexity. A simplified DAF calculation method is adopted from **ECL (2008)** and the formulas used are as shown below;

$$DAF = \frac{Max\ Dynamic\ Moment\ over\ Y\ axis\ \ @mudline}{Max\ Static\ Moment\ over\ Y\ axis\ \ @mudline}$$

where both the maximum dynamic and static mudline moments are both obtained from the SACS Wave Response program. The calculated DAFs will then be applied to their respective wave cases in the final analysis.

3.3.6 Coordinate System and Units

All calculations, simulations and results in this project will be performed and interpreted in the S.I Units (meter, kilogram, kilo Newton, seconds, etc…). Data originally in other forms of units such as the English Units will be converted accordingly by using appropriate conversion factors.

Global coordinate systems in all 3 Dimensions are defined in SACS is as shown in Figure 7. Note that this differs from the local coordinate system where the x axis is always represented by the longitudinal member axis.

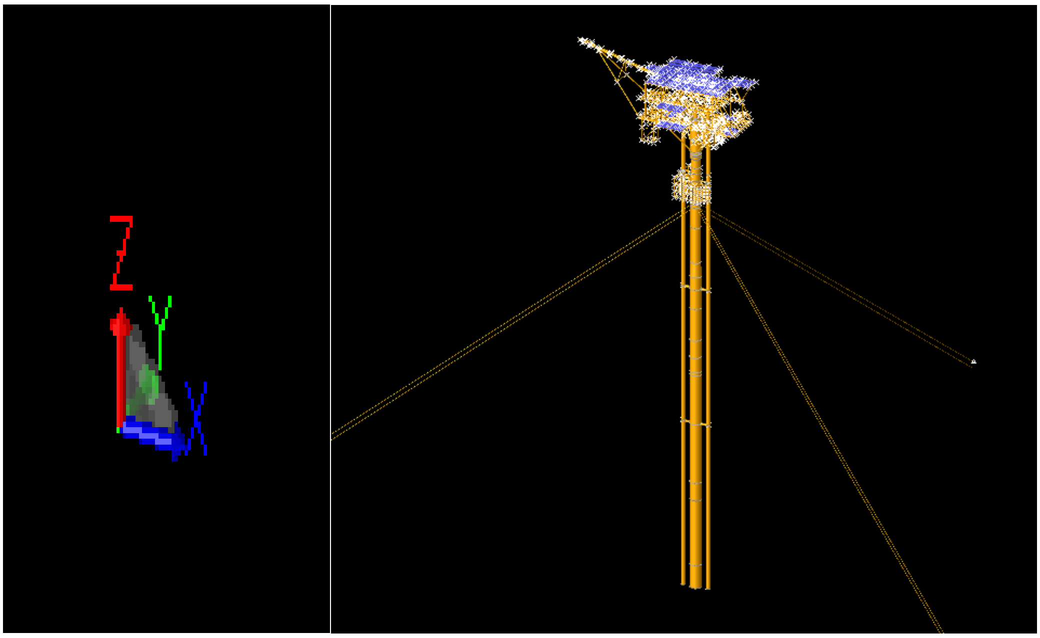

Figure 7 SACS Global Coordinate System

3.3.7 Two-directional Environmental Loading

For simplicity, a single environmental loading direction is chosen for the entire simulation. The direction was derived based on the basic principles of fundamental force equilibriums with simplistic assumptions that which would induce the maximum tension force in one set of guy cable. Mathematically, the force investigations are performed with respect to force summations in the x and y plane coupled with assumptions of 'neutral' (minimum force taking) and 'slack' (unable to function in compression) cables. Figure 8 depicts the unidirectional environmental loading. The derivation logic of the unidirection is available in the Appendix.

Note that the sequence of guy cable reduction sequence for each approach is given in the Appendix.

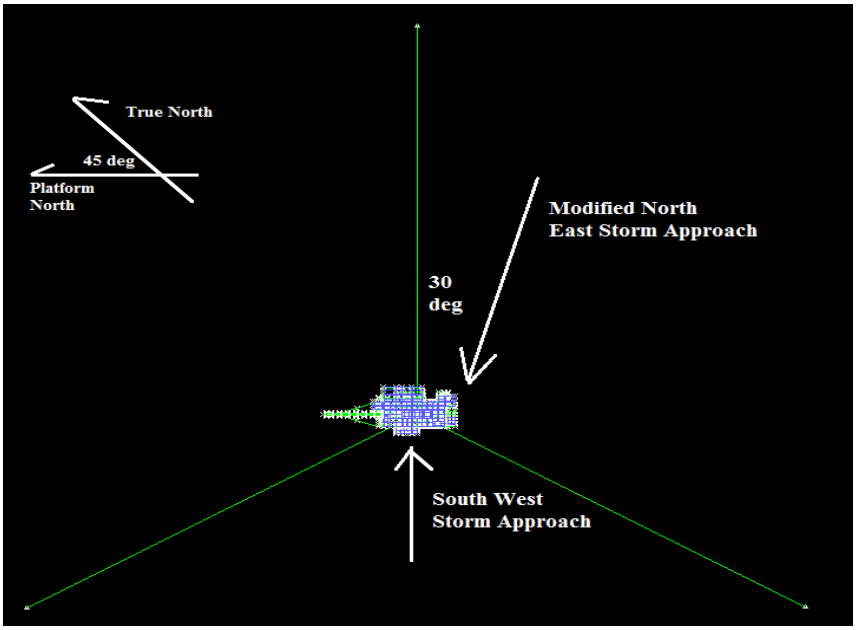

Figure 8 Worst Case Environment Load Approach

In Figure 8, it is useful to note that the derived direction of wave approach is towards the east face of the Platform. This is modelled in SACS with a angle of attack of 300 degrees. The guy cable reduction sequence is provided for in the Appendix.

3.3.8 SACS LDP-A Model

The previous existing SACS Model topside Data is one that was incomplete in its structural member definitions. To correct this fault, the model is subjected to detailed scrutiny in tandem with the As Built Documents (see reference) after which, it is then completed, checked and deemed ready for use in the simulations. The soil pile interaction input file is checked and modified accordingly (as in the case for the 'degraded' soil condition) with reference to the Soil Investigation Report BH-ANOA L1. The model is then customized for the project for simulation in its damaged condition by reducing the number of mooring / guying cables. Figure 7 depicts the modelled structure with its full guying system in place (all 3 sets of cables). A full 3D platform view is provided in the Appendix. The cable pretension of 100 kips will be modelled using temperature loading as the author's attempts to simulate pretension forces via the MEMB2 line proved to be futile. The details of the temperature loading calculations will be given in the Appendix.

3.3.9 Load Combinations

The load combinations for the static analysis are defined exclusively to encompass the maximum topside operating weight and the respective environmental loadings alongside substructure appurtenances loadings, as briefly summarized in Table 9 (given in terms of load condition number). Refer to Table 10 for load condition definitions.

Table 9 Static Analysis Load Combinations

Category	Seastate load	Topside load	Substructure load
PTS Load Condition 240^0 90^0	21,31 24,34	1 , 2, 3, 5, 8	15,50,51
As designed Load Condition	22,32 25,35	1 , 2, 3, 5, 8	15,50,51
Joint Density (t=8s) Load Condition	23,33 26,36	1 , 2, 3, 5, 8	15,50,51
Joint Density (t=6s) Load Condition	27,33 28,36	1,2,3,5,8	15,50,51

Table 10 Load Condition Definitions

Load Condition	Description
1	SACS Generated Self Weight
2	Topside structural appurtenances weight (rails, grating, ginpole, stairs, etc)
3	Open area live load
5	Equipment Operating Weight
8	Piping/instrument/electrical Operating Weight
15	Cable Pretension
21	PTS Metocean Load 240 DEG (NE)
22	As Designed Metocean Load 240 DEG (NE)
23	Joint Density (t=8s) Metocean Load 240 DEG (NE)
24	PTS Metocean Load 90 DEG (SW)
25	As Designed Metocean Load 90 DEG (SW)
26	Joint Density (t=8s) Metocean Load 90 DEG (SW)
27	Joint Density (t=6s) Metocean Load 240 DEG (NE)
28	Joint Density (t=6s) Metocean Laod 90 DEG (SW)
31	Wind joint loading for PTS 240 DEG
32	Wind joint loading for As Designed 240 DEG
33	Wind joint loading for Joint Density 240 DEG
34	Wind joint loading for PTS 90 DEG
35	Wind joint loading for As Designed 90 DEG
36	Wind joint loading for Joint Density 90 DEG
50	Substructure appurtenances dead load
51	Substructure appurtenances buoyancy load

CHAPTER 4

RESULTS & DISCUSSION

The results that will be included in this report will have heavy emphasis oo the interpretation and discussion of the response of the Tarpon as defined by the reaction of its main structural caisson to external loadings.

4.1 Load Summation and Member Stresses

(Note that all caisson internal forces are taken as the resultant of the force components in the plane of deflection. Tables 11 and 12 summarizes the worst Caisson internal forces experienced by extracting the Tarpon's response to the As Designed (AD) condition.)

It was found that in the As designed condition, the extreme wave height induces such a deflection on the Tarpon that the supposedly 'slack' (such cables are unable to take compression) cable in the fully guyed scenario took a credible amount of compressive forces – hence this resulted in a misleading deviation from the actual stiffness of the platform. Hence, the x 2 guys scenario will be assumed as the effective fully guyed response; and this fits logically with the notion that in any one storm direction, there has to be one 'slack' cable (unable to contribute to the platform in compression).

Table 11 Design caisson mudline internal forces (modified NE –AD in place)

	x 1	x 2 (also taken as fully guyed equivalent)	Freestanding
Axial (kN)	-4653.7	-5364.3	-4127.9
Shear (kN)	710.2	250.1	1767.5
Bending (kN.m)	-28518.2	-5828.1	106000

Table 12 Design caisson mudline internal forces (SW –AD in place)

	x 1	x 2 (also taken as fully guyed equivalent)	Freestanding
Axial (kN)	-6982.9	-5872.3	-3330.9
Shear (kN)	-1371.7	241.0	5589.8
Bending (kN.m)	78831.7	6605.8	65933.6

It is also worth mentioning at this stage that the mudline caisson internal forces are defined as the maximum of the internal forces developed in the caisson within 5m from the mudline and does not necessarily refer to the mudline overturning moment or shear.

It is apparent that the freestanding Tarpon suffers from huge bending moments and that, as will be detailed in the proceeding sections, the Tarpon structure fails in all of its freestanding scenarios. The tables 11 and 12 showcase the worst maximum caisson internal forces at the mudline.

4.2 Intact vs weak soil

Soil sensitivity studies shows that the Tarpon's In place response depends very little on the soil condition beneath it with the exception of relatively bad scenarios such as a combination of maximum loading (As Designed metocean) coupled with lost of guy wires. In fact, the Tarpon platform in its intact form (x 3 guy wires) shows negligible differences between the intact and weak soil scenarios. Interestingly, the platform's sensitivity to the soil beneath it increases as the modelled cases become worse – i.e ; maximum design storm coupled with loss of guy wires. The following graphs illustrate this.

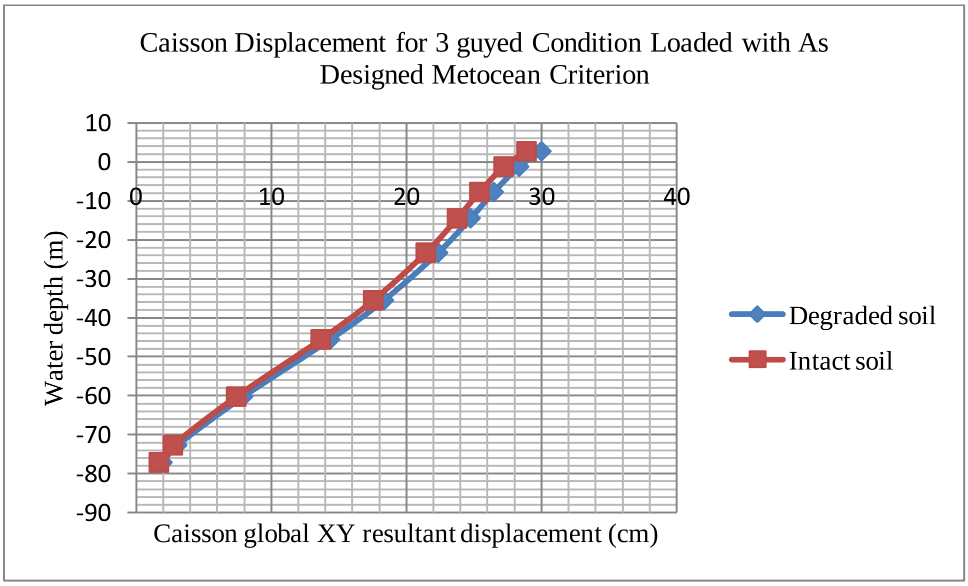

Figure 9 Intact caisson insensitive to soil

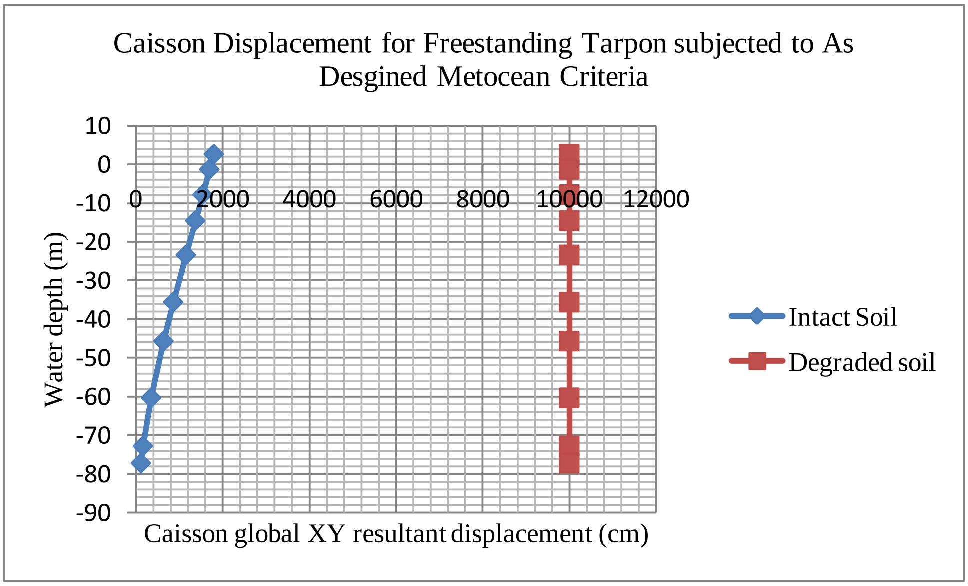

Figure 10 Caisson soil sensitivity

The main take away from this section is that for most conceivable cases, the Tarpon platform is relatively insensitive to the soil foundation it is resting on. Hence, heretofore, the results will be centered mostly on intact soil as the constant condition, whilst varying the other variables – like that of metocean, storm direction and guying cables.

4.3 Caisson Unity Check

This section highlights the interpretation of the plots of caisson unity checks throughout its entire length when subjected to both storm directions on intact soil.

4.3.1 Modified North East Storm Approach

The modelled Tarpon was subjected to storms approaching from the modified North East direction. Unity check ratios were taken throughout the length of the Caisson ,from the mudline up, after condensing the data file obtained from SACS In Place with PSI analysis. It should be noted that UC values greater than unity would deem the member to have failed. The maximum UC ratios are as summarized in table 11.

Table 13 Caisson UC summary for Modified NE Approach (intact soil)

Metocean	x 1	x 2	x 3	Freestanding
As Designed	1.66 @MDL	0.48 @MDL	0.37 @MDL	6 @MDL

PTS	1.04 @MDL	0.29 @MDL	0.31 @MDL	2.46 @MDL
Joint Density 8 sec	0.54 @MDL	0.23 @MDL	0.24 @TC/S	1.12 @ MDL
Joint Density 6 sec	0.59 @MDL	0.23 @MDL	0.24 @TC/S	1.34 @MDL

** MDL – Mudline   TC/S – Terminator Clamp to Splash Zone*

An important note to take at this point is that the Tarpon platform fails in its freestanding mode in ALL modified north east metocean loadings. This dismisses any notion that the Tarpon might stand a chance without its guying system. Also see the unique case where the unity check for the three guyed – joint density scenarios is maximum not at the mudline (UC = 0.21 for both joint densities) but at the region between the cable terminators and the splash zone. This goes to show that the cable terminators is actually one of the more critical elements in the Tarpon's structural system and that its design, positioning and maintenance should be reviewed in depth to see if better alternatives exist, instead of merely accepting it as it is.

In its fully guyed mode, the platform survives even the worst metocean loadings (as designed). An interesting note is that the 2-guyed scenario produces nearly the same UC values as the fully guyed Tarpon – all except the As Designed metocean criteria. [It was found that in the As designed condition, the extreme wave height induces such a deflection on the Tarpon that the supposedly 'slack' (such cables are unable to take compression) cable took a credible amount of compressive forces – hence this resulted in a misleading deviation from the actual stiffness of the platform. :– *repeated from section 4.1*]. Hence for the As designed case, it would be advisable to omit the 3 guyed – As Designed UC value and take its effective fully guyed UC as the 2 guyed scenario – which is sensible noting that in actual fact, from the modified NE direction, only such two cables will be taking the lateral loads while the other, is unable to contribute in compression (slack). As for the other metocean cases, it is seen from SACS member review that the guy elements supposedly in compression still retain a credible amount of tension (from their pretension loading)- due to the lesser deflections imposed as compared to the As Designed - hence still accurately describing the platform's stiffness to the best possible accuracy. This is evident when their UC values are almost the same as their 2 guyed scenario counterparts. The singly guyed Tarpon fails in the As Designed condition and may yet marginally survive (if not failed) the PTS metocean criteria whereas it comfortably takes on the Joint Density loadings.

4.3.2 South West Storm Approach

The second storm direction is simulated to approach the Tarpon from the true South West direction. The maximum UC values are summarized in Table 12.

Table 14 Caisson UC summary for SW Direction (Intact soil)

Metocean	x 1	x 2	x 3	Freestanding
As Designed	4.4 @MDL	0.55 @MDL	0.36 @ MDL	7.65 @ MDL
PTS	2.46 @MDL	0.44 @MDL	0.3 @MDL	2.79 @MDL
Joint Density 8 sec	1.11 @MDL	0.35 @MDL	0.24 @ TC/S	1.28 @MDL
Joint Density 6 sec	1.17 @MDL	0.35 @MDL	0.24 @ TC/S	1.51 @MDL

** MDL – Mudline   TC/S – Terminator Clamp to Splash Zone*

Again, as with the modified NE, the South West storm sees to the failure of the Freestanding Tarpon in all metocean criterion.

4.4 Caisson Deflection (full plots available in appendix)

This section highlights the lateral deflection plots along the length of the caisson when subjected to both storm directions on intact soil. To avoid overloading in this results section, the bulk of the extensively plotted caisson deflection graphs will be included in the Appendix for the reader's reference. Both the unity checks and maximum deflections will be united in a result triangulation in section 4.5.

4.4.1 Modified North East Storm Approach

Table 15 is the succinct summary on the Tarpon's maximum displacement. Note that, in comparison, when the pushover analysis was conducted for this particular storm direction, the structure fails though the plasticity of its anchor piles at a maximum pre collapse deflection of 196.6 cm.

Table 15 Tarpon Max Deflection for MNE Approach

Guy	Metocean	Max lateral Caisson Deflection (cm)
x 0	As Designed	1800.6
	PTS	578.3
	Joint Density 8 sec	232.9
	Joint Density 6 sec	291.8
x 1	As Designed	315.2

	PTS	179.8
	Joint Density 8 sec	82.2
	Joint Density 6 sec	91.5
x 2	As Designed	46.6
	PTS	11.0
	Joint Density 8 sec	14.2
	Joint Density 6 sec	12.9
x 3	As Designed	46.6
	PTS	15.5
	Joint Density 8 sec	7.1
	Joint Density 6 sec	9.2

4.4.2 South West Storm Approach

Table 16 is the succinct summary on the Tarpon's maximum displacement when loaded from South West. Note that, in comparison, when the pushover analysis was conducted for this particular storm direction, the structure fails though the plasticity of its anchor piles at a maximum pre collapse deflection of 273.4 cm.

Table 16 Tarpon Max deflection for SW approach

Guy	Metocean	Max Caisson Deflection (cm)
x 0	As Designed	13016.8
	PTS	684.6
	Joint Density 8 sec	284.1
	Joint Density 6 sec	348.3
x 1	As Designed	1135.4
	PTS	557.9
	Joint Density 8 sec	222.5
	Joint Density 6 sec	238.5
x 2	As Designed	62.1
	PTS	13.1
	Joint Density 8 sec	11.2
	Joint Density 6 sec	9.1
x 3	As Designed	62.1
	PTS	16.4
	Joint Density 8 sec	8.1
	Joint Density 6 sec	10.3

4.5 Pushover Analysis

The pushover analysis for the modelled guyed caisson monopod was performed in SACS via the Collapse module and interpreted using the corresponding results output file and the graphical results interpreter, Colvue. An attempt was made to perform the pushover analyses for all guy configurations (from freestanding to fully guyed) and for both degraded and intact soil conditions. However, only several modelled scenarios passed the simple logic test devised by the author specifically to eliminate possible erroneous analyses due to the inherent uncertainties in modelling a wire rope cable element as a standard prismatic beam member. The logic test is coupled with SACS's built in error warning systems and together, form a sound basis in assuring reasonable reliability in the result data. The simple logic test comprise of a quick check on the failed structure based on several criterion;

- Main load taking cables in tension
- Designated failure cable experience local buckling at the first two wave load increments (for non-fully guyed conditions)
- Pile connected to the purposely failed cable experiences relatively negligible forces

It is found that for most cases with degraded soils, SACS computed negative structural matrices (error in solution) for the piles and hence resulted in unreliable results.

Table 17 Pushover Summary

Collapse scenario	With pretension		
	Load factor	Caisson Maximum displacement (cm)	Reacted base shear (kN)
Fully guyed, Intact Soil , NE	11.78	-196.6	4329
Fully guyed, Intact Soil , SW	12.76	273.4	6493

The collapse scenario highlighted blue in the table above are to be nullified, and replaced by results from their respective x 2 guy counterparts. This is due to the fact that both x 3 guyed scenarios calculated enormous compressive forces on the guy cable that was designated to be slack in the direction of the force. In real operating conditions, this would not happen as wire ropes/ cables have no compressive strength. Hence to simulate a fully guyed condition subjected to load increments till failure, the two load taking guy wires in the direction of the force (NE or SW) are maintained while the ‘slack’ cable was given cross sectional properties so small that its effect on the overall stiffness of the Tarpon can be necessarily neglected.

SACS Event history – Mode of failure;

For all the pushover analyses conducted, failure of the structure was initiated at the anchor piles, where for most instances, pile plasticity occurred and in some scenarios, SACS recorded pile pullout events in the degraded soil models (which are not included here due to excessive error warnings from SACS).

This is further enhanced by the fact that the author has modelled the guy wire ropes to be of Fy = 24. 8 kN/cm2 and 129.5 kN/ cm2 steel grades respective and found that both yields the same results in terms of caisson moment , displacement and cable axial stresses. Pile failure / plasticity for both instances are equal and this enforces the notion that the integrity of the Tarpon structure is controlled primarily by the anchor piles.

Of all the degraded soil models, only one case stands out as a usable accurate interpretation – that which is listed in the table below as “x2 guys , Degraded Soil, SW”.

Table 18 Logic check for usable Pushover Analysis

Collapse scenario	Special Event Description (passes the logic test)	Cable Axial force		
		P1CS26 (kN)	P2CS26 (kN)	P3CS26 (kN)
Fully guyed, Intact Soil , NE	Local Buckling for P1CS26 (LF =4) P3 – Pile Plastic	-	1239.63	2580.08
Fully guyed, Intact Soil , SW	Local buckling for P3CS26 (LF =3) P1 and P2 – Pile plastic	2657.44	2685.52	-
Fully guyed, Degraded soil , SW	Local buckling for P3CS26 (LF=3) . P1 and P2 – Pile pull out and plastic	1708.46	1666.12	-

4.6 Unity Check - Deflection Results Triangulation

The results from the Unity checks are then superimposed with the caisson’s in place and ultimate lateral resistance as tabulated in Table 12. The ultimate caisson lateral deflection signifies its deflection at collapse (plastic failure) in its simulated fully guyed mode.

(Note : the orange highlights plastic caisson failure while the yellow highlights signifies that the Tarpon is marginally surviving on its plastic reserve strength , if not failed already.)

Table 19 Key Result Triangulation for modified NE approach (intact soil)

Guy	Metocean	Max lateral Caisson Deflection (cm)	Max Caisson Unity Check	Full lateral deflection capacity (cm)
x 0	As Designed	1800.6	6 @MDL	196.6 (Structure has collapsed) (Initiated by Pile plasticity)
	PTS	578.3	2.46 @MDL	
	Joint Density 8 sec	232.9	1.12 @ MDL	
	Joint Density 6 sec	291.8	1.34 @MDL	
x 1	As Designed	315.2	1.66 @MDL	
	PTS	179.8	1.04 @MDL	
	Joint Density 8 sec	82.2	0.54 @MDL	
	Joint Density 6 sec	91.5	0.59 @MDL	
x 2	As Designed	46.6	0.48 @MDL	
	PTS	11.0	0.29 @MDL	
	Joint Density 8 sec	14.2	0.23 @MDL	
	Joint Density 6 sec	12.9	0.23 @MDL	
x 3	As Designed	46.6	0.48 @MDL	
	PTS	15.5	0.31 @MDL	
	Joint Density 8 sec	7.1	0.24 @TC/S	
	Joint Density 6 sec	9.2	0.24 @TC/S	

It is obvious here that the freestanding modes have all failed indefinitely. The singly guyed condition also fails under the extreme As Designed metocean criteria and marginally survives with its plastic reserve strength (if not failed already) when loaded with the PTS metocean criteria while it comfortably survives the joint densities. While the remaining scenarios are in favour of the platform's survival against the storms, it is worth noting the red text in the 3 guyed As designed metocean scenario, where a significant amount of compression was induced in the third (supposedly slack cable) guy element. As this is a trivial situation, the particular result will be omitted and the 2 guyed As Designed condition will be used in its place for future interpretations. The fully guyed PTS and Joint density conditions all show reserve tensions (residual pretension) in the 'slack' cable due to lesser deflections than the As Designed conditions – signifying that the cable is still exerting a 'pulling' force on the platform and is not taking any compression- and that this pulling force might be of significance to the lateral stiffness of the platform. Hence, the third cable should be included in the analysis.

Table 20 UC Result Triangulation for true SW approach (intact soil)

Guy	Metocean	Max Caisson Deflection (cm)	Max Caisson Unity Check	Full deflection capacity (cm)
x 0	As Designed	13016.8	7.65 @ MDL	273.4
	PTS	684.6	2.79 @MDL	
	Joint Density 8 sec	284.1	1.28 @MDL	(Structure has collapsed)
	Joint Density 6 sec	348.3	1.51 @MDL	
x 1	As Designed	1135.4	4.4 @MDL	
	PTS	557.9	2.46 @MDL	(Initiated by Pile plasticity)
	Joint Density 8 sec	222.5	1.11 @MDL	
	Joint Density 6 sec	238.5	1.17 @MDL	
x 2	As Designed	62.1	0.55 @MDL	
	PTS	13.1	0.44 @MDL	
	Joint Density 8 sec	11.2	0.35 @MDL	
	Joint Density 6 sec	9.1	0.35 @MDL	
x 3	As Designed	62.1	0.55 @MDL	
	PTS	16.4	0.3 @MDL	
	Joint Density 8 sec	8.1	0.24 @ TC/S	
	Joint Density 6 sec	10.3	0.24 @ TC/S	

Like in the NE direction, the SW storm approach induces failure in all freestanding Tarpons. The singly guyed Tarpon in the SW approach fails indefinitely for As designed and PTS metocean criterion while banks on its reserve plastic strength to marginally survive the joint density storms (if not failed). Notice the red coloured text for the fully guyed – As Designed metocean scenario as a similar situation to its NE counterpart (the lengthy explanation will not be repeated here again – please refer the latter paragraphs). Herein, it would be advisable to take the fully guyed response to the As Designed metocean condition to be its two guyed scenario.

4.7 Wire Rope (guy cable) Forces

Here, we assume the nominal breaking strength of the wire ropes to be 713 tons (ECL, 2008). Taking $g = 9.80665\ m/s^2$, that equates to 6992 kN. Here, we discuss the prevailing two most extreme analyses conducted. The guy wires are analysed to act in their pairs and in the condition that one of the wire in the pair snaps.

Table 21 Max Guy wire tension- strength check for AD Metocean

	NE load (kN)	SW Load (kN)	NE FOS	SW FOS
x 1	761.1	742.5	9.2 (pair) 4.6 (single)	9.4 (pair) 4.7 (single)
x 2	1250.3	1135.3	5.6 (pair) 2.8 (single)	6.2 (pair) 3.1 (single)
x 3	Equivalent to x 2			

Table 22 Max guy wire tension during pushover

	NE load (kN)	SW Load (kN)	NE FOS	SW FOS
Simulated fully guyed	2580.1	2685.5	2.7 (pair) 1.4 (single)	2.6 (pair) 1.3 (single)

It is clear from tables 17 and 18 that the guy cables will not fail in axial tension. Even with the pushover analysis, the guy cables still possess relatively large reserve strengths. Hence, should to any set of guy cables be observed to have failed (no longer in position), attention should be given to its connections at the terminator clamps and anchor piles while investigating the potential role of corrosion , creep and fatigue in its failure.

4.8 Preliminary Dynamic analysis

4.8.1 Eigenvalue

The natural periods of the Tarpon structure were simulated via SACS Dynpac module. Here, two sets of analyses are performed – one with a fixed base assumption and the other includes structure-soil-pile interaction effects by means of pile foundation super element creation. The tarpon model was simulated via SACS generated mass and the author's choice of load to mass conversion for dynamic eigenvalue analysis. The Caisson and Conductors below the mean sea level are designated as flooded members to account for the added mass from the displaced water column. The dynamic mass system was selected as 'consistent/continuous mass' in contrast to the lumped mass model. A total of 10 modes of vibration were obtained for the fixed assumption while the author opted for 20 modes in the case of the structure-pile-soil interaction.

Table 23 Natural periods derived with the fixed base assumption

Modes	Freestanding (s)	X 1 (s)	X 2 (s)	X 3 (s)
1	10.17 (X)*	7.540 (Y)*	2.934 (Y)*	1.907 (X)*
2	7.474 (Y)*	2.203 (X)*	1.897 (X)*	1.898 (Y)*
3	1.694	1.673	1.652	1.639
4	1.459	1.460	1.426	1.387
5	0.915 (T)*	0.910 (T)*	0.912 (T)*	0.909 (T)*
6	0.652	0.642	0.639	0.638
7	0.633	0.634	0.628	0.622
8	0.323	0.321	0.322	0.323
9	0.306	0.308	0.308	0.307
10	0.274	0.274	0.275	0.275

* (X) – First X bending mode , (Y) – First Y bending mode , (T) – First torsional mode

Table 24 Natural periods derived with pile superelement

Modes	Freestanding (s)		X 1 (s)		X 2 (s)		X 3 (s)	
	Good soil	Bad soil	Good soil	Bad soil	Good soil	Bad soil	Good soil	Bad soil
1	16.430 (X)*	0	11.224 (Y)*	11.772 (Y)*	3.746 (Y)*	3.841 (Y)*	2.418 (X)*	2.451 (X)*
2	11.8466 (Y)*	0	2.805 (X)*	2.854 (X)*	2.403 (X)*	2.444 (X)*	2.354 (Y)*	2.405 (Y)*
3	2.606	14.267 (X)*	2.377	2.414	2.358	2.408	2.344	2.389
4	2.272	10.156 (Y)*	2.143	2.234	2.055	2.088	2.020	2.048
5	1.361 (T)*	1.925	1.354 (T)*	1.355 (T)*	1.354 (T)*	1.355 (T)*	1.352 (T)*	1.353 (T)*
6	0.944	1.723	0.875	0.903	0.842	0.854	0.828	0.840
7	0.919	1.399 (T)*	0.839	0.851	0.824	0.837	0.809	0.821
8	0.593	1.393	0.480	0.531	0.442	0.459	0.439	0.453
9	0.584	1.364	0.480	0.527	0.434	0.458	0.427	0.448
10	0.571	1.309	0.446	0.461	0.433	0.451	0.426	0.444
11	0.562	1.288	0.440	0.460	0.425	0.448	0.419	0.441
12	0.460	0.672	0.410	0.431	0.400	0.413	0.397	0.410
11	0.445	0.657	0.403	0.413	0.384	0.395	0.379	0.388
14	0.368	0.385	0.363	0.365	0.362	0.363	0.361	0.362
15	0.367	0.357	0.362	0.362	0.361	0.362	0.361	0.362
16	0.314	0.345	0.294	0.316	0.294	0.294	0.294	0.295
17	0.293	0.312	0.281	0.294	0.281	0.287	0.279	0.286
18	0.280	0.294	0.263	0.287	0.248	0.275	0.247	0.284
19	0.231	0.280	0.230	0.272	0.233	0.253	0.239	0.249
20	0.223	0.230	0.214	0.217	0.215	0.218	0.215	0.218

* (X) – First X bending mode , (Y) – First Y bending mode , (T) – First torsional mode

As expected, the assumed fixed bases at the mudline will logically incur an idealistic picture on the natural period of the Tarpon structure, in the sense that the fixed connection would generically increase the stiffness of the system, hence decreasing its natural period/increasing its frequency as compared to the case where the soil stiffness is taken into consideration. The fixed base assumption also introduces a slight liberty into the analysis, which may offset several conservative parameters applied in the static analysis. The dynamic amplification factors (DAF) are calculated for both the data in table 1 and 2, showcased in tables 3 and 4 with the streamlined/summarized DAFs to be employed in the linear static analysis in table 5.

It is worth noting that the results from this dynamic investigation on the Tarpon structure should not be used/considered as an accurate dynamic sensitivity measure of the system. Nonetheless, this first level dynamic study does provide a useful insight into the comparative dynamic behaviour of the Tarpon platform. In essence, this serves as a rough input for the SACS Wave Response module to generate dynamic and static structural response values to obtain the dynamic amplification factors to be used in amplifying the seastate in the final analysis for simulating increased loadings due to dynamic effects. A full scale inertia load set generation, foundation pile stub and/or superelement creation alongside a complete dynamic deterministic or spectral wave analysis is beyond the scope of this project – the author recommends for it to be performed exclusively as a separate study on the dynamic response /sensitivity of the Tarpon platform.

4.8.2 DAF Computation (full calculation in Appendix)

It can be observed from the Appendix that the results of the SACS Wave Response analysis are rather eratic and chaotic in nature, dotted with non converging values. The author attributes this to the inaccuracy of describing the model in SACS by representing the guy cables with prismatic cylindrical beam elements which can take both compression and tension. Ideally, the guy cables should be modelled as cable elements capable of handling tension only. The built in GAP function in the SACS programme module to model tension only members does not apply for in place pre/post tensioned cables, and as other literatures have noted, would cause severe errors if forcefully applied in the analysis **(ECL,2008)**.

It would be useful to note that the detailed wave response results, DAF calculation and filtering are included in the appendix.

Despite having performed the analysis up to 1000 iterations, there are several result excerpts that show non convergence of the analysis; some to severely high percentages of non convergence. This is normally observed for the less than 3 (full) guyed conditions and the author infers here that the complex cable-structure-soil interaction is over simplified in this first level dynamic analysis. Nonetheless, with the combined result data from both tables 3 and 4, appropriate values of DAFs can be extracted with liveable accuracy for use in the linear static analysis; table 5 summarizes this. The author recommends that other software should be brought into the fray to aid future works for in depth dynamic behavioural research for Tarpon platforms, such as ORCAMOOR (to model the guy cables), and the like.

The dynamic analysis for the liquefied soil condition failed due to a severe error caused by the minutely small values of the soil properties which caused negative matrices and subsequently led to the automated process termination of the post processor. This is not of crucial importance, as the author reasons that the structure modelled on liquefied soil will fail regardless of the DAFs applied. The main purpose of modelling liquefied soil is merely to provide a worst case scenario in the linear static analysis for a more holistic project scope, where structural failure is imminent. Hence it is logical to assume that the DAFs used for the bad soil conditions can simply be adopted for the liquefied soil.

In the light of very possible erroneous dynamic data, a novel simplistic result filtering methodology is employed by the author specifically devised for this project, as highlighted in bullet form at the end of this paragraph.

- Outlier values are not to be taken into consideration. An outlier data is defined simply as the DAF value with an obvious deviation from the group (i.e , a value of 40 amongst values ranging from 1.0 to 2.0.)
- The jest of this methodology lays in taking the arithmetic mean to be used as the filtered DAFs.
- With reference to existing literatures on DAF calculations, it is found that generically, the DAF for fixed offshore steel structures approximately lie within the range of 1.0 to 1.8 (**Mourad et.all , 2005**). The substructure design of the

Tarpon structure performed by ECL also revealed applied DAFs within the range of 1.006 to 1.4. Hence for any calculated DAFs with values less than unity, the value 1.0 shall be assigned to these cases whilst 1.8 is used as a practical capping upper limit, which would give a rather conservative final linear static analysis.

- The author infers that non convergence coupled with extremely large DAFs might signify structural resonance, but will not dwell further into the matter in the scope of this paper.
- For standardization and removal of illogical data, the calculated DAFs are then filtered out by use of deductive reasoning and logic, prioritizing converged data and omitting non converged data where possible. Here, the average of the good-bad soil data pair for each case is calculated. From table 20, it can be seen that the soil condition has a small (negligible) effect on the natural periods of the structure. It can also be seen that the DAF varies very little between good and bad soils as well as between both wave approach direction. Hence, for the sake of DAF value selection, the variables mentioned will be omitted – in other words, their values are merged in a simple arithmetic mean taking union that which greatly simplifies this section. The filtered data is showcased in Table 21.

Table 25 Filtered DAF values

	Freestanding	X 1	X 2	X 3
PTS (240 deg)	1.00	1.05	1.15	1.08
AD (240 deg)	1.49	1.07	1.22	1.10
JD (240 deg)	1.00	1.06	1.14	1.12
JD6 (240 deg)	1.00	1.00	1.05	1.21
PTS (90 deg)	1.02	1.09	1.06	1.07
AD (90 deg)	1.65	1.12	1.22	1.08
JD (90 deg)	1.00	1.10	1.19	1.12
JD6 (90 deg)	1.00	1.00	1.11	1.23

It can be seen that the fully guyed condition (x 3) possesses the least DAF values , which can be rationalized by the fact that an intact Tarpon is stiffer than that of its cable reduced models. A stiffer structure will come with it, a higher first mode frequency, hence reducing its dynamic response to the waves investigated in this study (hence the lower DAF values). The freestanding model captured DAFs of less than unity, which maybe a result of incorrect modelling in terms of reducing the cables or a result of the fact that the freestanding caisson has a large 1st mode period of 16 seconds, which makes it highly compliant ,therefore effectively reducing the structural stresses. This would also help explain the DAF resulting from the As Designed load case, which has the highest wave period (closest to the high natural period of the freestanding Tarpon) amongst all the metocean criterions. However, with regards to the author's DAF data filtering/selection criteria, a value of 1.0 is taken for all such cases for added conservativeness. Also, it is worth noting that where the Joint Density wave parameters induce a slightly greater DAF than the PTS- This is perhaps due to the JD's wave period which is closer to that of the first natural period of vibration for the platform.

Other than the above, there are no clear observed trends / relationships between the DAF values for different the soil-structure models. This first level dynamic response estimation may not be viable in describing the actual dynamic characteristics of the platform, as the cables in this study are modelled as rigid beam elements. Nonetheless, for the scope of this project, the values as showcased in column 5 of Table 5 will be applied to the linear static in place analysis utilizing nonlinear wave theories to obtain the Tarpon's structural response to a certain degree of conservativeness.

4.9 Global Structural Stiffness

In this section, a combination of both software (SACS) and manual calculations will be employed to obtain an approximate on the global Tarpon structural stiffness in its fully guyed and freestanding mode; that which will be used consequently to manually compute its 1st mode of natural vibration to be compared with the SACS Dynpac values.

4.9.1 Global Stiffness Approximation

The Tarpon's stiffness would, logically by first inspection be considerably lower than that of conventional fixed platforms. But how much lower it really is? This sub section seeks to debunk the later – taking the stiffness of a conventional jacket to be in the range of 4000 – 5000 kN/m.

The author has attempted numerous methodologies of analyses in SACS to obtain a force to displacement relation where the corresponding stiffness k = F/x is the gradient of the force – displacement graph. It was then concluded that one particular methodology with its accompanying assumptions best describes the Tarpon's structural stiffness – that which is documented in considerable detail herein.
First, the centre of the lateral forces (for sea going structures – is mainly derived from wind, wave and current) is determined using the forces summary generated in the SACS output listing file as shown below.

For the worst case metocean condition (as designed) ;

Modified NE approach - The sum of forces at the origin are:

Fx = -967.69 Fy = -1801.33 Fz = -6158.46

Mx = -26450.7 My = 13569.76 Mz = -3139.76

The center of forces is:

For X forces: X = 2.504 Y = -0.822 Z = -14.212

For Y forces: X = 1.302 Y = -1.15 Z = -12.444

For Z forces: X = -0.03 Y = 0.655 Z = -7.36

True SW approach - The sum of forces at the origin are:

Fx = -3.57 Fy = 2023.76 Fz = -6238.86

Mx = 28541.96 My = -2578.61 Mz = -399.33

The center of forces is:

For X forces: X = -274.276 Y = -12.836 Z = -2.621

For Y forces: X = -0.175 Y = -1.428 Z = -12.422

For Z forces: X = -0.415 Y = -0.545 Z = -7.52

To better approximate the in place Tarpon global stiffness, it is crucial to take the centre of the lateral forces as it modelled in the software , instead of defining it arbitrarily at the top of the caisson (which would undoubtedly give a very small-conservative value – effectively underestimating its in place stiffness). It can be seen from the SACS output excerpt above that the resultant lateral forces are located roughly 12m below the water line. It is here that an artificial joint (coined CSTF) is created to enable a point loading at z = -12 m.

The point load is incremented gradually and the corresponding caisson deflection is recorded - and ultimately the graph of F-x is plotted, both for the freestanding and fully guyed condition. The displacements due to the loading however, will be taken at the top of the caisson at joint C001. The results are as summarized in the following graph plots.

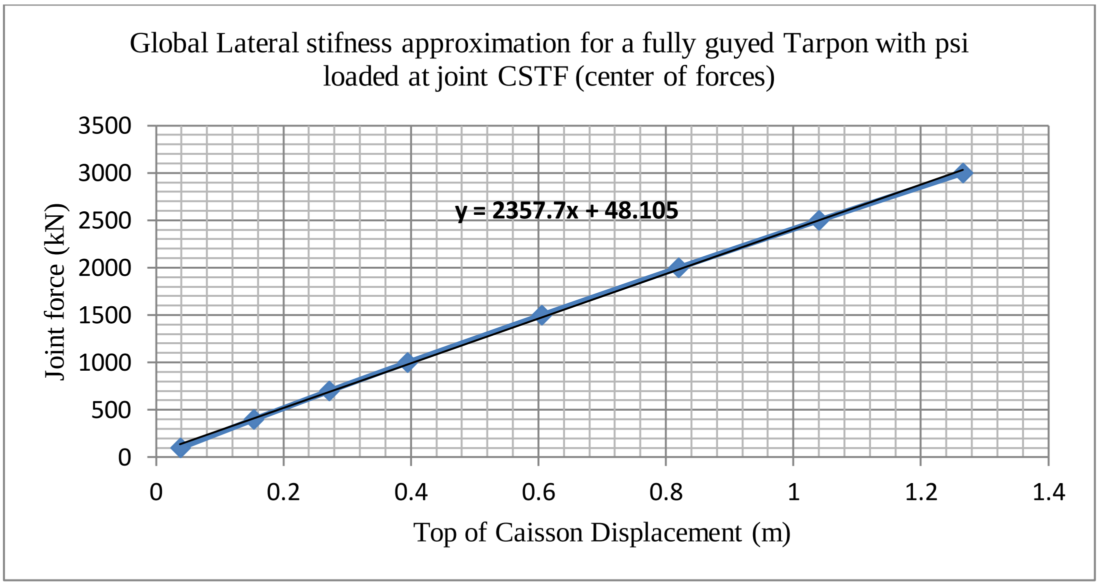

Figure 11 Fully guyed tarpon stiffness

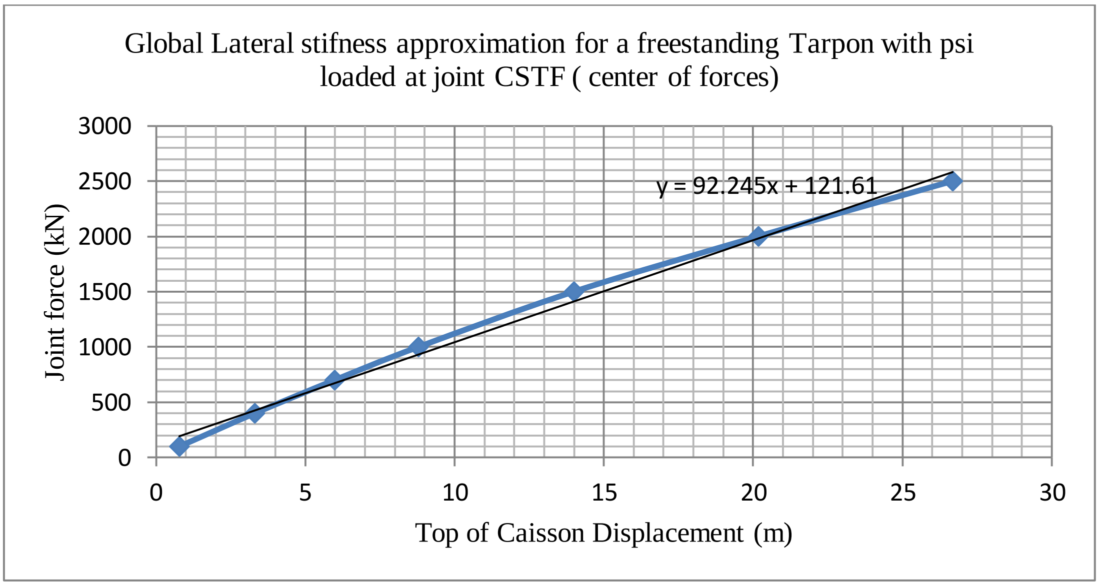

Table 12 Freestanding tarpon stiffness

It can be readily observed from the excel generated best fit line equation that the in place stiffness of the fully guyed Tarpon can be taken as 2400 kN/m while the value for freestanding is approximately 93 kN/m.

Even in its fully guyed condition, the Tarpon's stiffness is a far cry lesser than that of a conventional jacket platform – but nonetheless, this would deem a rather unfair comparison as the Tarpon is fundamentally a somewhat 'small scale' marginal field platform designed to support minimal field development / production equipment.

4.9.2 Manual 1st Mode Approximation (full calculations in the Appendix)

Utilizing the in place global stiffness values from 4.8.1, manual computations to determine the platform's first mode of natural vibration (period) for its freestanding and fully guyed condition were performed to be compared with the SACS Dynpac generated eigenvalues.

Table 26 Global Stiffness

Condition	Approximated stiffness (kN/m)	Calculated natural period (s)	SACS Dynpac Generated (s)
Fully guyed	2400	2.67	2.42
Freestanding	92	13.7	16.43

It can be observed that the computer generated values more or less agrees with the manually calculated natural period.

4.10 Reserve Strength Ratio (RSR)

The Tarpon's RSR is defined herein as the ratio of the ultimate structural caisson mud line moment (generated via pushover analysis) and the maximum design mud line moment. For an unmanned platform, the target RSR value based on PETRONAS Carigali's recommendations is a minimal of 1.32.

Table 27 RSR values summary

<table>
<tr><td rowspan="4">Guy/Design Load</td><td rowspan="2">Design</td><td colspan="2">Structure collapse (pushover)</td><td rowspan="4">RSR</td></tr>
<tr><td colspan="2">(intact structure)</td></tr>
<tr><td>Caisson Mudline MAX resultant moment</td><td rowspan="2">Load factor</td><td>Caisson Mudline MAX resultant moment</td></tr>
<tr><td>kN.m</td><td>kN.m</td></tr>
</table>

1 guy/PTS 240 deg	16985.86954	11	17829.57393	1.05
1 guy/AD 240 deg	28518.22864	11	17829.57393	0.63
1 guy/JD 240 deg	7708.92799	11	17829.57393	2.31
1 guy/ JD6 240 deg	8496.354884	11	17829.57393	2.10
1 guy/PTS 90 deg	43586.81375	13	23090.25224	0.53
1 guy/ AD 90 deg	78831.66705	13	23090.25224	0.29
1 guy/ JD 90 deg	18429.40382	13	23090.25224	1.25
1 guy/ JD6 90 deg	19437.85523	13	23090.25224	1.19
2 guy/PTS 240 deg	2652.580487	11	17829.57393	6.72
2 guy/AD 240 deg	5828.082539	11	17829.57393	3.06
2 guy/JD 240 deg	1598.166539	11	17829.57393	11.16
2 guy/ JD6 240 deg	1618.497358	11	17829.57393	11.02
2 guy/PTS 90 deg	2411.539335	13	23090.25224	9.57
2 guy/ AD 90 deg	6605.815279	13	23090.25224	3.50
2 guy/ JD 90 deg	867.7219131	13	23090.25224	26.61
2 guy/ JD6 90 deg	948.1089491	13	23090.25224	24.35
3 guy/PTS 240 deg	2920.898713	11	17829.57393	6.10
3 guy/AD 240 deg	4132.164267	11	17829.57393	4.31
3 guy/JD 240 deg	1010.561013	11	17829.57393	17.64
3 guy/ JD6 240 deg	926.6838781	11	17829.57393	19.24
3 guy/PTS 90 deg	2701.40858	13	23090.25224	8.55
3 guy/ AD 90 deg	3840.031039	13	23090.25224	6.01
3 guy/ JD 90 deg	808.3884741	13	23090.25224	28.56
3 guy/ JD6 90 deg	728.5119628	13	23090.25224	31.70
0 guy/PTS 240 deg	43945.33725	11	17829.57393	0.41
0 guy/AD 240 deg	110866.0532	11	17829.57393	0.16
0 guy/JD 240 deg	18656.44664	11	17829.57393	0.96
0 guy / JD6 240 deg	22788.39215	11	17829.57393	0.78
0 guy/PTS 90 deg	50260.31864	13	23090.25224	0.46
0 guy/ AD 90 deg	65933.63539	13	23090.25224	0.35
0 guy/ JD 90 deg	21728.61604	13	23090.25224	1.06
0 guy/ JD6 90 deg	26130.14422	13	23090.25224	0.88

*Note that the orange colour code highlights RSR values that are less than 1.32.

It is once again exceedingly obvious that the Tarpon platform does not survive in its freestanding mode. It is also worthy to observe that in the case of a singly guyed Tarpon, its survival actually highly depends on the storm directionality and magnitude. In fact, the platform gradually exhibits increasingly sensitive behaviour to the metocean conditions as its guy wires are reduced sequentially from fully guyed to freestanding. The Tarpon can also be said to be highly dependent on its guying system – without which it will indefinitely fail in any storm condition. And as the number of guy wire sets are reduced, the platform becomes increasingly sensitive to storm directionality – something that it is relatively insensitive to in its fully guyed condition. One will also observe several extreme RSR values especially when in its fully guyed condition, loaded with the mildest of storms (i.e. 3 guy/JD – 31.7). To the trained person, this RSR value is seemingly trivial in nature and it goes to show how the value can fluctuate so wildly from a whopping 31 to a meagre 1 (or even lesser) when varied from a fully guyed condition to its freestanding mode – which suggests that the Tarpon has in fact, very little redundancy to begin with. Whereas the fixed jacket platform has relatively more alternative load pathways due to indeterminacy (redundancy), the Tarpon's integrity is solely dependent on its guying cables and anchor piles – all three sets of them – and this really does not provide much redundancy. Even the failure of a single guy cable may initiate imminent failure- this is elaborated further in 4.11.

4.11 Worst Conceivable Scenario

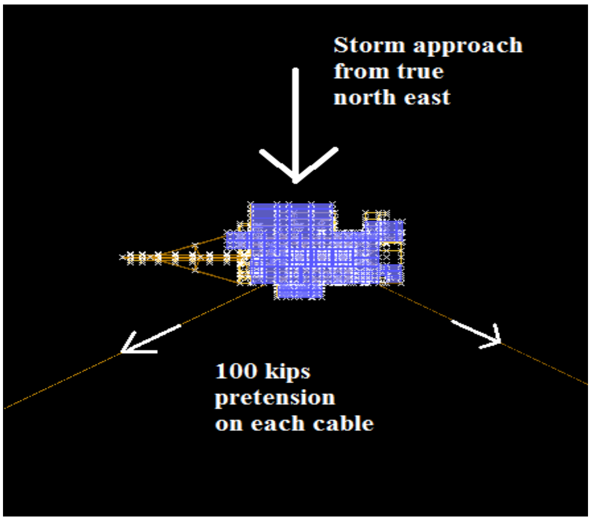

Figure 13 Worst conceivable scenario

When scrutinized, the Tarpon's worst case scenario is actually not in its freestanding mode. This goes to show that with the culmination of a series of unfortunate events, like that the loss of a single set of guy cables coupled with an unlucky storm direction, the platform may fail indefinitely. To illustrate, even in the event of failure for a single guy cable set (leaving the caisson to be doubly guyed), given an unfortunate storm direction, the resulting condition may actually be worse than that of a freestanding caisson (as shown in figure 13). The figure below illustrates the storm approaching the Tarpon along the lines of its lost cable. One can observe that the pretension in the remaining two cables actually aids the storm to topple the Tarpon.

4.12 Results Verification Checks

4.12.1 Logic Check – in place with intact soil condition

To eliminate improbable conditions like that of cables in compression from affecting the accuracy of the results, the results obtained from SACS were first grouped and filtered using two logic tests as shown in the table below. Note that the SACS Tarpon reduced guy model was simulated by changing the cross sectional properties of the particular 'eliminated' cable by modelling it to be inestimably small with negligible contribution (if any at all) to the overall stiffness and hence the response of the structure. The terms used here 'minor compression', 'residual tension' and 'large compression' are debunked as below;

'minor compression' refers to relatively mild compression on the cable when compared to the tension of the remaining guy wires. 'residual tension' refers to the remaining tensile forces due to the pretension loading on a cable which is supposed to slack in the direction of the lateral load. 'large compression' refers to cases where the compression in the presumably slack cable exceeds that of the tension of the main design tension cable – which would require scrutiny and analysis modifications to correct the error.

Table 28 Logic check - Intact soil _ NE

Metocean	Guy	Did Group GFL fail as designated?	Main tension cable as predicted?
As designed	X1	YES *	YES *
	X2	YES	YES
	X3	NOT APPLICABLE	YES-minor compression in P1 cables
PTS	X1	YES*	YES*
	X2	YES	YES
	X3	NOT APPLICABLE	YES – residual tension in P1 cables

Joint density 8 second period	X1	YES*	YES*
	X2	YES	YES
	X3	NOT APPLICABLE	YES – residual tension in P1 cables
Joint density 6 second period	X1	YES*	YES*
	X2	YES	YES
	X3	NOT APPLICABLE	Yes – residual tension in P1 cables

* x1 guying conditions show possible erroneous results

Table 29 Logic check - Intact soil _SW

Metocean	Guy	Did Group GFL fail as designated?	Main tension cable as predicted?
As designed	X1	YES	YES
	X2	YES	NO –large compression in P3 cables
	X3	NOT APPLICABLE	YES-minor compression in P3 cables
PTS	X1	YES	YES
	X2	YES	NO – large compression in P3 cables
	X3	NOT APPLICABLE	YES-residual tension in P3 cables
Joint density 8 second period	X1	YES	YES
	X2	YES	NO- large compression in P3 cable
	X3	NOT APPLICABLE	YES- residual tension in P3 cables
Joint density 6 second period	X1	YES	YES
	X2	YES	NO- large compression in P3 cables
	X3	NOT APPLICABLE	YES – residual tension in P3 cables

4.12.2 Cable pretension (proved)

A linear temperature model is used to simulate a pretension force of 444 kN (100 kips) on each guy cable. The workings of the calculations and methodologies used for determining the temperature differential are included in the appendix. In order to ensure that the manually calculated model works as intended, a separate load case containing only the topside dead and live loads and the cable temperature loading was created and analysed.

The Tarpon in its fully guyed condition was used as the test model as all three symmetrical cables loaded with the pretension forces would be in equilibrium and hence, provide an accurate picture on the internal loads generated by the temperature loading input. Say, if a 2 guyed condition Is used, under no lateral loading, the pretension will cause the Tarpon to deflect in the direction of pull of the cables, hence creating a false impression on the total internal force generated by the temperature differential.

The snippet in figure 14 shows a portion of the SACS output report detailing the internal loads in the respective cable pairs P1 – CS26 and P1 – CS27 and the like (note that each symmetrically positioned cable system consists of a pair of cables for redundancy) . Notice the absence of any torsional and bending loads with inestimably small shear (negligible). Axial tension loads take precedence here, with values in the range of 443 kN to 445 kN , a good estimate of the 100kips design pretension. The values per cable differ from one another as they vary (though very slightly) in length and hence the calculation of the temperature differential was performed for each length value to yield very slight changes in the temperature input on each cable.

DATE 24-JU

LEDANG (GUYED CAISSON STRUCTURE) REASSESSMENT - INPLACE ANALYSIS

SACS-IV SYSTEM MEMBER INTERNAL LOADS SUMMARY REPORT

MEMBER	GRP	MAX. UNITY CHECK	CRIT COND	LOAD COND NO.	DIST FROM END m	AXIAL kN	SHEAR Y kN	SHEAR Z kN	TORSION kN-m	BENDING Y-Y kN-m	BENDING Z-Z kN-m
						* * * * * * * * * * I N T E R N A L		L O A D S * * * * * * * * *			
P1-CS26	GUY	0.61	TN+BN	TEST	0.0	443.84	0.11707E-26	0.90873E-26	0.0000	0.0000	0.0000
P1-CS27	GUY	0.61	TN+BN	TEST	0.0	443.82	-0.11548E-26	-0.91177E-26	0.0000	0.0000	0.0000
P2-CS26	GUY	0.61	TN+BN	TEST	0.0	445.66	-0.64664E-27	-0.74539E-26	0.0000	0.0000	0.0000
P2-CS27	GUY	0.61	TN+BN	TEST	0.0	445.65	0.64722E-27	0.74941E-26	0.0000	0.0000	0.0000
P3-CS26	GUY	0.61	TN+BN	TEST	0.0	445.02	-0.44748E-27	0.21330E-26	0.0000	0.0000	0.0000
P3-CS27	GUY	0.61	TN+BN	TEST	0.0	444.94	-0.26623E-26	0.12843E-25	0.0000	0.0000	0.0000

Figure 14 SACS Snippet on cable pretension

CHAPTER 5

CONCLUSION

5.1 Conclusion

5.1.1 Concluding overview

This project addresses the pressing issue of a need for design models and assessments for the Tarpon platforms owned by PCSB in both PMO and SBO waters by providing a sensitivity insight into redefining the Tarpon monopod design with regards to differing metocean criteria, intact/damaged conditions and soil foundation characteristics (intact/degraded). The preliminary analyses conducted showed very little correlation between the soil condition and the internal forces of the Caisson-that which will be subjected to scrutiny. Nonetheless, the difference between the metocean criteria is very clear, with a maximum forces coming from the As Designed condition, followed by PTS and finally the Joint Density; that which poses potential force/material savings. The guying system condition (whether it is fully guyed or partially guyed) also plays a primary role in determining the robustness of the Tarpon Monopod. By means of a pushover analysis, the initiating mode of failure was also determined.

The author envisions that this project can be of high use for PETRONAS as a valuable addition to their stock of literatures detailing the sensitivity of the Tarpon to varying guy and soil conditions subjected to different sets of metocean criteria. The end result expected is summarized as follows. This project delivers a comprehensive report detailing the structural response of the Tarpon Monopod when subjected to different metocean conditions,

5.1.2 Results Executive Summary

- The Tarpon Monopod has relatively low structural redundancy.
- It is a structure whose integrity is highly dependent on its guying system.

- Even one set of missing guy cable may initiate structural failure during unfavorable storm approach directions.
- Its response is vastly sensitive to different storm directions and guy wire configuration, especially in its damaged conditions (removed guy wires).
- It may survive with only two or even one guy wire pair given that the storm approach is favorable for utilizing the full capabilities of the remaining cables.
- In its freestanding mode, the Tarpon structure fails in all simulated storm conditions in this study.
- The initiating mode of failure is the anchor pile plasticity.
- The wire ropes / guy wires would not fail in tension, given that they are in good condition (no significant corrosion etc..)
- Should the guying system fail, attention should be given to its connections at the anchor piles and terminator clamp.

5.2 Way Forward

It is obvious herein that the Tarpon is not exactly the most robust available option for 70-80m water depth offshore marginal field exploitations. Although its patented design is well thought of, below are some of the finer points for further improvements to the Tarpon structure that the designer would like to consider and incorporate it in future developments;

- Increase redundancy in the Tarpon guy system – This can be achieved by increasing the stiffness of the structural caisson, say by provision of grout to a certain length ,the insertion of ring stiffeners or simply a caisson section with higher/tougher cross sectional properties.
- Improve pile capacity as to avoid plasticity. – consider different pile technologies such as steel –concrete grouted piles or suction piles, instead of conventional hollow steel piles.
- Form a dedicated inspection and maintenance system for the Tarpon platform.

- Look at alternative marginal field platform designs (minimal gravity based platform , mini floaters , etc)
- Place simple axial strain/stress monitoring gauges on each guy cables to effectively observe as to how the tension in each cables fair alongside its pre-tension of 100 kips.

5.2.1 Future undertakings

This project can be taken a step further to include a full detailed dynamic analysis of the Tarpon platform. Critics may comment that this project's scope of application is limited to Tarpon placed in the water depth range of 70m-80m , while Tarpon platforms in shallower water may well exhibit relatively different behaviours.

This would be the recommended future work that can be done on this project whereby a similar methodology is performed for Tarpons in shallower water to attempt and try to correlate the results of such platforms in varying water depths. The author hypothesizes that the differences may be insignificant and that the Tarpon's key responses are essentially the same – so long as the design seastate is not for that of breaking waves.

As the engineering practices in Malaysia slowly shifts towards one that includes the provision of seismic design, it would only be logical to perform a seismic study on the Tarpon platform. This is also one of the areas that perhaps maybe of significant importance in future Tarpon design considerations.

A detailed study using Finite Elements or the like on the anchor piles of the Tarpon platform should be performed to truly analyse and design a piling system that is most suited to the Tarpon's configuration. Along the same line, the cable terminators at the caisson should be analysed for its most optimum placement (below the water line) and design.

Another concern is that the Tarpon platform has a tendency to induce motion induced discomfort in humans. This is also another interesting area to conduct a study on.

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APPENDIX

(Note that the various tables and figures in the Appendix are not listed in the list of tables and figures due to their inherent self-explanatory headlines)

A. Unidirectional Loading Determination

B. Wind Averaging Interval Conversion

C. Wind Forces as Joint Load

D. Soil Model Data

E. Cable Pretension by Temperature Loading Calculations

F. Modelled cable effective properties

G. Caisson deflection and unity check plots

H. Wave response – DAF calculation

I. Dynpac model- Retained degrees of freedom

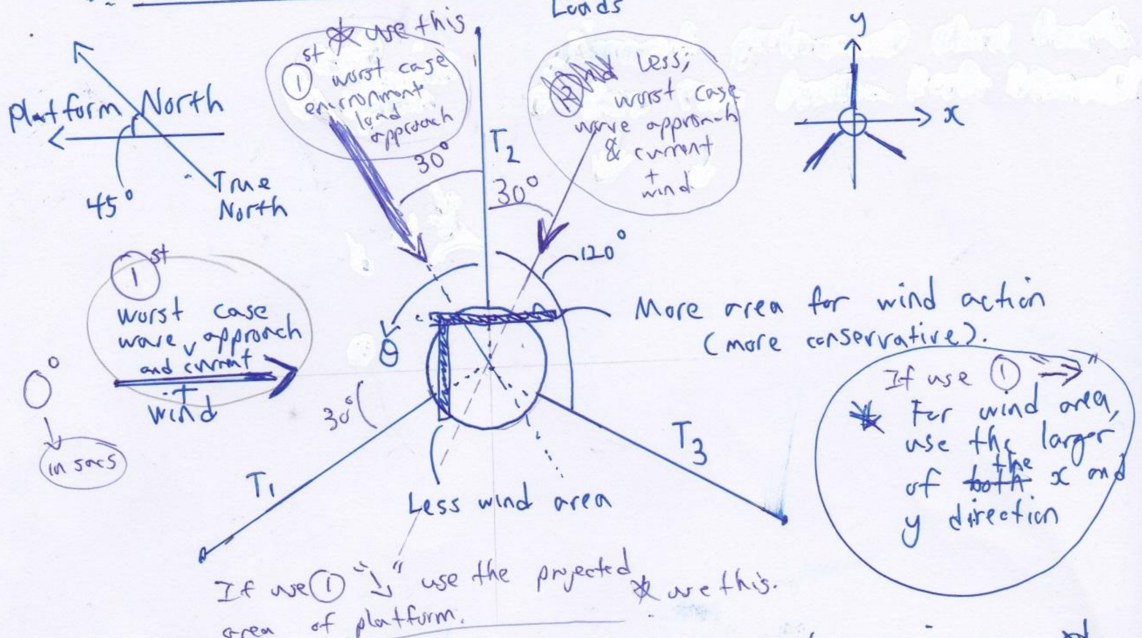
J. Manual stiffness -eigenvalue calculation

K. Guy cable reduction sequence

L. LDP-A SACS Model

A. Unidirectional Loading Determination

Derivation of Environmental Loads Approach Direction.



Exaggerated view (plan) of the caisson and its guying systems.

If $\theta = 45°$; T_1 minimal load (neutral), T_3 cannot take mainly compressive loads (slack)

$T_2 = F \cos 45 = 0.71F$

If $\theta = 90°$: ~~T_2 slack~~, $T_1 = F/\cos 30 = 1.15F$,

T_2 neutral cable (minimum load),

T_3 slack cable (cannot take compression)

If $\theta = 0°$; $T_2 = F$, T_1 and T_3 slack.

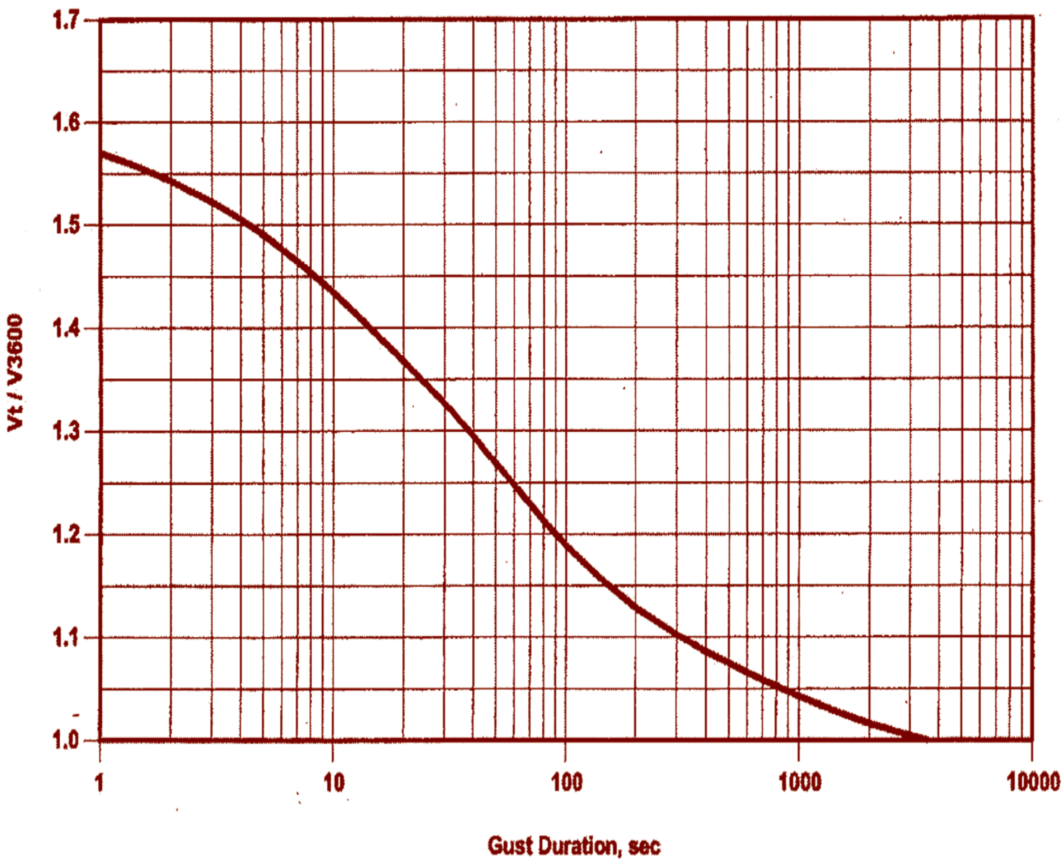
The above shows a simplistic force breakdown for the guying system, for $0 < \theta < 120°$ ~~(a quadrant)~~.

Hence the direction of approach of environmental loads are as shown in the figure above.

* USE ① from  direction

B. Wind Averaging Interval Conversion

In order to convert various wind averaging intervals to the standard 1 hour mean used for analysis in this project, conversion factors obtained from the Durst Curve will be used.



The Durst Curve, 1960 (Source: Dregger,2005)

The table below shows the wind averaged speed interval conversion to 1 hour mean.

Load Scenario	Given data (m/s)	Conversion Factor	1 hour mean (m/s)
PTS	29 (1 min)	1/1.25	23.2
Joint Density	19.9 (3 second gust)	1/1.51	13.2
As Designed	39 (10 min)	1/1.07	36.4

C. Wind Forces as Joint Load

API RP 2A -21st Edition recommends the use of the formula below for wind force calculation.

$$F = \frac{1}{2}\,\rho\, v^2\, C_s\, A$$

Where p = mass density of air taken as 1.184kg/m^3 at 25^{0}C

V = wind speed in m/s

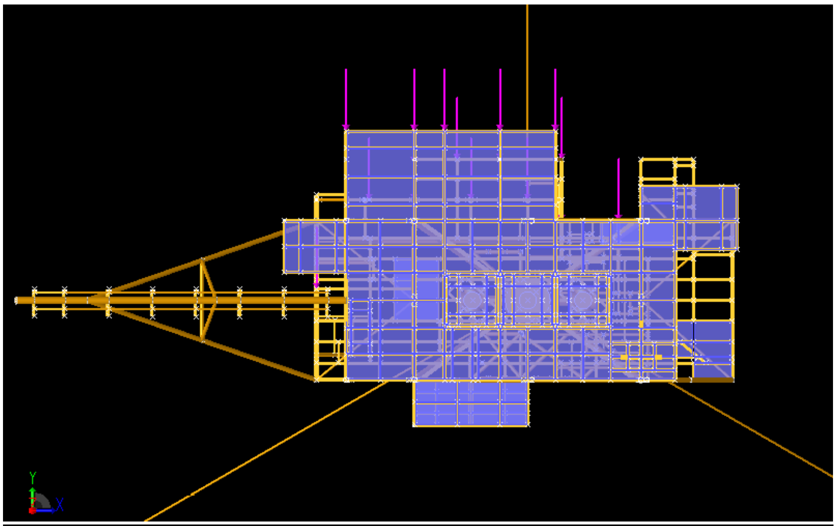
C_s = Shape factor taken as unity as per API recommendations

A = Wind area, taken as 70% of the projected platform area. *

* for a conservative analysis, the largest area of the platform will be taken.

Wind forces will act in the positive x direction towards platform south.

Load Scenario	Effective wind area (assumed 70% enclosed) (m^2)	Total Force, F (kN)	Force per joint, F/16 (kN)
PTS	18.3 x 10 = 183m^2	40.8	2.55
Joint Density	Assume 70 %	13.2	0.83
As Designed	enclosed ; A= 128m^2	100	6.25



The figure above shows the wind force on platform east face simulated as joint loads (see pink labelled arrows)

D. Soil Model Data

Stratum	Soil Description	Penetration (m) From	Penetration (m) To	Submerged Unit Weight (kN/m^3)	Design Shear Strength (kPa)	Unit Skin Friction (kPa)	Unit End Bearing unchanged (Mpa)
1	Very Soft	0.0		5.0 3.5	4 2.8	0 0	0.002
	CLAY		0.8			36 25.2	
2	Firm	0.8		7.5 5.25	25 17.5	225 157.5	0.008
	CLAY		5.9		40 28	360 252	0.022
3	Stiff	5.9		8.0 5.6	55 38.5	346.5 495 ~~347.5~~	0.026
	CLAY		11.0				0.034
4	Very Stiff	11.0		8.0 4	120 ~~60~~ 84	1080 540	0.055
	SILT		17.9				0.065
5	Very Stiff	17.9		8.0 5.6	90 63	810 567	0.056
	CLAY		25.0				0.066
6	Medium Dense	25.0		8.0 4	$\phi = 25^0$ $\phi = 20$	2340 1170	0.056
	SAND		27.2			2546 1273	0.062
7	Very Stiff	27.2		8.0 5.6	90 63	810 567	0.069
	CLAY		30.0				0.073
8	Stiff to Very Stiff	30.0		8.0 5.6	70 49	630 441	0.064
	CLAY		75.2		150 105	1350 945	0.148
9	Stiff	75.2		7.5 5.25	130 91	1170 819	0.130
	CLAY		95.3		160 112	1140 798	0.160
10	Very Stiff	95.3		8.0 5.6	160 112	1140 798	0.160
	CLAY		130.0		165 115.5	1485 1039.5	0.165
11	Hard	130.0		7.8 5.46	165 115.5	1485 1039.5	0.165
	CLAY		150.0		246 172.2	2214 1549.8	0.246

* "Blue" (colour) marks the modification to the data for "bad" soil.

TABULATION OF DESIGN PARAMETERS
BH ANOA-L1
ANOA LEDANG LOCATION

E. Cable Pretension by Temperature Loading Calculations

There is a need for alternative pretension modelling for the guy cable elements in SACS after attempts to simulate it via the normal MEMB2 line failed. Hence, a negative temperature loading is simulated on the cables which will induce a certain strain, calculated to produce a reasonable estimate of the 100 kips/444.8kN pre tension.

Assumptions:

- Linear thermal expansion
- Thermal expansion coefficient, $\alpha = 0.000012$ (typical for steel)

Given parameters:

- $E = 9652.660$ kN/cm^2
- $A_{eff} = 48.94$ cm^2
- Dimensions per cable: 101.6mm diameter × 50.79mm thick

E= Stress/Strain

Strain = (444.8kN ÷ 48.94 cm^2) / 9652.660 kNcm^{-2}

= 9.416×10^{-4}

Strain = Δ_{length} / L

$\Delta_{length} = 9.416 \times 10^{-4}$ (12924)

= 12.169cm

$\Delta_{length} = L_0\alpha\,(t_{final} - t_{initial})$

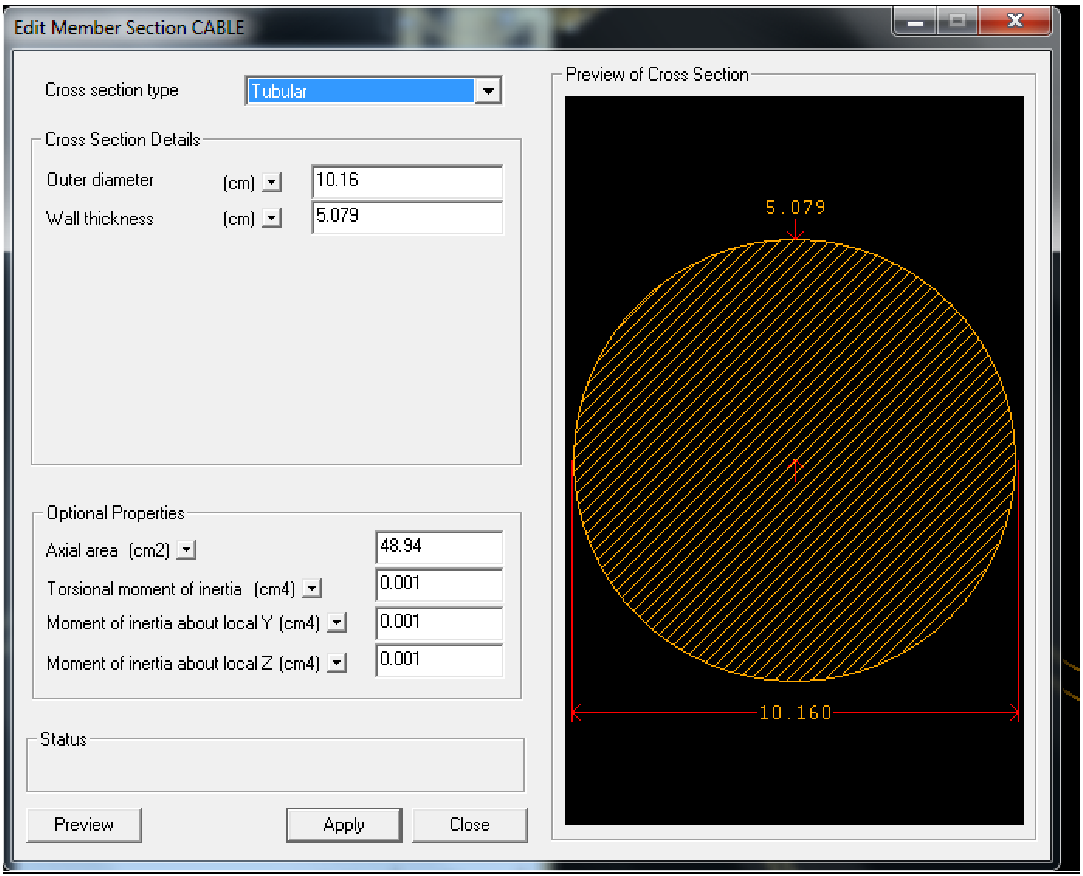
For simplicity, let $(t_{final} - t_{initial}) = \Delta_{temp}$

$12.169 = 12924\,(0.000012)\,\Delta_{temp}$

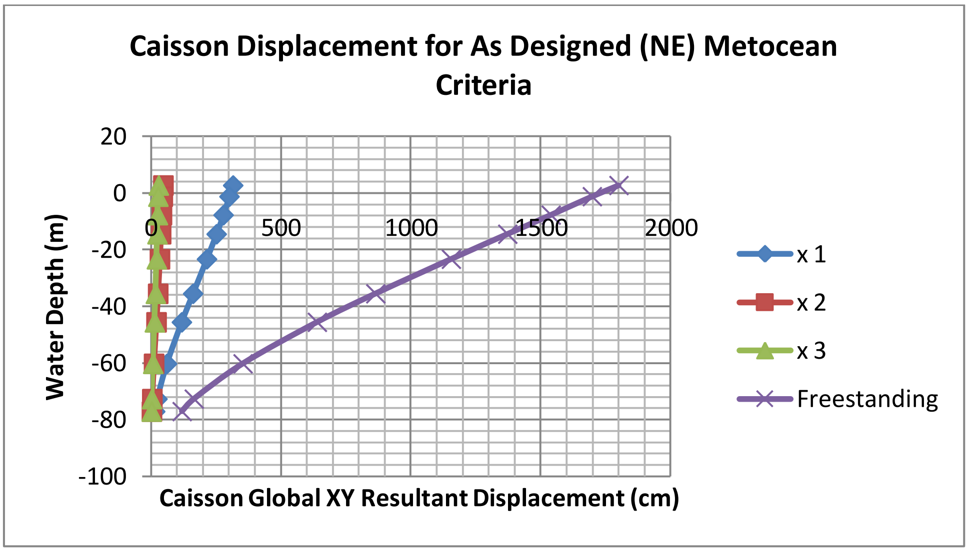
$\Delta_{temp} = 78.47\ ^oC + 10\ ^oC$ (to ensure no under estimation, after trial and error runs on SACS software) = -88.47^{o}C * - the same is repeated for the slightly differing cable lengths , hence the slight variation in the SACS temperature input.

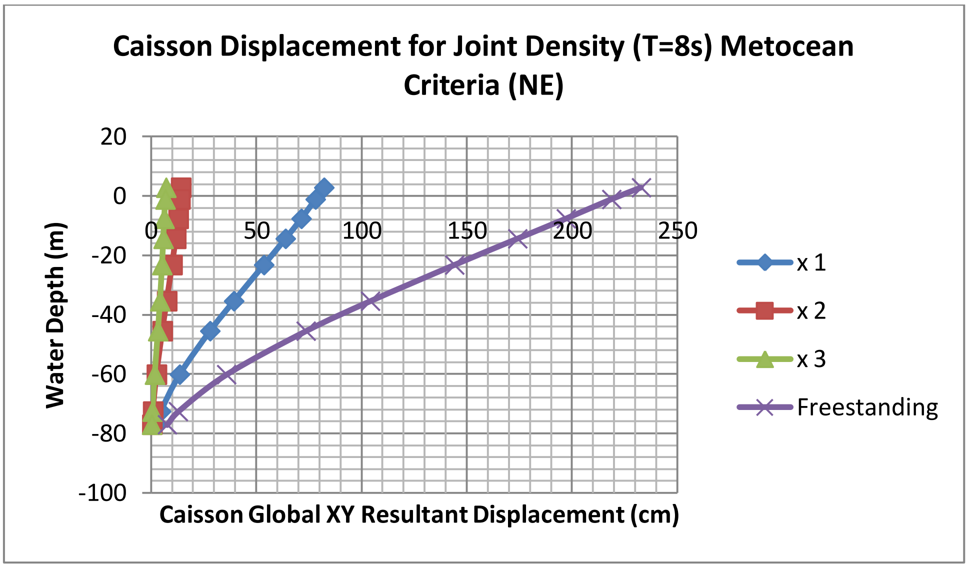
*the negative sign is to highlight contraction due to simulated temperature reduction.

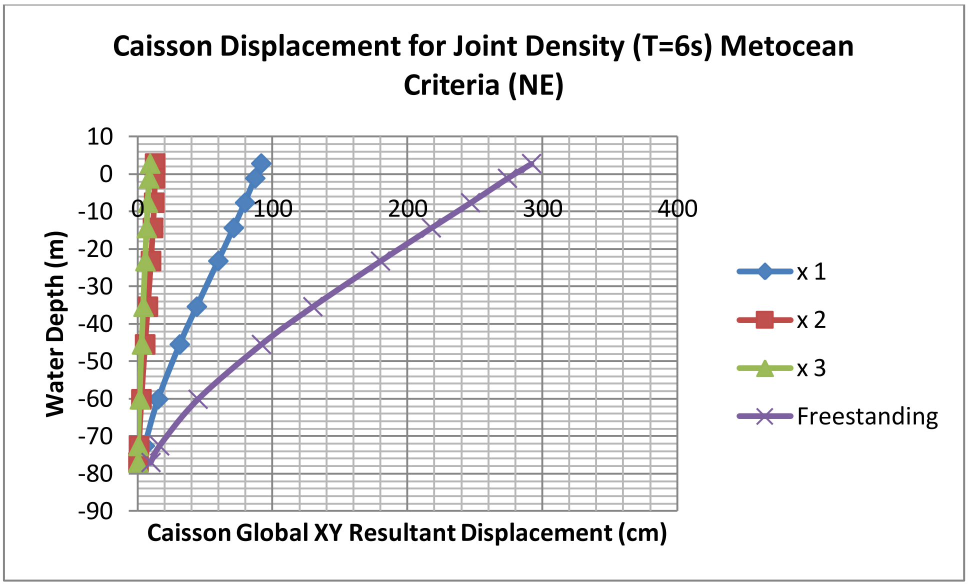
F. Modelled cable effective properties

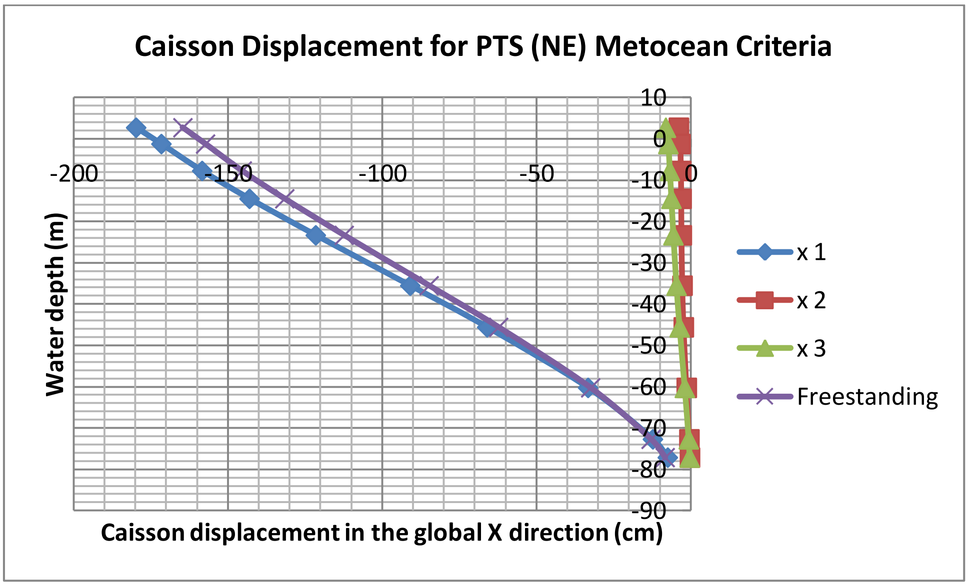


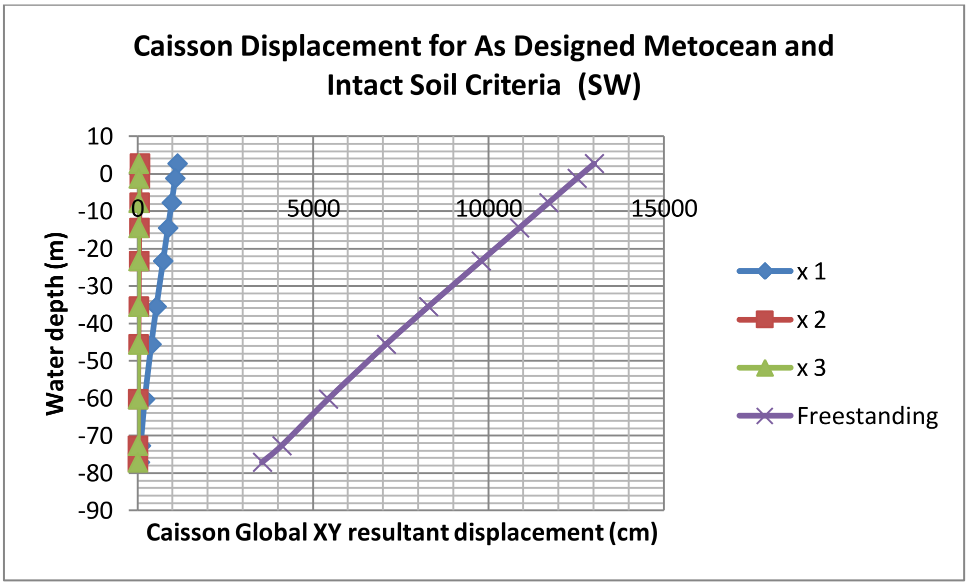
G. Caisson deflection and unity check plots

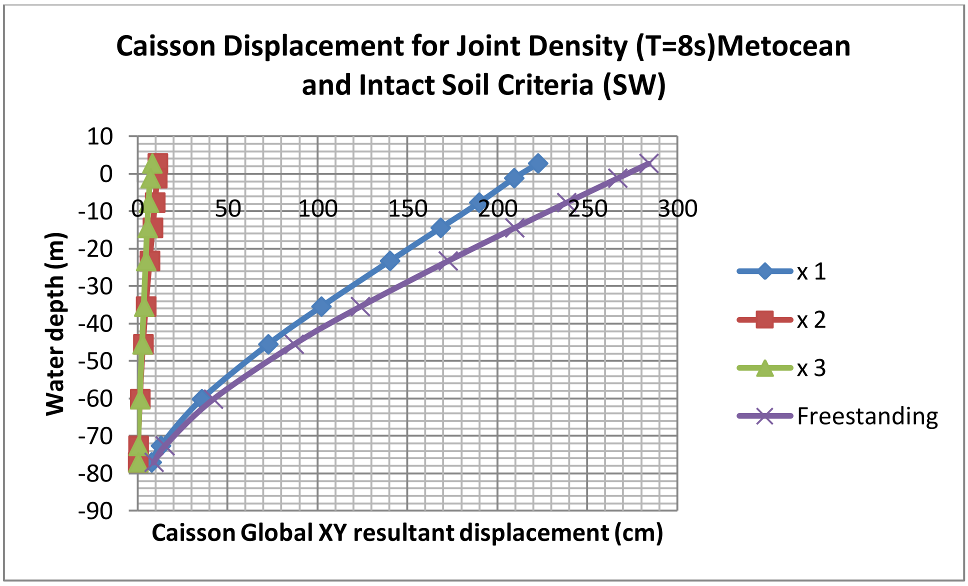


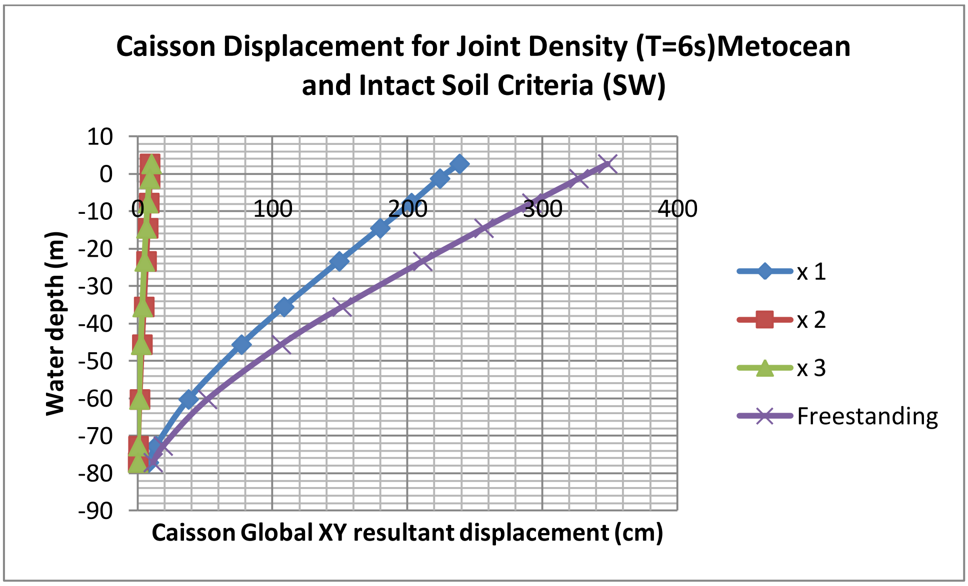




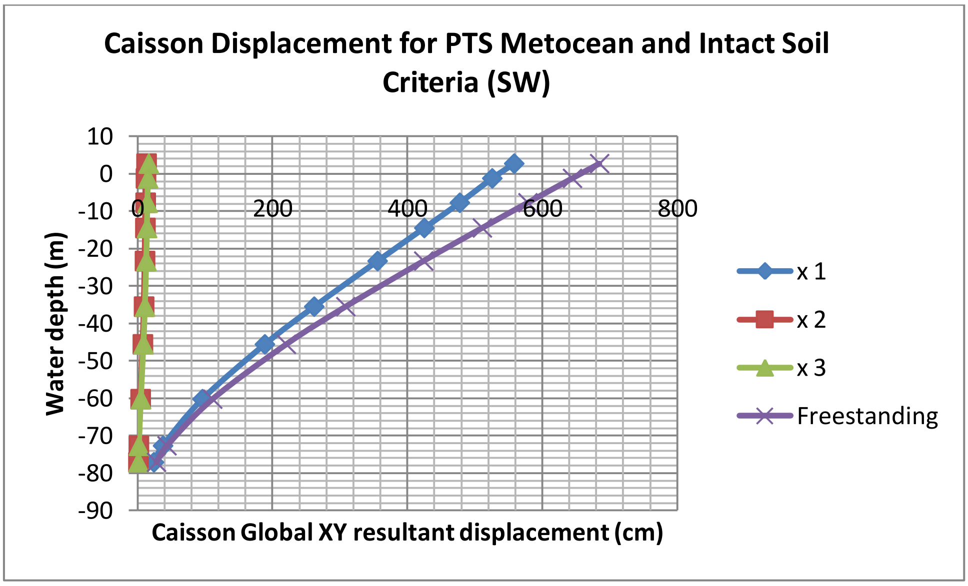

Caisson Displacement for PTS (NE) Metocean Criteria
Water depth (m)
Caisson displacement in the global X direction (cm)
-200
-150
-100
-50
0
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
x 1
x 2
x 3
Freestanding


Caisson Displacement for As Designed Metocean and Intact Soil Criteria (SW)
Water depth (m)
Caisson Global XY resultant displacement (cm)
0
5000
10000
15000
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
x 1
x 2
x 3
Freestanding

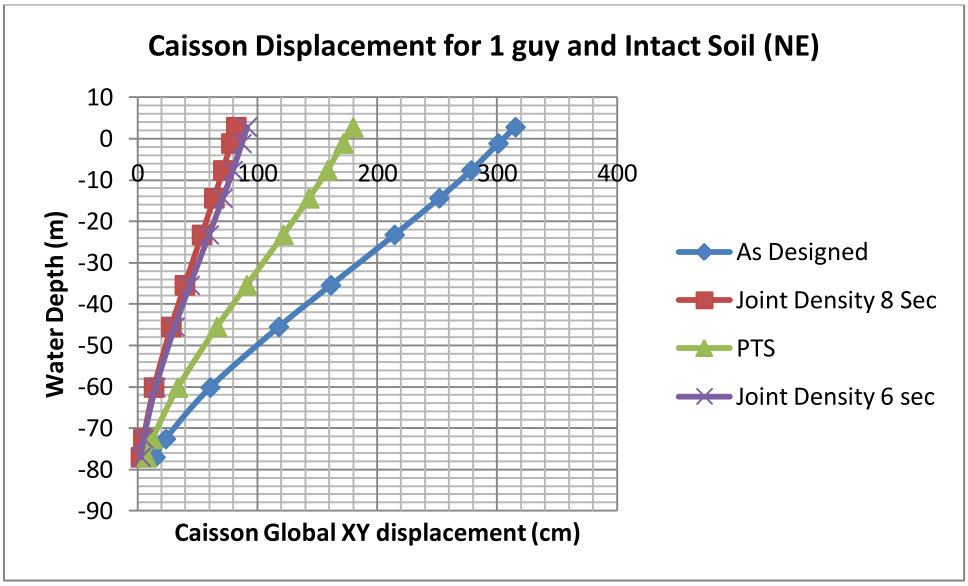

Caisson Displacement for Joint Density (T=8s)Metocean and Intact Soil Criteria (SW)
Water depth (m)
Caisson Global XY resultant displacement (cm)
0
50
100
150
200
250
300
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
x 1
x 2
x 3
Freestanding



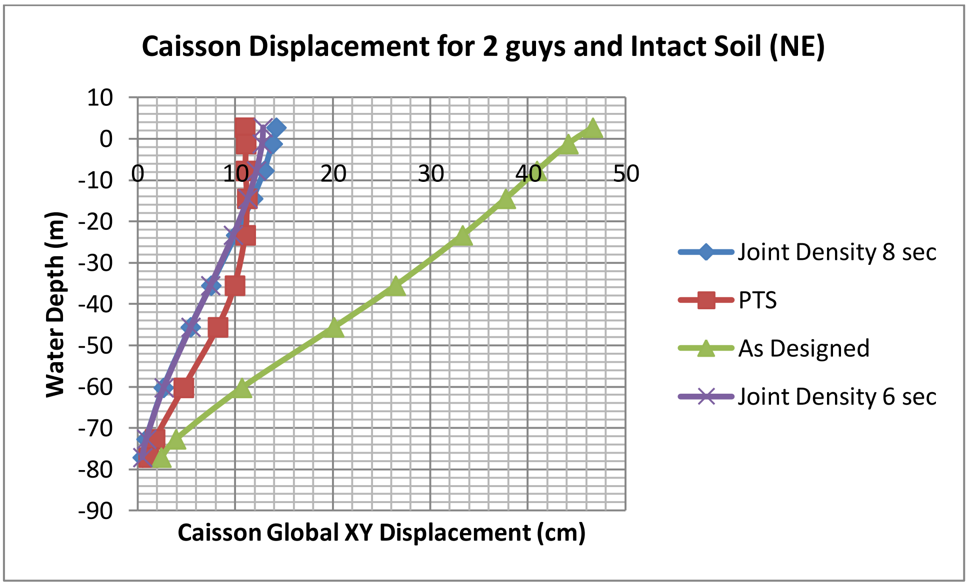
Caisson Displacement for Joint Density (T=6s)Metocean and Intact Soil Criteria (SW)
Water depth (m)
Caisson Global XY resultant displacement (cm)
x 1
x 2
x 3
Freestanding

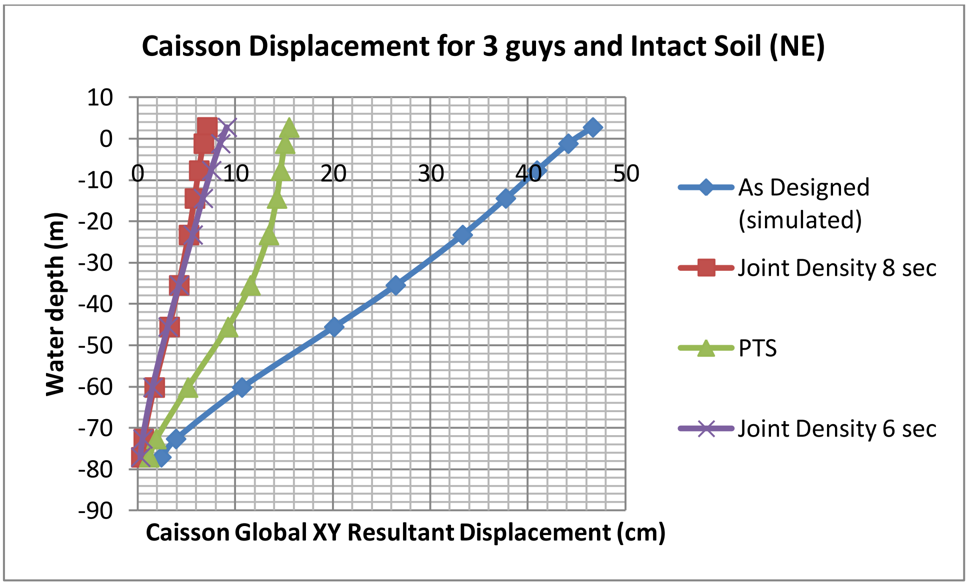


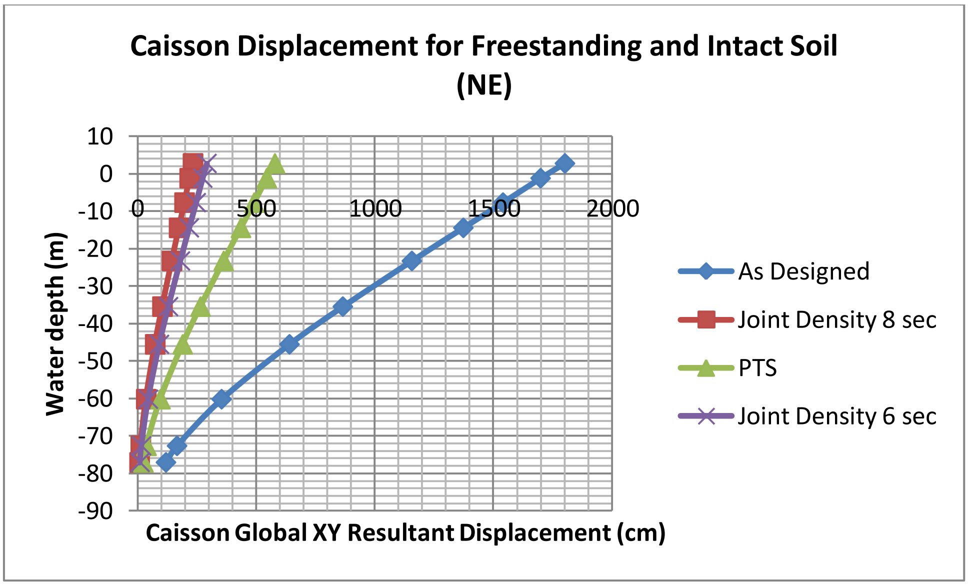
Caisson Displacement for PTS Metocean and Intact Soil Criteria (SW)
Water depth (m)
Caisson Global XY resultant displacement (cm)
x 1
x 2
x 3
Freestanding

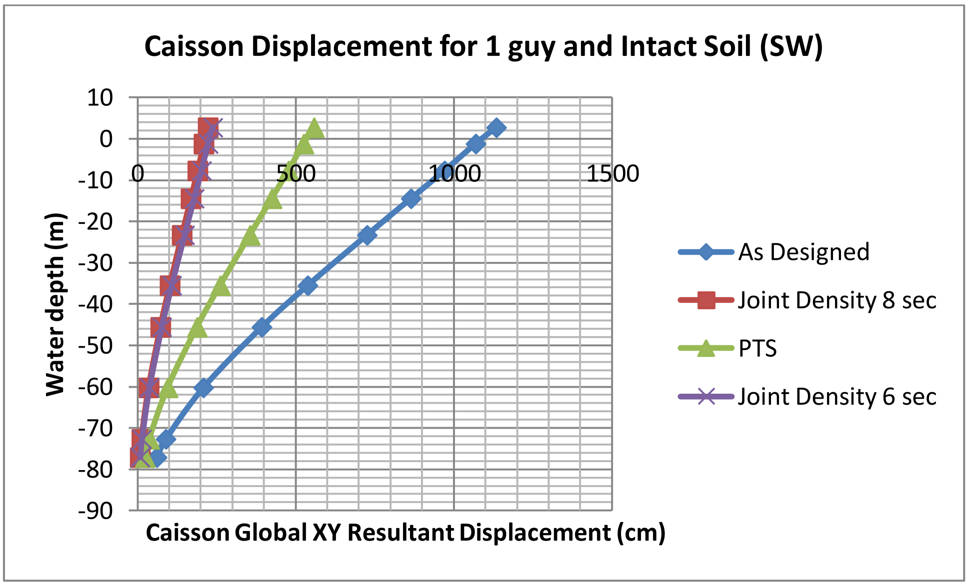


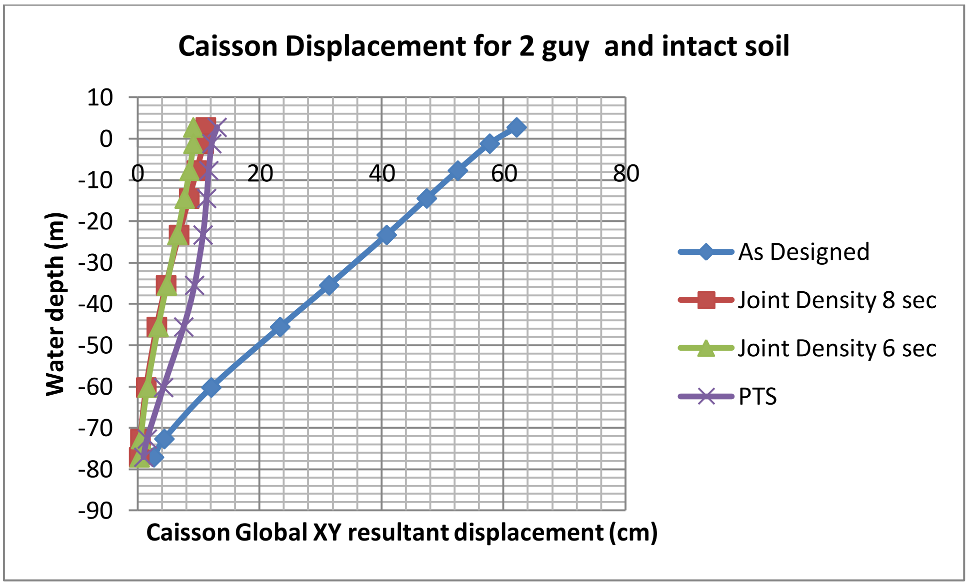
Caisson Displacement for 1 guy and Intact Soil (NE)
Water Depth (m)
Caisson Global XY displacement (cm)
As Designed
Joint Density 8 Sec
PTS
Joint Density 6 sec

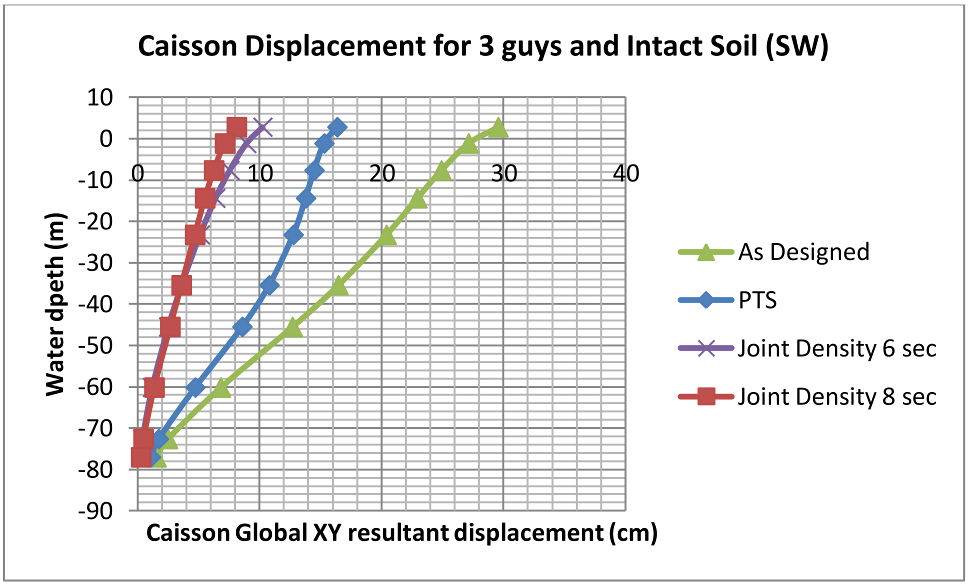

Caisson Displacement for 2 guys and Intact Soil (NE)
Water Depth (m)
Caisson Global XY Displacement (cm)
Joint Density 8 sec
PTS
As Designed
Joint Density 6 sec

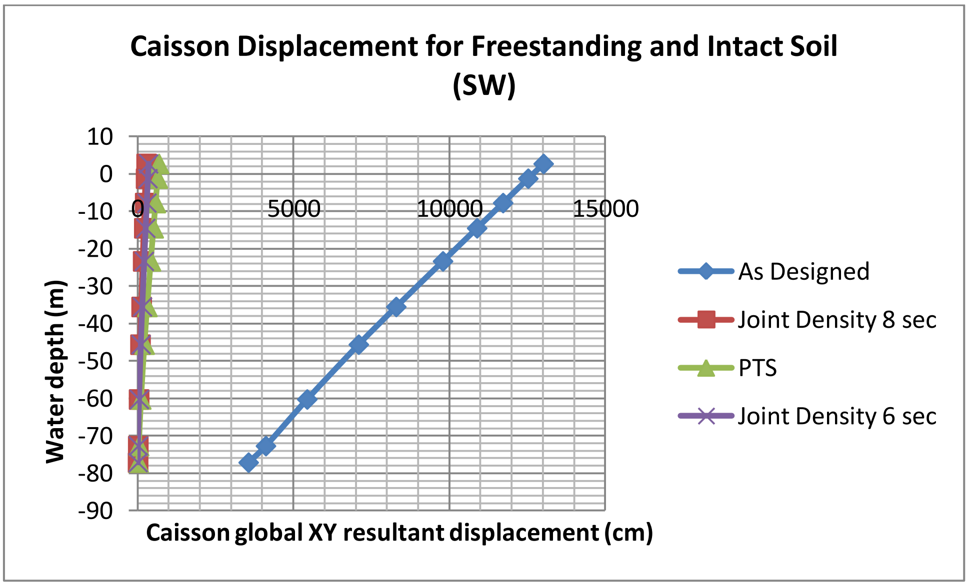

Caisson Displacement for 3 guys and Intact Soil (NE)
Water depth (m)
Caisson Global XY Resultant Displacement (cm)
As Designed (simulated)
Joint Density 8 sec
PTS
Joint Density 6 sec

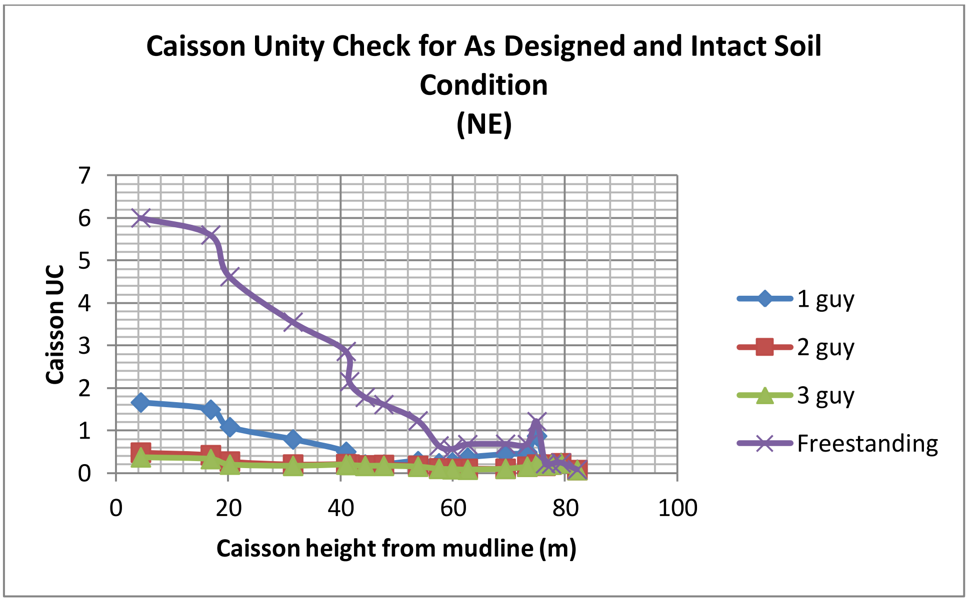

Caisson Displacement for Freestanding and Intact Soil (NE)
Water depth (m)
Caisson Global XY Resultant Displacement (cm)
As Designed
Joint Density 8 sec
PTS
Joint Density 6 sec

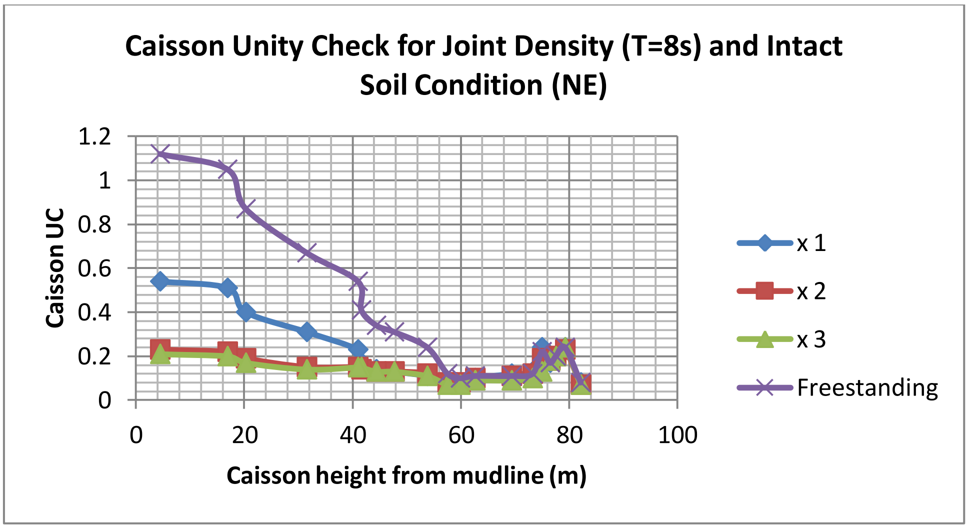

Caisson Displacement for 1 guy and Intact Soil (SW)
Water depth (m)
Caisson Global XY Resultant Displacement (cm)
As Designed
Joint Density 8 sec
PTS
Joint Density 6 sec

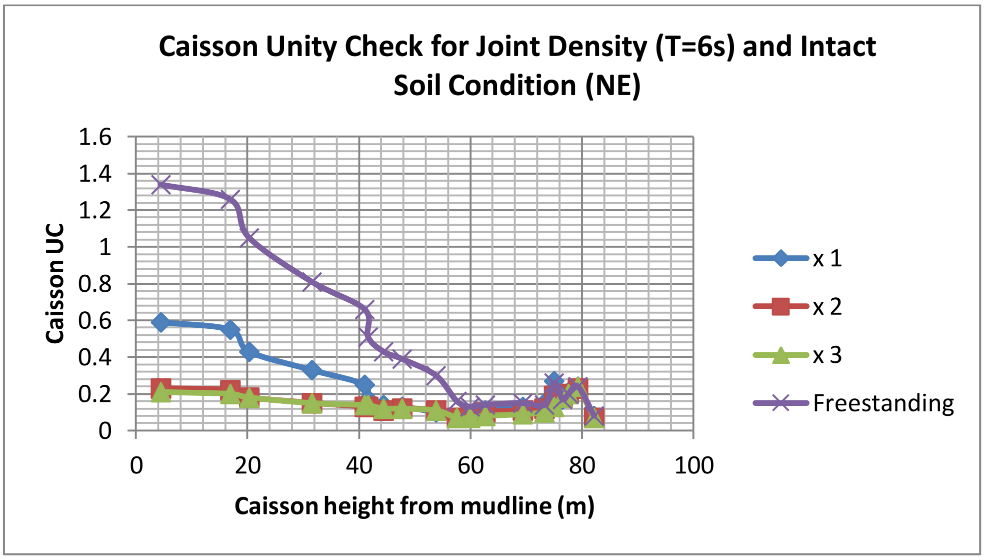

Caisson Displacement for 2 guy and intact soil
Water depth (m)
Caisson Global XY resultant displacement (cm)
As Designed
Joint Density 8 sec
Joint Density 6 sec
PTS

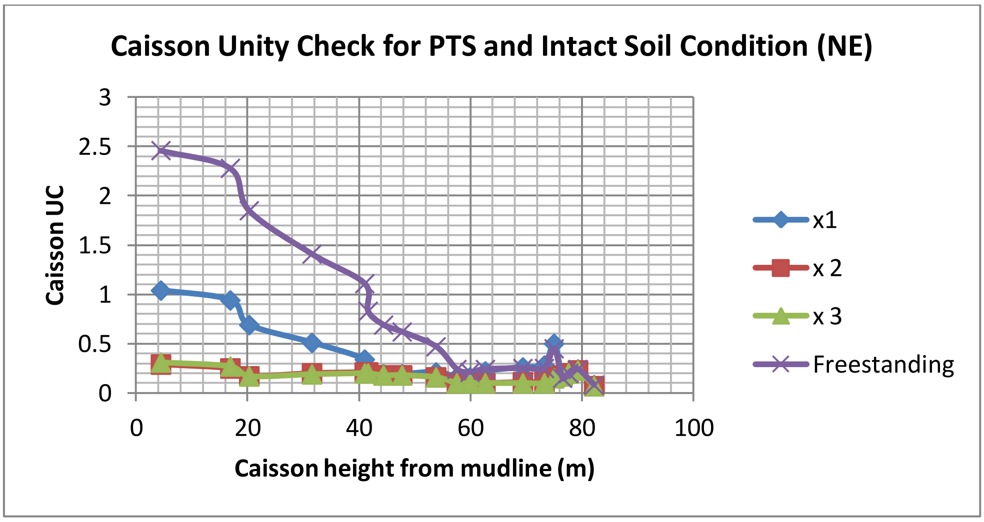

Caisson Displacement for 3 guys and Intact Soil (SW)
Water dpeth (m)
Caisson Global XY resultant displacement (cm)
As Designed
PTS
Joint Density 6 sec
Joint Density 8 sec

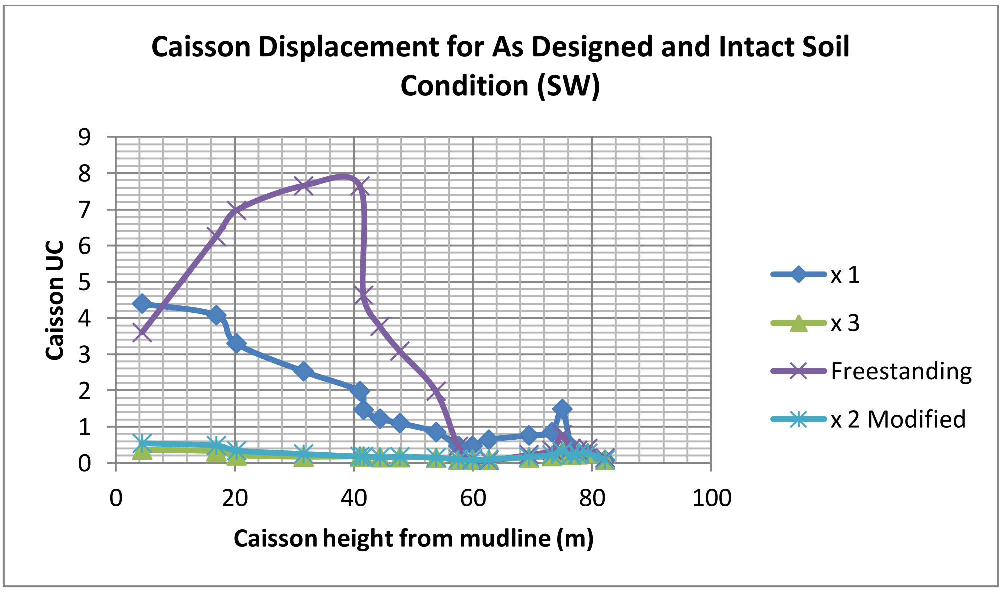

Caisson Displacement for Freestanding and Intact Soil (SW)
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
0
5000
10000
15000
Water depth (m)
Caisson global XY resultant displacement (cm)
As Designed
Joint Density 8 sec
PTS
Joint Density 6 sec

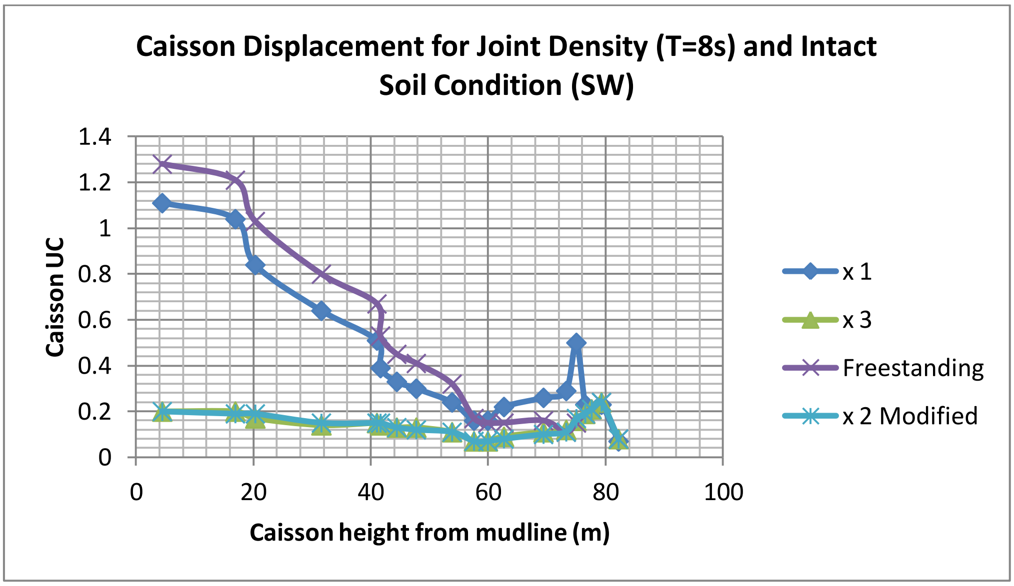

Caisson Unity Check for As Designed and Intact Soil Condition (NE)
7
6
5
4
3
2
1
0
0
20
40
60
80
100
Caisson UC
Caisson height from mudline (m)
1 guy
2 guy
3 guy
Freestanding

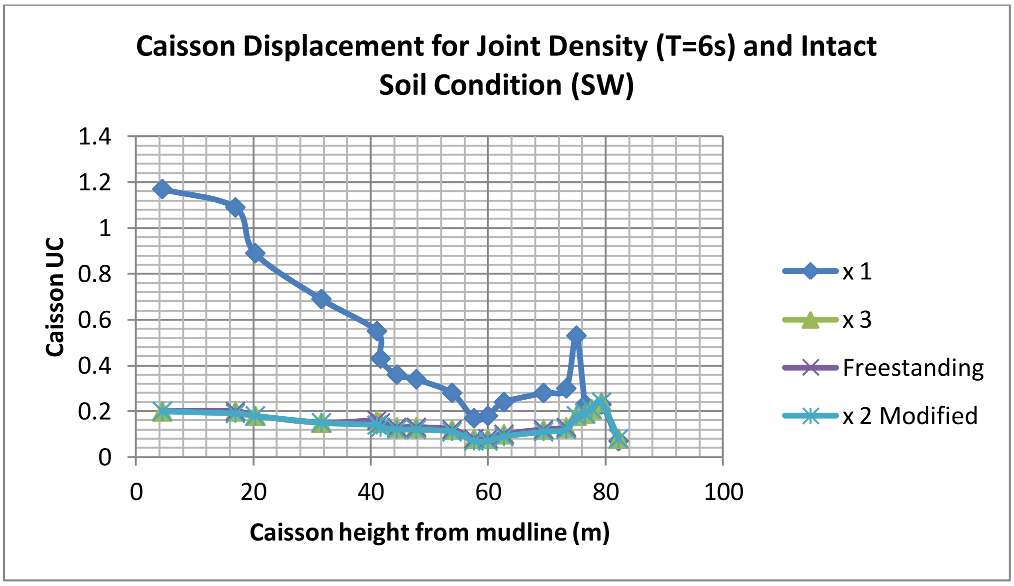

Caisson Unity Check for Joint Density (T=8s) and Intact Soil Condition (NE)
1.2
1
0.8
0.6
0.4
0.2
0
0
20
40
60
80
100
Caisson UC
Caisson height from mudline (m)
x 1
x 2
x 3
Freestanding

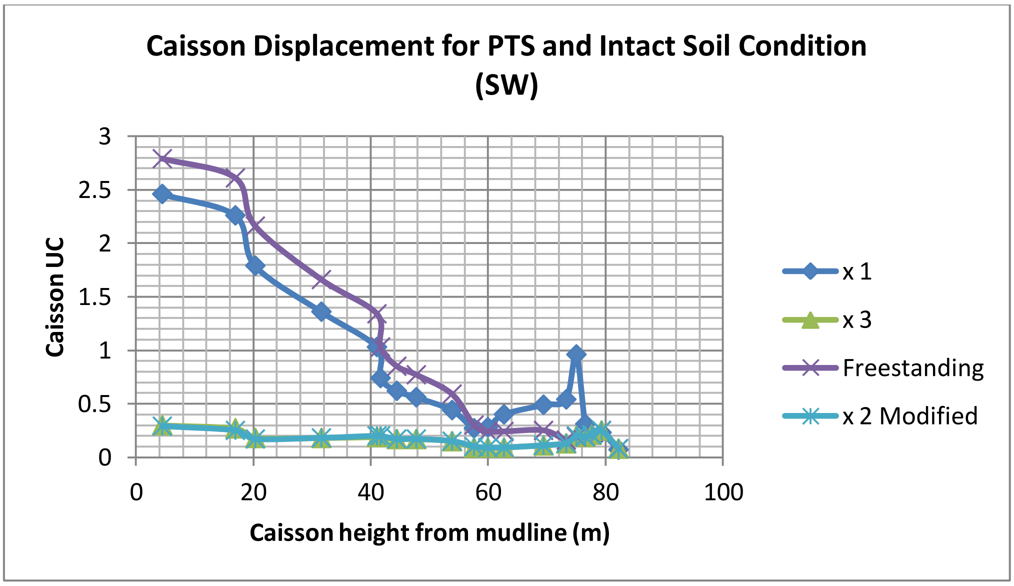

Caisson Unity Check for Joint Density (T=6s) and Intact Soil Condition (NE)
Caisson UC
Caisson height from mudline (m)
x 1
x 2
x 3
Freestanding


Caisson Unity Check for PTS and Intact Soil Condition (NE)
Caisson UC
Caisson height from mudline (m)
x1
x 2
x 3
Freestanding


Caisson Displacement for As Designed and Intact Soil Condition (SW)
Caisson UC
Caisson height from mudline (m)
x 1
x 3
Freestanding
x 2 Modified


Caisson Displacement for Joint Density (T=8s) and Intact Soil Condition (SW)
Caisson UC
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
20
40
60
80
100
Caisson height from mudline (m)
x 1
x 3
Freestanding
x 2 Modified


Caisson Displacement for Joint Density (T=6s) and Intact Soil Condition (SW)
Caisson UC
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
20
40
60
80
100
Caisson height from mudline (m)
x 1
x 3
Freestanding
x 2 Modified


Caisson Displacement for PTS and Intact Soil Condition (SW)
Caisson UC
3
2.5
2
1.5
1
0.5
0
0
20
40
60
80
100
Caisson height from mudline (m)
x 1
x 3
Freestanding
x 2 Modified

H. Wave response – DAF calculation

Comparison of dynamic and static mudline moments for DAF calculation (for fixed assumption)

Guying system	Metocean criteria	Max dynamic Moment X (kN-m)	Max static moment X (kN-m)	Dynamic Amplification Factor (DAF)	Remarks *
Freestanding	240 deg				
	PTS	44184.3	47055.9	0.94	NC – 23.499%
	AD	156180.2	79900.1	1.96	NC – 157.065%
	JD	20243.1	22543.9	0.90	NC – 35.984%
	90 deg				
	PTS	-56958.0	-55371.7	1.03	NC – 44.122%
	AD	-242160.5	-93488.0	2.59	NC-495.934%
	JD	-23707.0	-26568.7	0.89	CA
x 1	240 deg				
	PTS	Infinity	48502.8	Infinity	NC - infinity %
	AD	87035.0	82233.8	1.06	CA
	JD	112411.8	23139.3	48.58	NC
	90 deg				
	PTS	-60539.8	-57016.9	1.06	CA
	AD	-101371.2	-97309.1	1.04	CA
	JD	-31765.1	-27352.0	1.16	CA
x 2	240 deg				
	PTS	57858.4	54333.9	1.07	AC – 1.229%
	AD	97428.7	91443.3	1.07	AC – 7.158%
	JD	27723.9	25252.2	1.10	CA
	90 deg				
	PTS	-65713.3	-61921.7	1.06	CA
	AD	-119733.3	-104653.7	1.14	AC -4.561%
	JD	-31879.7	-29090.4	1.10	CA
X 3	240 deg				
	PTS	59279.6	57017.6	1.04	CA
	AD	103019.8	95942.4	1.07	CA
	JD	28304.0	26351.1	1.07	CA
	90 deg				
	PTS	-69597.0	-66843.7	1.04	CA
	AD	-117846.6	-112018.7	1.05	CA
	JD	-33280.5	-30837.3	1.08	CA

* NC – Not converged , CA – Convergence achieved , AC – Acceptable convergence margin

Comparison of dynamic and static mudline moments for DAF calculation (with pile superelement)

Guying system	Metocean criteria	Max dynamic Moment X (kN-m)	Max static moment X (kN-m)	Dynamic Amplification Factor (DAF)	Remarks *
Freestanding Good soil	240 deg PTS AD JD	 22336.1 29560.4 8364.7	 47194.2 80.154.3 22603.3	 0.47 0.37 0.37	 CA CA CA
	90 deg PTS AD JD	 -26751.8 -35315.2 -9800.8	 -55532.3 -93780.8 -26637.5	 0.48 0.38 0.37	 CA CA CA
Freestanding Bad soil	240 deg PTS AD JD	-	-	-	-
	90 deg PTS AD JD	-	-	-	-
x 1 Good soil	240 deg PTS AD JD	 50640.1 Infinity 22742.9	 48309.5 81910.2 23019.4	 1.05 Infinity 0.99	 CA NC – infinity % CA
	90 deg PTS AD JD	 -63192.9 -114227.7 -27392.2	 -56761.1 -96855.8 -27199.3	 1.11 1.18 1.01	 AC – 1.331% CA CA
x 1 Bad soil	240 deg PTS AD JD	 50599.3 88260.9 23565.8	 48286.1 81866.3 23003.0	 1.05 1.08 1.03	 CA CA CA
	90 deg PTS AD JD	 -62493.1 -116095.4 -28548.6	 -56722.6 -96794.9 27175.9	 1.10 1.20 1.05	 CA CA CA
X 2 Good soil	240 deg PTS AD JD	 64690.1 140201.6 32058.8	 54125.8 91090.7 25122.2	 1.20 1.54 1.28	 AC – 6.694% NC – 15.98% CA
	90 deg PTS AD JD	 -70218.2 7020722.5 -36642.2	 -61587.9 -104076.8 -28891.7	 1.14 67.46 1.27	 CA NC – 4192% CA
X 2 Bad soil	240 deg PTS AD JD	 66907.7 106655.6 27089.6	 54074.6 91001.7 25090.9	 1.24 1.17 1.08	 NC – 10.095 % CA NC – 11.891 %

	90 deg PTS AD JD	 --59765.2 -149009.0 -36842.2	 -61518.6 -103964.2 -28850.6	 0.97 1.43 1.28	 NC – 14.067% NC -16.745% CA
X 3 Good soil	240 deg PTS AD JD	 62574.8 107392.9 29910.7	 56677.1 95347.3 26150.1	 1.10 1.13 1.14	 CA CA CA
	90 deg PTS AD JD	 -73374.6 -122571 -34758.9	 -66438.1 -111336.1 -30603.3	 1.10 1.10 1.14	 CA CA CA
X 3 Bad soil	240 deg PTS AD JD	 63096.4 105848.3 30392.0	 56614.9 95239.5 26113.3	 1.12 1.11 1.16	 CA CA CA
	90 deg PTS AD JD	 -71284.6 -123768.3 -35444.3	 -66366.5 -111218.9 -30562.8	 1.07 1.11 1.16	 CA CA CA

DAF calculation

Guying system	Metocean criteria	DAF from table 3	DAF from table 4	Calculated DAF for each case
Freestanding Good soil	240 deg PTS AD JD	 0.94 NC 1.96 NC 0.90 NC	 0.47 0.37 0.37	 Avg = 0.71 <1.0 , hence use 1.0 Avg = 1.17 Avg = 0.64 < 1.0 , hence use 1.0
	90 deg PTS AD JD	 1.03 NC 2.59 NC 0.89	 0.48 0.38 0.37	 Avg = 0.755 <1.0 , hence use 1.0 Avg = 1.49 Avg = 0.63 <1.0 , hence use 1.0
Freestanding Bad soil	240 deg PTS AD JD	 0.94 1.96 0.90	-	 0.94 < 1.0, hence use 1.0 1.96 >1.8 , hence use 1.8 0.9 <1.0 , hence use 1.0
	90 deg PTS AD JD	 1.03 2.59 0.89	-	 1.03 2.59 > 1.8, hence use 1.8 0.89 <1.0 , hence use 1.0
x 1 Good soil Ok	240 deg PTS AD JD	 Infinity NC 1.06 48.58	 1.05 Infinity NC 0.99	 Ignore outlier, hence use 1.05 Ignore outlier , hence use 1.06 Ignore outlier, hence use 1.0

		NC		
	90 deg PTS AD JD	 1.06 1.04 1.16	 1.11 1.18 1.01	 Avg = 1.09 Avg = 1.11 Avg = 1.09
x 1 Bad soil ok	240 deg PTS AD JD	 Infinity NC 1.06 48.58 NC	 1.05 1.08 1.03	 Ignore outlier, hence use 1.05 Avg = 1.07 Ignore outlier, hence use 1.03
	90 deg PTS AD JD	 1.06 1.04 1.16	 1.10 1.20 1.05	 Avg = 1.08 Avg = 1.12 Avg = 1.11
X 2 Good soil ok	240 deg PTS AD JD	 1.07 1.07 1.10	 1.20 1.54 NC 1.28	 Avg = 1.14 Avg = 1.31 Avg = 1.19
	90 deg PTS AD JD	 1.06 1.14 1.10	 1.14 67.46 NC 1.27	 Avg = 1.10 Ignore outlier, hence use 1.14 Avg = 1.19
X 2 Bad soil	240 deg PTS AD JD	 1.07 1.07 1.10	 1.24 NC 1.17 1.08 NC	 Avg = 1.16 Avg = 1.12 Avg = 1.09
	90 deg PTS AD JD	 1.06 1.14 1.10	 0.97 NC 1.43 NC 1.28	 Avg = 1.02 Avg = 1.29 Avg = 1.19
X 3 Good soil	240 deg PTS AD JD	 1.04 1.07 1.07	 1.10 1.13 1.14	 Avg = 1.07 Avg = 1.10 Avg = 1.11
	90 deg PTS AD JD	 1.04 1.05 1.08	 1.10 1.10 1.14	 Avg = 1.07 Avg = 1.08 Avg = 1.11
X 3 Bad soil	240 deg PTS AD JD	 1.04 1.07 1.07	 1.12 1.11 1.16	 Avg = 1.08 Avg = 1.09 Avg = 1.12
	90 deg PTS AD JD	 1.04 1.05 1.08	 1.07 1.11 1.16	 Avg = 1.06 Avg = 1.08 Avg = 1.12

Filtered DAF values

	Freestanding	X 1	X 2	X 3
PTS (240 deg)	1.00	1.05	1.15	1.08
AD (240 deg)	1.49	1.07	1.22	1.10

JD (240 deg)	1.00	1.06	1.14	1.12
JD6 (240 deg)	1.00	1.00	1.05	1.21
PTS (90 deg)	1.02	1.09	1.06	1.07
AD (90 deg)	1.65	1.12	1.22	1.08
JD (90 deg)	1.00	1.10	1.19	1.12
JD6 (90 deg)	1.00	1.00	1.11	1.23

Additional metocean criterion – Joint Density with wave period 6 seconds

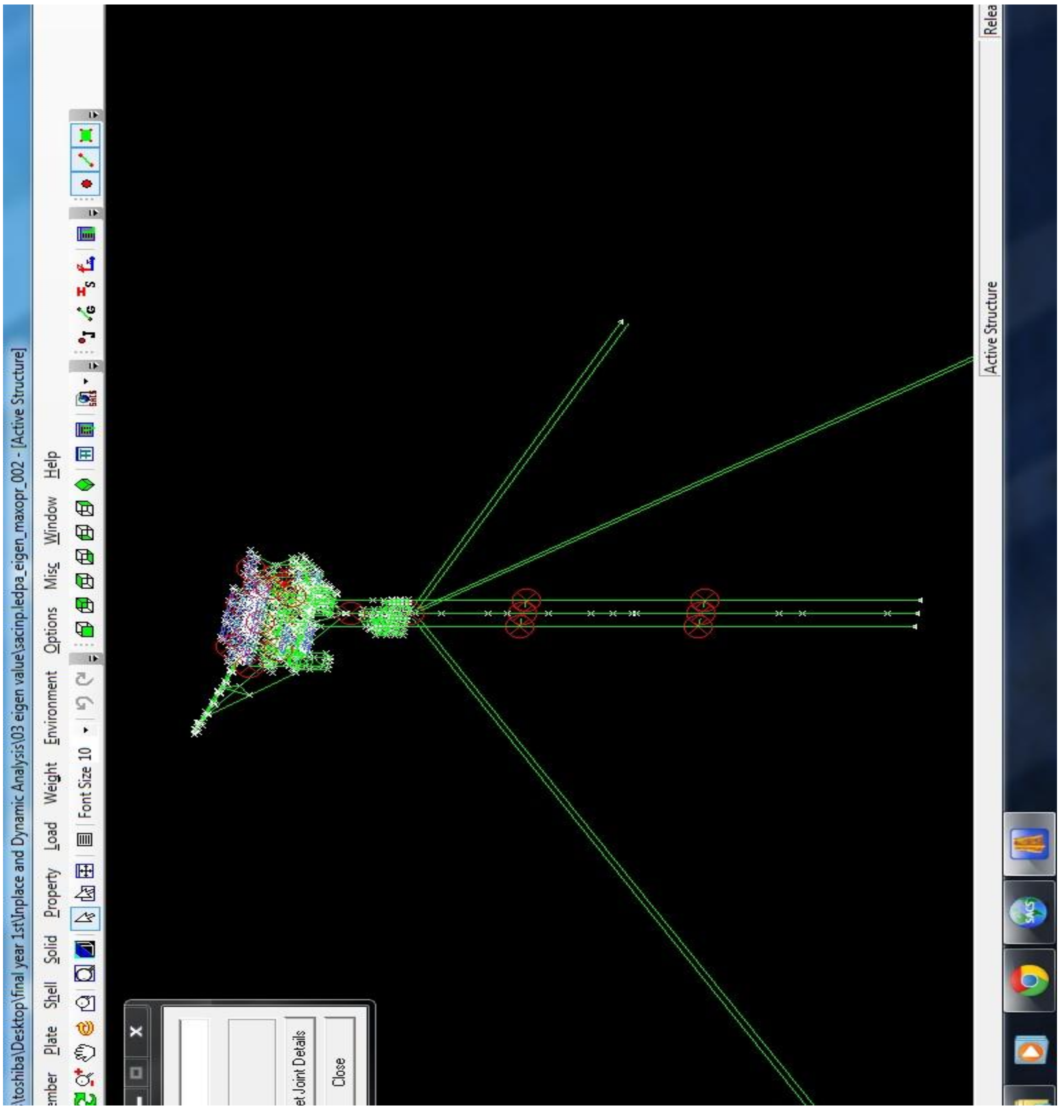
Guying system	Metocean criteria	Max dynamic Moment X (kN-m)	Max static moment X (kN-m)	Dynamic Amplification Factor (DAF)	Remarks *
1 guy good soil	JD 6 sec T 240 deg	22742.9	23019.4	1.00	Less than unity, use 1.0 , CA
1 guy good soil	JD 6 sec T 90 deg	-27392.2	-27199.3	1.00	CA
2 guy good soil	JD 6 sec T 240 deg	30960.3	29588.0	1.05	CA
2 guy good soil	JD 6 sec T 90 deg	-38569.2	-34591.5	1.11	CA
3 guy good soil	JD 6 sec T 240 deg	36769.6	30372.7	1.21	CA
3 guy good soil	JD 6 sec T 90 deg	-44421.1	-35972.4	1.23	CA
Freestanding good soil	JD 6 sec T 240 deg	8562.8	27626.1	0.31	Less than unity, use 1.0 , CA
Freestanding good soil	JD 6 sec T 90 deg	-9987.6	-32935.7	0.30	Less than unity, use 1.0 ,CA

‘Frequency domain methods may be used for extreme wave response analysis to calculate the dynamic amplification factor to combine with the static load, provided linearization of the drag force can be justified; for guyed towers, both the drag force and non-linear guyline stiffness would require linearization….’ **API RP 2A WSD**

It can be seen that the fully guyed condition (x 3) possesses the least DAF values , which can be rationalized by the fact that an intact Tarpon is stiffer than that of its cable reduced models. A stiffer structure will come with it, a higher first mode frequency, hence reducing its dynamic response to the waves investigated in this study (hence the lower DAF values). The freestanding model captured DAFs of less than unity, which maybe a result of incorrect modelling in terms of reducing the cables or a result of the fact that the freestanding caisson has a large 1st mode period of 16 seconds, which makes it highly compliant ,therefore effectively reducing the structural stresses. This would also help explain the DAF resulting from the As Designed load case, which has the highest wave period (closest to the high natural period of the freestanding Tarpon) amongst all the metocean criterions. However, with regards to the author's DAF data filtering/selection criteria, a value of 1.0 is taken for all such cases for added conservativeness. Also, it is worth noting that where the Joint Density wave parameters induce a slightly greater DAF than the PTS- This is perhaps due to the JD's wave period which is closer to that of the first natural period of vibration for the platform.

Other than the above, there are no clear observed trends / relationships between the DAF values for different the soil-structure models. This first level dynamic response estimation may not be viable in describing the actual dynamic characteristics of the platform, as the cables in this study are modelled as rigid beam elements. Nonetheless, for the scope of this project, the values as showcased in column 5 of Table 5 will be applied to the linear static in place analysis utilizing nonlinear wave theories to obtain the Tarpon's structural response to a certain degree of conservativeness.

I. Dynpac Model– Retained degrees of freedom



J. Manual Stiffness - Eigenvalue Calculation

For a fully guyed Tarpon, the calculated global stiffness k = 2400 kN/m

For a freestanding Tarpon, the calculated global stiffness k = 92 kN/m

Formulas used herein;

Natural cyclic frequency, w = $\sqrt{\frac{k}{m}}$

And $w = 2\pi f$

Assumptions herein;

The lumped mass formula to be used shall consist of the sum of the half mass of the substructure (m_s) above the mudline and the documented topside mass (m_d).

Given $m_d = 184.8$ t and $m_s = [1 – (34.5 / 86.614)] \times 800$ t

$M = m_d + m_s /2 = 184.8 \text{ t} + 500 \text{ t} /2 = 435$ tons

Then for the fully guyed Tarpon;

w = (2400 / 435)^0.5

= 2.35 rad/s

And f = 0.37 hz or T = 2.67 seconds

For the freestanding Tarpon;

w = (92/435) ^0.5

= 0.46 rad/s

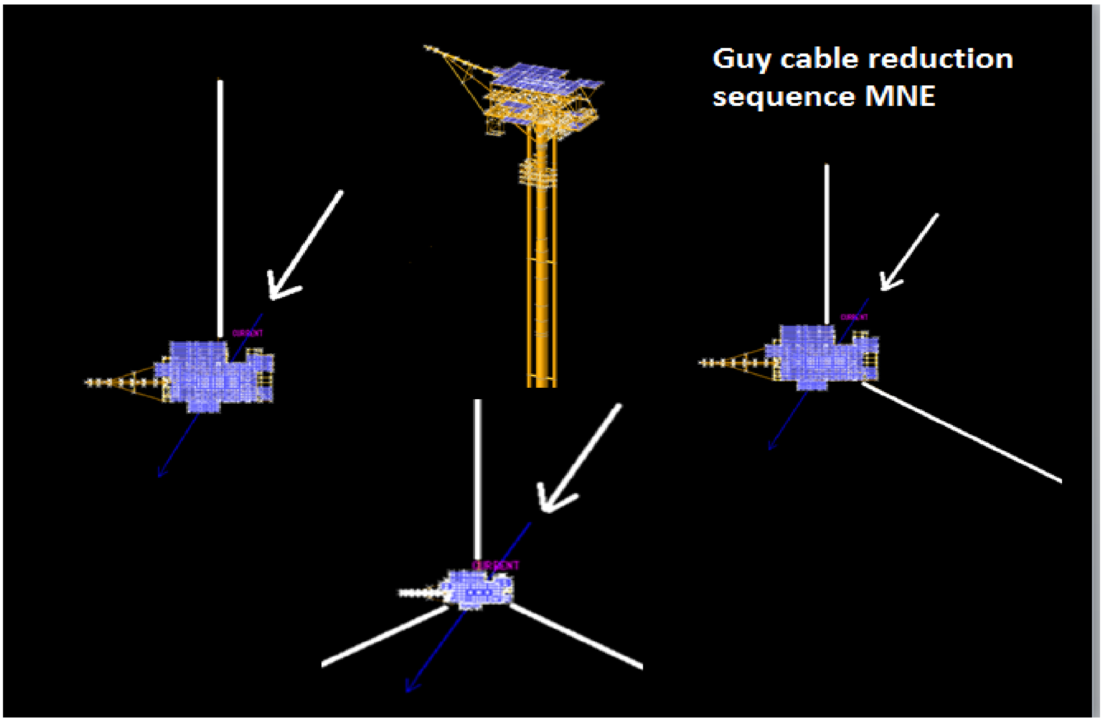
And f = 0.073 hz or T = 13.7 seconds

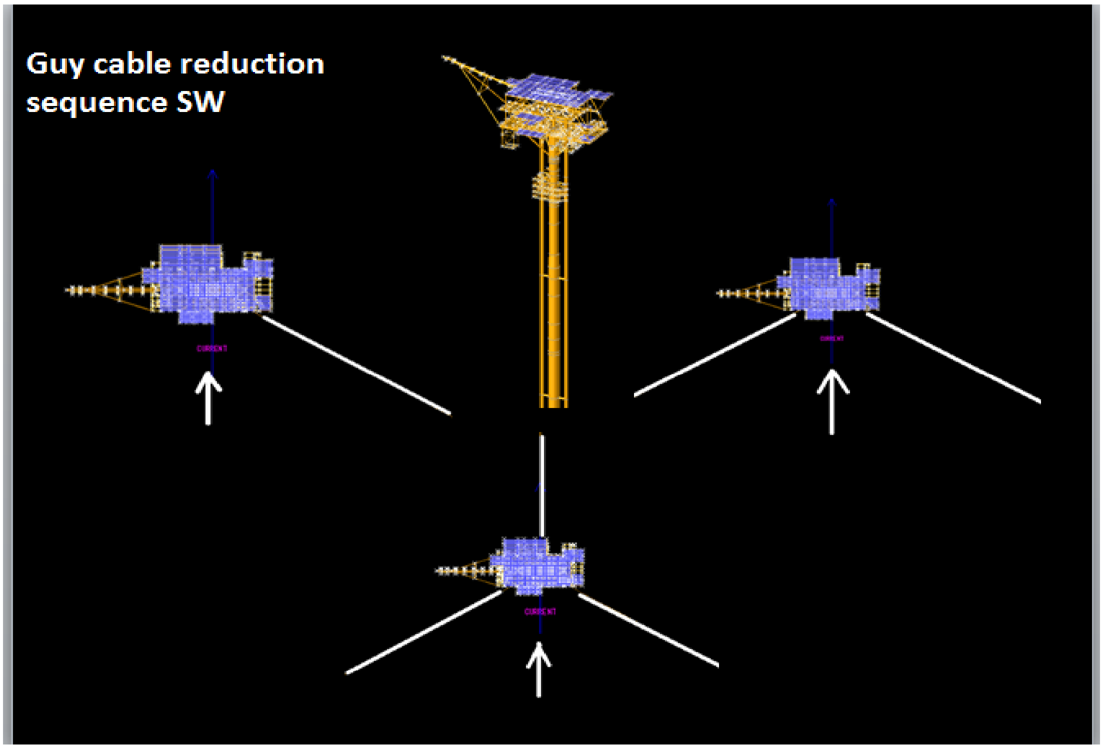
Checking using rule of thumbs , T = 0.1N where N is number of storeys. Assuming 1 storey to 3 meters.

Caisson height = 86.6m. , equivalent to approximately 28 storeys.

Then T = 0.1 (28) = 2.8 seconds.

K. Guy cable reduction sequence





L. LDP-A SACS Model