

**PILE-SOIL STRUCTURE INTERACTION IN JACKET PLATFORM
UNDER LATERAL LOADINGS**

by

Fatin Najwa binti Kamaruddin

A project dissertation submitted to the

Civil Engineering Programme

Universiti Teknologi PETRONAS

in partial fulfilment of the requirement for the

BACHELOR OF ENGINEERING (HONS)

CIVIL ENGINEERING

CERTIFICATION OF APPROVAL

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Approved by,

(Prof. Kurian V. John)

CERTIFICATION OF ORIGINALITY

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

(FATIN NAJWA BINTI KAMARUDDIN)

ABSTRACT

It is essential for a fixed offshore platform supported by pile foundation to resist lateral loading due to wind, wave and current forces subjected to the platform since minimal movement is required to provide stable work place. Thus, the design of the pile foundation should satisfy the complicated and uncertain environmental magnitude. The response of the pile foundation to the environmental load is strongly affected by pile structure and soil structure interaction. Since the development of p-y curve represents the pile foundation and soil structure interaction, it will be used to model the soil resistance to the pile movements and the results obtained will be analyzed. Structural Analysis Computer System (SACS) will be employed in this paper to model the structural member of the platform and the pile foundation. A changed in parameter will be tested to account for responses of the pile foundation to soil structure interaction which is the pile thickness. The responses of pile due to these transient loadings have been obtained.

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CHAPTER I

INTRODUCTION

1.1 Background of Study

Offshore jacket structures have been used in oil and gas exploration and production for decades. They are installed at moderate depth up to 400m depth. In the North Sea area, the existing jackets have typically been designed for a life of 20 years [1]. In the present days, engineering practices has allowed more critical and complicated structures to be designed. Thus, offshore structures should be designed to experience minimal movement to provide a stable work station for operations such as drilling and production of oil.

The design of offshore structures should satisfy the complicated and, in most cases, combined environmental phenomenon of extremely uncertain magnitude of transient loading (eg. Wind, wave, current, operational loads etc.) [14].

In recent studies, there have been experimental and analytical investigations carried out to determine response of offshore structures through Finite Element Analysis Methods. Advanced methods have been developed, such as the Stochastic Response Surface Method and Spectral Stochastic Finite Element Method [14].

Overall structural response and capacity of this type of platform greatly depends on the member behavior in the non-linear range of deformation and non-linear interaction of the foundation with the soil [9].

Foundation piles give a significant substantial effect on the response of fixed offshore structures [11]. The response of the pile foundation and soil structure interaction is determined through p-y curve. The most commonly used methods for p-y curve development are based on a relatively small amount of data in specific soil conditions [15].

1.2 Problem Statement

In general, the magnitude of vertical and lateral loads subjected to onshore pile groups is substantially lower compared to that supported by offshore pile groups. [16].

Existing offshore structures are relatively safe with regards to overload from wave and current loading, provided that the load pattern assumed in design is not significantly altered [1]. As mentioned before, offshore structures have been designed for a specific designed life. Some offshore structures will still be used beyond its designed life.

Thus, it is necessary to examine severe wave forces acting on the offshore structure, in order to perform reliable design of an offshore structure in ocean. In this case, only maximum wind, wave and current load will be verified in the determination of p-y curve. In addition, since large offshore platforms have heavy dead loads, the reaction forces on the foundation become severe and very firm foundations should be required [4].

Current knowledge of the soil-pile interaction under lateral loading needs development because characteristics of lateral load-deflection relations of pile under lateral loading, fundamental behavior of the soil-pile-structure system, and the

effects of soil nonlinearity on the soil-pile-structure response are not well understood [3].

The major task in this research is to do investigation on pile-soil structure interaction due to lateral loadings subjected to jacket type offshore structures in order to achieve a reliable design. In order to determine the pile-soil structure interaction, p-y curve first must be developed as it indicates the relationship between pile foundation and soil interaction. The analysis of p-y curve can be performed using various existing methods.

1.3 Objectives of the Study

The objectives of this study are:

- To determine the response of the pile foundation and soil structure interaction which is the curves needed for pile design.
- The study the change in soil properties with depth and the correlation with pile-soil interaction in offshore jacket structures.

1.4 Scope of the Study

In this project, the focuses are on

- Constructing lateral load-deflection relationships of piles embedded in heterogeneous and nonlinear soil layers.
- Examine the maximum loads of the waves, wind and current acting on the platform to determine the response evaluations for offshore structures considering the pile-soil interaction.

- Investigate the response offshore structures considering pile-soil interaction in nonlinear soil layers.

The case study for this project is F9JT-A, Kumang Cluster Development Project. This platform is located in F9 Field, offshore Bintulu, Sarawak. The field is part of block PM318 with the water depth of 94.6 m.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

In the literature review, we have reviewed four major parts which are the general explanation about jacket platforms, geotechnical characteristics of some offshore region, soil mechanics and pile soil interaction in offshore platforms and the previous researches in pile soil interaction in jacket type offshore platforms.

2.2 Types of Offshore Platforms

Offshore platform is a massive structure that is meant to support the exploration and production of oil and gas from beneath the seafloor. There are two types of offshore platforms which are fixed and floating structures.

2.2.1 Fixed Structures

Fixed structures are structures that extend to the seabed which are Jacket, Gravity Based Structure (GBS), Compliant Tower and Jack-Up offshore platforms.

- **Jacket Platform**

Based on Chakrabati, fixed jacket platforms consist of tubular members (typically 8 in to 48 in diameter) interconnected to form a three dimensional truss. These structures usually have four to eight legs with the outside battered to achieved better stability against toppling. This structure is usually used in moderate water depth up to 400m. These type of structures are used both as exploratory and production structures. [5]

2.2 Geotechnical characteristics of some offshore regions

Characteristics of soil conditions in some offshore regions arise from the depositional environment. Also, the geomorphic and ocean processes have shaped the bathymetry and work sediments.

Fine-grained sediments prevail further from shore and in deeper water. This is because of the coarser terrigenous sediments cannot be transported this far, therefore pelagic sediments prevail.

In extremely broad terms, some common seabed conditions in the major areas of oil and gas exploration can be characterized as:

- Gulf of Mexico – soft, normally consolidated, medium high plasticity clays ($\sim 30 < PI < 70$), often with interbedded sand layers.
- Campos and Santos Basins offshore Brazil – sands and clays with high carbonate content.
- West Africa – soft, normally consolidated, very high plasticity clays ($\sim 70 < PI < 120$, Puech *et al.* 2005), often deposited rapidly by river deltas.
- South-East Asia – desiccated crusts of stiff soil, which are remnants of low sea levels during Pleistocene, with strengths one or two orders of magnitude greater than underlying soil.
- Australia's North-West Shelf and Timor Sea – carbonate sands, silts, clay-sized soils, often with variable cementation.

Different soil conditions are available across each region and each of these area is vast.

However, the foregoing descriptions broadly characterize the type of deposit that dominates design decisions – such as field architecture and foundation solutions – of offshore oil and gas developments [7].

2.3 Soil Properties

It is difficult to predict the mechanical behavior of soils because of the structure of the soil may be highly inhomogeneous which is caused by its geological history, and it is often not possible to determine the detailed behavior of the soil by tests in the laboratory or in situ. Usually, the mechanical properties are strongly non-linear, with the material exhibiting irreversible plastic deformations when loaded and unloaded and often showing anisotropic behavior.

2.3.1 Strength and Stiffness

The shear strength and stiffness of soil determines whether or not soil will be stable or how much it will deform. The strength of soil is necessary to determine the stability of a slope, the settlement and the limiting pressure. It is important to differentiate between failure of a soil element and the failure of a geotechnical structure. The shear strength of soil depends on many factors including the effective stress and the void ratio.

The shear stiffness is important, for example, for evaluation of the magnitude of deformations of foundations and slopes prior to failure and because it is related to the shear wave velocity. The slope of the initial, nearly linear, portion of a plot of shear stress as a function of shear strain is called the shear modulus.

2.4 Load Transfer Mechanism from Pile to Soil

The load transfer mechanism from a pile to soil is a complex system. Consider a pile of length L . The load applied on the pile is gradually increased from zero to $Q_{(z=0)}$ at the ground surface. Part of these loads will be resisted by the side friction evolved along the shaft, Q_1 , and part by the soil below the tip of the pile, Q_2 . The *frictional resistance per unit area* at any depth z may be determined as

$$f(z) = \frac{\Delta Q(z)}{(p)(\Delta z)}$$

p = perimeter of the cross section of the pile.

At ultimate load, $Q_{(z=0)} = Q_u$, $Q_1 = Q_s$ which is the skin friction, and $Q_2 = Q_p$, the maximum point resistance.

2.5 Procedure employed for p - y curve, t - z curve and q - z curve

The pile foundation should be designed to sustain the lateral and axial loads. The designer should consider overload cases which the design of pile should be increased with an appropriate factor of safety.

i. P-y curve

Based on API-RP-2A-WSD [19], lateral soil resistance deflection (p - y) curves should be constructed using stress-stain data from laboratory soil samples. For the lateral bearing capacity for soft clay, p_u has been found to vary between $8c$ and $12c$ except at shallow depth. P_u increases from $3c$ to $9c$ as X increases from 0 to X_R according to:

$$p_u = 3c + \gamma X + J \frac{cX}{D} \quad (6.8.2-1)$$

$$p_u = 9c \text{ for } X \geq X_R \quad (6.8.2-2)$$

Where,

P_u = ultimate resistance, psi (kPa)

c = undrained shear strength for undisturbed clay soil samples, psi (kPa)

D = pile diameter, in. (mm)

γ = effective unit weight of soil, lb/in² (MN/m³)

J = dimensionless empirical constant with values ranging from 0.25 to 0.5 having been determined by field testing. A value of 0.5 is appropriate for Gulf of Mexico clays.

X = depth below soil surface, in. (mm)

X_R = depth below soil surface to bottom of reduced resistance zone in in. (mm). For a condition with depth, Equations 6.8.2-1 and 6.8.2-2 are solved simultaneously to give:

$$X_R = \frac{6D}{\frac{\gamma D}{c} + J}$$

Where the strength varies with depth, Equations 6.8.2-1 and 6.8.2-2 may be solved by plotting the two equations, i.e., p_u vs. depth. The point of first intersection of the two equations is taken to be X_R . These empirical relationships may not apply where strength variations are erratic. In general, minimum values of X_R should be about 2.5 pile diameter.

The p - y curves for the short-term static load case may be generated from the following Figure 1:

p/p_u	y/y_c
0.00	0.0
0.50	1.0
0.72	3.0
1.00	8.0
1.00	∞

Figure 1: P-y curve for short-term static load case

Where

P = actual lateral resistance, psi (kPa)

Y = actual lateral resistance deflection, in. (mm)

$Y_c = 2.5\epsilon_c D$, in. (mm)

ϵ_c = strain which occurs at one-half the maximum stress on laboratory undrained compression tests of undisturbed soil samples

ii. *T-z curve*

The recommended curve for non-carbonated soils is shown in the Figure 2 below:

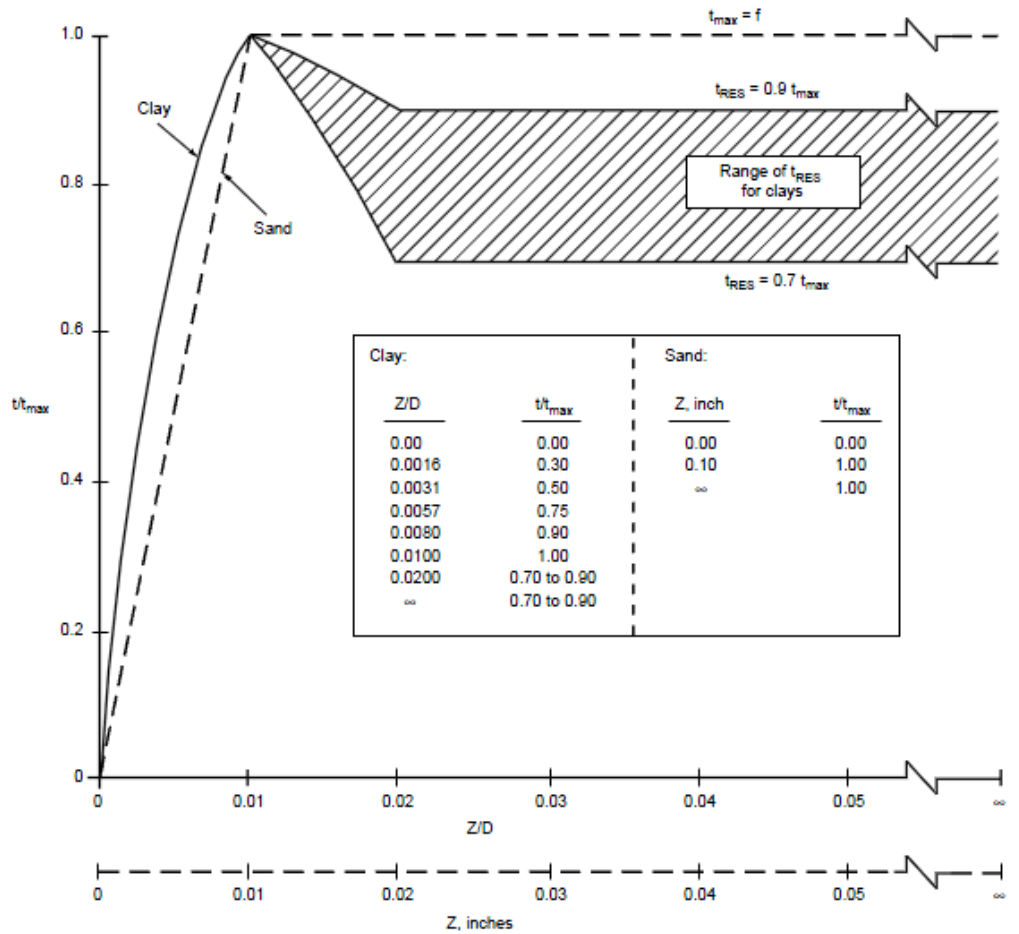


Figure 2: Recommended *t-z* curve for non-carbonated soils

Where

t = local pile deflection, in. (mm)

D = pile diameter, in. (mm)

t = mobilized soil pile adhesion, lb/ft² (kPa)

t_{max} = maximum soil pile adhesion or unit skin friction capacity computed according to Section 6.4, lb/ft² (kPa)

iii. Q - z curve

The end bearing or tip load capacity should be determined as described in 6.4.2 and 6.4.3. However, relatively large pile tip movement are required to mobilize the full end bearing resistance. A pile tip displacement of 10 percent of the pile diameter may be required for full mobilization in both sand and clay soils. The following curve in the Figure 3 is recommended for both sands and clays.

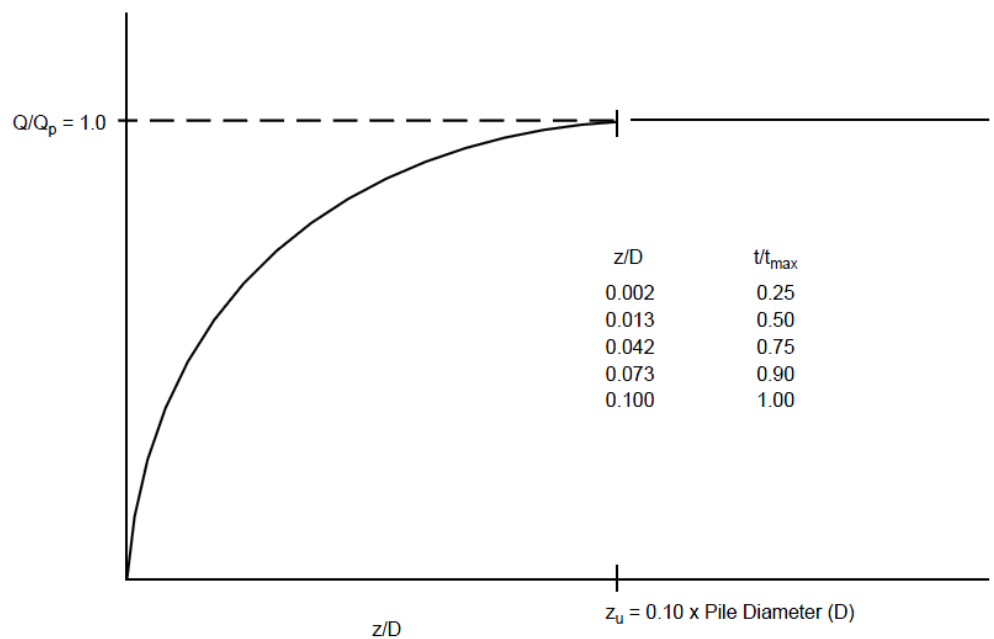


Figure 3: Recommended q - z curve for sands and clays

Where

z = axial tip deflection, in. (mm)

D = pile diameter, in. (mm)

Q = mobilized end bearing capacity, lb (kN)

Q_p = total end bearing, lb (kN), computed according to Section 6.4.

2.6 Previous study on similar type of research

There are some studies have been done for this similar type of research. There are five studies that have been reviewed with different kinds of method used and parameters tested on.

Mostafa & El Naggar (2003) [11] have studied on the response of fixed offshore platforms to wave and current loading including soil-structure investigation. The objective of this study is to investigate the response of offshore structures subjected to loadings supported by clusters of piles and this study is focused on an efficient approach to model the response of pile groups supporting a jacket structure to transient loading.

Methods that have been applied by Mostafa & El Naggar (2003) is finite element package (ASAS) to calculate wave forces on the tower and tower members and employ the concept of dynamic *p-y curves* and dynamic *p-multipliers*, *t-z curves* and *q-z curves* to model the soil reaction to pile movement.

Parameters that they have tested on are foundation flexibility, dynamic soil resistance, pile-soil-pile interaction, soil stiffness and platform deck mass.

As a conclusion, this study has achieved the objectives and the results obtained are as follows:

- i. Foundation flexibility increases the natural period of the platform, increase in the response of offshore towers and increases the velocity and acceleration at the top node of the tower.
- ii. Pile-soil-pile interaction increases the response along the offshore tower height and along the pile length and it also has a significant effect on the

stresses along the pile shaft especially in the bending moment, one of the most important parameters in the design.

- iii. Dynamic pile soil resistance decreases the response of the tower and the supporting piles.
- iv. A decrease in the resistance of the upper soil layers results in an increase in the response at the tower base and along the pile shaft and a decrease in the shear force and bending moment along the pile shaft.

Asgarian & Lesani (2007) have conducted a research on pile-soil-structure interaction in pushover analysis of jacket offshore platforms using fiber elements. The objective of this study is to perform a study on pushover analysis with the application of “fiber elements” which are capable of modelling post-buckling behaviour of braces.

Agarian & Lesani (2007) have conducted the simulation of two-dimensional models of the platforms in Persian Gulf region using “DRAIN-3DX” software which has the capability if accounting non-linear geometric and material properties. They also have developed numerical models that incorporated different foundations conditions for considering pile-soil-structure interaction to perform a pushover analysis of jacket platform.

Different from Mostafa & El Naggar (2003), this study have tested on the pile-soil interaction analysis with actual soil in-situ characteristics, pile head fixed below mud-line elevation, pile head hinged below mud-line elevation, application of linear and non-linear pile stubs and combination of linear and non-linear pile-soil characteristics.

The results of the study by Agarian & Lesani (2007), they have found out that the most favourable capacity is achieved when pile-soil interaction is considered in the analysis. The usage of linear pile stubs is an equivalent solution that is recommended for the initial stiffness assessment and it is not recommended for the ultimate

assessment. The combination of the non-linear pile, points out the key role of the surrounding soil resistance, especially for the first layers, which affect the static non-linear behaviour. Last but not least, the lateral displacements of the linear pile stubs are combined linear and non-linear pile are significant at the lowermost platform elevation due to lack of fixity and are not practical.

This paper by Asgarian, Fiouz & Talarposhti (2008) is about the effect of soil-pile-structure interaction non-linear response of jacket type offshore platforms. This paper describes the effect of seismic soil-pile-structure interaction (SSPSI) on through Incremental Dynamic Analysis (IDA) method and shows suitable length to model offshore with equivalent dummy pile for more accuracy.

They have applied IDA method which involves performing a series of nonlinear time history analyses under a suite of ground motion records. Also, they used a computer program for Non-linear Earthquake site Response Analyses of layered soil deposits (NERA) for the non-linear response of soil layers.

Asgarian, Fiouz & Talarposhti (2008) found that the behaviour of each models in both direction is almost similar but it is depended to frames interstory stiffness, which behaviour of the models in each direction is different. In X-direction, which frames has more stiffness, flatline in model without SSPSI has almost linear behaviour meanwhile in Y direction because of conditions of deck installation with float-over method and less frames stiffness, flatline in model without SSPSI is lower than SSPSI model. The difference of the model behaviour with and without SSPSI is depended to equivalent pile stiffness (length), frames stiffness and interstories stiffness.

The objectives of paper by Park, Kawano, Choi & Koo (2010) are:

- i. To calculate the effects of both waves and seismic forces on the platform accurately.
- ii. To examine the effects of soil-structure interaction.

- iii. To recognize the importance of dynamic soil-structure interaction for offshore platforms founded on soft soils.
- iv. To clarify the effects of uncertainty due to dynamic forces for the reliability estimation of the offshore platforms
- v. To clarify the effects of uncertainties for the reliable design of offshore platforms

In their study, they have used eigenfunction method to calculate the wave forces on the offshore platform which is having permeable cylindrical structures. They also used modal analysis to evaluate dynamic response of the idealized three-dimensional platform. They have also used Monte Carlo Simulation (MCS) to access uncertainty effects of dynamic forces and structural properties play very important roles on the reliability evaluations of the platform.

In this study, they have proven that the dynamic response of the offshore platform with porosity is considerably decreased with an increase of the porosity rate. The effect of seismic forces should be evaluated accurately because the structure is significantly more influenced by the seismic forces than by the wave force. Last but not least, for offshore structures with pile-soil foundation system, it is important for the reliability estimation to understand the relation between dominant frequency of seismic motions and the natural frequency of the structure system.

The last paper which was prepared by Agaskar & Metcalf (1971) is about the interaction between structural pile foundations of offshore platforms. The objective of this paper is to describe method of achieving convergence between the displacements of the platform at the foundation level and those of the piles in successive analyses without reanalyzing the entire structure in each cycle.

Agaskar & Metcalf (1971) have applied matrix condensation to reduce stiffness and cation matrices to involve only the terms corresponding to the displacements of the structures at foundation level.

The parameters that they did the research on are wind, wave, current and earthquake. The final result that they obtained is that the convergence is rapid and the offshore platform design involves the consideration of non-linear pile-soil response in the automated design process.

2.7 Gantt chart & Key Milestones

Table 1: FYP 1 Gantt Chart & Key Milestones

No.	Detail/Week	1	2	3	4	5	6	7		8	9	10	11	12	13	14	
1	Selection of Project Topic	█	█						Mid-semester Break								
	Pile-Soil Interaction in Jacket Platform	█	█														
2	Preliminary Research Work		█	█	█	█											
	Research on relevant literature review on related study		█	█	█	█											
	Research on suitable methodology to be applied		█	█	█	█											
3	Proposal Defence										█	█					
											█	█					
4	Project work continues												█	█	█		
	Research on particular platform to be a case study												█	█	█		
	Study on environmental data and geotechnical information												█	█	█		
	Learn how to operate software SACS and Finite Element Analysis												█	█	█		
Project Milestones																	
1	Completion of Extended Proposal Defence						█										
2	Completion of Interim Draft Report															█	
3	Completion of Interim Report															█	

Table 2: FYP 2 Gantt Chart & Key Milestones

No	Detail/Week	1	2	3	4	5	6	7		8	9	10	11	12	13	14	15	
1	Project Work Continues								Mid-Semester Break									
	Gather information on platform description																	
	Gather and collect data for environmental data geotechnical information																	
	Start the modelling soil reactions																	
2	Project Work Continues																	
	Start the using SACS software for the simulation of structures																	
	Analyse pile structure interaction of the structure																	
	Obtain results on platform response to lateral loading																	
Project Milestone																		
3	Submission of Progress Report																	
4	Pre-EDX																	
5	Submission of Draft Report																	
6	Submission of Dissertation (soft bound)																	
7	Submission of Technical Paper																	
8	Oral Presentation																	
9	Submission of Project Dissertation (Hard Bound)																	

CHAPTER 3

METHODOLOGY

3.1 Research Methodology

In this paper, the methodology proposed by Mostafa & El Naggar [11] will be applied in this research methodology. This paper presents a methodology describes the efficient approach to model the response of pile groups of a jacket structure to transient loading. This method applies the concepts of p-y curves to model the soil reaction subjected to pile movement.

Several parameters that will be tested on which are foundation flexibility, soil resistance, pile-soil-pile interaction, soil stiffness and platform deck mass. These parameters are ones that affect the characteristics of the platform and the responses of the platform due to wind, wave and current loading will be investigated.

The proposed process flow for this methodology is shown in the Figure 4.

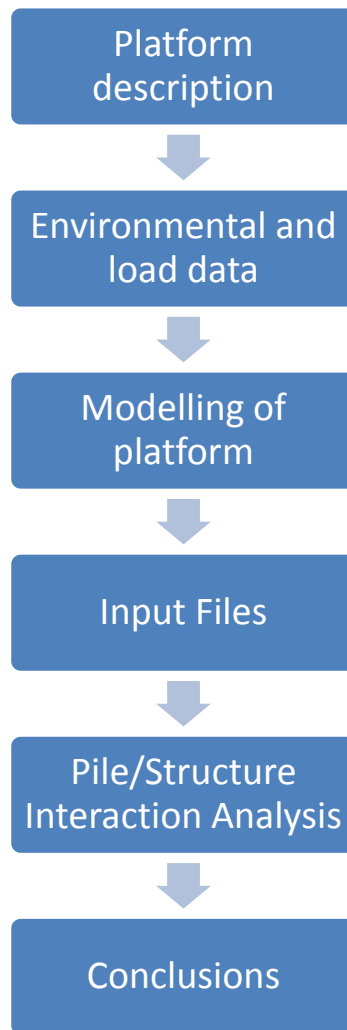


Figure 4: Methodology process flow

3.1.1 Platform Description

First of all, the information on the platform considered will be gathered. The case study for this research is F9JT-A, Kumang Cluster Development Project. This platform has been completed, located in F9 Field, offshore Bintulu, Sarawak . The water depth at site is 94.6m and the substructure is a pile steel jacket. The F9JT-A platform is designed for a 4-pile jacket.

3.1.2 Environmental and Load Data

In this project, as far as the environmental loads are concerned, wave is the most significant due to its magnitude. Correlative to this, current, wind and live loads form a combination of loads that should be considered for the reliable design of offshore structures. An offshore structure is subjected to different types of loading depending on its service and environmental conditions during its operation. These loads can be categorized as dead loads, live loads and environmental loads.

Dead loads are the weight of the entire platform including topside loads and any permanent equipment and machineries mounted on the platform which does not change the mode of operation. Live loads on the other hand are the loads imposed on the structure when operation takes place, which include the weight of any drilling and production equipment, heliport, utilities equipment and as such. Live loads are usually idealized as uniform distributed loads [17]. Environmental loads are loads enforced on the platform by the natural phenomena including wave, wind and current and this type of load is the most important in the design of offshore structures due to high level of randomness.

The F9JT-A Platform model has been preloaded with in-place dead and live load and environmental storm water conditions. Pile/structure interaction analyses have been carried out to determine the response of pile under lateral loading considering the soil interaction. Load combinations are considered in the analyses, which are:

Table 3: Load combinations

Load Case Name	No.	Load
LL01	6	Area Live Load
	13	Piping & Equipment Operation Weight
	13A	Future Piping & Equipment Operating Weight
	14	Electrical & Instrumental Operating Weight
	26	Storm Reacting @well #0145
	37	Upward LL
	42	Vent Boom Operation Wind +X-Direction
	51	Rig @#0145 Operation Wind X-Direction
ST02	40	Topside Operating Wind +X-Direction
	41	Topside Operating Wind +Y-Direction
	63	Storm Wave & Current Direction 45 Degree
	94	Inertia Storm Wave & Current Direction 0 Degree
	95	Inertia Storm Wave & Current Direction 90 Degree
DL	1	Primary Steel Weight
	4	Topside Miscellaneous Weight
	7	Crane Dead Load
	2A	Jacket Appurtenance Submerged Weight
	3A	Jacket Post Installed Appurtenance Submerged Weight
	11A	Future Piping & Equipment Dry Weight
	101	Helicopter Dead Load
	18A	Riser/J-Tube Submerged Weight
	11	Piping & Equipment Dry Weight
	16	

The load condition 63 is obtained from Metocean Department and is associated with the storm water condition. The criteria for this load condition are shown in table 4 and 5. Figure 5 shows the load condition 63 acting on the structure.

Table 4: Design wave criteria for load condition 63

Design Wave	
Wave Type	Stokes
Wave Height (m)	11.7
Wave Period (s)	10.6
Wave Direction (degree)	45
Kinematic Factor	0.95

Table 5: Values for associated current for load condition 63

Distance (from the mudline, up) (m)	Velocity (m/s)
0.00	0.00
9.46	0.55
47.29	0.95
94.60	1.20

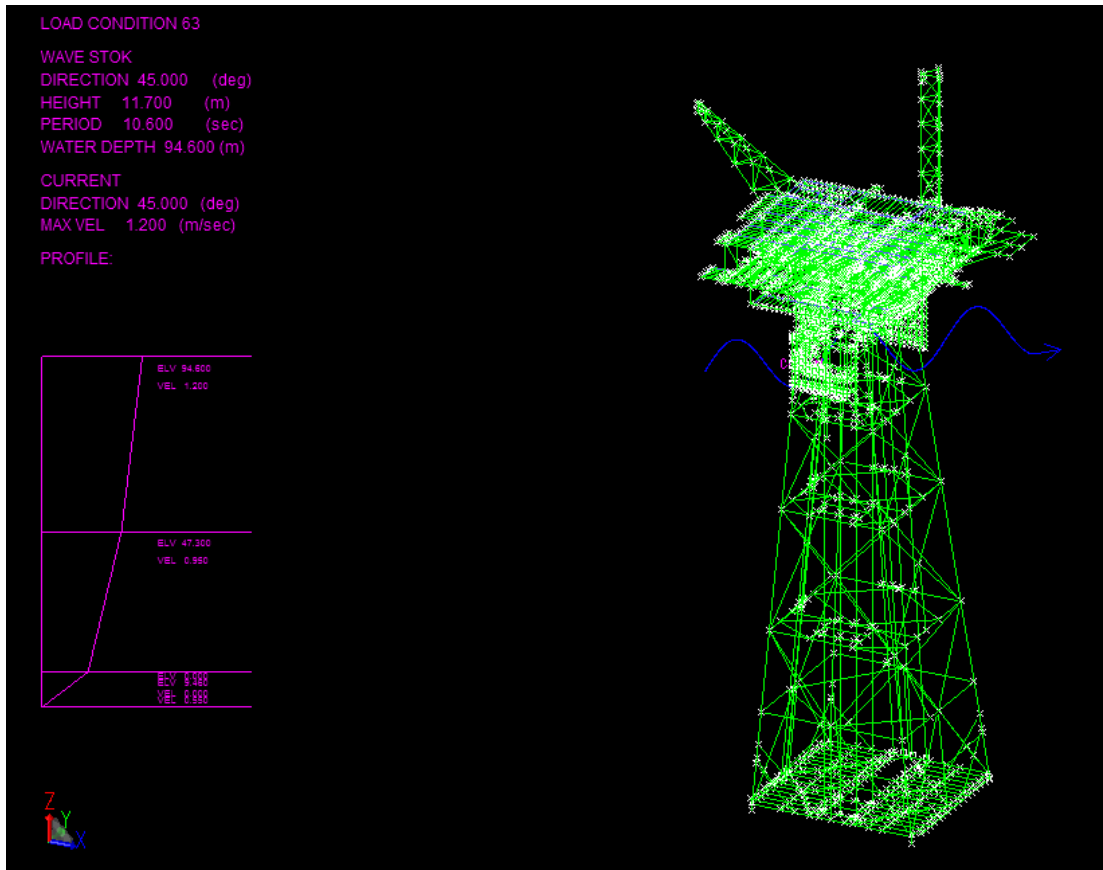


Figure 5: Load condition 63 acting on the structures

3.1.3 Modelling Platform

The structural members of the F9JT-A platform and foundation piles are modelled using space frame elements in the SACS software. This program provides special handling of structures that are jacket oriented. The modelling session starts with opening Precede and creates a new 4 main legs structure. Precede is an interactive full screen color graphics modeller and it has the capabilities of generating models include geometry, material and section properties and loading. Also, it has the

capabilities to generate SEASTATE condition. SEASTATE is an environmental loads generator with full implementation of API 20th edition and support five wave theories. Other than that, Precede automatically generate offshore jacket and deck, load including gravity, pressure and skid mounted equipment loads.

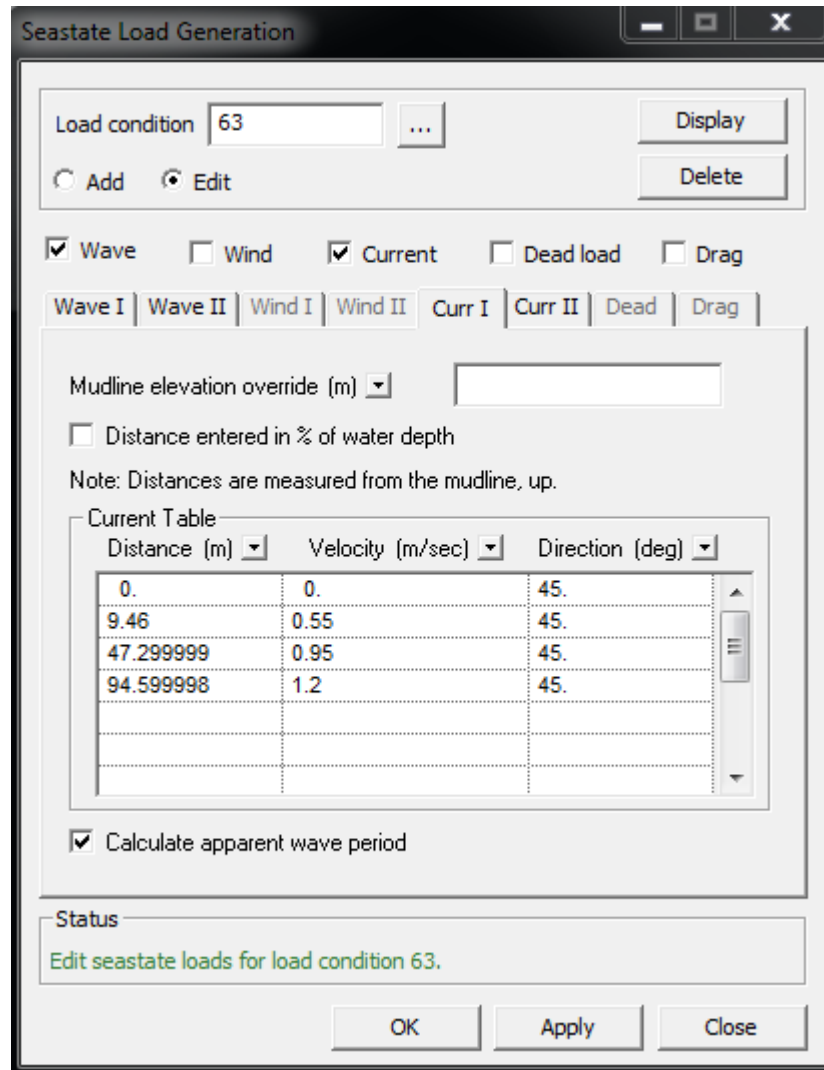


Figure 6: SEASTATE Load Generation

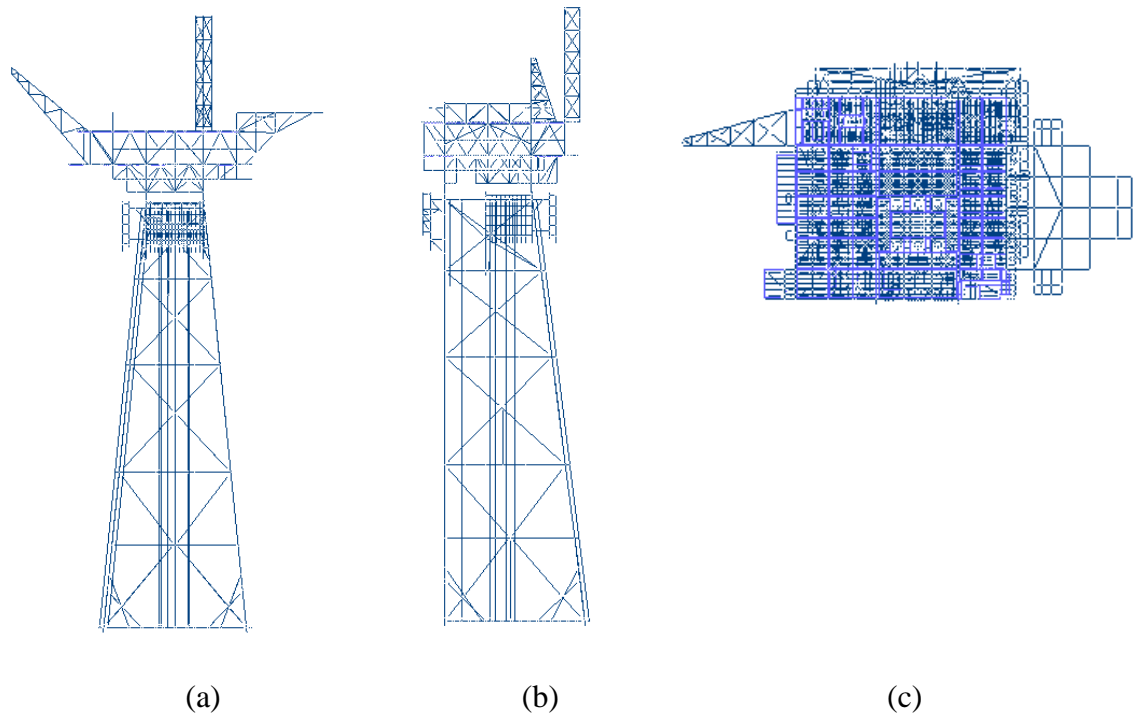


Figure 7: F9JT-A platform modelled in Precede. (a) XZ Front View (b) YZ Right View (c) XY Top View

Figure 6 shows the SEASTATE Load Generation dialog box. The wave is user defined and it generates load due to wind, gravity, buoyancy and mud flow. Meanwhile, Figure 7 shows the model of F9JT-A which was modelled in Precede. This model is shown in wireframe mode which does not display the material and section properties.

3.1.4 Input Files

In this study, analysis of Pile/Structure Interaction (PSI) requires two input files which are:

- i. SACS input file (sacinp.txt*)
- ii. PSI input file (psiinp.txt*)

SACS input file is auto-generated when the modelling of platform has been done. The information added into the system will be displayed in numerical form in the SACS input file. This input file can be displayed using Data Generator. Data Generator on the other hand is an interactive data generator for all programs. It is an editor which labels and highlights data fields, and provides help for data input. Figure 5 below shows the SACS input file for the F9JT-A platform.

```

SACS Data Generator - [D:\Final Year Project 2\F9\F9JT-Jacket.Trial\sacinp.f9jt-1.txt]
File Edit Search Tools Window Help
D:\Final Year Project 2\F9\F9JT-Jacket.Trial\sacinp.f9jt-1.txt
DOPT      NF+Z      1.030      7.85      -94.80      94.80GLOBMN      HYD      MPT
KUMANG CLUSTER DEVELOPMENT PROJECT
F9JT-A - SUBSTRUCTURE IN-PLACE ANALYSIS
*****
* CLIENT : PETRONAS CARIGALI SDN. BHD.
* FILE NAME : sacinp.f9jt-a18
* DATE UPDATED : 31 march 2008
* ENGINEER : MFJ
* DESCRIPTION: COMBINED SUBSTRUCTURE/TOPSIDE GEOMETRY
* MODEL FOR SUBSTRUCTURE IN-PLACE ANALYSIS
* 1) Topside Model incorporated with
* latest additional piping weight (14/3/08)
* 2) Initial Pile Depth Penetration = 130m
* 3) Modified from sacinp.f9jt-a14 (AFDD)
* 4) Add New Boat Landing Model (30.8.07)
* 5) Appurtenance Weight Updated (17.9.07)
* 6) Add New Riser Guard Model (21.9.07)
*****
OPTIONS   MN  PI  SDUC  4 4  DC  C  PTPTPTPTPT  PT
LCSEL ST  DL
LCSEL ST  LL01
*LCSEL ST  LL01 LL02 LL03 LL04 LL05 LL06 LL07 LL08
LCSEL ST  ST02
*LCSEL ST  ST01 ST02 ST03 ST04 ST05 ST06 ST07 ST08 ST09
*LCSEL ST  CS01 OP01 OP02 OP03 OP04 OP05 OP06 OP07 OP08 OP09 SM01 SM02
*LCSEL ST  SM03 ST01 ST02 ST03 ST04 ST05 ST06 ST07 ST08 ST09 SX01 SX02
*LCSEL ST  SX03 SX04 SX05 SX06 SX07 SX08 SX09
UCPART    0.00 0.80 0.80 1.00 1.00
HYDRO Z APEXTMNRGPAI 94.800 -94.800 2.000 1.025 1.000 0.250
HYDRO2    0.90ML 0.80 2.00
AMOD
AMOD ST02 1.333
*AMOD ST02 1.333ST03 1.333ST04 1.333ST05 1.333ST06 1.333ST07 1.333
*AMOD ST08 1.333ST09 1.333SX01 1.333SX02 1.333SX03 1.333SX04 1.333SX05 1.333
*AMOD SX06 1.333SX07 1.333SX08 1.333SX09 1.333
SACS Model File | LN 01, Col 01

```

Figure 8: SACS Input File

In the SACS input file, the water weight density, structure weight density, mudline elevation and water depth, are all displayed, refer Figure 6. The load case selection can also be viewed in this data generator. For this study, three (3) load combinations are selected to be included in the analysis, refer Figure 7. Other than that, SACS input file also tells the joints, members, plate information which has been defined during modelling of the platform.

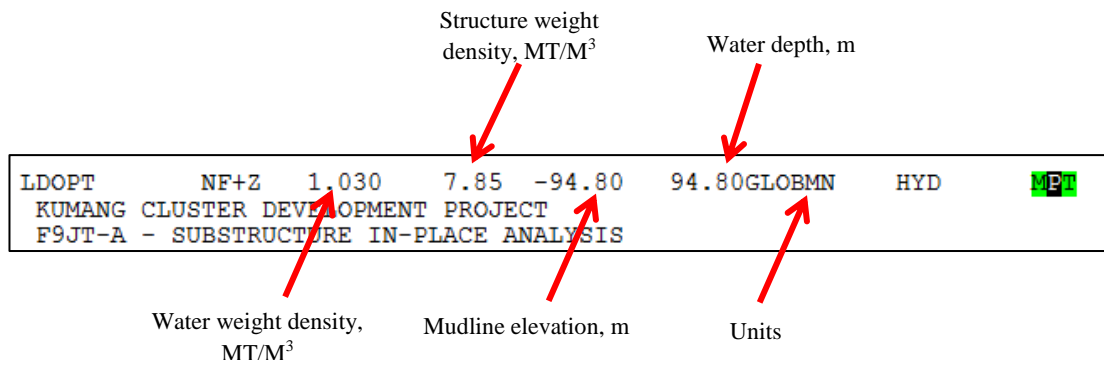


Figure 9: SEASTATE Option

```
LCSEL ST      DL
LCSEL ST      LL01
*LCSEL ST     LL01 LL02 LL03 LL04 LL05 LL06 LL07 LL08
LCSEL ST      ST02
```

Figure 10: Load Case Selection

For Pile Structure Interaction (PSI), the input file has to be generated using data generator. The nonlinear foundation model, including the pile and the soil properties, is specified separate from the model information in a PSI input file. To create a PSI input file, the interface joints between the linear structure and the nonlinear foundation must be designated in the SACS model by specifying the support condition ‘PILEHD’ on the appropriate JOINT input line. In this PSI input file, there are three (3) pile group defined which is as displayed in the table below.

Table 6: Pile group properties

Pile Group Label	Outside Diameter (m)	Wall Thickness (m)	Pile Segment Length (m)	Elastic Modulus (Mpa)	Shear Modulus (Mpa)	Yield Stress (kN/m2)
PL1	1.524	0.06	23.2	200,000	8,000	340,000
PL1	1.524	0.05	106.8	200,000	8,000	340,000
CN1	0.66	0.19	80	200,000	8,000	345,000

For pile group labelled PL1, the data for t - z soil axial is added and there are 50 numbers of soil strata. The soil stratum is defined by the distance to top of stratum. The t - z axial values also added into the input file at the corresponding strata. Then, the t - z axial bearing data is also added into the input file and there are 21 numbers of soil strata. For p - y data, there are 70 numbers of soil strata and its reference pile diameter is 1.524 m. Next is for pile group labelled CN1, the t - z soil axial, t - z axial bearing and lateral soil data also added into the input file.

3.1.5 Pile/Structure Interaction Analysis

Pile Structure Interaction (PSI), analyses the behaviour of a pile supported structure subject to one or more static load conditions. Finite deflection of the piles which is called as “P-delta” effect and nonlinear soil behaviour both along and transverse to the pile axis are accounted for. The program uses a finite difference solution to solve the pile model which is represented by a beam column on a nonlinear elastic foundation. The structure resting on the piles is represented as a linear elastic model.

PSI first obtains the pile axial solution, then uses the resulting internal axial forces to obtain the lateral solution of the piles. In general, soils exhibit nonlinear behaviour for both axial and transverse loads, therefore an iterative procedure is used to find the pile influence on the deflection of the structure.

PSI is designed to use pile and soil data, specified in an input file, in conjunction with linear structural data produced by the SACS IV program. Earlier, two input files have been created which are SACS and PSI input files. The analysis category that is employed in this project is Statics meanwhile the analysis class falls under Static Analysis With Non-Liner Pile/Structure Interaction. Once these input files have been selected, run the analysis to obtain the results.

CHAPTER 4

RESULT AND DISCUSSION

4.1 P-Y Curve

The pile foundation should be designed to resist lateral loads and the overall structural foundation system should satisfy under the overloads. *P-values* show the lateral soil resistance applied per unit length along the depth of the pile, meanwhile *y-values* show the displacement of the pile [19]. *P-y curve* shows the nonlinear relationship where lateral pile displacement *y* mobilizes under lateral soil reaction *p* per unit length. The *p-y curve* obtained is as the Figure 11 shown below.

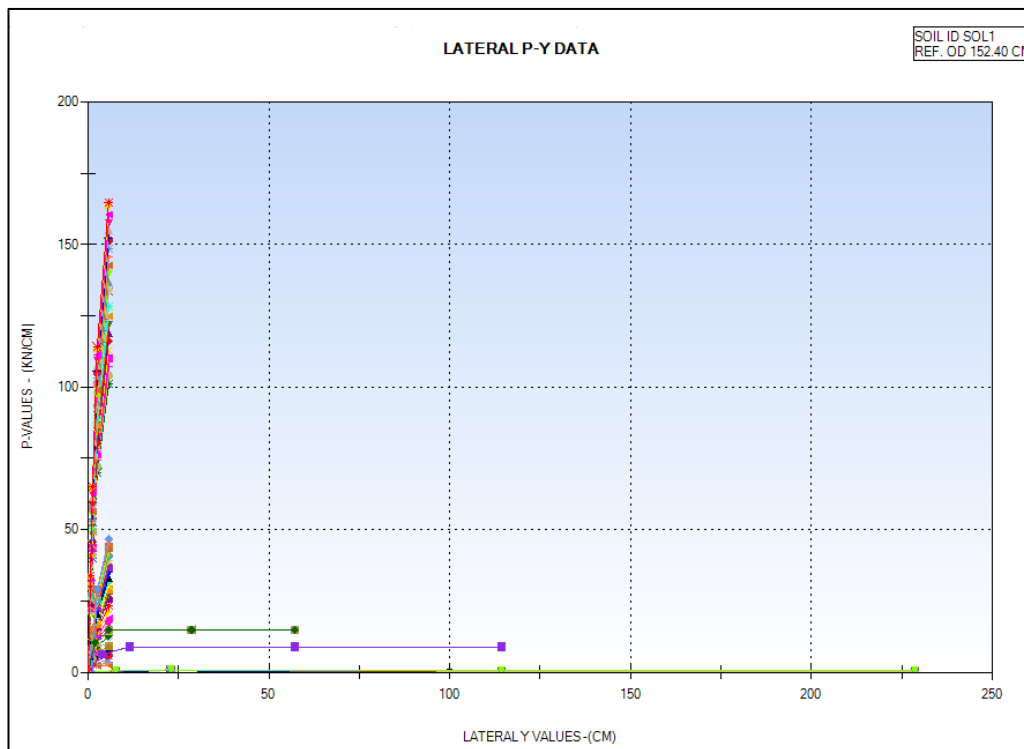


Figure 11: General p-y curve obtained for the whole soil layers

The Figure 11 above shows the $p-y$ curve for the pile diameter 152.40 cm. For close up, the Figure 12 shown below is for the soil strata from 2.0 m till 4.10 m below the mudline.

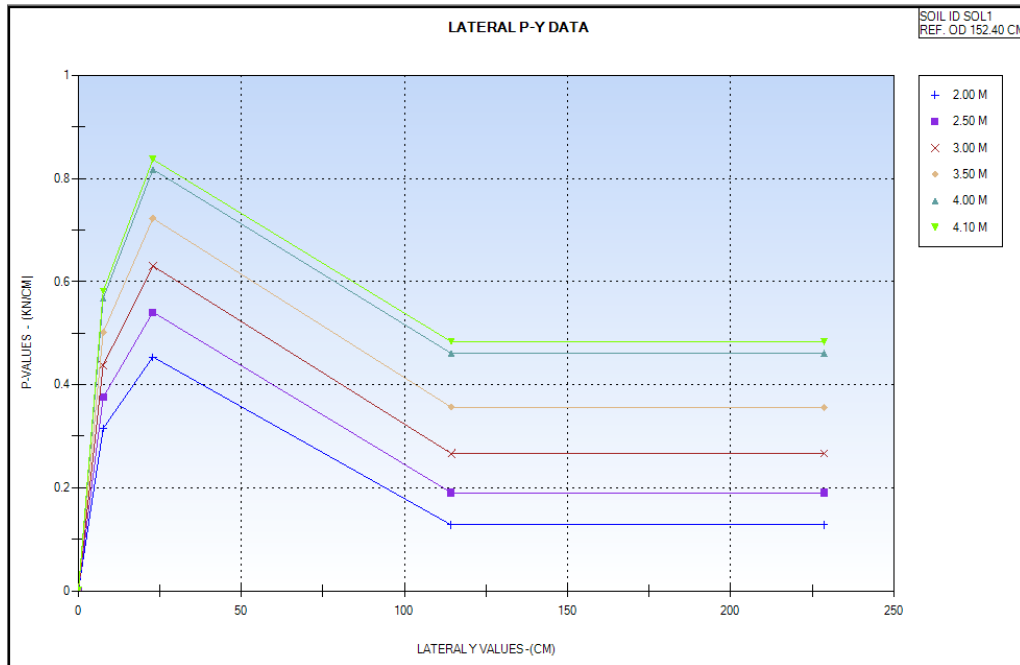


Figure 12: P-y curve for soil layers 2.00 m until 4.10 m

The Figure 12 shows that the pile is embedded in soft clay from 2.0 meters to 4.10 meters below the ground. For the soil layer at 2.50 m below the ground, the displacement is constant at 114.30 cm when the soil resistance is 0.1903 kN/cm. This region is known as plastic region. The Figure 12 is formed due to cyclic loading, applied to the pile. Cyclic loading [22] is the inertial loadings developed by environmental conditions such as storm waves and earthquakes. These repetitive loadings can cause temporary or permanent decrease in load-carrying resistance meanwhile rapidly applied loadings can cause increase in load carrying resistance. Since the environmental loadings defined in SACS software is the storm water condition, this graph has shown the pile responses under cyclic loading, which is storm waves in soft clay soil.

Based on API [19], to calculate ultimate resistance, p_u at a specific depth, equation 6.8.2-1 is used. The calculated value for p_u at soil layer 2.0 m is 1.294 kN/cm, which is larger than the value at the graph, 0.454 kN/cm. However, for the y -value, theoretical and experimental values are the same, which is 22.86 cm for soil layer at 2.0 m below the ground

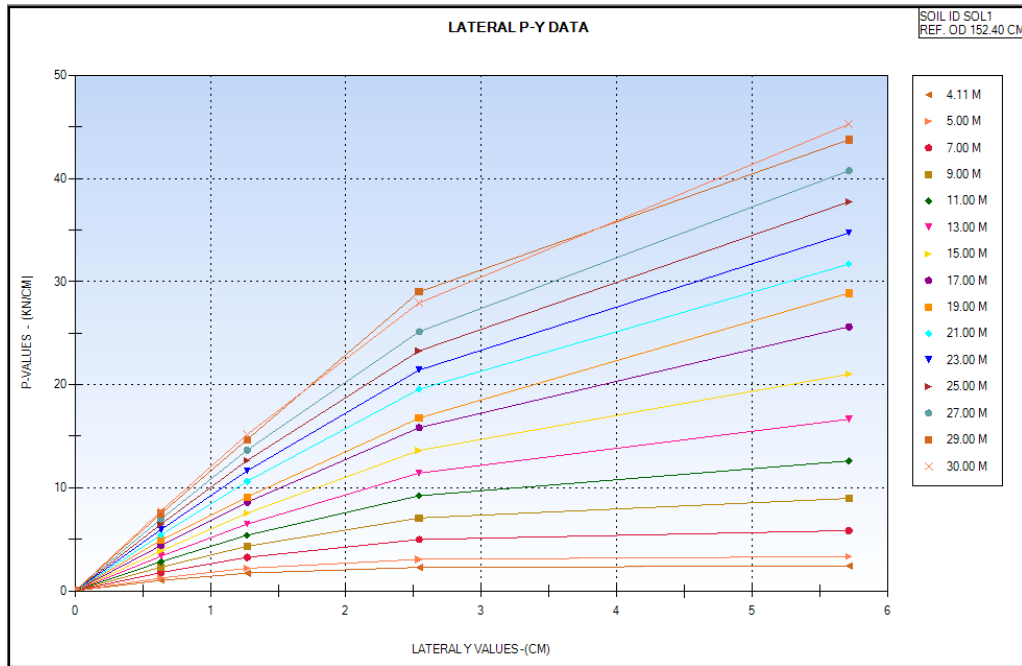


Figure 13: P-y curve for soil layers 4.11 m until 30.00 m

Figure 13 shows the pile response from soil layer 4.11 m until 30.00 m below the ground. The pattern of this Figure 13 is different with the Figure 12 because the soil layers represented are sand and this prediction is made based on He's et. al. [20] findings.

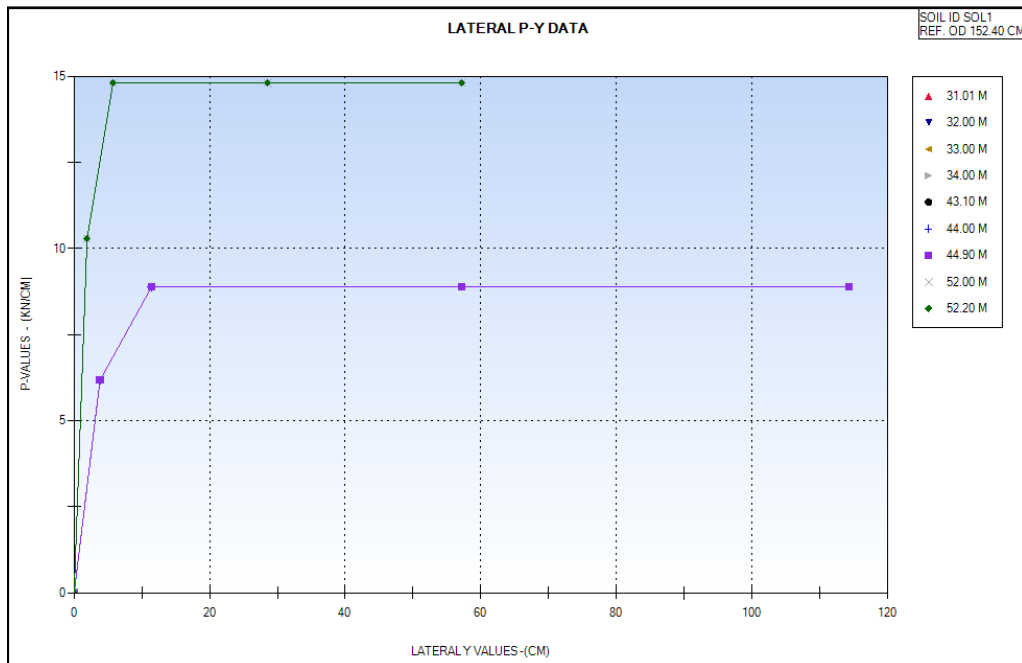


Figure 14: P-y curve for soil layers 31.01 m until 52.02 m

The Figure14 shown above is for the soil layers from 31.01 m till 52.20 m. The lower curves indicate the soil layers for 31.01 m till 44.90 m below the ground, meanwhile the upper curves indicate the graph for layers 52.00 m till 52.20 m. These layers have same graph pattern due to loading conditions. The upper curves illustrate that the piles are subjected to static loadings and in the meantime, the lower curves are subjected to cyclic loading. The lower curves have larger deflection, and this is related to the cyclic loadings applied. These curves also have been compared with the ISO 19902 [22]. Although the values compared are not equal, this can be explained as the code has been made according to Gulf of Mexico condition. Malaysia on the other hand, has quite mild conditions compared to Gulf of Mexico. The ultimate soil resistance according to the curves are 8.888 kN/cm and 14.813 kN/cm respectively. As the soil goes down deeper, the ultimate resistance is increasing.

For this Figure 14, the theoretical value should be calculated using equation 6.8.2-2 since X value is more than X_R . The curves give p -value of 8.888 kN/cm which is quite close to the theoretical value, 9.60 kN/cm. The y -value gives the ratio of y/y_c is 1.5, meanwhile the theoretical value should be 0.72. Same goes to the upper curve, the theoretical p -value is 13.716 kN/cm and the ratio of y/y_c is 60.96 cm which has a huge difference compared to the API standard.

4.2 Axial T-Z Curve

Axial $t-z$ curve on the other hand, represents the relationship between the induced shear stress due to load transfer and the vertical movement of the pile, along the pile shaft [21]. The following Figure 15 shows the result for this analysis.

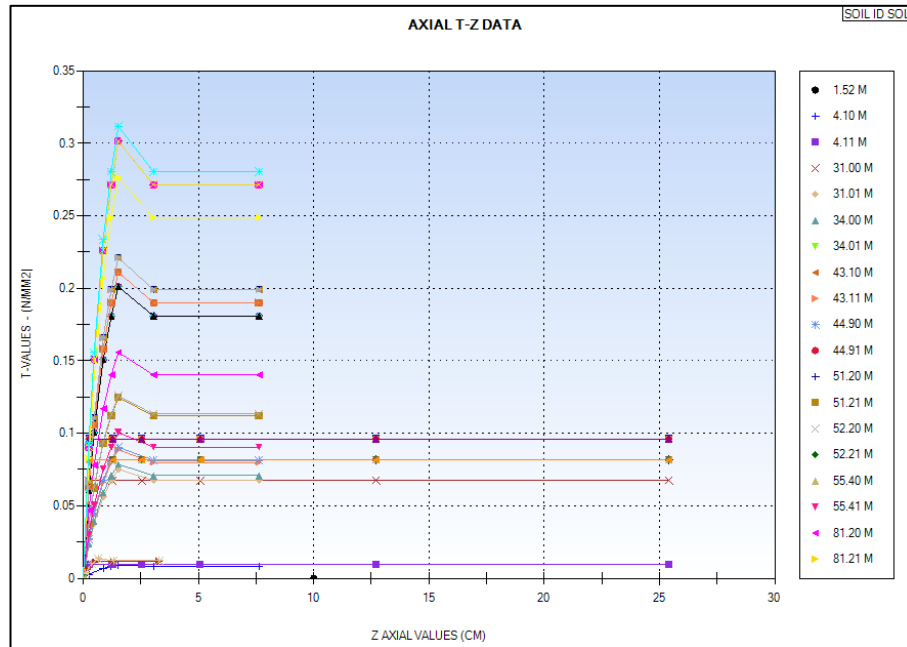


Figure 15: General t-z curve for whole soil layers

There are two types of curve that can be found in this t - z curve, the Figure 16 below represents the pile response in sand.

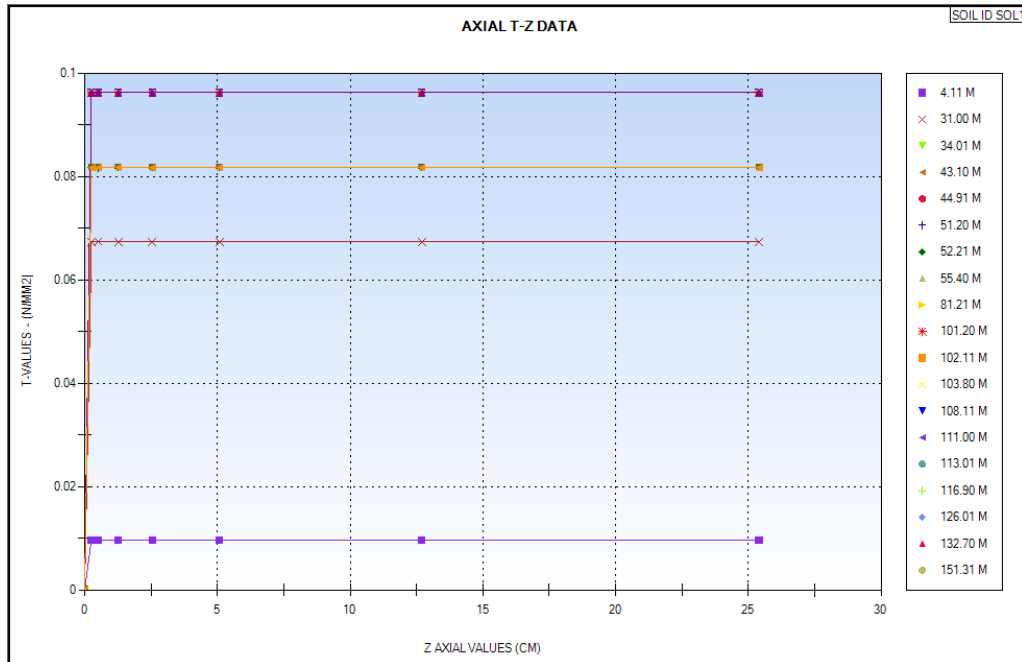


Figure 16: T-z curve for soil layers 4.11 m till 151. 31 m

The t -values obtained at the specified soil layers, as recommended in the API should be equal with the unit skin friction that can be computed according to section 6.4. Once the mobilized soil pile adhesion, t reached the maximum value, the displacement will keep constant. The Figure 17 shown below represents the pile response in clay.

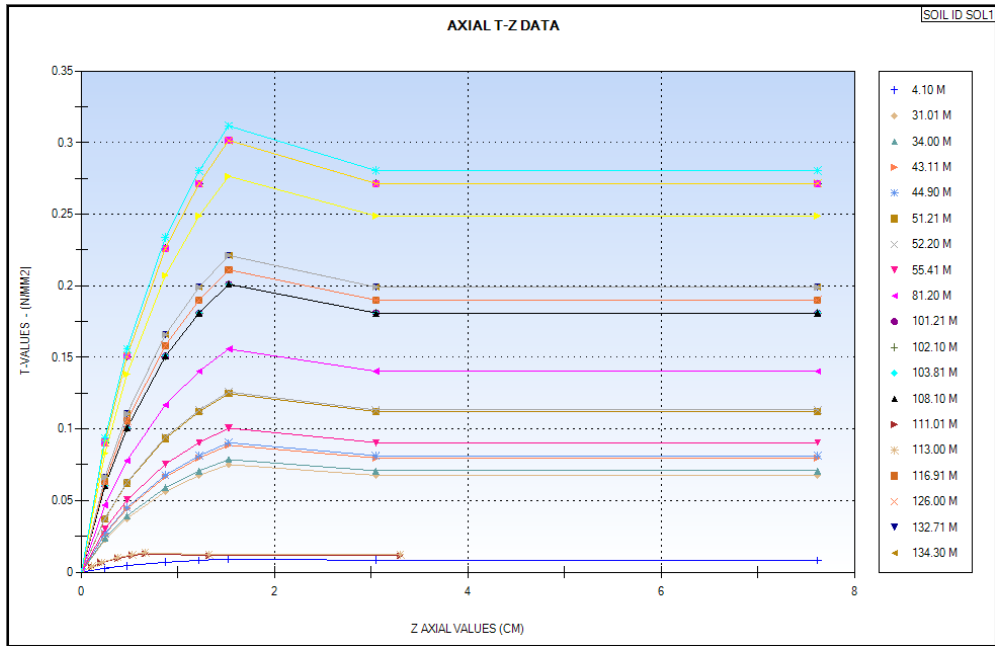


Figure 17: T-z curve for soil layers 4.10 m until 134.30 m

The Figure 17 has been compared with API and ISO standard, and the values shown are equals. All the z-values achieved at maximum *t-values* are the same for all soil layers. According to API standard, the ratio for z/D should be 0.01, and it is proven equal with the result obtained.

4.3 End Bearing T-Z Curve

Q - z curve shows the relationship between mobilized end bearing resistance and axial tip deflection. This graph tells the displacement at the axial tip due to the end bearing resistance from the soil. The result obtained is as per Figure 18 shown below.

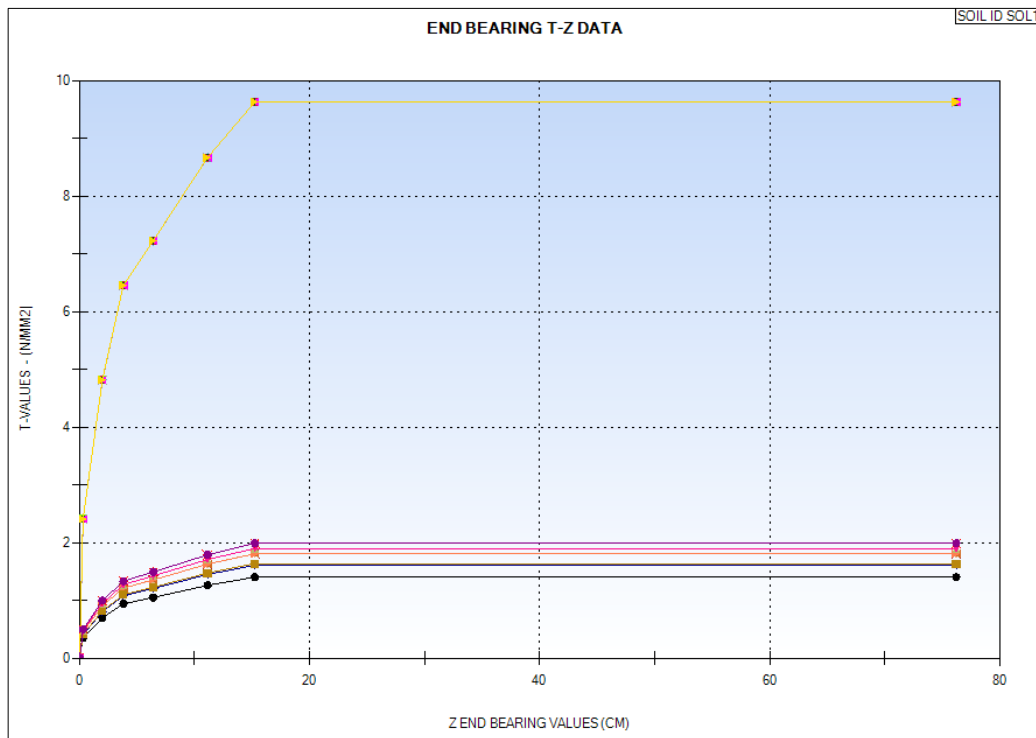


Figure 18: General q-z curve for the whole soil layers

The result obtained is as per the recommended graph in the API standard. The z -values when the maximum end bearing resistance is achieved should be equals to $0.10 \times$ pile diameter. The result shown above, gives the value of z which is 15.24 cm, as per recommended graph.

4.4 Changing the parameter

In the analysis, the wall thickness has been reduced from 6.0 cm to 4.0 cm. The results have shown an expected outcome, which can be concluded that the thinner the wall thickness of a pile, the larger the deflections occur. However, the differences cannot be compared with the curves obtained, due to very small number changed. From the output listings obtained from SACS software, the differences can be shown in terms of pile head coordinates.

FINAL DEFLECTIONS FOR LOAD CASE ST02							
IN							
PILE HEAD COORDINATES							
PILE JOINT	BATTER JOINT	DEFL(X) CM	DEFL(Y) CM	DEFL(Z) CM	ROT(X) RAD	ROT(Y) RAD	ROT(Z) RAD
101	201	1.3773080	2.7041368	0.0129181	-0.0002062	0.0000955	0.0019687
102	202	-0.0721351	2.8713570	-0.0150145	-0.0002982	0.0000935	0.0021656
103	203	-1.5642630	2.5209710	-0.0293009	-0.0001549	-0.0000030	0.0019530
104	204	0.0000155	-2.4063400	0.0157212	-0.0000042	-0.0000037	-0.0019027
105	305	-0.0114412	2.4894208	-0.0095648	-0.0003245	0.0000583	0.0046196
107	307	-0.0114412	2.5176772	-0.0093953	-0.0001933	0.0000620	0.0046801
108	308	-0.0114412	2.4287945	-0.0136868	-0.0003675	0.0000564	0.0045316
110	310	-0.0114412	2.4515155	-0.0134011	-0.0002154	0.0000651	0.0045723
111	311	-0.0114412	2.4425238	-0.0099903	-0.0002782	0.0000615	0.0045574
113	313	-0.0114412	2.4810577	-0.0097950	-0.0001968	0.0000613	0.0046254

Figure 19: Final deflections for load case ST02 in pile head coordinates for wall thickness 6.0 cm

The Figure 19 above shows the final deflection for load case ST02 in pile head coordinates for wall thickness 6.0 cm. These coordinates tell us the final deflection, not the new pile head coordinates.

FINAL DEFLECTIONS FOR LOAD CASE ST02							
IN							
PILE HEAD COORDINATES							
PILE JOINT	BATTER JOINT	DEFL(X) CM	DEFL(Y) CM	DEFL(Z) CM	ROT(X) RAD	ROT(Y) RAD	ROT(Z) RAD
101	201	2.0943982	3.0830515	0.0121890	-0.0002334	0.0001561	0.0022243
102	202	-0.0799458	3.3159613	-0.0238434	-0.0003593	0.0001231	0.0024622
103	203	-2.2953076	2.8363465	-0.0379286	-0.0001731	-0.0000022	0.0022050
104	204	-0.0100930	-2.7863205	0.0212896	0.0000120	-0.0001169	-0.0021973
105	305	-0.0114412	2.8689575	-0.0186020	-0.0003333	0.0000797	0.0053175
107	307	-0.0114412	2.9009413	-0.0186524	-0.0002022	0.0000831	0.0053847
108	308	-0.0114412	2.8007681	-0.0217941	-0.0003761	0.0000755	0.0052177
110	310	-0.0114412	2.8266078	-0.0217047	-0.0002242	0.0000839	0.0052637
111	311	-0.0114412	2.8181754	-0.0171176	-0.0002871	0.0000799	0.0052493
113	313	-0.0114412	2.8622367	-0.0170863	-0.0002057	0.0000793	0.0053265

Figure 20: Final deflections for load case ST02 in pile head coordinates for wall thickness 4.0 cm

The Figure 20 shown above on the other hand shows the final deflection for load case ST02 in pile head coordinates for wall thickness of 4.0 cm. Comparing both of the figures above, the deflection for wall thickness 4.0 cm is slightly larger than the wall thickness 6.0 cm.

Table 7: Comparison of deflections for wall thickness 6.0 cm and 4.0 cm

Wall thickness	X	Y	Z
6.0 cm	1.377308	2.7041368	0.0129181
4.0 cm	2.0943982	3.0830515	0.012189
Difference	0.7170902	0.3789147	-0.0007291

The Table 7 above shows the difference in pile head deflections between wall thickness for 6.0 cm and 4.0 cm.

Table 8: Percentage difference between wall thickness 6.0 cm and 4.0 cm respectively

	X	Y	Z
Pile head coordinates, original (cm)	-17.138	-10.35	-94.8
Deflections, 6.0 (cm)	1.377308	2.7041368	0.0129181
Deflections, 4.0 (cm)	2.0943982	3.0830515	0.012189
Percentage, 6.0 (%)	8.0365737	26.126926	0.01362669
Percentage, 4.0 (%)	12.2207854	29.787937	0.01285759

The Table 8 above shows the percentage difference between the two wall thickness, and as anticipated, the percentage deflections for the thinner wall thickness is higher than the thicker wall thickness.

4.5 Unity check ratio

The unity check ratio is the ratio of applied and allowable stress interaction. The value of unity check ratio should be less than 1.0.

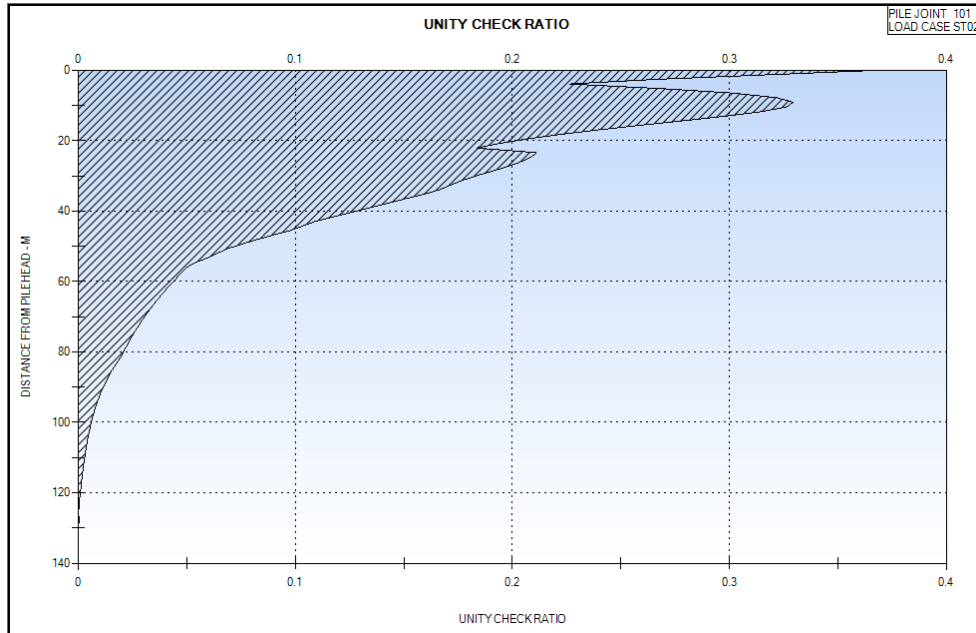


Figure 21: Unity check ratio for pile joint 101 under load case ST02 with wall thickness of 6.0 cm

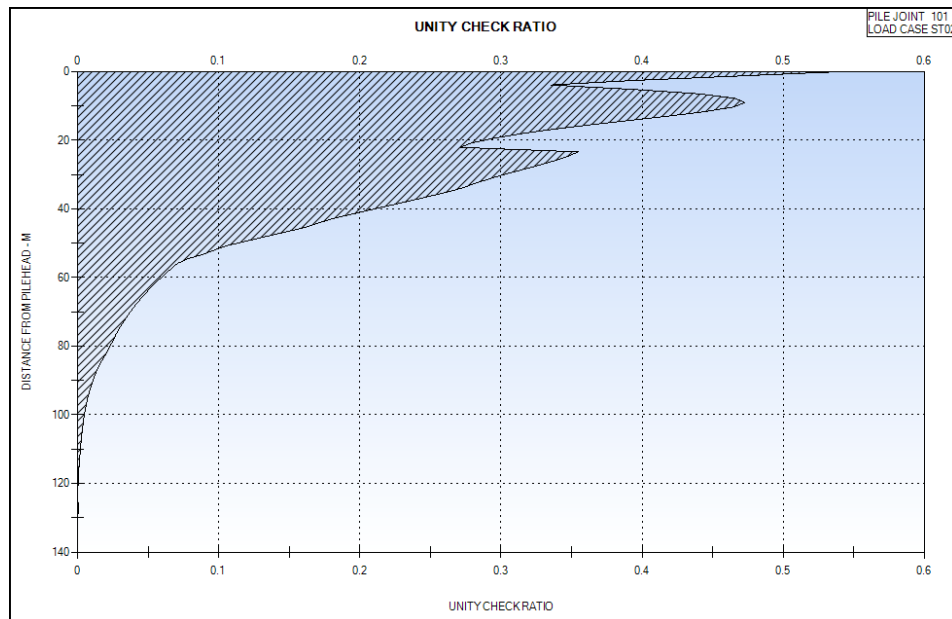


Figure 22: Unity check ratio for pile joint 101 under load case ST02 with wall thickness of 4.0 cm

The Figure 21 and 22 above show the unity check ratio for the pile joint 101, under load case ST02 with 6.0 cm and 4.0 cm wall thickness respectively. Pile with 6.0 cm wall thickness has the highest value of unity check ratio of 0.35 meanwhile for pile with 4.0 cm wall thickness has the highest value of unity check ratio of 0.525. This comparison has shown that the thinner wall thickness, the higher the unity check ratio is.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

Based on the results obtained, the objective for this project has achieved. The pile responses has been illustrated into three (3) types of graph, each indicates different relationship. *P-y curve* indicates the relationship between the lateral soil resistance per unit length of pile and deflections of pile due to the resistance. *T-z curve* conversely specifies the relationship between the soil-pile shear transfer and local pile deflections at any depth meanwhile *q-z curve* shows the relationship between the mobilized end bearing resistance and axial pile tip deflections.

These graphs also has shown different pile responses in different soil properties that changes with depth. This has shown that the pile behaves differently under different soil properties. These findings has satisfy the author's objectives.

Deep foundations must be designed in such a way to support the substantial lateral loads as weel as axial loads. The design for axially loaded deep foundations may be adequately designed by simple static methods, however for the lateral loads is more complex [19]. The soil-structure interaction compatibility should be satisfied and the nonlinear soil response complicates the solutions. Therefore, *p-y curve* is needed in partial of pile design. The behaviour of pile depends on stiffness of the pile and soil mobilization of resistance in the surrounding soil.

Based on the author's judgement, according to the result achieved, the wall thickness of the pile can be reduced in order to accomplish an optimized design. In the future, the pile diameter and pile length can be considered into changing the parameters because these parameters also affects the pile behaviour in terms of deflections and also unity check ratio. These parameters can contibute to obtain an optimized and reliable design of a jacket structures if it is considered in the analysis.

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