Dynamic Behavior of Jacket Type Offshore Structure

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ABSTRACT

Unlike structures in the air, the vibration analysis of a submerged or floating structure such as offshore structures is possible only when the fluid-structures is understood, as the whole or part of the structure is in contact with water. Using the commercial F.E.A. program ANSYS (v.12.0) (to model the Winkler and Brick-full bond models) and program ABAQUS(v.6.9) (to model the Brick-interface model), the stress matrix considering a dynamic load was superposed on the stiffness matrix of the structure. A time domain solution is recommended, using the generalized Morison's equation by FORTRAN90 program to construct a program to calculate the wave forces, and Airy's linear and Second Order Stoke's wave theories are employed to describe the flow characteristics by using MAPLE13 program (to solve and apply the boundary conditions of the problems on Laplace's equation), and the results are compared and discussed. Both free and forced vibration analyses are carried out for two case studies.

KEYWORDS: Natural frequency, Fixed jacket offshore, Finite element method, Validity of wave theory, Dynamic analysis.

INTRODUCTION

The development of the offshore oil industry led to numerous installations of offshore platforms. The major use of these platforms is in the drilling for oil and gas beneath the seafloor. Other uses include, but are not limited to, military applications, navigational aid to ships and generating power from the sea.

Modal test which is one of the examination assessments is the method to analyze the dynamic characteristics. It is a method to measure frequencies, finding the natural frequency of the fixed jacket platform and forecasting the vibration phenomenon for mode shape. In case of domestic applications, the study of fixed jacket platform system has been actively conducted in some big corporations, small and medium enterprises and national researches. But it was impossible to obtain systematic data.

Based on this design, we calculated the complex load on the offshore (fixed jacket platform). Calculations in the offshore have to take into account wave load(caused by waves). However, since current load is insignificant compared to wave load, it can be ignored.

LOAD CALCULATION IN OFFSHORE

The loads that strongly affect offshore structures can be classified into the following categories;
1. Permanent (dead) loads. 2. Operating (live) loads.
3. Environmental loads. 4. Construction-installation loads. 5. Accidental loads.

Whilst the design of buildings onshore is usually influenced mainly by the permanent and operating loads, the design of offshore structures is dominated by environmental loads, especially waves, the impact and loads of which arise in the various stages of constructional installation.

Environmental loads are those caused by environmental phenomena such as wind, waves, currents, tides, ice and marine growth. Their characteristic parameters, defining design load values, are determined in special studies on the basis of available data.

In this study, the effect of wave, impact and ground motion will be taken into consideration in the dynamic analysis of the offshore structures.

For an offshore platform, the most important loads are the hydrodynamic loads and impact loads which are included in this study. These hydrodynamic forces are governed by sea waves, while impacts usually occur during berthing of ships. The most widely used approach to determine the hydrodynamic loads (water wave forces) acting on the members of a structure is normally the semi-empirical Morison's equation. This equation is originally developed to compute the hydrodynamic forces acting on a cylinder at a right angle to the steady flow, and is given as:

$$dF = \rho\pi \frac{D^2}{4} C_m \, a \, ds + \frac{1}{2} \rho D \, C_d \, V \, |V| \, ds \quad (1)$$

In this equation, it is assumed that the wave force is acting on the vertical distance (ds) of the cylinder due to the velocity (v) and acceleration (a) of the water particles, where (ρ) is the density of water, (D) is the cylinder diameter, (C_m) and (C_d) are inertia and drag coefficients (Madhujit and Sinha, 1988).

Various methods exist for the calculation of the hydrodynamic loads on an arbitrarily oriented cylinder by using Morison's equation. The method adopted here assumes that only the components of water particles and accelerations normal to the member produce loads (Qian and Wang, 1992).

To formulate the hydrodynamic load vector F_w, consider the single, uniform, cylindrical member (i) between nodes I and J as shown in Fig. (1). The forces are found by the well-known semi-empirical Morison's formula (Equation(2)). It also represents the load exerted on a vertical cylinder, assuming that the total force on an object in the wave is the sum of drag and inertia force components. This assumption (introduced by Morison) takes the drag term as a function of velocity and the inertia force as a function of acceleration (Zienkiewicz et al., 1978; Dean and Dalrymple, 1984; McCormick, 1973), so that:

$$F_n = \rho.\frac{\pi}{4}D^2.C_m.v_n' - (C_m-1)\rho.\frac{\pi}{4}D^2.u_n'' + \qquad (2.a)$$
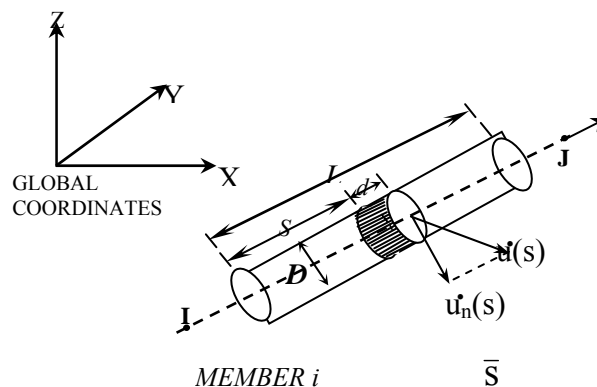$$\frac{1}{2}\rho.D.C_d.(v_n - u_n').|(v_n - u_n')|$$

MEMBER i $\overline{S}$
Figure 1: Water particle velocities along member i

which can be simplified to:

$$F_n = \rho.\frac{\pi}{4}D^2.C_m.v'_n + \frac{1}{2}\rho.D.C_d.(v_n).|(v_n)| \qquad (2.b)$$

where:

F_n: nodal hydrodynamic force normal to the cylinder.

D: Outer diameter of cylinder. ρ: Sea water density.

C_d: Drag coefficient. v'_n: Water particle acceleration.

C_m: Inertia coefficient. v_n: Water particle velocity.

u'_n: Structural velocity. u''_n: Structural acceleration.

Equation (2.b) neglects the non-linear terms of drag coefficient (Al-Jasim, 2000; Sarpakaya and Issacom, 1981), water particle velocity and acceleration can be evaluated by potential velocity computed from wave theories, the absolute value of velocity is needed to preserve the sign variation of the force.

Generalizing the one-dimensional form of Morison's equation to the three-dimensional from of the hydrodynamic force per unit length along the beam element at location (s) measured from its end to the nearest node is given as (Zienkiewicz et al., 1978):

$$\{F(s)\} = \rho.\frac{\pi}{4}.D^2.C_m.\{v'_n(s)\} + \frac{\rho}{2}.D.C_d.\{v_n(s)\}.\{|v_n(s)|\} \qquad (3)$$

The hydrodynamic force per unit length vector is given as:

$$\{F_w(s)\} = \begin{bmatrix} F_x(s) \\ F_y(s) \\ F_z(s) \end{bmatrix} \qquad (4)$$

and the normal water particle velocity and acceleration vectors are given as:

$$\{V_n(s)\} = [s]\{u(s)\} \text{ and } \{a_n(s)\} = [s]\{a(s)\} \qquad (5)$$

$$[s] = I - \bar{S}.\bar{S}^T = \begin{bmatrix} 1-S_1^2 & -S_1S_2 & -S_1S_3 \\ -S_1S_2 & 1-S_2^2 & -S_2S_3 \\ -S_1S_3 & -S_2S_3 & 1-S_3^2 \end{bmatrix} \qquad (6)$$

where; I: is the (3x3) identity matrix, $\bar{s}$: is the unit directional vector along the member and S_1, S_2 and S_3

are directional cosines in x, y and z directions, respectively (Zienkiewicz et al., 1978), and:

$$\{V(s)\} = \begin{bmatrix} v_x(s) \\ v_y(s) \\ v_z(s) \end{bmatrix} \qquad (7.a)$$

$$\{a(s)\} = \begin{bmatrix} a_x(s) \\ a_y(s) \\ a_z(s) \end{bmatrix} \qquad (7.b)$$

These velocity and acceleration components are derived in detail in the next section.

Now, to calculate the load vector in global coordinates system, the element is divided into two parts by using equation (4) distributing the wave effects on the beam element equally to the end nodes as nodal forces. Therefore, the element of hydrodynamic load vector $\{f^e\}$ corresponding to the element nodal displacement vector $\{q\}$ can be expressed as follows:

$$\{f^e\} = \begin{pmatrix} f_{x1} \\ f_{y1} \\ f_{z1} \\ 0 \\ 0 \\ 0 \\ f_{x2} \\ f_{y2} \\ f_{z2} \\ 0 \\ 0 \\ 0 \end{pmatrix} \qquad (8)$$

ENVIRONMENTAL LOADS

The external loads include hydrostatic pressure, wind, waves, currents, tide, ice, earthquakes,

temperature, fouling, marine growth and scouring.

Hydrodynamics of Water Waves

Ocean waves are undulations in the water's surface resulting in the transfer and movement of energy. The disturbance is propagated by the interaction of disturbing (e.g. wind) and restoring (e.g. gravity) forces. The energy in most ocean waves originates from the wind blowing across the water's surface. Large tsunami or seismic sea waves are generated by earthquakes, space debris, volcanic eruptions or large marine landslides. On the other hand, tides, the largest of all ocean waves, result from the combined gravitational force exerted on the oceans by the sun and the moon (Glossary, 2002).

Dean (1968, 1974) presented an analysis by defining the regions of validity of wave theories in terms of parameters H/T and d/T, since T is proportional to the wavelength. Le Mehaute (1969) plotted a figure to aid in the selection. This figure shows the validity of different water waves based on the water depth(d), the wave height(H) and the wave length (L) as shown in Fig.(2).

For the sake of comparison, two wave theories are used to evaluate the velocity and acceleration of water particles: Airy wave theory (linear) and Stoke's second order theory (non-linear). The loads resulting from using these theories with Morison's equation are applied on the platform and the resulting bending moments, axial forces and displacement variations with time of the platform (at a point on the top of the platform and near the sea bed on the platform) are plotted to show the differences between the theories.

Both Le Mehaute and Dean recommend cnoidal theory for shallow-water waves of low steepness and Stoke's higher order theory for steep waves in deep water. For low steepness waves in transitional and deep water, linear theory is adequate but other wave theories may also be used in this region. Fenton's theory is appropriate for most of the wave parameter domain (Demirbilek and Vincent, 2002).

Airy linear theory is the common linear wave theory. Stoke's theory assumes that wave motion properties such as velocity potential (ϕ) can be represented by a series of small perturbations. The linear Airy wave theory can be used when the wave height to wave length $(\frac{H}{L}) \leq \frac{1}{50}$ as given in (McCormick, 1973).
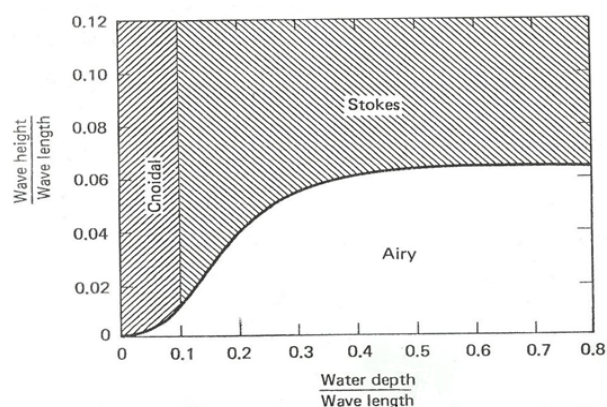

Figure 2: Validity of water wave theory (Al-Jasim, 2000)

Hydrodynamic Forces

In order to calculate the hydrodynamic forces acting on the members of an offshore structure, it is required first to describe the sea state as occurring in nature which determines the wave surface profile and then the characteristics of the water wave particles hitting the structure. It is always assumed that the water waves are represented as two-dimensional plane waves that propagate over a smooth horizontal bed in water of a constant undisturbed depth (h).

It is also assumed that the waves maintain a permanent form, which means that there is no underlying current and that the free surface is uncontaminated. The fluid (water) is taken to be incompressible and inviscid and the flow to be irrotational (Dean and Dalrymple, 1984).

Fig.(3) indicates the general form of the xz-plane wave train conforming to these assumptions. Here, the wave is progressive in the positive x-direction and the z-axis measured positive upwards from the mean water level, the wave height being H, the wave length L, the wave period T and η the elevation of the water above the mean water level (Al-Salehy, 2002).
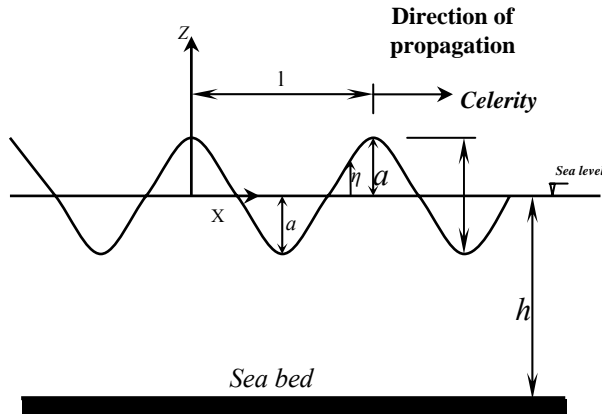

Figure 3: Definition of terms used in the wave equations

The surface must satisfy the special linear form of the wave equation of Laplace solution to obtain the velocity potential (ϕ) and is subjected to the above conditions and linearized boundary conditions.

$$\frac{\partial^2 \phi}{\partial X^2} + \frac{\partial^2 \phi}{\partial Z^2} = 0 \qquad (9)$$

To solve Equation (9), some BCs must be satisfied (Al-Salehy, 2002; Dean and Dalrymple, 1984; McCormick, 1973):

1- At the Sea Surface (Air-Water Interface)

(a) The velocity of a particle must be tangential to the surface, the kinematic condition is:

$$\frac{\partial \eta}{\partial t} + \frac{\partial \phi}{\partial x} \cdot \frac{\partial \eta}{\partial x} - \frac{\partial \phi}{\partial z} = 0 \qquad \text{at } z = \eta \quad (10.a)$$

(b) The pressure is zero and the energy equation must be satisfied, the dynamic condition is:

$$\frac{\partial \phi}{\partial t} + g\eta + \frac{1}{2}\left[\left(\frac{\partial \phi}{\partial x}\right)^2 + \left(\frac{\partial \phi}{\partial z}\right)^2 \right] = f(t) \qquad \text{at } z = \eta \quad (10.b)$$

2- At the Sea Floor Where the Vertical Velocity is Zero

That is:

$$\frac{\partial \phi}{\partial z} = 0 \qquad \text{at } z = -h \qquad (10.c)$$

The components of water particle velocity can be given as (Essa and Al-Janabi, 1997):

$$v_x = \frac{\partial \phi}{\partial x} \qquad (11.1a)$$

$$v_z = \frac{\partial \phi}{\partial z} \qquad (11.1b)$$

whereas, the components of the local particle acceleration, which are only taken into account in the computation of the hydrodynamic force are given as:

$$a_x = \frac{\partial v_x}{\partial t} \qquad (11.2a)$$

$$a_z = \frac{\partial v_z}{\partial t} \qquad (11.2b)$$

The major problem in solving for (ϕ) arises from the boundary conditions to be applied at the air-water interface, η (t), being itself part of the solution sought. Therefore, there are several solutions in common use. These are linear wave theory in deep water, Stoke's higher order wave theories, stream potential function theory and Cnoidal theories in shallower water (Sarpakaya and Issacom, 1981).

In the present study, only the linear (Airy) wave

theory and Stoke's second order theory are considered to compute the characteristics of water particles and the hydrodynamic forces.

To find velocities and accelerations that are used in Morison's equation, Laplace equation must be solved by considering the BCs at the sea surface $\eta(t)$, being itself part of the solution sought. Therefore, different wave theories are used as mentioned.

The linear wave theory (Airy theory) is used only to find velocities and accelerations at different depths, locations and times.

Wave Theories

All wave theories obey some form of wave equation in which the dependent variable depends on physical phenomena and boundary conditions (Al-Salehy, 2002). In general, the wave equation and the boundary conditions may be linear or non-linear.

Some of these theories are shown in profiles Fig.(4):

1- Airy Wave Theory: (Sinusoidal Waves)
This theory:
- is most accurate for low amplitude waves in deep water.
- is less accurate for predicting wave behavior in shallow water.
- is the most commonly used wave theory because it is the least mathematically complex one.
- does not take into account the effect of wave height in determining wave velocity.

Stoke's Wave Theory (Trochoidal Waves)
This theory:
- can be used for deep, intermediate and shallow water waves.
- is mathematically complex.
- takes into account the effect of wave height on velocity.

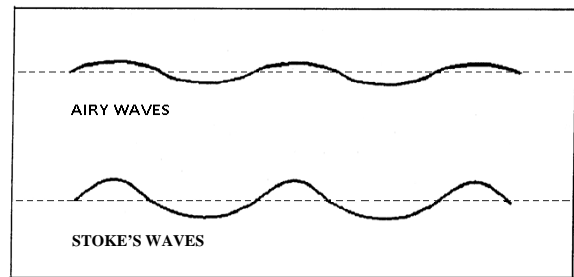- more accurately describes orbital velocity asymmetries.

Figure 4: Wave profile shapes of different progressive gravity waves (Demirbilek and Vincent, 2002)

Airy Theory (Linear Wave Theory)

This theory is termed following a linearization of the boundary conditions at the air-water interface. In this theory, the essential idea or restriction is that the wave height (H) must be much smaller than both the wave length (L) and the still water depth (d); that is (H<<L, d). The linear wave theory for two-dimensional, free, periodic, waves is developed by linearizing the equations that define the free surface boundary conditions. With these and the bottom BCs, the periodic velocity potential is sought that satisfies the requirements of irrotational flow. The free surface BCs may now be applied directly at the still water level (Dean and Darlymple, 1984).

Therefore, the free surface BCs, as expressed in Equation (12.a) and Equation (12.b), are reduced to:

$$\frac{\partial \phi}{\partial z} - \frac{\partial \eta}{\partial t} = 0 \qquad \text{at } z=0 \qquad (12.a)$$

$$\frac{\partial \phi}{\partial t} + g.\eta = 0 \qquad \text{at } z=0 \qquad (12.b)$$

By using separation of variables and BCs (Equation 12.a, b), the velocity potential (ϕ) can be found as (Othman and Dawood, 2002):

$$\phi = \frac{\pi.H}{k.T} \cdot \frac{\cos[k(z+h)]}{\sin(k.h)} \cdot \sin(kx - \Omega.t) \qquad (13)$$

in which:

k: wave number ($k = \frac{2.\pi}{L}$); T: wave period;

Ω: wave circular frequency ($\Omega = \frac{2.\pi}{T}$);

h: depth of water; H: wave height.

Now, it is simple to obtain the velocities and accelerations as:

$$V_x = \frac{\partial\varphi}{\partial x} = \frac{\pi.H}{T} \cdot \frac{\cosh[k(z+h)]}{\sinh(kh)} \cdot \cos(kx - \Omega t) \qquad (14.a)$$

$$V_z = \frac{\partial\varphi}{\partial z} = \frac{\pi.H}{T} \cdot \frac{\sinh[k(z+h)]}{\sinh(kh)} \cdot \sin(kx - \Omega t) \qquad (14.b)$$

$$a_x = \frac{\partial v_x}{\partial t} = \frac{\partial^2\varphi}{\partial t \partial x} = \frac{2\pi^2.H}{T^2} \cdot \frac{\cosh[k(z+h)]}{\sinh(kh)} \cdot \sin(kx - \Omega t) \qquad (15.a)$$

$$a_z = \frac{\partial v_z}{\partial t} = \frac{\partial^2\varphi}{\partial t \partial z} = \frac{2\pi^2.H}{T^2} \cdot \frac{\cosh[k(z+h)]}{\sinh(kh)} \cdot \sin(kx - \Omega t) \qquad (15.b)$$

These velocities and accelerations in Equation (14) and Equation (15) are used in Morison's equation to calculate load vectors of hydrodynamic loading by using linear Airy wave theory after being transformed from global coordinates for each member of the offshore platform.

Stoke's Second Order Wave Theory

Stokes (1874) employed perturbation techniques to solve the wave boundary value problem and developed a theory for finite—amplitude wave that he carried to the second order. In this theory, all the wave characteristics (velocity potential, celerity, surface profile, particle kinetics…etc) are formulated in terms of a power series in successively higher orders of the wave steepness (H/L).

A condition of this theory is that (H/d) should be small so that the theory is applicable only in deep water and a portion of the intermediate depth range.

For engineering applications, the second-order and possibly the fifth-order theories are the most commonly used (Sorensen, 2006).

Stoke's wave expansion method is formally valid under the conditions (Iraninejad, 1988):

H/d << (kd)2 for kd < 1 and H/L << 1.

Stoke's wave theory is considered most nearly valid in water where the relative depth (D/L) is greater than about (1/10) (Patal, 1989).

Stoke's theory would be adequate for describing water waves at any depth of water. In shallow water, the convective terms become relatively large, the series convergence is slow and erratic and a large number of terms is required to achieve a uniform accuracy (Muga, 2003).

The fluid particle velocities are then given by:

$$V_x = \frac{\pi.H}{T} \cdot \frac{\cosh[k(z+h)]}{\sinh(kh)} \cdot \cos(kx - \Omega t) + \frac{3(\pi.H)^2}{4TL} \cdot \frac{\cosh[2k(z+h)]}{\sinh^4(kh)} \cdot \cos 2(kx - \Omega t) \qquad (16.a)$$

$$V_z = \frac{\pi.H}{T} \cdot \frac{\sinh[k(z+h)]}{\sinh(kh)} \cdot \sin(kx - \Omega t) + \frac{3(\pi.H)^2}{4TL} \cdot \frac{\sinh[2k(z+h)]}{\sinh^4(kh)} \cdot \sin 2(kx - \Omega t) \qquad (16.b)$$

The fluid particle accelerations are then given by:

$$a_x = 2\frac{\pi^2.H}{T^2} \cdot \frac{\cosh[k(z+h)]}{\sinh(kh)} \cdot \sin(kx - \Omega t) + \frac{3\pi^3.H^2}{T^2L} \cdot \frac{\cosh[2k(z+h)]}{\sinh^4(kh)} \cdot \sin 2(kx - \Omega t) \qquad (17.a)$$

$$a_z = -\frac{2\pi^2 . H}{T^2} . \frac{\sinh[k(z+h)]}{\sinh(kh)} . \cos(kx - \Omega t) -$$

$$\frac{3\pi^3 H^2}{T^2 L} . \frac{\sinh[2k(z+h)]}{\sinh^4(kh)} . \cos 2(kx - \Omega t) \quad (17.b)$$

These velocities and accelerations in Equation(16) and Equation(17) are used in Morison's equation to calculate load vectors of hydrodynamic loading by using Stoke's wave theory after being transformed from global coordinates for each member of the offshore platform.

DYNAMIC ANALYSIS

In offshore structures, the applied loads (environmental loads) generally have a dynamic nature. To study the behavior of these structures, free vibration and forced vibration must be considered in order to understand the actual (as possible) behavior and response.

Free Vibration Analysis

The equations of motion for a freely, undamping system can be obtained by omitting the damping matrix and applied load vector from Equation(2) to get:

$$[M]\{\ddot{U}\} + [K]\{U\} = 0 \qquad (18)$$

The solution of this equation, which is known as an eigenproblem, can be postulated to be of the form:

$$\{U\} = \{\overline{U}\} \quad sin \ \omega(t-t_0) \qquad (19)$$

where $\{\overline{U}\}$ is the vector of amplitudes of motion (which does not change with time). The substitution of equation (19) into the equation of free vibration equilibrium (18) gives:

$$-\omega^2 [M]\{\overline{U}\} sin\, \omega(t-t_0) + [K]\{\overline{U}\} sin\ \omega(t-t_0) = \{0\}$$
$$(20)$$

Since the sine term is arbitrary and may be omitted, the above equation is reduced to the form:

$$[[k] - \omega^2 [M]]\{\overline{U}\} = \{0\} \qquad (21)$$

According to the Cramer's rule, the solution of this set of simultaneous equations is of the form:

$$\{\overline{U}\} = \frac{\{0\}}{\left| [K] - \omega^2 [M] \right|} \qquad (22)$$

Hence, a nontrivial solution is possible only when the denominator determinant vanishes. In other words, finite amplitude free vibrations are possible only when:

$$\left| [K] - \omega^2 [M] \right| = 0 \qquad (23)$$

In general, equation (23) results in a polynomial equation of degree N in ω^2 for a system having N degrees of freedom. This polynomial is known as the characteristic equation of the system. The N roots of this equation, which are called the eigenvalues, represent the frequencies of the N modes of vibration which are possible in the system (Paz, 1980).

Corresponding to each eigenvalue, there will be an eigenvector, or natural mode $\{\overline{U}_n\}$, where:

$$\{\overline{U}_n\} = \begin{bmatrix} \overline{U}_1 \\ \overline{U}_2 \\ \vdots \\ \vdots \\ \overline{U}_{N-1} \\ \overline{U}_N \end{bmatrix} \qquad n=1,2,3,...N. \qquad (24)$$

The modes are determined only within a constant multiplier. Thus, modes can be normalized in any convenient manner. Therefore, if the value of one of the elements of the natural mode vector $\{\overline{U}_N\}$ is assigned as a specified value, say a unity for the first element, then the remaining (N-1) elements are uniquely determined. The process of normalizing a

natural mode is called normalization and the resulting modal vectors are called normal modes. For convenience, these normal mode vectors are usually expressed in a dimensionless form by dividing all the components by one reference component. The resulting vector corresponding to the nth eigenvalue is called the nth mode shape $\{\phi n\}$ (Clough and Penzien, 1975), thus:

$$\{\phi_n\} = \begin{bmatrix} \phi_{1n} \\ \phi_{2n} \\ \phi_{3n} \\ \vdots \\ \vdots \\ \phi_{Nn} \end{bmatrix} = \frac{1}{\overline{U}_{kn}} \begin{bmatrix} 1 \\ \overline{U}_{2n} \\ \overline{U}_{3n} \\ \vdots \\ \vdots \\ \overline{U}_{Nn} \end{bmatrix} \tag{25}$$

In which $\overline{U}_{kn}$ is the reference component.

The square matrix made up of the N mode shapes will be represented by$[\phi]$:

$$[\phi] = \begin{pmatrix} \phi_{11} & \phi_{12} & \phi_{13} & \cdots & \phi_{1N} \\ \phi_{21} & \phi_{22} & \ddots & \ddots & \phi_{2N} \\ \vdots & \ddots & \ddots & \ddots & \vdots \\ \vdots & \ddots & \ddots & \ddots & \vdots \\ \phi_{N1} & \cdots & \cdots & \cdots & \phi_{NN} \end{pmatrix}$$

These mode shapes have certain special properties, which are very useful in the structural dynamic analysis. These properties are called the orthogonallity relationship (Biggs, 1964), which can be expressed for any system as:

$$\{\phi_m\}^T [M] \{\phi_n\} = 0 \qquad \text{for } m \neq n \tag{26}$$

and

$$\{\phi_m\}^T [K] \{\phi_n\} = 0 \qquad \text{for } m \neq n$$

ANSYS12.0 and ABAQUS6.9 programs use a method of subspace iteration, this method requires the Jackobi method, Ritz reduction functions and iterative procedure as detailed in (Zienkiewicz et al., 1978; Bathe and Wilson, 1977).

Forced Vibration Analysis

To understand the response of offshore structures subjected to a load of dynamic nature, forced vibration analysis is used to get the response of the platforms to these forces.

There are different methods available for evaluating the structural response to dynamic loads, such as frequency domain analysis, direct integration method,…etc. These are Newmark's implicit, most flexible step-by-step integration methods in time domain, presented by Newmark (Zienkiewicz et al., 1978; Bathe and Wilson, 1977; ANSYS, 1997; Vugts and Hayes, 1979). This method (including an improved algorithm called HHT (Chung and Hulbert, 1993) and using finite difference expansions in the time interval (Δt)) is based on the following expressions for the velocity and displacement at the end of the time interval (Bathe and Wilson, 1977).

$$\dot{u}_{t+\Delta t} = \dot{u}_t + \Delta t (1-\delta) \ddot{u}_t + \Delta t \delta \ddot{u}_{t+\Delta t} \tag{27}$$

$$u_{t+\Delta t} = u_t + \Delta t \dot{u}_t + (\Delta t)^2 (\frac{1}{2}-\alpha) \ddot{u}_t + (\Delta t)^2 \alpha \ddot{u}_{t+\Delta t} \tag{28}$$

where α,δ are selected to produce the desired accuracy and stability. One of the most widely used methods is the constant average acceleration method when (δ=0.5, α=0.25) which is a conditionally stable method without numerical damping. This method is called an (implicit integration method), since it satisfies the equilibrium equation of motion at time t+Δt, or:

$$M \ddot{u}_{t+\Delta t} + C \dot{u}_{t+\Delta t} + Ku_{t+\Delta t} = F_{t+\Delta t} \tag{29}$$

This equation can be solved by iteration; however Equations (27), (28) and (29) can be combined into a step by step algorithm which involves the solution of a set of equations. Each time step is of the form:

$$K^* . U_{t+\Delta t} = F^* \tag{30}$$

Since K* is not a function of time, it can be triangularized only once at the beginning of the calculation. The computer solution time for this type of algorithm is basically proportional to the number of time steps required.

LOAD CALCULATION IN OFFSHORE

In order to investigate the dynamic response method in this study, a jacket type model of an offshore platform type structure has been considered in this paper, and for the sake of comparison, two wave theories are used to evaluate the velocity and acceleration of the water particles; namely the Airy wave theory (linear) and Stoke's second order theory (non-linear). The loads resulting from using these theories with Morison's equation are applied on the platform and the resulting bending moments, axial forces and displacement variations with time of the platform (at a point on the top of the platform and near the sea bed on the platform) are plotted to show the differences between the theories.

Finite element method is used for both spatial and temporal coordinate systems.

To take the effect of soil-structure interaction into account, the two models are analyzed using Winkler, Interface (F.E.) and Full Bond (F.E.) methods which have different treatments in representing the soil-structure relation as discussed earlier. The length of pile embedment in the soil, the end conditions of the pile: spring, hinged and fixed, and modeling of inertia forces as lumped mass are taken into account.

Structure Description

Fig. (5) shows that the fixed jacket offshore platform model described in (Al-Salehy, 2002; Carpon et al.) is adopted. The frame is descritized into (178) beam elements for superstructure and (240) beam elements embedded in elastic soil used to model the four piles embedded to a depth of (60m) below mudline in the sea bed that support the platform and (303) nodes.

A FORTRAN program is constructed to calculate the water particle velocity and acceleration by using two theories: Airy theory and Stoke's second order theory. Also, this program calculates the wave forces at each node in the superstructure embedded in the fluid medium by using Morison's equation.

The forces on each member are calculated and then distributed on the nodes for x, y and z-directions.

The deck mass is modeled using the lumped mass in five nodes forming a pyramid (Al-Salehy, 2002; Carpon et al.). This model is the same model adopted by Al-Salehy (2002) taking the Winkler model with isolated springs at nodal points only to represent the soil resistance taking into account normal and tangential moduli of subgrade reaction in three directions for each node. Table (1) contains the properties of the piles, while the properties of the steel and physical properties of the soil are defined in Table (2).

Table 1. Properties of piles

Member Number	Shape	Outer Dimension (m)	Thickness (mm)
1	Circular Pipe	2.0	55
2	Circular Pipe	1.525	38
3	Circular Pipe	0.91	25
4	Circular Pipe	0.75	25

Table 2. Properties of material

Material	Modulus of elasticity (Es) (kN / m^2)	Density (ton / m^3)	Poisson's ratio (μ)
Steel	200E6	7.8	0.3
Soil	45E3	1.9	0.3

Soil Properties

The effect of soil type on the dynamic response is studied by using three types of soil in addition to the soil of the case study 1. The soil properties including elastic modulus, Poisson's ratio and density are given in Table (3) (Rubin and Coppolina, 1983).

Table 3. Physical properties of soils

Soil	Modulus of elasticity (Es) (kN / m^2)	Density (ton / m^3)	Poisson's ratio (μ)
Soft clay	15E3	1.8	0.3
Medium clay	30E3	1.9	0.4
Hard clay	60E3	1.9	0.3

Free Vibration Analysis

Free vibration analysis is carried out taking into account the length of pile embedment in the soil, the end conditions of the pile: spring, hinged and fixed, and modeling of inertia forces as lumped mass.

Table (4) compares the fundamental four modes of natural frequencies of the present work for spring pile tip in Winkler model and for fixed pile tip in [Brick model(Full bonding) and Brick model with interface element] with that reported by Al-Salehy (2002). Al-Salehy used isolated springs to represent the soil-structure interaction. The tabulated data shows that the results obtained by the present work for Winkler model are higher than those given by Al-Salehy for sway,

bending and axial modes due to stiffer model adopted in the present study. But the values of natural frequencies are close in torsion mode. This is because the pile legs of the structure prevent the superstructure from rotation and there is no effect of the torsional stiffness of the piles at different lengths of the piles on the torsional stiffness of the entire structure.

The main variables adopted in the free vibration analysis are soil properties (according to type of soil). The first four values of cyclic natural frequencies obtained from the different three models (Winkler, Full bond and Interface) (Al-Salehy, 2002) are shown in Table (5).

Table 4. Natural frequencies for basic modes of vibration for different models of soil-structure interaction for case study

Mode	Present work (ANSYS and ABAQUS) Lumped (Hz)			Al-Salehy (2002) (STAADIII) (Hz)
	Winkler	Brick (Full Bonding)	Brick (Interface)	
Sway	0.38297	0.37947	0.31137	0.207
Torsion	0.69980	0.69789	0.57251	0.725
Bending	1.51347	1.47543	1.52865	1.507
Axial	1.53777	1.51501	1.55023	0.940

Table 5. First four natural frequencies for different types of soils (case study)

Soil Type	Model		
	Winkler (Hz)	Brick (Full Bond) (Hz)	Brick (with Interface Element) (Hz)
Soft clay	0.11534	0.08553	0.06384
	0.3401	0.32121	0.2397
	1.09171	1.05369	1.0756
	1.3455	1.19533	1.13723

Medium clay	0.28723	0.2400	0.19694
	0.3499	0.34196	0.28053
	1.51347	1.3879	1.43796
	1.5378	1.4469	1.48054
Hard clay	0.62233	0.52192	0.44202
	0.7155	0.60223	0.50966
	1.9457	1.85244	1.6004
	2.08617	1.9361	1.96207

Forced Vibration Analysis

Different computer programs are used in calculating the dynamic response of the adopted case studies. Software ANSYS12 for Winkler and Brick (Full Bond) models and software ABAQUS6.9 for Brick (with interface element) model are used.

The effect of wave theory used was studied in the forced vibration analysis of offshore jacket platform with soil–structure interaction subjected to wave loading only.

To study the action of wave forces on the dynamic behavior of the offshore platform model shown in Fig. (5), the following wave parameters have been considered (Al-Salehy, 2002):

Wave height = 21 m, Wave period = 12 sec,

Wave length = 225 m, Water depth = 115 m,

Water density = 1025 kg/m^3.

The assumed value of the viscous damping ratio is (5%) for all modes of vibration considering (2%) as hydrodynamic damping, whereas the remaining (3%) simulates energy dissipation from sources other than hydrodynamic, called structural damping (Al-Salehy, 2002; Al-Jasim, 2000).

The inertia coefficient (C_M) and drag coefficient (C_D) are estimated from laboratory experiments indicating a general range from 1.2 to 2 for C_M and from 0.6 to 1.2 for C_D depending on flow conditions and surface roughness (Rubin and Cappolina, 1983; Faltinsen, 1990). In this study, these coefficients are

taken as (2.0) and (0.8), respectively.

Fig. (7) shows the force-time curve for node (A) due to wave loads in three directions using Airy theory and Morison's equation {Equation (2.b)}.

Fig. (8) shows the comparison between the variation of wave forces at node (A) in x-direction with time for Airy and Stoke's theories.

Figs. from (9) to (20) show (for the case of lumped mass approximation using Airy and Stoke's theories, respectively) for Winkler, Brick (full bonding) and Brick (with interface element) models, respectively, and (spring support for pile tip) with Winkler model, (fixed support for pile tip) with Brick (full bonding) and Brick (with interface element) models, the dynamic response for both the bending moment at deck level (member AB) and axial force in seabed level (member CD), respectively. It is shown that the two curves have a similar behavior and both of them reached a steady state condition after one period of time (only 24 sec), and it is shown that there is little increase in (max. value of the bending moment at deck level and in max. value of the axial force in sea bed level) when using the Stoke's theory .

In addition, the structure response in the case of full bond model is less than that resulting from using interface model, and the least is for Winkler model. This is because the interface element considers relative displacement or slip movement between soil and structure.

EFFECT OF WAVE THEORY USED ON THE PLATFORM DISPLACEMENT

In this case, the platform model is analyzed to evaluate the effect of using different wave theories on the variation of displacement with time at point A (as shown in Fig.(5)) on the platform due to the variation of wave forces calculated by the different wave theories.

Depending on the parameters (D/L) and (H/L) and from Fig. (2) (Al-Jasim, 2000), the appropriate wave

theory for the cases studied is given in Table (6).

Table (7) shows the maximum deck displacements for all end conditions of pile tip (spring, hinge and fixed) and for all soil-structure interaction representations (Winkler, Brick (full bonding) and Brick (with interface) models).

The comparison of the results of using other theories to using the most suitable theories is given in Table (8) for all end conditions of pile tip and for all soil-structure interaction representations.

The comparison between Airy and Stoke's theories by the plot of variation of displacements at node A with time for spring end condition of pile tip in Winkler model, and for fixed end condition of pile tip in Brick (full bond) and Brick (with interface element) models and for all soil-structure interaction representations is given in Fig. (21). It is obvious that there is a little difference in time variation of the displacement between the two theories for all soil-structure interaction representations.

Table 6. The most suitable wave theory for waves used

Case Study	Wave length (m)	$\frac{D}{L}$	$\frac{H}{L}$	Wave Theory
1	225	0.5	0.09	**Stoke's**
2	261	0.08	0.04	**Airy**

Table 7. Maximum deck displacement with end conditions and soil representations

Theory	Soil-Structure interaction model	End conditions of pile tip	Max.displacement (mm)
	Winkler	Spring	9.1075
		Fixed	8.3555
		Hinge	8.3555

Airy	Brick (full bonding)	Fixed	15.0972
		Hinge	15.0972
	Brick (with interface)	Fixed	19.9381
		Hinge	19.9381
Stoke's	Winkler	Spring	9.654
		Fixed	8.8568
		Hinge	8.8568
	Brick (full bonding)	Fixed	16.0332
		Hinge	16.0407
	Brick (with interface)	Fixed	21.1942
		Hinge	21.1942

Table 8. Comparison of maximum deck displacement with end conditions and soil representations from different theories

Winkler Model	
Case of comparison	**Displacement ratio**
$\frac{SAT}{SST}$	0.943
$\frac{FAT}{FST}$	0.943
$\frac{HAT}{HST}$	0.943
Brick (full bonding) model	
$\frac{FAT}{FST}$	0.942
$\frac{HAT}{HST}$	0.942
Brick (with interface) Model	
$\frac{FAT}{FST}$	0.941
$\frac{HAT}{HST}$	0.941

SAT: Spring end condition of pile tip, Airy theory.
FAT: Fixed end condition of pile tip, Airy theory.
HAT: Hinge end condition of pile tip, Airy theory.
SST: Spring end condition of pile tip, Stoke's theory.
FST: Fixed end condition of pile tip, Stoke's theory.
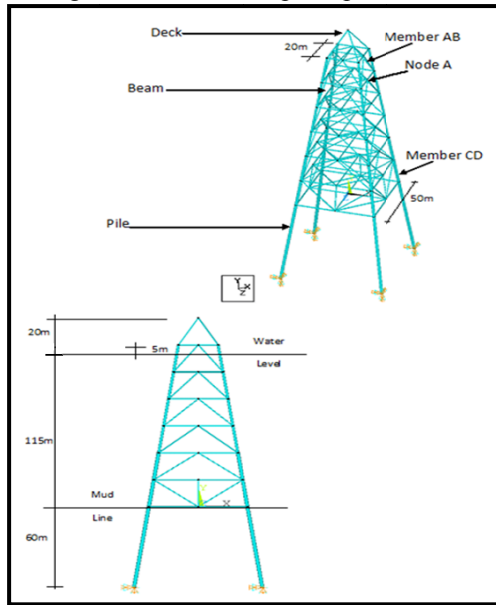
HST: Hinge end condition of pile tip, Stoke's theory.

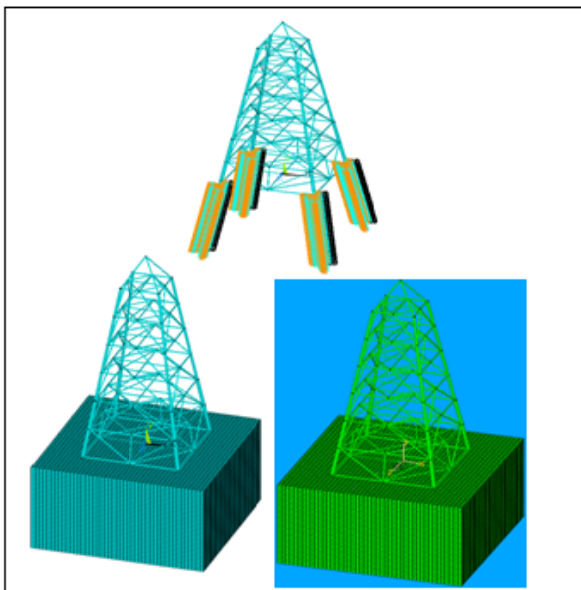**Figure 5: Geometry and dimensions of jacket type of offshore platform**

Figure 6: Model of the structure with soil-structure-interaction (3-D) (a,b,c)

Based on Table (6), the appropriate selection of a wave theory for the case study is Stoke's theory. However, Airy theory can be applied for this case study. The variation in the resulting displacements is less than 6% for the ratio between Airy and Stoke's theories, respectively, therefore, with such range of difference Airy wave theory can be used to reduce the computation effort, especially in the first design stages.
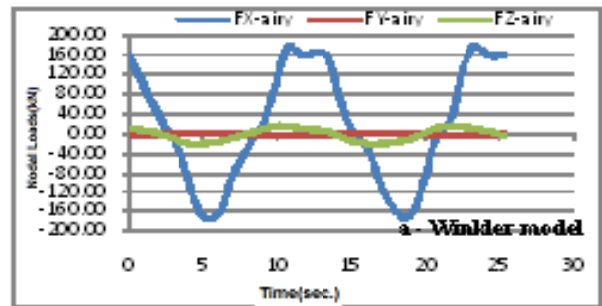
Figure 7: Wave forces at node (A) in x, y and z-directions with time for Airy theory in case study 1
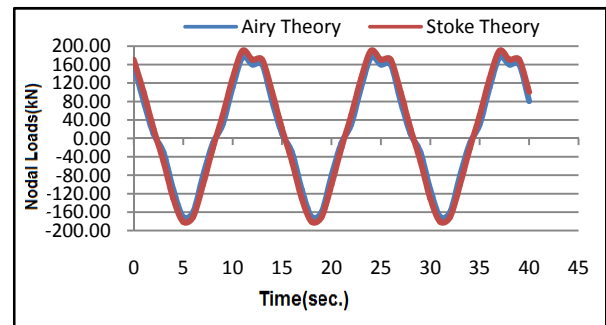
Figure 8: Comparison in variation of wave forces at node (A) in X- direction with time for case study 1 between Airy and Stoke's theories
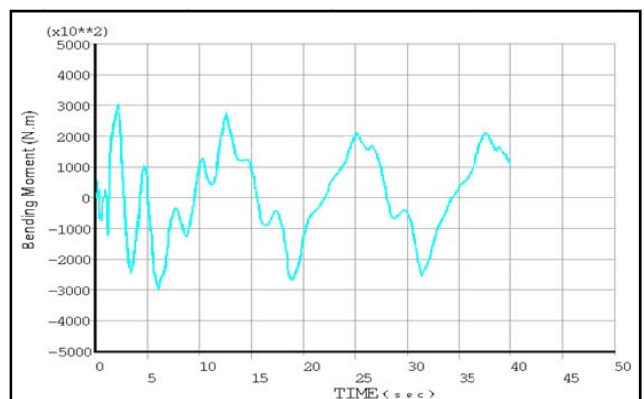
Figure 9: Time variation for the maximum bending moment at the level of deck due to wave forces for (Winkler) model by Airy theory
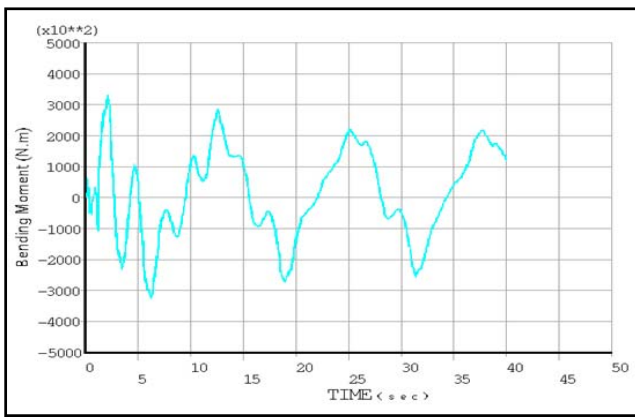
Figure 10: Time variation for the maximum bending moment at the level of deck due to wave forces for (Winkler) model by Stoke's theory

Figure 11: Time variation for the maximum axial force at the level of sea bed due to wave forces for (Winkler) model by Airy theory
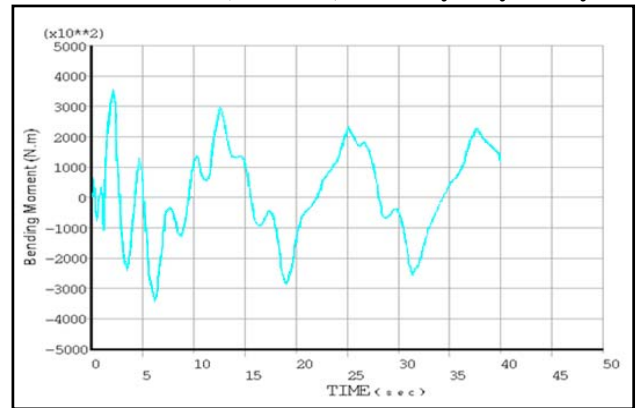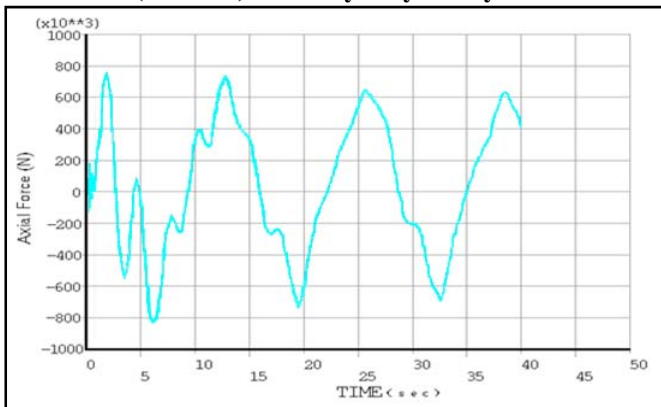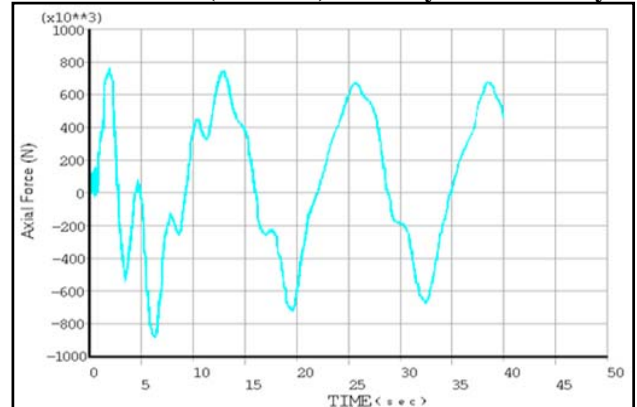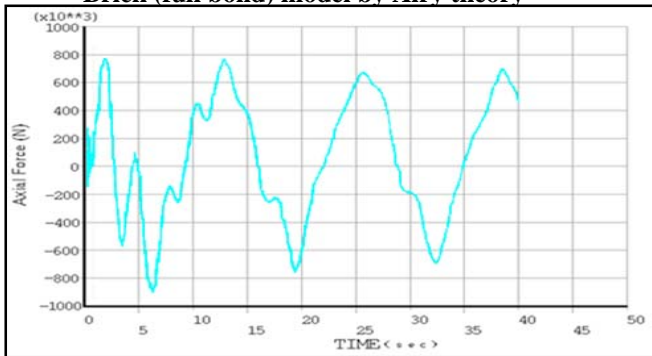

Figure 12: Time variation for the maximum axial force at the level of sea bed due to wave forces for
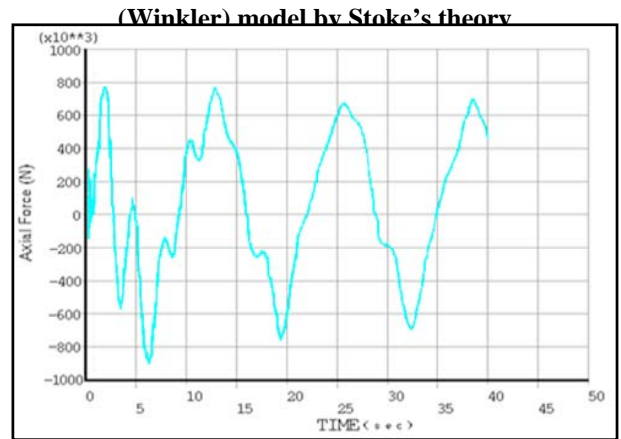
Figure 13: Time variation for the maximum bending moment at the level of deck due to wave forces for Brick (full bond) model by Airy theory

Figure 14: Time variation for the maximum bending moment at the level of deck due to wave forces for Brick (full bond) model by Stoke's theory

Figure 15: Time variation for the maximum axial force at the level of sea bed due to wave forces for

Brick (full bond) model by Airy theory

Figure 16: Time variation for the maximum axial force at the level of sea bed due to wave forces for Brick (full bond) model by Stoke's theory
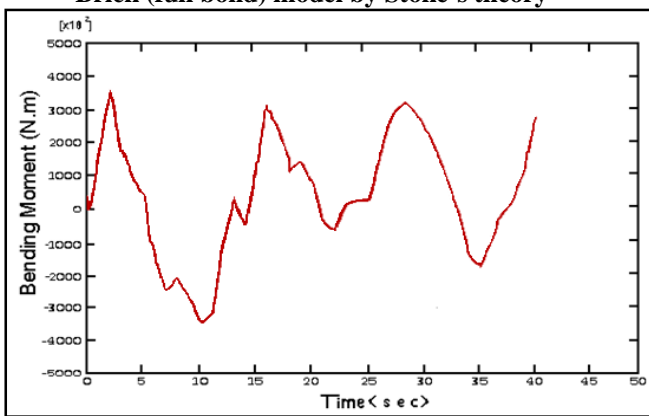

Figure 17: Time variation for the maximum bending moment at the level of deck due to wave forces for Brick (interface) model by Airy theory
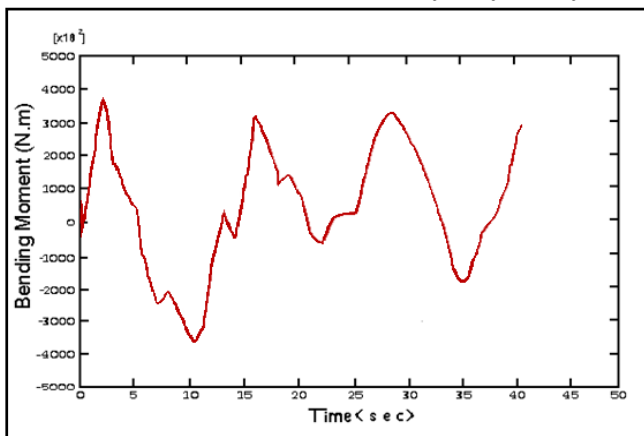

Figure 18: Time variation for the maximum bending moment at the level of deck due to wave

forces for Brick (interface) model by Stoke's theory
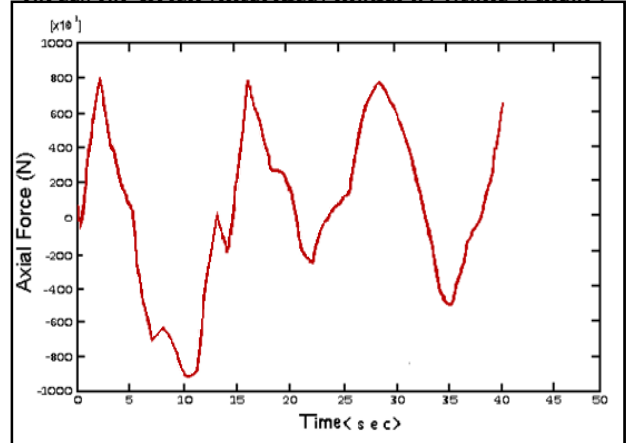

Figure 19: Time variation for the maximum axial force at the level of sea bed due to wave forces for Brick (interface) model by Airy theory
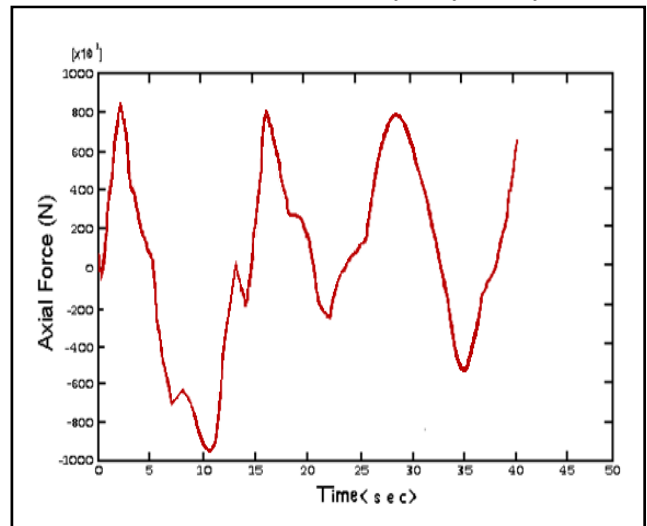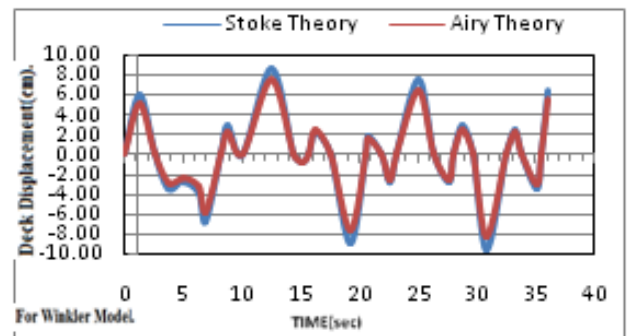

Figure 20: Time variation for the maximum axial force at the level of sea bed due to wave forces for Brick (interface) model by Stoke's theory
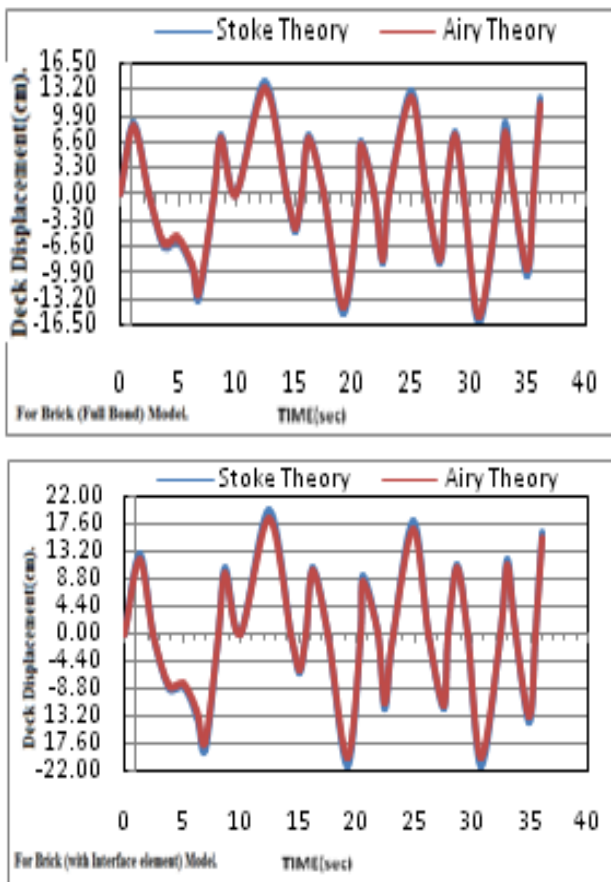
Figure 21: Time variation of deck displacement for different theories (Airy and Stoke's) and for different soil-structure interaction models

CONCLUSIONS

This research compared the results of forced vibration analysis that applied the periodic load to exciting force expressed by natural frequency results.

This research shows the concept of dynamic design about two types of wave theories. Therefore, the following concluding remarks can be drawn:

1. The overall dynamic response of the structure (offshore structure) is sensitive to the soil-structure interaction model. The natural frequencies of the interface model adopted in this work are less than natural frequencies of other models considered in this study, because the interface model considers the soil block in addition to the pile foundation leading to an increase in the model mass and a decrease in the natural frequencies.
2. The interface element model gave more acceptable and reasonable results than the other models considered in this study.
3. The interface model gives more realistic results in comparison with the Winkler model. This is because the Winkler model concentrates the physical properties of soil at the locations of springs, while in the interface model the soil parameters are distributed among the block elements, thus giving the true representation of soil behavior.
4. Wave characteristics represented by wave theories used in the present work have a smaller effect on behavior and response of the offshore platform.
5. The results showed a variation in maximum deck displacement of about (6%). For the case of different wave theories on case study 1, therefore, with such range of difference, Airy wave theory can be used to reduce the computation effort, especially in the first design stages.

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