SEISMIC RESPONSE ANALYSIS OF OFFSHORE STRUCTURE SUPPORTED BY GROUP PILE FOUNDATION BY CONSISDERING WAVE AND CURRENT ACTION

A Thesis Submitted
in Partial Fulfillment of the Requirements
for the Degree of

MASTER OF TECHNOLOGY

In

Structural Engineering

By

Mohd Raish Ansari
(University roll No. 1170444008)

Under the Guidance of

Mr. Shubhranshu Jaiswal
(Assistant Professor)

BABU BANARASI DAS UNIVERSITY
LUCKNOW

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CERTIFICATE

This is to certify that the thesis entitled titled "**Seismic Response Analysis of Offshore Structure Supported by Group Pile Foundation By Considering Wave and Current Action"** by **Mohd Raish Ansari** Under the guidance of Assistant Professor **Mr. Shubhranshu Jaiswal** to the Babu Banarasi Das University, Lucknow for the award of the degree of Master of Technology from Structural Engineering is a bonafide record of research work carried out by him under our supervision. The contents of this thesis, in full or in parts, have not been submitted to any other Institute or University for the award of any degree or diploma.

Thesis Supervisor
Mr. Shubhranshu Jaiswal
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Dated:-

DECLARATION

I, **Mohd Raish Ansari** hereby to certify that the work which is being, presented in the M.Tech project report entitled "**Seismic Response Analysis of Offshore Structure Supported By Group Pile Foundation By Considering Wave and Current Action**" in the fulfillment of the requirement for the award of Master of Technology in Structural Engineering and Submitted to the Department of Civil Engineering of BABU BANARASI DAS UNIVERSITY, Lucknow (U.P) is the authentic record of my own work carried out under the guidance of Assistant Professor **Mr. Shubhranshu Jaiswal**, Civil Engineering Department.

The matter presented in this thesis has not been submitted by us for the award of any other degree elsewhere.

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ABSTRACT

The basic purpose for this paper to obtains the approximate value of the response spectrum for offshore structure or platform which is subjected by wave, current, and other loads. Also comparing the various values such as displacement, base reaction and velocity-acceleration by defines the response spectrum method and time history method with the help of SAP2000 software. In this paper there are two code used, first one is Indian Standard (IS) 1893 part-1: 2016 for response spectrum and time history. Last one is American Petroleum Institute - Recommended Practice -2A-Working Stress Design (API-RP-2A-WSD) for defining the wave load at offshore platform. There is only one model created whose depth of the water considered about 72m and this platform is supported by the group pile foundation and its is fully steel space frame. By using the code API-RP-2A-WSD, the seismic force is automatically applied to the platform and its only take Primary wave (P-wave) into consideration because this wave enough to move in the fluid. The response spectrum curve of each member at join is discussed in details. The model is Jacket platform which is type of the fixed platform. The total number of the pile in this model is 32 which have 0.90m diameter and in one platform of the model is 8 numbers of the piles

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CHAPTER 1

INTRODUCTION

1.1. GENERAL

Offshore structure is defined as structure which is mainly installed in the marine environment usually for production and transmission of oil, gas, electricity etc. In this paper we are analyzing the seismic response of offshore structure supported by group pile foundation by considering the wave and current action simultaneously. The analysis of offshore structure is a most demanding set of tasks faced by Structural Engineering. Over and above the usual condition and situations met by land-based structure, offshore structure have the added complication of being placed in an ocean environment where hydrodynamic interaction effect and dynamics response become major considerations in their design. The foundation support condition and character of the dynamic response of not only the structure itself but also of the riser system for oil extraction adopted by them. Invariably, non-linearity's in the description of the hydrodynamic loading characteristics of the structure- fluid interaction and in the association structural response can assume importance and need be address.

The total number of the offshore structure/platform in the various bays, gulf and ocean of the world is increasing every years, most of the which are the fixed jacket type platform and it is mainly install when the depth of the water is from 30m to 200m for mainly oil and gas exploration purposes. The fixed offshore platform is subjected to the different environmental loads during their lifetime. These loads are imposed on the platform through natural phenomenon such as wind, current, wave, earthquake and snow. Among the various types of the environmental loading, wave loading is dominated loads. According to the API-RP2A environmental loads with the exception of the earthquake; should be combined in a manner consistent with the probability of their simultaneous occurrence during the loading condition being considered. The standard design of the structure is carried out using the allowable stress method. However it is important to the clarify the effect on the nonlinear response for an offshore structure under the severe wave condition.

The offshore structure may be analyzed by using the static or dynamic analysis method. The static analysis method is sufficient for the structure, which is rigid enough to neglect the dynamic force associated with the motion under the time dependence environmental loading. On the other hand, structure which are flexible due to their particular form and which are to be used in the deep sea must be checked for dynamic loads. Dynamics analysis is particularly important for the wave of moderate height is they make the greatest contribution of the fatigue damage and reliability of the offshore structure. The dynamic response evaluation due to wave force has significant roles on the reliable design of the offshore structure.

In the design and analysis of the fixed offshore structure many nonlinear physical quantities and mechanisms exist that is difficult to quantify and interpret in the relation of the hydrodynamics loadings. The calculation of the wave loads on the vertical tubular member is always of the major concern to the engineers, especially recently when such study are motivated by the need to build solid offshore structure in the connection with the oil and natural gas production. The effect of the various wave patterns on the offshore structure have been investigated by numerous researchers.

The energy use and availability are important driver for national development today and the per capita energy use is now gradually becoming accepted as a part of the globally development indices. In 2000 fossil fuel supplied 90% of global energy with the crude oil accounting for 40% of the total 25%. Several studies have been done on the future energy demand and supply from 2013 onwards.

Fig-1.1:- Offshore Structure.

1.2. DESCRIPTION OF THE SIMULATION METHOD

In this paper a method is described that in principle can handle two point need for simulation of the green water phenomenon. The method, incorporated in the computer program based on the Navier- Stokes equation, which describe the motion of incompressible, viscous fluid. Originally, the method was developed to simulate liquid sloshing on the board spacecraft. In this application the surface tension is the driving force in absence of gravity. Also an accurate description of the free surface is essential.

1.2.1. Free Surface Displacement

A very important aspect of the application is the presence of the free liquid surface. Many methods for the treatment of the free surface are described in the literature; often the method foe flow calculation with the free surface is classed by the method for the interface treatment. The most popular ones are the level set method and volume of the fluid method, which is adopted in the current method.

In level set formulation a distance ϕ(x, t) is introduced denoting the distance from x to the initial interface location at t=0. The interface corresponds to the contour ϕ=0 at any instant. The volume of the fluid (VOF) method, VOF function F is introduced with the value between zero(0) and one(1), indicating the fractional volume of the cell that is filled with a certain fluid. Based on the volumetric data, the free surface is reconstructed and displaced where the method is termed a volume tracking method. The VOF method is extremely suitable in the fixed grid simulation method, where the free surface should be able to have an arbitrary complex topology. For example, in the wave simulation the wave is sometime overturning such that the interface intersects itself and merges. The Volume tracking method is developed by Noh and Woodward which is given below and this equation states that the interface is moving with the liquid velocity.

$$\frac{DF}{Dt} = \frac{\partial F}{\partial t} + (u.\nabla)F = 0$$

Where

u=(u,v,w) the velocity vector.

t = time.

∇ = gradient operator.

1.2.2. Moving Objects

Moving object in the domain can be accounted for in the different ways. Commonly known in the fixed grid field of the method are the immersed boundary method, the fictitious domain method and cut-cell method. The first two methods are the treating the boundary of the object as a special region in a single phase. So the whole domain is filled with liquid and body forces (in the cell containing the moving object boundaries) account for the presence of the moving object.

In the cut-cell method the object is solid and sharp object boundary is cutting through the grid cell. This method differs from the other method in that the interface stays sharp and is not smeared over a few cell widths. A sharp interface method is needed in the application studied, since the peak of the water impact pressure should not be flattened due to swearing the interface over a few cell widths. The disadvantage of this cut-cell method is the sudden change of the nature of the cell, from the fluid to the body cell and vice-versa, introduce the discontinuities, but by avoiding the smearing of the interface, the velocity of the fluid along moving object, that is important in the application at hand, is not smoothened over the object interface.

In the fictitious domain method, introduced by Glowinki, the flow computation is done on a fixed space region, which contains the moving object using the finite element method. Lagrange multiplier is defined on the region occupied by rigid body to match the fluid flow and rigid body motion velocity over the interface between the region. A variation formulation is derived involving Lagrange Multipliers to the force rigid body motion inside the moving object.

1.2.3. Generation and Propagation of the waves

For the calculation of load on the offshore structure a wave generation option in the simulation method is essential. Some parts of the load calculation can be done without presence of the wave. The parts of the calculation of the load due to green water can be done without wave by modeling the water around the bow using a breaking dam model.

There are three different possibilities to the model wave generation in simulation method like Comflow. First possibilities that the wave can be generated using a wave maker as is also done in a wave tank. The water maker is modeling by moving flap that can move horizontally and or can be rotate about different axes. The second possibility that the wave can be generated by at the inflow boundaries by prescribes the velocity and water height. The velocity and water height can be calculated by description method of the wave. The third possibilities of wave generated are that to use other simulation method that calculate the wave field, and prescribe velocity can be calculated by Comflow domain method.

1.3. PLANNING OF THE OFFSHORE PLATFORM

There are mainly three step for planning of the offshore structure according to the API-RP2A, which is given below:-

1.3.1. Planning

The designing and constructions of the new platform and for the relocations of the existing platform used for the drilling, developments and storage of hydrocarbons in the offshore area. In addition, guidelines are provided for the assessment of the existing platform in the event that it becomes necessary to make a determination of the ‘safe for purpose’ of the offshore platform/structure. Adequate planning should be done before actual design is started in order to obtain a workable and economical offshore structure to the platform a given function. The initial planning should include the determination of all criteria upon which the design of the platform is based.

1.3.2. Design Criteria

The design criteria used in the offshore structure is most Working Stress Method which include all operational requirement and environmental data which could affect the detail design of the platform.

1.3.3. Codes and Standards

The code used for designing and analysis of the offshore structure is API-RP2A: WSD and made maximum use of the existing codes and standard that have been found acceptable for the engineering design and practice from the standpoint of the public safety.

1.4. Operational Consideration

There are so many operational considerations on the basis of the API-RP2A, which is given below:-

1.4.1. Function

The main aims of use function words is that for which purpose the platform to be design in usually categorized as drilling, producing, storage, material handling, live quarters, or a combination of these. The platform configuration should be determined by a study of layout of the equipment to be located on the deck.

1.4.2. Location

The location of the platform should be specific before the design is completed. Environmental condition varies with geographic location; within a given geographic area, the foundation condition will vary as will such parameter as design wave height and tide and periods.

1.4.3. Orientation

The orientation of the platform refers to its position with plan referenced to a fixed direction such as true north. Orientation is usually governed by the direction of prevailing seas, wind, current and operational requirement.

1.4.4. Water Depth

The information on the water depth and tide is needed to select appropriate oceanographic design parameter. The water depth should be determined as accurately as possible so that elevation can be established for boat landing, fenders, deck and corrosion protection.

1.4.5. Fire Protection

The safety of the personnel and possible destruction of the equipment require attention to the protection method. The selection of the system depends upon the function of the platform. Procedure should conform of all federal, state and local regulation where they exist.

1.4.6. Deck Elevation

The large force and overturning moment results when wave strike a platform' lower deck and equipment. Unless the platform has been designed to resist these forces, the elevation of the deck should be sufficient to provide adequate clearance above the crest of the design wave. In addition, consideration should be given to providing an 'air gap' to allow the passage of the wave larger than the design wave.

1.4.7. Wells

The exposed well conductor adds environmental forces to a platform and requires support. Their number size and spacing should be known early in the planning stage. The conductor pipes may or may not assist in the resisting the wave force. If the platform is to be set over an existing well with the wellhead above water, information is needed on the dimensions of the tree size of the conductor pile, and the elevation of the casing head flange and top of wellhead above the mean low water

1.5. Environmental Consideration

According to the API-RP-2A:WSD there are following consideration for designing and analysis of the offshore structure:-

1.5.1. General Meteorological and Oceanographic Consideration

The expert of designing and analysis of the offshore structure should be consulted when defining the pertinent meteorological and oceanographic condition affecting a platform site. The following summary present the information that could be required. Measured and/or model generated data should be statistically analyzed to develop the description of the normal and extreme environmental condition which is given below:-

The normal environmental condition means condition of the environment expected to occur frequently during the life of the platform are important both during the construction and service life of the platform.

The extreme condition is also important in the designing of the offshore structure. All data used should be carefully documented. The estimated reliability and source of all data should be noted, and the method employed in developing available data into the desired environmental value should be defined.

1.5.2. Winds

The wind forces are exerted upon the portion of the structure that is above the water as well as on any equipment, deck house and derricks that are located on the platform. The wind speed may be classified as:-

Gusts wind speed that average less than one minute in the duration and its return periods is about 100years.

Sustained wind speeds that average one minute or longer in duration.

The wind data should be adjust to the standard elevation, such as 10m above the means sea level, with a specific averaging time, such as one (1) hour.

The spectrum of the wind speed fluctuations about the average should be specific in some instances. For example compliant structure such as Guyed Tower and Tension Leg platform in the deep water may have natural sway periods in the range of one (1) minute, in which there is significant energy in the wind speed fluctuations.

The following should be considered in the determining appropriate design wind speed, which is given below:-

For Normal Conditions:-

a) Frequency of occurrence of the specified sustained wind speed from various directions for each month or season.

b) The persistence of the sustained wind speed above specified threshold for each month or season.
c) The probable speed of the gust associated with sustained wind speeds.

For extreme conditions:-

The projected extreme wind speed of specified direction and averaging times as a function of their recurrence interval should be developed. Data should be given concerning the following:-

a) The site measurement, data of the occurrence, magnitude of the measured gusts and sustained wind speed, and wind direction for the recorded wind data during the development of the projected extreme winds.
b) The projected number if the occasion during the specified life of the structure when sustained wind speed from specific direction should exceed a specific lower bound winds speed.

1.5.3. Waves

The wind driven waves area a major source of the environmental force on the offshore structure. Such waves are irregular in shape, vary in height and length and may approach a platform from one or more direction simultaneously. For these reason the intensity and distribution of the force applied by wave are difficult to determine. Because of the complex nature of the technical factor that must be considered in developing wave dependent criteria for design of the platform, experienced specialist knowledge in the field of the meteorology, oceanography and hydrodynamics should be consulted.

In that area where prior knowledge of oceanographic condition is insufficient the development of the wave dependent design parameter should include at least the following steps:-

i. Development of all necessary meteorological data.
ii. Projection of the surface wind field.
iii. Prediction of deep water general sea-states along storm track using the analytical model.
iv. Definition of maximum possible sea state consistent with geographical limitations.

v. Delineation of the bathymetric effect on the deep water sea state.

vi. Introduction of probabilistic technique to predict sea state occurrence at the platform site against various time base.

vii. Development of design wave parameter through physical and economic risk evaluation.

There are two conditions for checking the wave, which is given below:-

1.4.3.1. For normal condition (for both sea and swells).

1.4.3.2. For extreme condition.

1.5.4. Tides

It is important consideration in the platform design. It is rise and fall of the sea level caused by the combined effect of the gravitational force exerted by Moon, Sun and rotation of the earth.

Tides are mainly classified into the two types, which is given below:-

1.5.4.1. Astronomical tide

The tidal level and character which would result from the gravitational effect e.g. sun, moon, earth without any atmospheric influence.

1.5.4.2. Wind tide

The vertical rise in the still water level on the leeward of a body of the water, particularly the ocean, caused by wind stress on the surface of the water. The difference in the windward and leeward sides of such a body caused by wind stresses.

1.5.5. Currents load

Current are important in the design of the any type of offshore platform. They affect:-

a) The location and orientation of boat landing and barge bumpers.

b) The force on the platform, where the possible that boat landing and barge bumper should be located, to allow the boat to engage the platform as it moves against the current.

Current load are also classified into the three types, which is given below:-

i. Tidal current.
ii. Circulatory current.
iii. Storm generated current.

The total current profile associated with sea state producing the extreme wave should be specified for platform design. The frequency of the occurrence of the total current speed and direction at different depths for each month and/or season may be useful for planning operation.

1.6. ACTIVE GEOLOGIC PROCESSES

it describe the ground motion and effect on the offshore structure, it have mainly three step to describe about it:-

1.6.1. General

In the many offshore areas, geologic processes associated with movement of the near surface sediment occur within time periods that are relevant to fixed platform design. The nature, magnitude and return intervals of the potential seafloor movement should be evaluated by site investigation and judicious analytical modeling to provide input for determination of the resulting effect on the structure and foundations.

1.6.2. Earthquake

The seismic force should be considered in the platform design for area that are determined to be seismically active on the basis of the previous records of the earthquake activity, both in the frequency of occurrence and in magnitude. Seismic activity of an area for purposed of the designing of the offshore structure is rated in terms of possible severity of the damage of these structures. Seismicity of an area may also be determined on the basis of the detailed investigation.

Seismic consideration should include investigation of the subsurface soil at platform site for instability due to liquefaction, submarine slides triggered by the seismic activity, proximity of the site to faults, the characteristic of the ground motion expected during the life of the platform, and the acceptable seismic risk for the type of the operation intended. The platform in shallow water that may be subjected to Tsunamis should be investigated for the effect of the resulting forces.

1.6.3. Faults

In some offshore areas, fault planes may extend to the seafloor with the potential for either vertical or horizontal movement. Fault movement can occur as a result of seismic activity, removal of the fluid from the deep reservoir or long term creep related to large scale sedimentation or erosion facilities is close proximity to fault plane intersecting the seafloor should be avoided if possible.

1.6.4. Seafloor Instability

The movement of the seafloor can occur as result of loads imposed on the soil mass by ocean wave pressure, earthquake, soil self weight or combination of these phenomenon. Weak unconsolidated sediments occurring in ares where wave pressure are significant at the seafloor are most susceptible to wave induced movement and maybe unstable under negligible slope angles. Earthquake induced force can induced failure of the seafloor slopes that are otherwise stable under the existing self-weight force and wave condition.

1.6.5. Scour

The scouting is removal of the seafloor soils caused by current and wave. Such erosion can be natural geologic process or can be caused by structural elements interrupting the natural flow regime near the seafloor.

From the observation, scour can usually be characterized as some combination of the following:-

a) Local scour: - steep sided pits around such structure element as piles and piles groups, generally as seen in flume models.
b) Global scour: - shallow scoured basin of the large extent around a structure, possibly due to overall structure effect, multiple structure interaction or wave/soil/structure interaction.

c) Overall sea-bed movement: - movement of the sand wave, ridges and shoal that would occur in the absence of structure. This movement can be caused by lowering or accumulation.

1.6.6. Shallow Gas

The presence of either biogenic or pathogenic gas in the pore water of near surface soil is an engineering consideration in offshore area. In addition to being a potential drilling hazard for both site investigation soil boring and soil well drilling, the effect of the shallow gas may be important to engineering of the foundation. The importance of the assumption regarding shallow gas effect on the interpreted soil engineering properties and analytical models of geologic process should be established during initial stage of the design.

1.7. SITE INVESTIGATION-FOUNDATION

During the site investigation process for the foundation, it include given step by step for investigation:-

1.7.1. Site Investigation Objective

Knowledge of the soil condition existing at site of the construction on any sizable structure is necessary to permit a safe and economical design. On the site soil investigation should be performed to define the various soil strata and their corresponding physical and engineering properties. Previous site investigation and experience at the site may permit the installation of the addition structure without additional studies.

The initial step for a site investigation is reconnaissance. Information may be collected through a review of available geographical data and soil boring data available in the engineering files, literature or government files. The purpose of this review is to identify potential problem and to aid in planning subsequent data acquisition phase of the site investigation.

1.7.2. Sea-bottom Surveys

The primary purpose of a geophysical survey in the vicinity of the site is to provide data for geologic assessment of the foundation soil and the surrounding area that could affect the site. Geophysical data provide evidence of slumps, scarps, irregular or rough, topography, mud and lateral variation in strata thickness. The areal extent of the shallow soil layer may sometimes be mapped if good correspondence can be established between the soil boring information and result from the sea-bottom survey.

The geophysical equipment used includes, which is given below:-

I. Sub-bottom profile fir definition of the bathymetry and structural features within near surface sediments.
II. Side scan sonar to define the surface features.
III. Boomer for definition of the structure to depth up to a few hundred feet below the seafloor.

The shallow sampling of near surface sediment using drop, piston, grab, samplers or vibro-coring along geophysical track lines may be useful for calibration of results and improved definition of the shallow geology.

1.7.3. Soil Investigation and Testing

If the practical soil sampling and testing program should be defined after a review of the geophysical results. On site, soil investigation should include one or more soil boring to provide samples suitable for the engineering properties testing and mean to perform in-situ testing, if required. The number and depth of the boring will depend on the soil variability in the vicinity of the site and perform configuration.

As a minimum requirement, the foundation investigation for pile supported structures should provide the soil engineering property data needed to determine the following parameter, which is given below:-

i. Axial capacity of pile in tension and compression.
ii. Load-deflection characteristic of axially and latterly loaded piles.
iii. Pile drivability characteristic
iv. Mud-mat bearing capacity.

1.8. SELECTING THE DESIGN ENVIRONMENTAL CONDITIONS

Selection of the environmental condition to which platform are design should be responsible of the owner. The design environmental criteria should be developed from the environmental information described above, and it also includes the risk analysis where prior experience is limited. The risk analysis may include the following:-

i. Historical experience.
ii. The planned life and intended use of the platform.
iii. Prevention of pollution.
iv. The possible loss of human life.
v. The estimated cost of the platform damage or loss when subjected to the environmental condition with various recurrence intervals.

1.9. PLATFORMS TYPES

According to coda provision API-RP2A, platform are classified into two types, which is given below in details:-

1.9.1. Fixed Platform

It is defined as platform extending above the water surface and supported at the sea-bed by means of piling, spread footing or other means with the intended purpose of remaining stationary over an extended period.

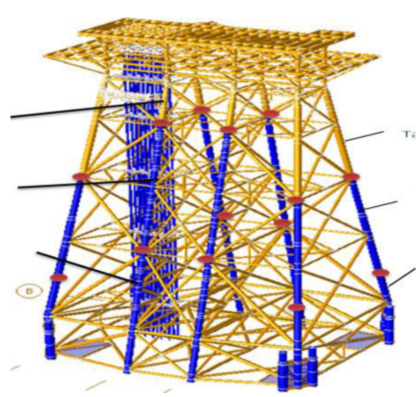

Fig-1.2: Jacket Type Platform

Fixed platform also classified into following types on the basis of API-RP2A which is given below:-

1.9.1.1. Jacket platform

Fig-1.3: Jacket Platform

It is generally consist of the following way:

i. Completely braced, the redundant welded tubular space frame extending from an elevation at or near the sea-bed to above water surface, which is design to serve as the main structural element of the platform, transmitting lateral and vertical force to the foundation.
ii. Piles or other foundation elements that permanently anchor the platform to the ocean floor, and carry both lateral and vertical loads.
iii. A superstructure provides deck space for supporting operational and other loads.
iv. It is installed when the depth of the water is upto 500m and time period is less than 4 sec.

1.9.1.2. Tower Platform

It is modification of the jacket platform that has relatively few large diameters (5m). the tower may be floated to location and placed in position by selective flooding. It may or may not be supported by piling. Where piles are used, they are driven through sleeve inside or attached to the outside of the legs. The piling may also serve as well conductors. If the tower, s support is furnished by spread footing instead of by piling, the well conductor may be installed either inside or outside the legs.

1.9.1.3. Concrete Gravity Structures

Fig-1.4: Concrete Gravity Structure

It is one of the fixed type platforms that relies on the weight of the structure rather than piling to resist environmental loads.

1.9.1.4. Non-Jacket Platform

Many structures have been installed and are serving satisfactorily that do not meet the definition of the jacket type platforms as defined above. In general, these structures do not have reserve strength or redundancy equal to the conventional jacket type structure. For this reason, special recommendation regarding design and installation are provided above definition. Minimum structure is defined as structure which have one or more of the following attributes:-

- Structural framing, which provide less reserve strength and redundancy than a typical well braced, three leg template type platform.
- Free standing and guyed caisson platform which consist of one large tubular member supporting one or more wells.
- Well conductor which are utilized as structural and/or axial foundation elements by means of attachment using the welded connections.

- ❖ Threaded pinned clamped connection to foundation elements.
- ❖ Braced caisson and other structure where single element structural system is major component of the platform, such as deck supported by the single deck-leg.

1.9.1.5. Compliant Platform

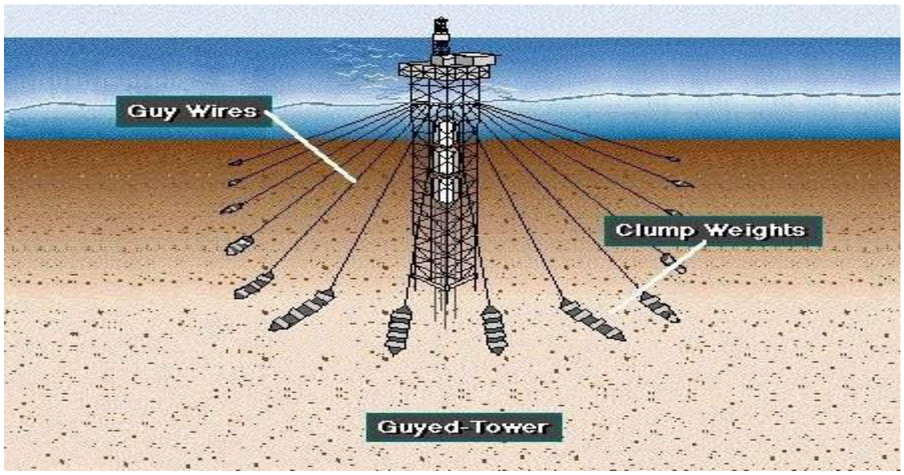

Fig-1.5: Compliant Platform

It is bottom founded structure having substantial flexibility. It is flexible enough that applies force are resisted in significant part by inertial force. The result is reduction in force transmitted to platform and supporting foundation. It is much like fixed platform. It consists of a narrow flexible tower and piles foundation that can support a conventional deck for drilling and production operation.

1.9.2. Floating Production System

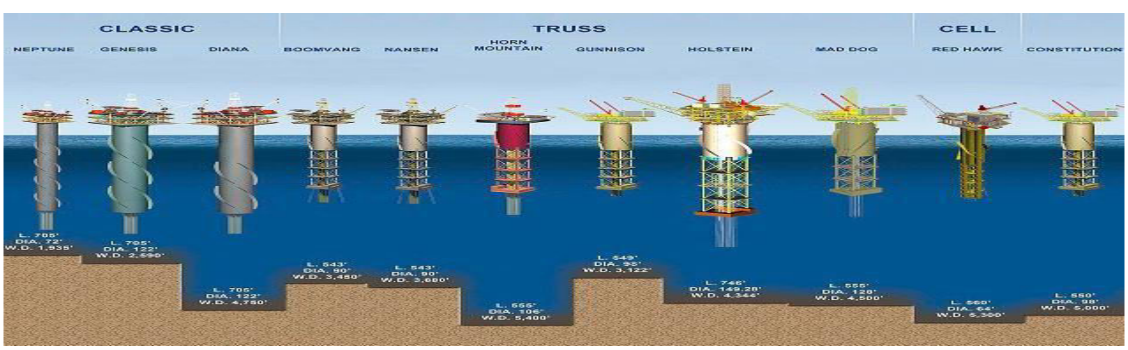

Fig-1.6: Floating Production System

A number of different floating structures are being developed and used as floating production system (e.g., tension leg platform, spars). Many aspect of this recommended practice are applicable to certain aspect of the design of the structure.

CHAPTER 2

LITERATURE SURVEY

After study various paper related to the seismic response analysis of offshore structure with different loading condition which conclusion is given below of all paper:-

Bor-Feng Peng [1]

i. The paper publish by above author is "Nonlinear Dynamic Soil-Pile Structure-Interaction Analysis of the Offshore Platform for the Ductility Level Earthquake under Soil Liquefaction condition" in the 2004 and they gave the conclusion is that the impact of soil liquefaction on the platform dynamic response and the foundation system design is significant and summarized as follows:-
ii. Higher pile thickness demand.
iii. Deeper pile penetration depth requirement.
iv. Larger topside and pile head displacement.
v. Longer periods of the platform foundation structural system.
vi. Reduced the number of the structural elements developing nonlinear hysteretic behavior if the platform is still stables.
vii. Greater bending moment of the pile developed at the deeper depth.
viii. Based on the nonlinear finite element analysis results of the pile to jacket leg connection with material plasticity and larger deformation effect.

Takeshi Maki [2]

The paper publish by Takeshi Maki[1] and Hiroshi Mutsuyoshi[2] is "Response Behavior of RC Pile under Severe Earthquake" in 2004 and conclusions are given below :-

Pile placed in soil ground has higher restoring force than that of the pile body itself due to effect of the earth pressure. In addition, the location of the plastic hinge shift accounting to the ratio of the stiffness of the pile and ground.

Under the reversed cyclic loading condition, passive earth pressure becomes high with the increase of the displacement amplitude. This is caused by the compaction of the soil particles around the pile body, and restoring force of the compaction soil ground may provide the remarkable progress of the damage pile.

Regarding the restoring force characteristic of the RC pile , applied analytical method can express the Skelton curve of the force displacement relationship at pile top, but cannot produce large hysteresis loop measured from experiment. This is because the model, which express the compaction effect, as state above is not installed structural system in the future, the damping characteristic of the system should be accurately estimated, and the material model of each component and model which can relate the phenomenon between pile and ground should be developed.

In order to complete development of the evaluation method, further investigation is needed on the restoring force characteristic of the RC pile, such as the difference between the static and dynamic response behavior, the effect of the strain rate dependence and frequency dependence.

Bai Degui [3]

The paper publish by the Bai Degui[1], Chen Guo Xing[2] and Wang Zhi Hua[3] is "Seismic Response Analysis of the Large Bridge Pier Supported by the Group Pile Foundation Considering the Effect of the Wave and Current Action" in 2006 and they use Morison Formula with Stokes' fifth order wave theory, the wave force, the nonlinear seismic response characteristic of the large pile group foundation pier structure deep water is analyzed, and conclusion is given below:-

i. The wave and ocean current has little effect on the acceleration response of the pile body.
ii. The wave and ocean current has great effect on the relative displacement of the pile body. The wave and ocean current make the relative displacement against the water-current direction decreasing, and make the one along the water current direction increasing and influence extent increase with flow velocity increasing, but the change the law with wave height is closely related to the earthquake ground motion characteristic.

iii. The effect of the wave and ocean current make the pile moment increasing, and influence extent increase with the flow velocity increasing, but the change law with wave height is closely related to the earthquake ground motion characteristic.

iv. In order to ensure safety and reliability of the large span bridge across channel and river it is necessary to take account of the influence of the wave ocean current in the seismic design of the large pile group foundation pier structure in the deep water.

Jing-Jong Jang [4]

The paper published by Jing-Jong Jang and Guo Jyh-Shinn is " Analysis of Maximum Wind Force for Offshore Structure Design" in the 2007 present the result if the study to statistically estimate the low frequency turbulent wind drag force (including the extreme value) acting on the floating offshore structure. In the calculating the drag force, force associated with the turbulent wind speed is considered using both linear and nonlinear (squared term) approach. From the result of the analysis, the following conclusion is given below:-

The probability density function of the turbulent drag force including the nonlinear term substantially deviates from the normal distribution in that probability density is greater for the larger force than is the case for the normal distribution.

Probability density function of peak drag force shift toward the larger value by taking higher order turbulent wind speed into consideration. In other word, the nonlinear approach yield higher probability density for the large drag force than linear approach. In evaluating the variance of the turbulent wind induced drag force from wind spectrum. It is necessary to consider the spectrum of the squared wind speed. The spectral density function of the squared wind speed is very large in the comparison with spectrum of the wind speed; however the contribution of the former to the variance of the drag force is extremely small. Extreme wind induced drag force are evaluated through both the linear and nonlinear approach on an offshore structure having a projected area of 2,000 m2 for the wind speed of 44.7m/sec at the 30m above the surface of the water. The wind spectral formulation developed broad on the wind data obtained from the measurement over the seaway is used in the computation. The result of the computation shows that nonlinear approach yield 8.0 percent increase in the design extreme drag force.

Toyoaki Nogami [5]

The paper publishes by Toyoaki Nogami is "Effect of Offshore Environment on Dynamics Response of Pile Foundation" in the 2008, the conclusion of this paper is given below:-

A semi-analytical method is developed for the dynamic response analysis of the fluid saturated porous medium. The plane strain condition in the Cartesian coordinate system and three dimensional conditions in the cylindrical coordinate system are considered. With the help of the separation of the variable, the method adopts the finite element discretization only along the depth and analytical form in the lateral direction. The method is found to be numerically very efficient and therefore it is particularly useful for two phase mixture problems since they generally require computation much larger than those for the single phase medium problems.

The pore fluid in the soil mass affect the dynamic response of the soil deposited by not by simply increasing the stiffness of the soil but also coupling the soil skeleton motion with pore fluid motion. All those are affected by loading rate relative to the pore fluid diffusion rate, boundary condition and stress gradients. The coupling effect is more predominant for higher permeability of the soil. When the soil permeability is low, a mode wave transmitted primarily to the fluid is distinctly different from those primarily transmitted to soil skeleton and decays very quickly with distance. Under the static and drained condition, the submersed soil response is identical to dry the soil response. Under the dynamic condition; however the transient pore fluid redistribution depends upon the rate of loading and permeability of the soil. The response of the submersed soil is closer to the untrained condition when the combinations of those are less favorable for the pore fluid movement. The larger pore fluid motion relative o the skeleton soil are higher damping generated. When the soil submersed below the water-table, the water above the soil deposit ca affect the dynamic response analysis of submersed soil under the water. It is well known that the Winkler model based on the cylindrical plane strain condition can produce the dynamic pi response reasonable well for pi in the single phase soil medium. It is found that the pore fluid and soil skeleton coupling effect in the transient motions also can be reasonable well produced by such Winkler model formulated for the fluid saturated porous medium.

Shehata E. Abdel Raheem [6]

The paper publish by Shehata E. Abdel Raheem[1], Elsayed M.A. Abdel Aal[2], Aly G.A. Abdel Shafy[3] and Fayez K. Abdel Seed[4] is “Nonlinear Analysis of Offshore Structure Under Wave Loadings” in 2009 the conclusion of this paper is given below:-

It is crucial to reduce the overall response of a jacket platform subjected to the environmental loads. In the general, the reduction of dynamic stress amplitude of an offshore structure by 15% can extend the service life over two times, and can result in decreasing the expenditure on the maintenance and inspection of the offshore structure. A finite element formulation has been developed for nonlinear response of the fixed offshore platform jacket. Where the three dimensional beam elements incorporating large displacement, time dependent wave force is considered. The time dependent wave force has been considered as drag component of the wave force, which is a function of the second order water particle velocity; hence the nonlinearity due to the wave force has been included.

The offshore structural analysis is used to obtain the platform displacement response under varying external loading. The deflection of the platform is studied for individual and combined wind and wave force. Jacket type offshore structure is displacement, axial force, bending moment, natural mode and frequencies of the free vibration are evaluated. A comparison of maximum displacement at all nodal point for various current incidence angle is introduced. The result indicates a significant effect of the current incidence direction. The displacement response (U1) increases nonlinearly with the height of the offshore platform, but there is a significant curvature to the displacement response (U2) along the height of the offshore structure. The large inter storey-drift of jacket leg is not allowed for jacket platform to satisfy the drilling and production requirements. Both maximum deck acceleration and maximum deck to top of the jacket displacement were important response parameter affecting the performance of the equipment, vessels and pipelines. On the other hand, low maximum deck acceleration was desirable for vessel and equipment.

Jianhong ye [7]

The paper publishes by the Jianhong ye[1] and Gang Wang[2] is "Seismic Dynamics of Offshore Breakwater on Liquefiable seabed foundation" in 2010 the conclusion of this paper is given below:-

The highly nonlinear dynamic interaction of the offshore breakwater and its seabed foundation is investigated using the coupled FEM numerical model. The dynamic behavior of the seabed soil is modeled by using a validated soil constitutive model-pastor-Zienkiewics mark. In this investigation, variation of the void ratio of the seabed soil and its corresponding permeability, as well as hydrostatic pressure acting on the surface of the seabed and offshore breakwater are updated in each time step in accordance to the seismically induced displacement of the offshore breakwater and deformation of the seabed foundation. The practice of the numerical implementation indicate y=that numerical solution will not coverage if these variation are not considered in the numerical modeling, especially in the situation of large deformation simulation.

Analysis in this study show the composite breakwater translated 12m in the X-direction and subsided 6m. This composite breakwater could not still serve as barrier to protect harbor or port. It is suggested construction of the offshore structure on the loose seabed foundation should be avoided in the practice. However, if the situation cannot be avoided, it is highly recommended to evaluate the seismic stability of the offshore structure using the advance computational tools, such as FSSI-CAS 2D program in this study. Ground improvement should be designed to reduce the potential and consequence of the seabed liquefaction. Again, advanced numerical tools should be used to evaluate the effectiveness of the mitigation measures.

Finally remarks on the two criteria that have been widely used to indicate the degree of liquefaction. Based on the physical consideration, the mean effective stress is a direct indicator of the liquefaction. In the practical, the soil is often regarded as liquefied when the residual pore pressure is equal to or greater than the initial effective pressure. According to the pore pressure generated within that area can well exceed the initial effective pressure even through the soil is not fully liquefied. Therefore the pore pressure ratio is not reliable indicator for the liquefaction, if the stress state of the soils is significantly affected by structure.

Torna Patil [8]

The paper is published by Torna Patil[1], Pratibha Alandkar[2] is “Dynamic Response of Offshore Structures” in the 2011 and conclusion is given below:-

a) The dynamic loads viz. Wind, Wave, Current, and Earthquake forces have a major impact on offshore structures. Hence their determination is of the utmost importance.
b) This paper focused on the procedure of analysis and design considerations for determination of wind force acting on offshore structures.
c) It is observed that during gusty winds, to avoid skidding of base of derrick on offshore platforms, interference and base flexibility can reduce the severity of extreme wind load effects. Hence, for times like extreme wind conditions, the design for deck components requires caution.
d) The dynamic loads viz. Wind, Wave, Current, and Earthquake forces have a major impact on offshore structures. Hence their determination is of the utmost importance.
e) This paper focused on the procedure of analysis and design considerations for determination of wind force acting on offshore structures.
f) It is observed that during gusty winds, to avoid skidding of base of derrick on offshore platforms, interference and base flexibility can reduce the severity of extreme wind load effects. Hence, for times like extreme wind conditions, the design for deck components requires caution.

Min-Chou Tsai [9]

The paper published by the Min-Chou Tsai[1], Hsien Hua Lee[2], Tinh Quoc Bui[3], and Chuanzeng Zhang[4] is “Offshore Structural Dynamic Analysis Considering Soil-Structure Interaction By A Coupled BEM And MESHFREE Method” in 2012 which conclusion is given below:-

Since seismic motions may cause serious damage to a bottom mounted platform, the effects of earthquakes on the dynamic response of the structure should be scrutinized in order to design reliable structures in seismic zones. This paper presents a methodology for the analysis of offshore structural system subjected to seismic excitations considering the soil structure interaction effect.

The proposed method is validated from the literature which shows the accuracy of the developed algorithm. The offshore system like structure, having the coupling effect due to the soil-pile foundation material during earthquake excitations is analyzed. The numerical results presented here prove the efficiency of the present algorithm to solve a soil-structure coupled problem of massive structures such as the typical offshore system. The advantage of using the present MKEFG-BEM method is that it requires less computational effort, in terms of both time and memory.

The responses of the soil-structure system considering an absorbing boundary indicate that the incident energy is effectively absorbed at the truncation boundary.

Another advantage of this method is that it requires less computational effort since it avoids evaluation of convolution integrals and Fourier transforms to calculate soil-structure interaction forces. The algorithm presented here is simple so that it may be programmed easily. The results show that the displacements and stresses have increased for the elastic as compared to the rigid base. Hence it is advisable to carry out the interaction analysis for pile structures like offshore under flexible base. It is also observed that the pile head is the most severely stressed zone; hence one may expect the appearance of cracks around the pile head region of the offshore structure system.

E. R. Johnson [10]

The paper published by the E. R. Johnson is "Horizontal Forces Due To Waves Acting On Large Vertical Cylinders In Deep Water" in 2012 which conclusion is given below:-

On the basis of the analytical expressions derived for the magnitude and distribution of the maximum forces, supported by the experimental data, some general guidelines can be formed fur the benefit of engineers interested in the horizontal wave forces on the vertical buoyancy cylinders of stable ocean platforms:-

a) The maximum horizontal force due to waves is proportional to the square of the cylinder diameter when the maximum force is inertial.
b) The force distribution is concentrated near the surface. One-half the possible maximum force on the cylinder occurs in the first one-tenth wavelength of depth from

the water surface. For the first one-tenth wavelength (200 ft for a 20-sec wave) of depth, the force distribution is almost linear with depth.

c) In the range of interest for platform buoyancy columns, the horizontal forces due to waves are about proportional to the column length and the square of the diameter. Therefore, wave force considerations should not be a factor in column proportioning since, for a given column buoyancy, the force would be about the same regardless of the length to diameter ratio.
d) However, the magnitude and distribution of the wave forces on the vertical cylinders need to be known for the structural design.

Behrouz Asgarian [11]

The paper published by the Behrouz Asgarian and Hamed Rahman Shokrgozar is "A new bracing system for improvement of seismic performance of steel jacket type offshore platforms with float over deck" in the 2013 which conclusion is given below:-

In this paper probabilistic performance was applied for the seismic performance assessment of a newly designed jacket type offshore platform with the float over deck installation system. It was observed from analysis results that due to lack of vertical bracing in the top bay of the jacket in the float over direction, design requirement were not satisfied. Probabilistic assessment indicated that MAF and confidence level these type of the platform are not satisfied for both immediate Occupancy and collapse prevention performance levels in the direction of the float over deck installation. Also from measured dynamic characteristics of the scaled model, it was observed that the dynamic characteristics in the float over direction

Kleefsman [12]

The paper publish by the Kleefsman, Kornelia Marchien Theresa is "Water impact loading on offshore structures. - A numerical study" in 2013 which conclusion is given below:-

The focus of this work has been on the simulation of wave impact loading on offshore structures. Especially, the event of high waves resulting in green water on the deck of an FPSO has been an important application. The method has been incorporated in a simulation program called Comflow. At the start of this project, the method was able to simulate fluid flow in complex geometries with the presence of a free liquid surface and moving bodies. At the domain boundaries no-slip or free-slip conditions were imposed and simple inflow and outflow conditions were implemented. To be able to perform wave impact simulations as aimed for in this project, wave generation options have been implemented. Also an investigation has been performed of outflow boundary conditions that are needed to prevent waves from reflecting at the domain walls. It has been investigated whether the waves are damped due to numerical choices, like the upwind discretisation of the convective terms in the Navier-Stokes equations. Since the impact is measured by pressure time traces, attention has been paid to the smoothness of the pressure signal. Further, attention has been paid to the robustness of the method, especially in handling the free surface that can be deformed severely in these kind of applications.

Yuejun LU [13]

The paper published by the Yuejun LU[1], Yanju PENG[2], Rongyu TANG[3] and Haijun SHA[4] is "Determination of Seismic Fortification Level of Offshore Platforms in China" in the 2013 which conclusion is given below:-

On the basis of seismic activity in the sea areas of China, structural characteristics of offshore platform, seismic design goal, and the seismic fortification experiences of related engineering, we recommend that the seismic fortification level of strength design takes a return period of 200 years, and that of deformation design takes a return period of 3000 years for offshore platforms in China. The return periods recommend in this paper are only used in the comparison between Chinese related Codes, has not been applied in structural analysis of offshore platform, also the reliability analysis of the design has not been done for difference platform structures. There is still a long way to go before the constitution of seismic code for offshore platforms, and many researches should be done in future.

Muhammad Al-Farisia [14]

In this study, "Reassessment of B1C offshore platform" in 2014 that owned by PHE ONWJ is numerically evaluated for service life extension in the next twenty years. This platform structure located in the Ardjuna Field, Northwest of Java and was installed on 1975. The reassessment analyses of B1C platform focus on in-place analysis, seismic analysis and fatigue analysis. These analyses refer to recommended practice Codes and Standards, Specifications, and Regulations issued by American Petroleum Institute (API RP2A). The results showed that the entire values of unity check for all members fulfill the requirements of API RP 2A - WSD. Meanwhile, fatigue analysis result showed that three joints have the fatigue life less than 59 years.

D. Boote [15]

The paper published by D. Boote[1] and D.Mascia[2] is "Anti-seismic Design Methodologies Applied to Offshore Structures" in the 2014 which conclusion is given below:-

The seismic behavior of a typical off-shore platform has been investigated by applying the criteria proposed by the main Classification Societies. Two different procedures were performed: spectral analysis and direct dynamic analysis. The study was limited to the linear domain and in this field the influence of the main parameters was enlightened.

At first, by a natural frequencies computation, two boundary conditions were examined for designed structure: clamped legs at ground level and deep piles foundations in elastic soil. Ground behavior was then described by Winkler elastic coefficients. Comparisons performed on natural frequencies and participation factors allowed to simplify the further analyses, by considering that, in case of rocky soil, the clamped model would satisfactory simulate the real structure.

Successively, through a spectral analysis, the damping coefficient influence was investigated. Response spectra of two seismic events (Taft and Elcentro) were built-in starting from the corresponding acceleration laws for different values of v. The high influence of results by v coefficient was observed and the need to further investigate damping phenomena emphatized. On this subject the most complete classification rules, that is API RP 2A, suggest three different design spectra, depending on the ground nature, which may be modified through a magnification factor D depending on v coefficient.

The conservative nature of the spectral investigation has been tested by a comparison with the results of direct dynamic analyses. Calculations have been performed through the application of acceleration time-histories of Taft and Elcentro earthquakes. In both the dynamic analyses performed the maximum stress levels and displacement values resulted to be lower than those of the corresponding spectral analyses. All the previous considerations are valid until the investigation is conducted in the linear field.

Nevertheless the effective reliability of the structure towards the limit state requires the investigation to be extended to the non linear domain. This further development of the study, which can be accomplished only by direct dynamic analysis, will be carried out taking into account the nonlinear behavior of the ground-piles complex, the ductility phenomena and the corresponding variation of material damping. For what concerns the non-linear soil behavior, the P-y curves proposed in API standards can be introduced while, for the structure ductility and the material damping, information is needed based on experimental investigations.

Abhijeet A. Maske [16]

The paper published by the Abhijeet A. Maske[1], Nikhil A. Maske[2] , Preeti P. Shiras[3] is "Seismic Response Of Typical Fixed Jacket Type Offshore Platform Under Sea Waves" in the 2014 which conclusion is given below:-

The response of fixed jacket type offshore platform was investigated using the time history analysis. As a result of the work that was completed in this study, the following conclusions were made:-

a) When the longitudinal components of the earthquake and wave are in different directions, an increase on the response of platform can be seen.
b) The displacement for earthquake load alone is less than the displacement for the combination of earthquake and wave loads.
c) This study shows significant difference between drift under simultaneously wave and earthquake loads compared with regulations criteria (for earthquake load).
d) It may also conclude that nonlinear response investigation is quite crucial for safe design and operation of offshore platform.
e) The time history analysis is a relatively simple way to explore the non linear behavior of offshore structures.

Khosro Bargi [17]

The paper published by Khosro Bargi[1], S. Reza Hosseini[2], Mohammad H. Tadayon[3], Hesam Sharifian[4] is "Seismic Response of a Typical Fixed Jacket-Type Offshore Platform (SPD1) under Sea Waves" in 2014 which conclusion is given below:-

The nonlinear dynamic behavior of Jacket-type platform under simultaneously acting of wave and earthquake loads was studied in this paper. The following results are obtained.

At first, the earthquake loads were applied alone at four different directions. Then wave and longitudinal component of earthquake were applied simultaneously in the same and different four directions.

The results comparison shows that the maximum displacement response of platform under combination of two loads (earthquake and wave loads) are more than maximum displacement response of earthquake load alone.

Y. YAMADA [18]

The paper published by Y. YAMADA[1] AND H. IEMURA[2] is "Seismic Response of Offshore Structures in Random Seas" in the 2015 which conclusion is given below:-

The dynamic response of offshore structures subjected to simultaneous wave and earthquake loadings is investigated. The results are compared using rms responses and reliability functions. The principal conclusions of this study may be summarized as below:-

a) The response of offshore structures mainly depends on the first few vibration modes. Therefore it is important to determine accurately these vibration modes and the corresponding natural frequencies.
b) The dynamic response analysis including the effects of soil-structure interaction requires the solutions of the governing equations of motion with many degrees of freedom. Application of the substructure method, which utilizes the reduction of the degrees of freedom of the superstructure using the modal matrix, seems to be very convenient and efficient for the response analysis of soil-offshore structure systems.

c) Responses due to earthquake motions vary with the intensity of the input ground acceleration. The responses are higher for the soil-structure interaction condition than for the rigidly supported base condition. In the absence of sea waves, the effects of linear hydrodynamic damping are small.

d) When the sea waves and the earthquake motions act simultaneously, the non-linear hydrodynamic damping forces are proportional to the relative velocities between the waves and the structure. Since the wave velocities are very much higher than the structural velocities, these damping forces become larger than those without waves. Sea waves act as a damping medium and reduce the seismic response of offshore structures.

e) Studies of the first passage probabilities indicate that small sea waves enhance the reliabilities of offshore structures against earthquake forces.

f) Earthquake forces provide significant contributions to the response evaluations of offshore structures in seismically active regions. Therefore it is important to examine the effects on the response evaluations due to earthquake forces as well as sea wave forces.

Antonio Cerami [19]

The paper published by Antonio Cerami is "Stochastic Seismic Analysis of Offshore Towers" in 2015 which conclusion is given below:-

In order to evaluate the mean square response of a MDOF system, like an offshore-tower structure, submerged in deep water and subjected to an earthquake ground motion, a simple step-by-step solution method has been presented.

The proposed procedure can be successfully applied for evaluating the dynamic response taking into account the nonlinearities due to the drag forces and the no stationary ground earthquake motions effects, without a remarkable increase in computational effort. The earthquake ground motion is simulated by considering the input as a random filtered noise multiplied by a deterministic shaping function of general shape.

In order to evaluate structure safety, the extreme value statistics are calculated, and in particular the probability of the maximum value of the response exceeding a certain threshold during a defined length of time T, byusing Rice's formulation. Using the same simple numerical examples, various structural responses have been evaluated, taking into account earthquake excitation and the effects due to drag forces. Different values of standard deviation of displacements have been obtained, and it is observed that the more the spectral power density of earthquakes increases, the more noticeable is the change in structural response.

The present method is being used in an ongoing further investigation aiming to evaluate structural response to both earthquake and sea-waves excitations. The results of this investigation, to be presented elsewhere, will account for the influence of the depth of the sea bottom.

Norberto Flores-Guzman [20]

The paper published by Norberto Flores-Guzman[1], Enrique Olivera-Villasenor[2], Andriy Kryvko[3] is "Seismic Pressures in Offshore Areas: Numerical Results" in the 2016 which conclusion is given below:-

The contribution and novelty of the present work is the use of integral equations (solved numerically by the Boundary Element Method) to study the effects of seismic actions in offshore and onshore areas. This formulation can be considered as a numerical implementation of Huygens' Principle in which the diffracted waves are constructed at the boundary from which they are radiated. Thus, mathematically it is fully equivalent to the classical Somigliana's representation theorem. In addition, seismic pressures due to the configuration of the sea bottom are highlighted. Several seabed configurations and materials are modelled to show seismic pressures along the water depth. Specific conclusions are given below:-

i. It has been found that the compression waves (P-waves) can produce greater seismic pressures than the distortional waves (SV-waves). Moreover, P-wave incidences generate greater pressures in remote locations from vertical cliffs. On the other hand, the maximum pressures caused by SV-waves are present in locations close to vertical cliff. The difference between the maximum pressure values obtained for a material with shear wave velocity of β =3000 m/s is approximately 9 times lower than those obtained for a material with a of β = 90 m/s, for the P wave incidence, and 2.5 times for the case of SV-waves. This result is relevant because the seabed material type has direct implications on the pressure field obtained. Results in time domain suggest that the calculated pressures are similar to those obtained for a horizontal configuration of the seafloor, for the case of P-waves and for the analyzed configurations. That is to say that the seafloor configuration does not cause great diffractions of P-waves. While, the obtained pressures, when a normal SV-wave excites a ramped configuration, are consequence of the diffractions of SV-waves by the seabed configuration, only. Another relevant finding is that the highest seismic pressure due to an offshore earthquake is almost always located at the seafloor

A. Ajamy [21]

The paper published A. Ajamy[1], M.R. Zolfaghari[2] is "Probabilistic seismic analysis of offshore platforms incorporates uncertainty in soil–pile–structure interactions" in 2016 which conclusion is given below:-

a) The Latin Hypercube Sampling (LHS) in conjunction with the Simulated Annealing (SA) optimization technique have been implemented as part of the Comprehensive Interaction IDA for the evaluation of structural systems. This approach allows modeling of a variety of uncertainties associated with different elements of a structural system. This is done by controlling the statistical correlations among random variables with a small number of simulations.

b) It is important to properly account for the uncertainties associated with seismic load and soil–pile–structure interactions when analyzing jacket type offshore platforms under a high level of ground shaking. This can help understand better when a structure is likely to collapse during an earthquake. The Comprehensive Interaction IDA approach is a systematic method to achieve this. The summary of the statistical data provides a good indication of the level of shaking associated to different limit states.

c) Simultaneous considerations of uncertainties associated with SPSI tend to increase the dispersion of the probability distribution for structural responses.

d) The Comprehensive Interaction IDA results can be combined with a site hazard curve to generate mean annual frequencies of exceedance at various levels, displaying the importance of integrating uncertainties in the seismic risk assessment.

e) The implementation of the proposed methodology to determine the structural response of the existing platform based on the above assumptions showed that this structure can provide reasonable seismic performance during earthquakes that have a 10% probability of occurrence in 50 years. However, for earthquakes with a 2% probability of occurrence in 50 years, the possibility of collapse is high.

CHAPTER 3

METHODOLOGY

3.1. Design Code for Offshore Structure

The code mainly used for design and analysis of the offshore structure is different edition of American Petroleum Institute (API) which is based on the Working Stress Method (WSM) and this code is also used in the Bombay High Field and the 20th edition of the API is 1993 was also issued in Load and Resistance Factor Design (LRFD) format, and was in 1997 supplemented with the section on the requalification of the offshore structure. American Petroleum institute(API) RP2A-LRFD, 1993 provision provide characterization of the environmental load and design requirement for the fixed offshore structure/platform for use in design, describe the analytical method to determine the force includes in the platform system by the ground motion, and give guidance for the sizing and configuring steel elements for the design force. The consideration of the environmental load consist earthquake loads in the earthquake ground motion, wind, wave and current loads. Design method for the structure, members or components under the static load to avoid failure, collapse, buckling are well defined in the code and standards, such equivalent code in other countries, whilst for the offshore structure the design code used almost invariably is API-RP2A(API 1993).

3.2. Software

i. The SAP name has been synonymous with state-of-the-art analytical methods since its introduction over 30 years ago. SAP2000 follows in the same tradition featuring a very sophisticated, intuitive and versatile user interface powered by an unmatched analysis engine and design tools for engineers working on transportation, industrial, public works, sports, and other facilities.

ii. From its 3D object based graphical modeling environment to the wide variety of analysis and design options completely integrated across one powerful user interface, SAP2000 has proven to be the most integrated, productive and practical general purpose structural program on the market today. This intuitive interface allows you to create structural models rapidly and intuitively without long learning curve delays. Now you can harness the power of SAP2000 for all of your analysis and design tasks, including small day-to-day problems.

iii. Complex Models can be generated and meshed with powerful built in templates. Integrated design code features can automatically generate wind, wave, bridge, and seismic loads with comprehensive automatic steel and concrete design code checks per US, Canadian and international design standards.

iv. Advanced analytical techniques allow for step-by-step large deformation analysis, Eigen and Ritz analyses based on stiffness of nonlinear cases, catenary cable analysis, material nonlinear analysis with fiber hinges, multi-layered nonlinear shell element, buckling analysis, progressive collapse analysis, energy methods for drift control, velocity-dependent dampers, base isolators, support plasticity and nonlinear segmental construction analysis. Nonlinear analyses can be static and/or time history, with options for FNA nonlinear time history dynamic analysis and direct integration.

v. From a simple small 2D static frame analysis to a large complex 3D nonlinear dynamic analysis, SAP2000 is the easiest, most productive solution for your structural analysis and design needs.

3.3. Foundation design

The recommended criteria given below for the pile foundation and more specifically to steel cylindrical (pipe) pile foundation.-

3.3.1. General

The foundation should be designed to carry static, cyclic and transient load without excessive deformation or vibration in the platform. Especially attention should be given to the effect of the cyclic and transient loading on the strength of the supporting soil as well as on the structural response of the piles. The guideline provided in sub-heading 3.3.3, 3.3.4, 3.3.5 is based upon the static, monotonic loading. Furthermore, this guideline does not necessarily apply to called problem soil such as carbonate material or volcanic sand or highly sensitive clay. The possible of the movement of the sea-floor against the foundation member should be investigated and force caused by such movement, if the anticipated should be considered in the design.

3.3.2. Pile Foundation

The type of the pile foundation in used in the offshore structure, which is given below:-

3.3.2.1. Driven Piles

The open ended pile is commonly used in the foundation for the offshore structure. These piles are usually driven into the sea-floor with the impact load by hammer which is operated by using the energy resources such as diesel, fuel or hydraulic power as the source of the energy. The pile wall thickness should be adequate to resist the axial and lateral load as well as stress during the pile driving. It is possible to predict approximately stress during pile driving using the principle of one dimensional stress wave transmission by selecting carefully parameter that govern the behavior of the soil, pile, cushion and hammer.

When a pile refuses before it reach design penetration, one or more of the following action can be taken:-

1. The review of hammer performance, all aspect of hammer performance, possibly with the aid of hammer and pile head instrumentation, may identify problem which can be solved by improved hammer operation and maintenance or by use of more powerful hammer.
2. Reevaluation of design penetration. Reconsideration of load, deformation and required capacities, of both individual piles and other foundation element, and the foundation as a whole, may identify reserve capacity available.
3. Modification of the piling procedure, usually the last course of action, may include one of the following:-
    i. **Plug Removal: - the** soil plug inside the pile is removed by the jetting and air lifting or drilling to reduce pile driving resistance. If plug removal result in inadequate pile capacities, the removal soil plug should be replaced by gravel grout or concrete plug having sufficient load carrying capacity to replace that of the removed soil plug. Attention should be paid to plug/pile

load transfer characteristic. The plug removable may not be effective in some circumstances particularly in the cohesive soil.

ii. **Soil Removal Below Pile Tip:-** It is removed either by the drilling an undersized hole or jetting equipment is lower through the pile which act as casing pipe for the operation. The effect on the pile capacity of drilling an undersized hole is unpredictability unless there has been previous experience under similar condition. Jetting below the pile tip should in general be avoided because of the unpredictability of the result

iii. **Two-State Driven Pile: -** a first stage or under pile is driven to a predetermined depth, the soil plug is removed and second stage on inner pile is driven inside the first stage pile is grouted to permit the load transfer and developed the composite action.

3.3.2.2. Drilled and Grouted Piles

It can be used in the soil which will hold an open hole with or without drilling mud. Load transfer between grout and pile should be design in accordance with API-RP2A. there are two types of the drilled and grouted piles as follows:-

1. **Single-Stage: -** an oversized hole is drilled to the require penetration, a pile is lowered into the hole and annulus between the pile and soil is grouted. This type of the pile can be installed in the soil which holds an open hole to the surface. Here is also use the alternative method in which the pile with expendable cutting tools attached to tip can be used as part of the drilled stem to avoid the time required to remove the drill and insert pile.
2. **Two-Stage: -** drilled and grouted pile consists of two concentrically placed piles grouted to become a composite section. A pile driven to penetration which has been determined to be achievable with available equipment and below which an open hole can be maintained. This outer pile becomes the casing for next operation which is to drill through it to the required penetration for the inner pile. The insert pile is lowered into drilled hole and annuli between insert pile and soil been two piles are grouted.

3.3.2.3. Belled Pile

Bells may be constructed at the tip of the piles to give increase bearing and uplift capacity through direct bearing on the soil. Drilling of the bell is carried out through the pile by under reaming with expander tool. A pilot hole may be drilled below the bell to act as sump for unrecoverable cutting. The bell and oil filled w concrete to a height sufficient t develop necessary load transfer between the bell and pile. Bells are connected to the pile to full uplift bearing load using steel ring such as steel members with adequate shear lugs, deformed reinforcement bars or pre-stresses tendons/ load into concrete should be designed with accordance with ACI-318. The steel ring should be enclosed for their full length below the pile with spiral reinforcement meeting the requirement of ACT-318. Load transfer been cover and pile should be design accordance with API-RP2A.

3.3.3. Pile Design

3.3.3.1. Foundation Size

When sizing a pile foundation, the following items should be considered Diameter, penetration wall thickness, type of tip, spacing, and number of pile, geometry, location, and mud line restrained, material strength, and installation method.

3.3.3.2. Foundation Response

A number of the different analysis procedure may be utilized to determine the requirement of a foundation. At a minimum, the procedure used should properly simulate the non-linear response behavior of the soil and assure load deflection compatibility been structure and pile soil system.

3.3.3.3. Deflection and Rotation

The deflection and rotation of the individual pile and total foundation system should be checked at all critical location which may include piles, top, point of the contra flexural, mud line etc. it should not exceed serviceability limit which wd render the structure inadequate for its intended function.

3.3.3.4. Pile Penetration

The designing of the pile penetrations be sufficient to develop adequate capacity to resist the maximum computed axial bearing and pull-out load with the appropriate factor of the safety. The ultimate pile capacity can be computed in accordance to API-RP2A. The allowable pile capacity are determine but dividing the ultimate pile capacity by the appropriate factor of safety which should not be less than the following values:-

Table-3.1: Factor of Safety for Pile Penetration

S.No	Load Condition	Factor of Safety
1	Design environmental condition with appropriate drilling loads.	1.5
2	Operating environmental condition during drilling operations.	2.0
3	Design environmental condition with appropriate producing load.	1.5
4	Operating environmental condition during producing operations.	2.0
5	Design environmental condition with minimum loads (for pullout).	1.5

3.4. Time history Analysis

Time history analysis is dynamic analysis of structure. In time history analysis we study the behavior of structure for load which is in time vs acceleration format. It's very difficult to do it manually.

It is applicable for the multi degree of freedom system, when the dynamic force is active on the structure in the form of seismic wave. It is applicable for both linear and non-linear analysis, and it is used to determine the dynamic structural response when structure subjected to the force which varies with time (force should not be constant). The model which analyze is Linear Time History. The data of the time history is taken from 1940 EI Centro Earthquake, acceleration vs. time which occurred at 19 may 1940 near the Imperial Valley of southeastern (Southern California) which near between the two borders that is Mexico and United States. The magnitude moment is 6.9 on the Mercalli intensity scale and it was the first strong ground motion because it near the fault rupture which recorded by the seismograph. Defining the time history function and fill the required value which is given below:-

Time history Analysis maybe in two forms which is given below:-

3.4.1. Linear Time History Analysis

Linear time history analysis calculates the solution to the dynamic equilibrium equation for the structural behavior (displacement, member force etc.) at an arbitrary time using the dynamic properties of the structure and applied loading when a dynamic load is applied. The Modal superposition method and direct method are used for linear time history analysis.

Because of linear analysis characteristics, nonlinearity is not considered. When using a nonlinear material, the material is converted to an equivalent linear elastic material for analysis.

The water level can be defined for the linear time history analysis and the effective stress results can be viewed. Also the drained/untrained effects of the material can be included in the analysis

3.4.1.1. Direct method

The direct method is a time history analysis that uses the DOF of the total analysis area as a variable. The dynamic equilibrium equation for the total DOF can be integrated gradually with time to find the solution. The solution is found for each time stage without any form change to the equilibrium equation and various integration methods can be used. The direct integration method conducts the analysis for all time stages and the number or time stages is proportional to the analysis time.

3.4.2. Nonlinear Time History Analysis

Nonlinear time history analysis is known for simulating a structure behavior under severe earthquake more proper than other methods. However for simplicity, most of the bridges in the category of Ordinary Standard Bridge (OSB) are being analyzed by a combined procedure which consists of a linear ARS analysis for earthquake response (demand) and a static nonlinear pushover for ultimate displacement (capacity) per the guidelines of many transportation agencies worldwide. The demand and capacity are then compared to determine the safety of the bridge. For the single degree of freedom (SDF) system, this procedure has been proven to be an effective method with satisfactory accuracy. For bridges in the category of OSB but with noticeable characteristics of multi-degree of freedom (MDF) system, large discrepancies between deformation patterns from linear analysis and nonlinear pushover are often observed by engineers. So, the accuracy of conclusion from this procedure is questioned. To explore nonlinear dynamic behavior of these bridges and investigate the adequacy of the popular combined linear with nonlinear analysis procedure, a series of bridges within the category of OSB ranging from slight to severe mass and stiffness unbalance was analyzed. The analysis methods used for each bridge include linear and nonlinear time history analysis, linear ARS analysis and nonlinear static pushover.

3.5. Response Spectrum Analysis

Response-spectrum analysis (RSA) is a linear-dynamic statistical analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response-spectrum analysis provides insight into dynamic behavior by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of damping. It is practical to envelope response spectra such that a smooth curve represents the peak response for each realization of structural period.

Response-spectrum analysis is useful for design decision-making because it relates structural type-selection to dynamic performance. Structures of shorter period experience greater acceleration, whereas those of longer period experience greater displacement. Structural performance objectives should be taken into account during preliminary design and response-spectrum analysis.

3.6. IS CODE 1893 Parts 1:2016

It is Indian Standard code for the earthquake resistant design of the structure, According to which we select the parameter of seismic analysis. Some seismic data are taken from IS CODE 1893 Part1:2016 which is given below:-

Table-3.2: Seismic Parameter

S.No	Parameter	Values
1.	Soil Type	Type II (medium soil)
2.	Zone (Z)	IV (0.24)
3.	Importance Factor (I)	1.2
4.	Response Reduction Factor (R)	5.0

3.7. Details Parameter of Model

The parameter of the offshore structure is given below which is used in the model:-

Table-3.3: Parameter of Model

S.No	Parameter	Numerical Values (m)
1.	Pile	0.900 diameter
2.	Connecting Rod	0.725 diameter
3.	Beam	0.400 diameter
4.	Stair Step Rod	0.075 diameter
5.	Height of 1st floor	80.0 from datum
6.	Height of 2nd floor	90.0 from datum
7.	Height of 3rd floor	98.0 from datum
8.	Height of roof	106.0 from datum
9.	Height of helipad	3.500 from roof
10.	Thickness of slab	0.120m
11.	Plan area of one platform	35mx35m

3.8. Live Load

The live load is taking according some general information:-

Table-3.4: live load

S.No	Load Name (Live)	Frame Load (KN/m)	Area Load (KN/m^2)
1.	Beam	22	-
2.	Deck	-	5
3.	Helipad	-	5
4.	Stair	-	2KN/m^2

3.9. Wave Load

Some wave parameter are taken by Code API-RP2A(API 1993) is given below:-

Table-3.5: Wave Load Parameter

S.No	Parameter	Details
1.	Wave Kinematics Factor	1.0
2.	Storm Water Depth	72.0 m
3.	Wave Height	3.0 m
4.	Wave Period	10 seconds
5.	Wave Theory	Airy Wave Theory
6.	Maximum Discretization Segment Size	1.524
7.	Mud line from Datum	-72m
8.	Number of Wave Crest Position Considered	1.0
9.	Water Weight Density	10.0536 KN/m^3

3.10. Current Load

The current load of the oceans are induced the drag forces on the offshore structure and it is generated by different number of the forces such as wind, breaking wave, salinity, etc. It is classified into the two categories; first one is Wind Driven Current whose nature is linear and it depend upon the velocity of the wind; second and last one is Tidal Current whose nature is non-linear.

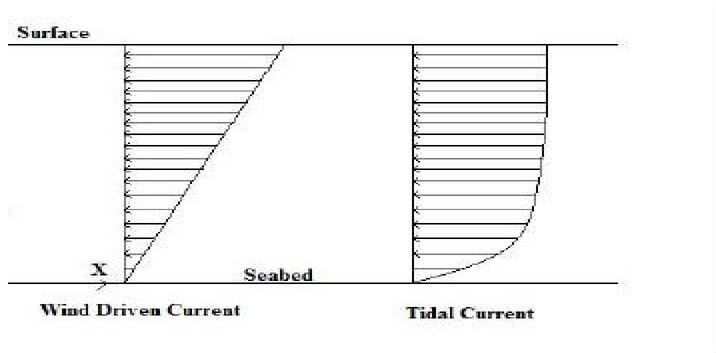

Fig-3.1: Type of Ocean Current

Using the approximate current velocity of with reference to the marine engineering and considering the wind driven current for calculation. Putting all value according to the API-RP-2A-WSD which is given below:-

Table-3.6: Parameter for ocean current.

S.No	Current Blockage Factor	Current Profile	Vertical from Datum	Current Velocity of water
1.	0.9	Linear	72 m	1.7 (m/s)

3.11. Different View

The different view of the model is given below:-

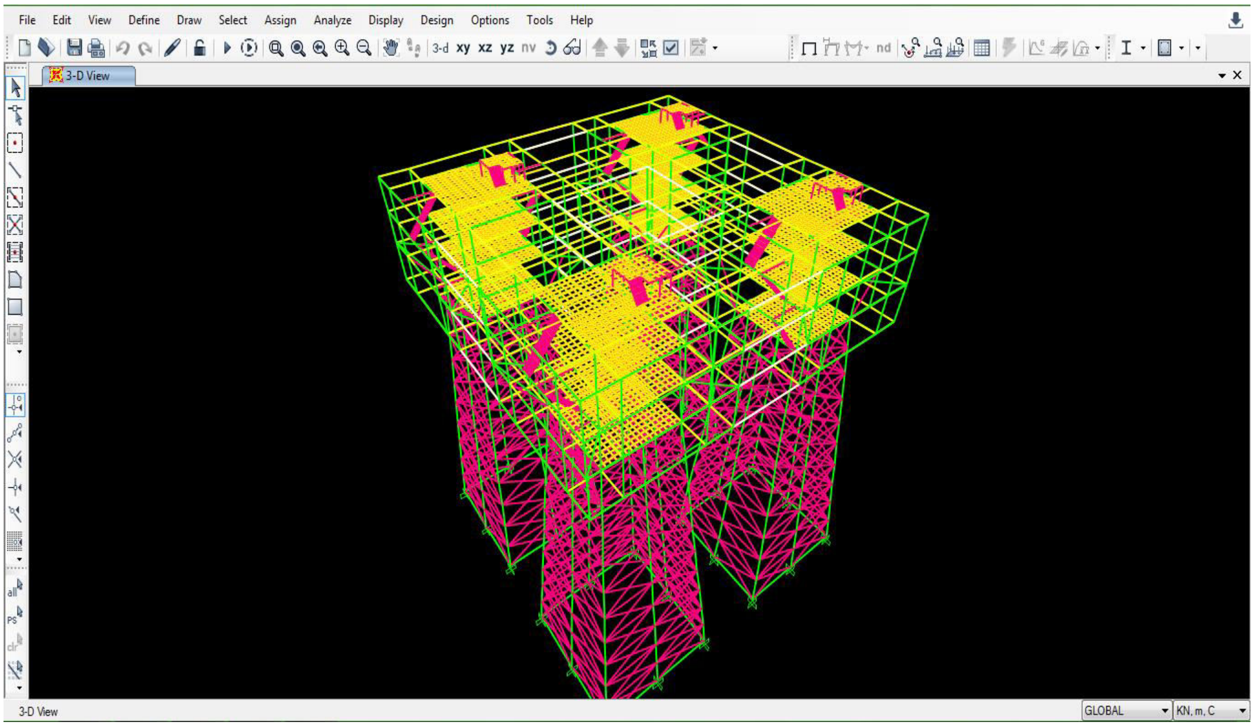

Fig- 3.2: 3D View

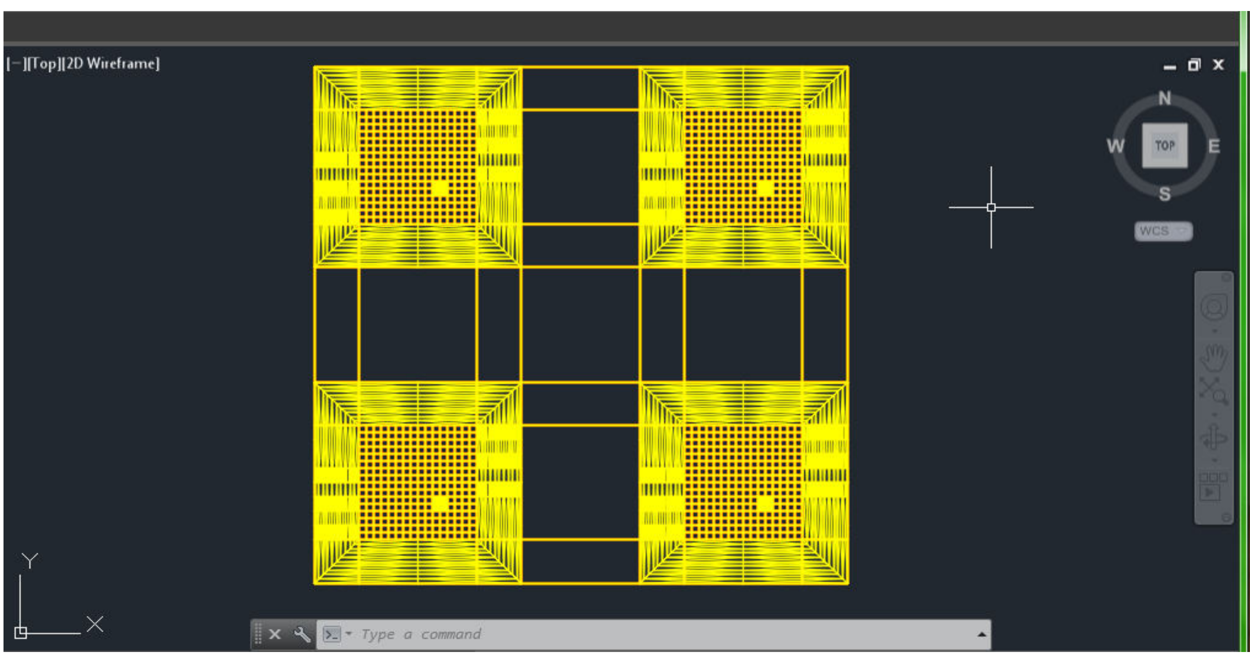

Fig- 3.3: Top View

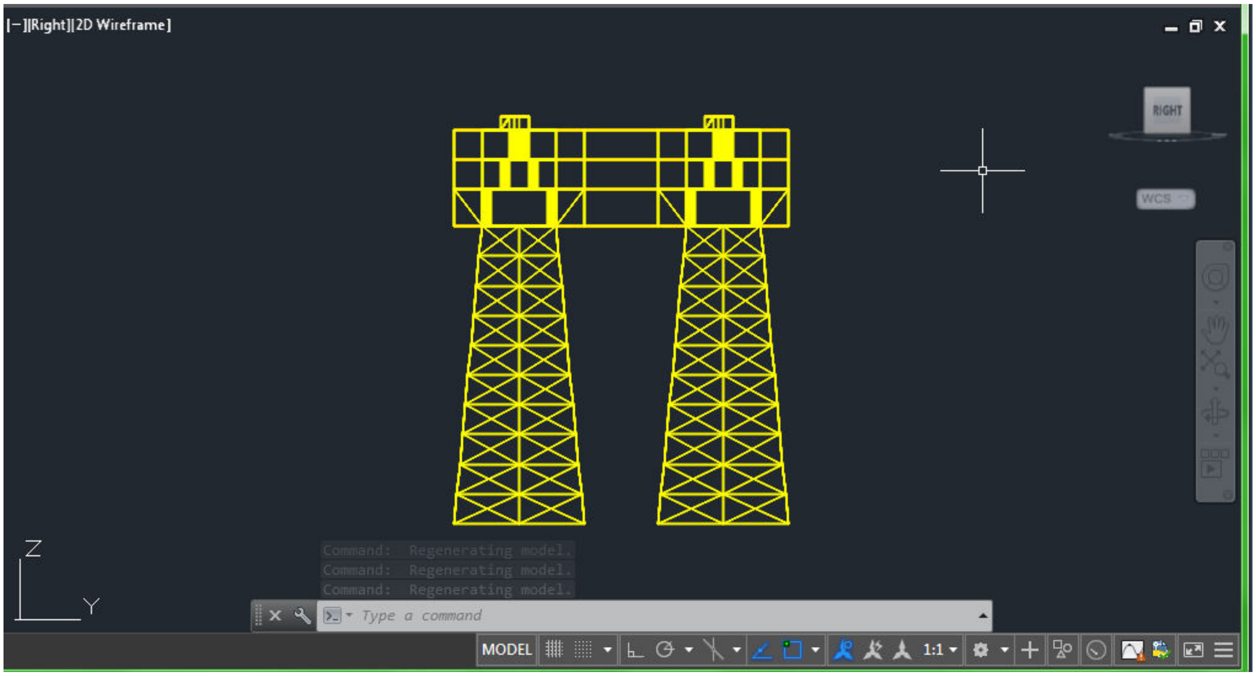

Fig- 3.4: Elevation View

CHAPTER 4

RESULT AND DISCUSSION

4.1. Modal Time Period and Frequency

The time period is defined as the time at which number of mode generate.

The table and graph of the modal time period and frequency of the model is given below:

Table-4.1: Modal Time Period

Mode	**Period (sec)**	**Frequency (cyc/sec)**	**CircFreq (rad/sec)**	**Eigenvalue (rad^2/sec^2)**
Mode1	1.726123	0.579332878	3.640055825	13.25000641
Mode2	1.720002	0.581394657	3.653010364	13.34448472
Mode3	1.503991	0.664897539	4.177674445	17.45296377
Mode4	1.052505	0.950113935	5.969741917	35.63781856
Mode5	0.869438	1.150167665	7.226716574	52.22543244
Mode6	0.784742	1.274304921	8.006693954	64.10714808
Mode7	0.610791	1.637221409	10.2869655	105.8216592
Mode8	0.541883	1.845416199	11.59509195	134.4461572
Mode9	0.376223	2.657995058	16.7006755	278.912562
Mode10	0.270118	3.702090032	23.2609177	541.070292
Mode11	0.158774	6.298240627	39.57301297	1566.023356
Mode12	0.094269	10.6079403	66.65165464	4442.443066

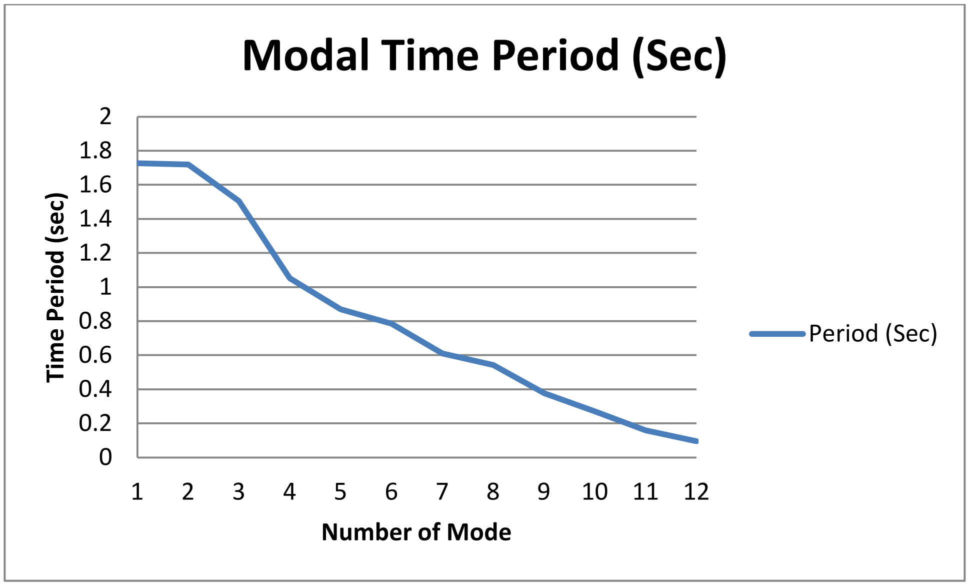

Fig-4.1: Modal Time Period

- In the above graph we found that the value of the time period is 1.726123 sec which is maximum showing at the mode-1.

4.2. Base Reaction

The base reaction is defined as the reaction which acts at the number of the mode of the structure.

The base reaction of the model is given below in the form of the mode number:-

Table-4.2: Base Reaction

No of Mode	GlobalFX (KN)	GlobalFY (KN)	GlobalFZ (KN)	GlobalMX (KN-m)	GlobalMY(KN-m)	GlobalMZ (KN-m)
Mode1	3350.142	137.169	0.625	-13294.9387	325312.465	-144259.963
Mode2	-137.662	3379.996	0.046	-327875.59	-13406.7522	160023.4476
Mode3	-7.105	-48.437	0.069	4528.3388	-666.0361	182566.2659
Mode4	12.351	-4.199	17.285	1047.4476	-1211.8596	-646.0176
Mode5	4019.118	-0.722	-3.494	-92.8514	134965.1164	-180850.574
Mode6	-3308.089	558.519	-5387.455	-322778.28	364773.2418	259241.2788
Mode7	-9669.574	31.936	113.804	2821.9821	-698711.11	436094.4829

Mode8	-13316.644	-33.227	156.147	9137.2561	-784206.01	597965.8573
Mode9	-380.705	-545.282	18870.063	876873.0279	-550876.46	-15684.5258
Mode10	23868.133	268.762	-6073.228	-281383.143	367527.4451	-1058599.67
Mode11	77590.828	-197.919	6393.406	299356.9379	73916.1446	-3505884.7
Mode12	-329283.52	-162.121	10560.736	502118.5486	-3406909.4	14800520.67

4.3. Velocity and Acceleration Due To Wave

Due to apply the wave loading at the pile of the offshore structure the velocity of the water increase at the pile of the structure.

The table of the velocity, acceleration, wavelength and pressure due to wave is given below:-

Table-4.3: Velocity and acceleration due to wave

S.No	**X_{wave} (m)**	**Z_{wave} (m)**	**V_{xwave} (m/sec)**	**V_{zwave} (m/s)**	**A_{xwave}(m/s^2)**	**A_{zwave}(m/s^2)**	**Pressure (KN/m^2)**
1.	10.7906	-9.1248	-0.2207	0.00757	0.00537	0.1228	372.2806
2.	118.3429	-28.0898	0.025	-0.295	-0.1962	-0.0148	282.801
3.	106.0949	-25.3805	-0.1399	-0.3024	-0.1989	0.0839	252.9402
4.	138.6286	-44.3454	0.1368	-0.0874	-0.068	-0.0694	448.0076
5.	64.7581	-35.2500	-0.208	0.1074	0.0747	0.1180	351.0802
6.	45.6207	-63.6974	-0.0297	0.034	0.0658	0.0060	639.9154

X_{wave} = Wavelength is X is equal to wavelength in Y-direction because the dimension in the Y- direction is same as X-direction and same property and loading.

Z_{wave} = Wavelength in the Vertical Direction.

V_{xwave} = Velocity of the wave in the X (horizontal) direction.

V_{zwave} = Velocity of the wave In the Z (vertical) direction.

A_{xwave} = Acceleration in X (horizontal) direction.

A_{zwave} = Acceleration on Z (vertical) direction.

- In the above table we found that the value of the wavelength in the horizontal direction is maximum as compared to the value of the wavelength in the vertical direction.

Using the Contour option to display the minimum and maximum velocity, acceleration and pressure in the generated wavelength, which is given below at default scaling ratio (0.5056). The figure given below which represent the maximum, minimum value of the wavelength which is produced by the applied wave.

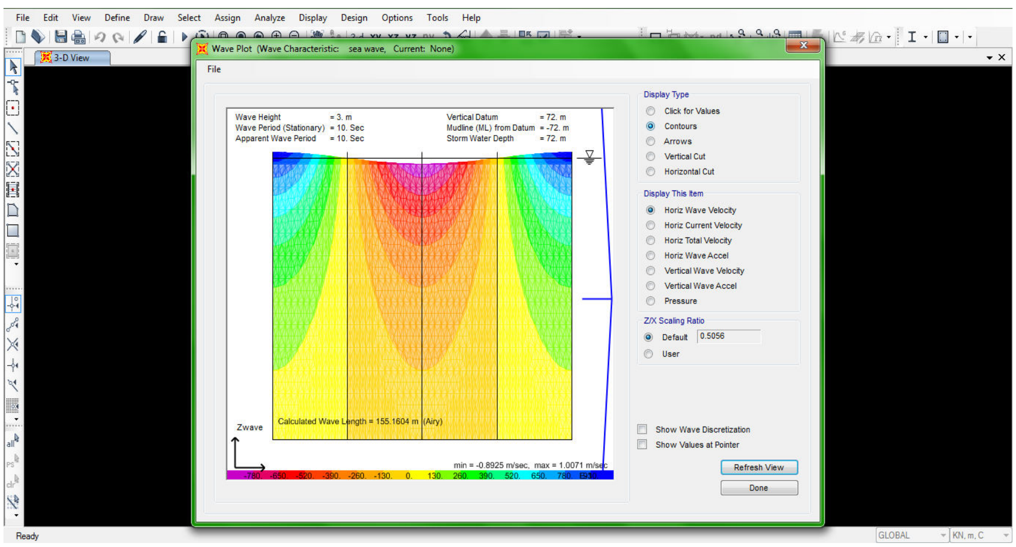

Fig-4.2: Contour of wave.

4.4. Maximum and Minimum Acceleration, Velocity and Pressure at Offshore Structure Due To Wave

Table-4.4: Maximum & Minimum Velocity and acceleration due to wave.

S.No	Parameter	Minimum	Maximum
1.	V_{xwave} (m/sec)	-0.8925	1.0072
2.	V_{zwave} (m/sec)	-0.9425	0.9425
3.	A_{xwave} (m/sec^2)	-0.5957	0.5957
4.	A_{zwave}(m/sec^2)	-0.6295	0.5571
5.	Pressure (KN/m^2)	0	725.4882

- In the above table, the maximum value of the velocity in the horizontal direction is maximum as compared to the maximum value of velocity in the vertical direction.

4.5. Forces due to Time History and Response Spectrum Function

The axial forces, shear forces, bending moment due to time history and response spectrum function is given below:

Table-4.5: Force due to Time History and Response Spectrum Method

`	Frame No.	Max/ Min	P (KN)	V_2 (KN)	V_3 (KN)	M_2 (KNm)	M_3 (KNm)	T (KNm)
TH	10948	Max	5.386	4069.48	0	48.085	0	7582.471
RS	10948	Max	9389.763	9022.328	1007.765	238	1914.491	19553.7541
TH	10949	Max	23.856	4570.762	0	0	39281.1857	0.0197
RS	10949	Max	6285.554	11893.53	1182.741	1394	101908.257	3932.916
TH	10950	Max	18.703	4190.867	0.025	11.54	33464.2028	0.0047
RS	10950	Max	5040.376	10892.10	209.258	14.143	86774.2282	2822.7076
TH	10951	Max	14.277	3956.701	14.741	40.499	28176.9405	0
RS	10951	Max	4151.672	10276.73	49.477	149	73035.7008	1825.1275
TH	10952	Max	9.635	3820.415	22.712	53.712	23202.4764	0
RS	10952	Max	3635.743	9921.634	50.271	108.32	60119.8617	1057.3743

P = Axial force in frame local 1 axis direction at the specific station.

V_2 = Shear force in frame local 2 axis direction at the specific station.

V_3 = Shear force in the frame local 3 axis direction at the specific station.

M_2 = Bending moment about frame local 2 axis direction at the specific station.

M_3 = Bending moment about frame local 3 axis direction at specific station.

T= Torsion moment about frame local 1 axis direction at the specific station.

The graph of the axial force and shear forces in local axis y direction is given below with respect to above data:-

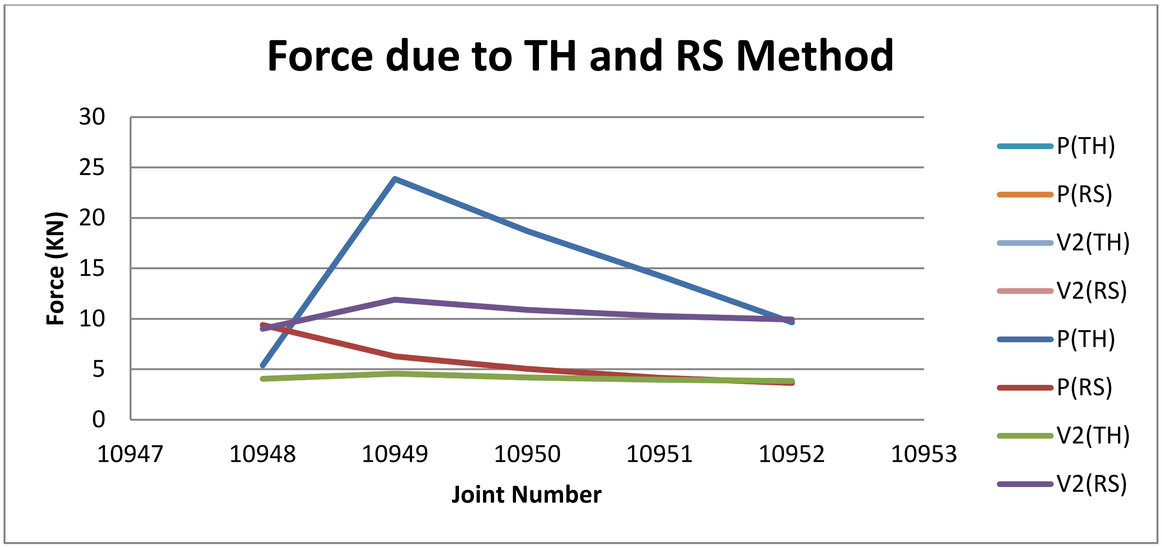

Fig-4.3: Force due to TH and RS

- In the above graph the value of the axial force due to applied the response spectrum is maximum because of value of the hydrodynamic forces is increasing and decreasing of the moving ship near the offshore structure.

4.6. Joint Displacement due to Response Spectrum Method

The value of the joint displacement between two frame member by defining the response spectrum function is given below:-

Table-4.6: Variation of the joint displacement by Response Spectrum Function

Joint No	U_1 (m)	U_2 (m)	U_3 (m)	R_1 (radians)	R_2 (radians)
10.	0.00628618	0.000519	0.0051289	0.002997	0.03851
11.	0.00627734	0.000265	0.0050816	0.000145	0.021415

12.	0.00625922	0.000269	0.008514	0.00043	0.005981
13.	0.00627786	0.000606	0.0035839	0.00075	0.02175
14.	0.00625466	0.000451	0.008468	0.000132	0.006049

U_1, U_2, and U_3= Joint displacement (relative to ground) in joint local 1, 2, and 3 axis direction respectively

R_1 and R_2 = Joint rotation (relative to ground) in joint local 1,and 2 axis direction.

The graph of the above table no-06 is given below. Which showing the variation of the joint displacement due to providing the response spectrum function:-

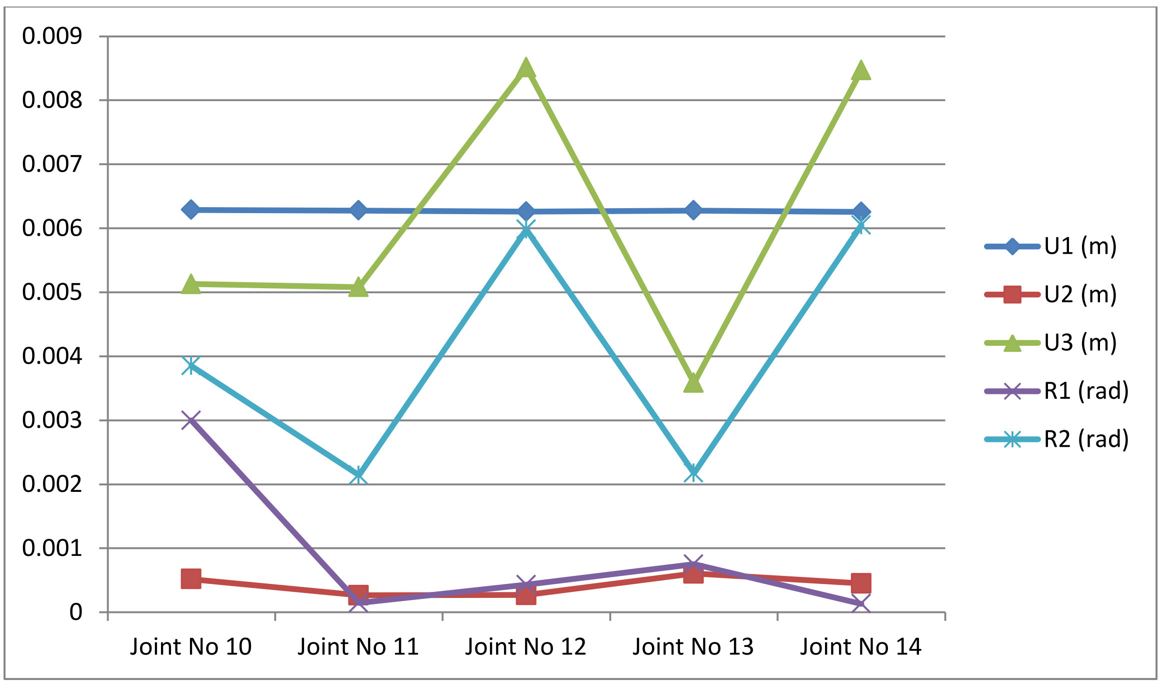

Fig-4.4: Variation of the joint displacement due RS Method.

- The value of the join displacement in local axis z direction increase and decrease because of the variation of the wavelength in the vertical direction.

4.7. Joint Relative Velocity due to Response Spectrum

The joint relative velocity due to response spectrum at the offshore structure is given below:-

Table-4.7: Relative velocity of the join due to response spectrum.

Joint No	U_1 (m/sec)	U_2 (m/sec)	U_3 (m/s)	R_1 (rad/sec)	R_2 (rad/sec)	R_3(rad/sec)
10.	3.737	0.0021	0.2208	0.012	0.168	0.001073
11.	3.7392	0.0016	0.1949	0.00124	0.145	0.0004919
12.	3.729	0.0024	0.0358	0.001759	0.052	0.00182
13.	3.7391	0.0055	0.1632	0.002772	0.147	0.0005281
14.	3.7209	0.0021	0.0359	0.001133	0.053	0.001482s

4.8. Modal Participation factor

The modal participation factor is a measure of how strongly a given mode contributes to the response of the structure when subjected to force/displacement excitation in a specific direction.

The modal participation factor is given by the defined "time history function" in the form of the table:-

Table-4.8: Modal Participation factor

Mode	UX (KN-m)	UY (KN-m)	UZ (KN-m)	RX (KN-m)	RY (KN-m)	RZ (KN-m)	Modal Stiff (KN-m)
Mode1	252.84081	10.35239	0.047143	-217.32	5303.653	22.033087	13.25001
Mode2	-10.3160	253.287	0.003453	-5285.89	-219.088	117.247	13.34
Mode3	-0.407	-2.7753	0.003958	47.978	-6.986875	10567.182	17.45
Mode4	0.008	-0.0640	0.0625	-1.51	-1.323	2.643	35.63
Mode5	76.958	-0.0141	-0.064	0.180	-3278.16	0.231	52.22
Mode6	-0.011	0.512173	-19.5430	-572.492	151.387	1349.491	64.107
Mode7	-91.45	0.313	0.979	1.23	411.188	-3.8314	105.82
Mode8	-98.891	-0.272	1.357	-3.053	1743.31	2.394	134.44
Mode9	7.2868	-3.330	78.4715	-45.921	223.85	-26.25	278.91
Mode10	38.312	1.418	-18.47	20.75	-2546.1	3.647	541.07

Mode11	62.652	-2.209	20.4664	2.442	-4984.48	1.2259	1566.02
Mode12	-47.620	-4.248	35.50	12.5	2064.962	8.5018	4442.44

CHAPTER 5

CONCLUSION

After analysis the model and study the result, the following conclusion are comes out:-

i. The maximum axial forces in the frame in local axis directions at the specific station is due to the defining the response spectrum function as compared to the defining the time history function. The shear force and bending moment in frame member in local first axis direction at the specific station is always low as compared to other local axis direction at the specific station.

ii. The relative joint displacement and relative velocity of join in local first axis direction is always more as compared to the other local axis direction by response spectrum function, but the relative rotation in local second axis is higher than compared to the other local axis.

iii. The spectral displacement is increasing with the increasing in the height of the pile of the offshore structure at the joint due to defining the response spectrum function, and maximum spectral velocity at the joint is linearly increase with increase height of the steel pile of the offshore platform.

iv. The joint reaction at support of front offshore platform is more than back offshore platform because the direction of the flow of water and effect of the moving ship near the platform. And minimum value of the joint reaction at support is due to providing the uniformly distributed load at the frame.

v. The value the wavelength produce in the horizontal direction is larger than the value of the wavelength produce in the vertical direction due to applied the wave loading.

vi. The According to the API-RP2A-WSD, if the maximum time period is less than 4 sec then design jacket type fixed platform and after analysis we found that the maximum value of time periods is 1.726123 sec so its validate that our structure is safe for jacket type offshore structure.

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