RESPONSE ANALYSIS OF BERTHING STRUCTURES FOR WAVE AND EARTHQUAKE INDUCED FORCES INCLUDING SOIL- STRUCTURE INTERACTION

Thesis

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By

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D E C L A R A T I O N

I hereby *declare* that the Research Thesis entitled "**Response analysis of berthing structures for wave and earthquake induced forces including soil-structure interaction**" which is being submitted to the **National Institute of Technology Karnataka, Surathkal** in partial fulfillment of the requirements for the award of the Degree of **Doctor of Philosophy** in **Civil Engineering** *is a bonafide report of the research work carried out by me*. The material contained in this thesis has not been submitted to any University or Institution for the award of any degree.

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C E R T I F I C A T E

This is to *certify* that the Research Thesis entitled "**Response analysis of berthing structures for wave and earthquake induced forces including soil - structure interaction**" submitted by **Shanthala B** , Register Number: AM04P06 as the record of the research work carried out by her is *accepted as the Research Thesis submission* in partial fulfillment of the requirements for the award of degree of **Doctor of Philosophy**.

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ABSTRACT

Countries surrounded by ocean can easily achieve tremendous progress in trade and industry provided proper planning of ports and harbours is made for transportation of goods and materials through sea transport. Rapid growth in the water transport system demands the construction of more port and harbour structures. Berthing structures are constructed in ports and harbours to provide facilities such as berthing and mooring of vessels, loading and unloading of cargo and embarking and disembarking of passengers. Berthing structures are classified as vertical face structures such as diaphragm walls and open piled structures such as jetties. These berthing structures are to be designed for berthing force, mooring force, wave force, current force, seismic force, active earth pressure and differential water pressure, in addition to self-weight of the structure and live load. The deck of berthing structure is generally supported by vertical piles Waves contribute to major loads on marine structures. Therefore it is important to quantify wave-induced load effects to ensure a reasonable, safe and robust design of berthing structures

The piled structures are the most commonly adopted structures in shallow water and deep water. Safe operations on such structures have pressed the necessity to design them to resist the disturbing wave environment, since wave forces are random and vary with time. In the present work, an attempt is made to study the response of vertical member and a simplified piled berthing structure with and without soil structure interaction. Miniature model of such structures are tested for finding natural frequency and response to forced vibrations. All of the experimental observations are reproduced quite accurately by the simulation. It was found that inclusion of water and the soil tend to increase the natural frequency of the structure.

In this research conceptual layout of jetty for berthing 200000 DWT ship is carried out based on the available environmental data and the ship dimensions at New Mangalore Port Trust, Mangalore. Static and dynamic wave response analysis is carried out using StruCAD 3D software, considering various load combinations. Responses are estimated

for various pile diameters and the results are compared. Responses are also found for 45º, 90º and 135º wave directions and it is seen that the structural response is maximum for wave direction perpendicular to the structural alignment. The detailed analysis of berthing structure, for the design significant wave height of 3. 2m and maximum wave height of 5.5 m is carried out for a full cycle of wave and responses are found out.

In order to carryout analysis of a structure under earthquake conditions a representation of the earthquake loading is essential. In such situation dynamic analysis of the structure will be required in which case accelerograms will be used given that they offer detailed representation of the ground motions during earthquakes. At the same time they provide the nature of the earthquake ground shaking. In this thesis three types of recorded accelerograms were used for the time history analysis of berthing structure with and without soil structure interaction. From the time history analysis optimum width was found for the deflection criteria. It was found that soil – structure interaction causes increase in displacements of the structure, which can cause large increase in natural period, leading to much larger relative displacements.

Keywords: Berthing structure, modal analysis, transient analysis, wave response, earthquake response, soil-structure interaction.

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LIST OF NOTATIONS

Symbol	Meaning
A_w	Windage area in m²
B	Beam of vessel
C_D	Drag coefficient
Ce	Eccentricity coefficient
C_m	Mass coefficient
C_s	Softness coefficient
C_w	Shape factor
D_m	Maximum base dimension of the structure
D'	Draught of the vessel m
D	Diameter of the pile
D_L	Average light draft
D_M	Molded depth in m
E	Berthing energy
F	Force due to wind in Kn
F_{wx}	wind forces along the x direction
F_{wy}	wind forces along the y direction
G	Acceleration due to gravity
H	Total height of main structure in meters
I	Importance factor
K	Wave number
K	Stiffness matrix
L	length of the vessel
Lv	Length between perpendicular in m
M	Mass

L	Wave length
P	Wind pressure
P	Mass density
P_z	Design wind pressure at height Z
R	response reduction factor
R	Radius of gyration
Sa/g	Average acceleration coefficient spectra
T	Wave period
T_s	Fundamental nature period
U	Instantaneous water particle velocity
u_0	Maximum horizontal water particle velocity
V	Berthing velocity
N	Kinematic viscosity
V_w	Wind speed
V_z	Design wind velocity
w	Unit weight of water
W_D	Displacement tonnage
W_m	Moulded Weight
ω	Wave frequency

CHAPTER 1

INTRODUCTION

1.1 PORT AND HARBOUR STRUCTURES

The basic function of a port is to provide a link between land and sea transport and to furnish means by which transfers of freight and passengers between the two systems can be made efficiently (Tull 1997). Indian subcontinent is gifted with a long coastline of around 7500 km. The major and minor ports located along the coastline play an important role in promoting commercial and industrial activities. More than 70% of the total trade of our country is through the water, hence port activities and their development is directly linked with the overall economy. Along the coastal belt, India is having 13 major ports and more than 187 minor ports. Due to the high growth rate of Indian economy and the continuous expansion of the maritime trade in the last decade, there has been an increasing need and demand for not only new mechanized ports but also the expansion and developments of the existing ports to cater to the needs of the present generation shipping vessels. Countries surrounded by ocean can easily achieve tremendous progress in trade and industry provided proper planning of ports and harbours is made for transportation of goods and materials through sea transport. Rapid growth in the water transport system demands the construction of more and more port and harbour structures.

Shore Protection Manual (1984) defines harbour as “any protected area affording a place of safety for the vessels”, while Port is defined as “a place where vessels may discharge or receive cargo, it may be the entire harbour including its approaches and anchorage, or any of the commercial part of the harbour where quay, wharves, facilities for transfer of cargo, docks to repair ships are provided”.

A port is constructed to provide facilities for the transshipment of ship cargo, transported to and from the inland locations by rail, road, inland waterways and pipeline. The basic requirement is to accommodate the ships safely along the berths or anchor. Mechanical handling equipments have to be provided for the efficient handling of cargo. Also, storage facilities have to be provided at the port. A port facility essentially consists of pier, wharfs, quays, bulkheads, dolphins and platform of structure, trestle and access bridge or catwalk and buildings. They are classified depending upon the type of service they provide as follows;

a) Harbour protection facilities.

b) Berthing, mooring and repair facilities.

c) Storage facilities.

1.1.1 Berthing structures

The facility constructed in the ports, which actually interacts with the incoming vessels while berthing, mooring and repairing is collectively called berthing structure. The berthing structure consists of jetties, moorings and berthing dolphins etc. The berthing structures and breakwaters constitute the most expensive structures constructed in any port. A berthing structure can be classified as an open type structure or a vertical face type structure. Open type structures consist of a rigid deck supported over vertical piles or combination of vertical and raker piles. The slope underneath the structure, established either by dredging or by filling should be constructed prior to the pile driving to avoid lateral forces to be developed on the piles due to the lateral movement of soil. Fig 1.1 shows an open type structure.

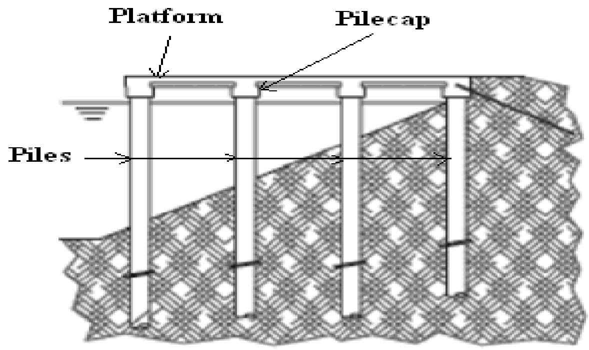


Fig 1.1 open type structure (Gaythwaite, 1981)

In vertical type structures, sheet pile walls, block wall, caissons and diaphragm walls are used. The diaphragm walls resist the active earth pressure and the load is absorbed by the tie rods anchored to the diaphragm walls or raker piles and by passive earth pressure in front of toe of wall. Fig 1.2 shows the vertical face type structure.

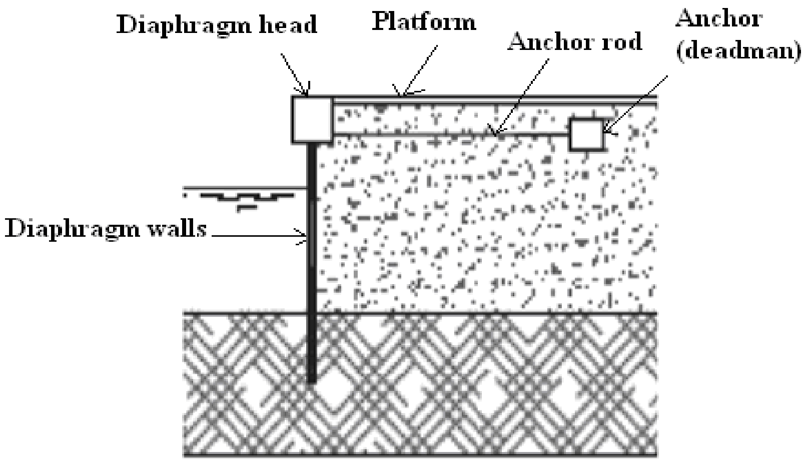


Fig 1.2 vertical face type structure (Gaythwaite, 1981)

1.1.2 Jetty

A pier or jetty is a structure projecting into water, in a harbour basin. Jetties are also located in open water outside actual harbours. A jetty consists of number of structures such as berthing dolphin, mooring dolphin, loading platform, and trestle to shore, each of which has a special type of function.

The mooring dolphins pick up the pull from hawsers. Mooring dolphins for breast line are located at the bow and stern at a distance (about the beam of the ship) from the berth tie, which will not make the moorings too steep. The berthing dolphins support fenders, which absorb berthing impacts. The berthing dolphins should be placed in such a way that the distance neither exceeds the length of straight side of the smallest vessel nor is less than approximately one third of the maximum length of the largest vessel. The jetty structures are further classified based on type of construction material and position of piles. Based on the material of construction, the jetty structures are classified as timber jetties, iron and steel jetties, reinforced concrete jetties. Nowadays reinforced concrete jetties are largely used. The reliability and life of the structure is dependent upon the method of analysis and material properties used.

1.2 WAVE – STRUCTURE INTERACTION

When a structure is subjected to periodic wave forces, it experiences corresponding periodic displacements with a frequency same as that of the wave loading. The associated back-and-forth accelerations of the structural mass induce, in turn, dynamic inertia forces on the structure. The effect of these dynamic inertia forces of the structure is generally to cause an increase in the displacements over those caused by the wave forces. If the natural frequency of the structure is considerably different than that of the wave loading, the dynamic amplification of the deflections will be small. This is the case for relatively short, stiff structures placed in relatively small water depths. In the case of taller structures, or in the case of structures having considerable flexibility because of their particular forms, the natural frequency of the structure may be close to that of the wave loading. This results in appreciable dynamic amplification. In such circumstances, a structure should be designed for dynamic effects in addition to the static considerations and a detailed dynamic analysis is essential.

The piled structures are the most commonly adopted offshore structures. Safe operations on such structures have pressed the necessity to design them to resist the disturbing wave environment. The wave forces are random and vary with time. In the present study, vertical cylinder and a simplified piled structure are analyzed for its dynamic characterization when it is exposed to wave forces. Experimental and numerical simulation is done in order to predict the free surface flow which can be helpful to determine the wave impact on the marine structures.

1.3 SEISMIC RESPONSE ANALYSIS

It may so happen that the marine structures are subjected to severe earthquakes leading to large losses. In such cases knowledge of the dynamic characteristics of the structures under earthquake loads becomes essential. Earthquakes are formed by sudden energy release in a volume of rock lying on a fault. The source of vibration normally located at large distance and significant depth from the site. During the ground shaking caused by an earthquake, the stress waves affect the waves in ocean. Historical references to the effect of local site conditions in the earthquake damage extend back to nearly two centuries. Kramer (2003) referred to the statement made by Mac Murdo in 1824 relating to the earthquake which occurred in Cutch, India in 1819 which stated that "buildings situated on rock were not by any means so much affected as those whose foundations did not reach to the bottom of the soil". Studies about the 1906 San Francisco earthquake showed that the intensity of ground shaking in the earthquake was related to local soil and geologic conditions. Site dependent amplification factors from recordings of microseisms at sites with different subsurface conditions were also developed. The Michoacan earthquake of magnitude 8.1 on September 19, 1985 caused only moderate damage in the vicinity of its epicenter near the Pacific coast of Mexico but caused extensive damage 350Km away in Mexico City. Structural damage was highly selective, while large parts of the city experienced no damage, other areas suffered pronounced damage. The greatest damage occurred in those portions of the Lake Zone underlain by 38 m to 50 m of soft soil. Studies of ground motions recorded at different sites in Mexico City illustrated the significant relationship between local soil conditions and damaging ground motions.

An earthquake of magnitude 7.1 occurred near Mt.Loma Preita located about 100 Km south of San Francisco and Oakland, California on October 19, 1989. The Loma Preita earthquake produced MMI VIII shaking in the epicentral region but resulted in higher intensities of MMI IX in portions of San Francisco and Oakland. The response of two seismographs located at Yerba Buena Island and Treasure Island, virtually at the same distance from the source, recorded dramatically different ground surface motions. The peak acceleration at Yerba Buena Island was 0.06g and the corresponding value at Treasure Island was 0.16 g. The Treasure Island seismograph was underlain by 45 ft of loose sandy soil of hydraulic fill and natural soils, over 55 ft of San Francisco bay mud. The Yerba Buena Island is a rock outcrop and the seismograph was located directly on rock. Significant amplification of the underlying bedrock motion was caused by the presence of soft soils at the Treasure Island site.

The devastating Bhuj earthquake that struck the Kutch area in Gujarat of magnitude 7.7 on 26th January 2001 was the most damaging one in the last 50 years in India. The epicenter of the earthquake was located at 23.4°N, 70.28°E and at a depth of about 25 km to the north of Bacchau town. A number of residential apartments suffered extensive damage and collapse in Ahmadabad city, which is nearly 300 km away from the epicenter. The city stands over deep deposits of cohesionless soil. The random distribution of such damage has been recorded from a number of localities scattered on the left and right banks of Sabarmati River. Govindaraju et al. (2004) carried out a site specific analysis and reported the settlement of soil deposit and soil amplification of a site close to Sabarmati river belt in Ahmadabad City during the earthquake in Bhuj. The high degree of damage to multistory buildings was essentially due to the transfer of large accelerations by soil amplification. Thus soil structure interaction also plays important roles under earthquake.

The seismic structural analyst should perform appropriate analyses to predict the earthquake response of the structure. This includes the selection of a method of analysis, formulation of structural mass and stiffness to obtain vibration properties, specification of damping, definition of earthquake loading and combination with static loads, and the computation of response quantities of interest. The analysis should start with the simplest method available and progress to more refined types as needed. It may begin with a pseudo-static analysis performed by hand or spreadsheet calculations, and end with more refined, linear elastic response-spectrum and time-history analyses carried out using appropriate computer programs. The required material parameters are formulated initially based on preliminary values from the available data and past experience or new test data. Damping values for the linear analysis should be selected, consistent with the induced level of strains. Seismic loads should be combined with the most probable static loads and may include multiple components of the ground motion when the structure is treated as a two-dimensional or three-dimensional model. In the modal superposition method of dynamic analysis, considerable number of vibration modes should be selected. In simplified procedures, the earthquake loading is represented by the equivalent lateral forces associated with the fundamental mode of vibration, where the resultant forces are computed from the equations of equilibrium.

1.4 SOIL - STRUCTURE INTERACTION (SSI)

The motion experienced by the base of a structure founded on rock is essentially identical to that occurring in the same point before the structure is built. The seismic analysis can thus be restricted to the structure excited by this specified motion. But in the case of a structure founded in soft soil site, two important modifications arise for the same incident seismic waves from the source. First, the ground motion recorded on the base of the structure will be different from that which would have been recorded had there been no building. Second, the response to earthquake motion of a structure founded on a deformable soil will not be the same as the structure supported on a rigid foundation.

The presence of the structure in the soil will change the dynamic system from the fixed base condition. The structure will interact with the surrounding soil, leading to a further change of the seismic motion at the base. These are the two aspects of the problem of building-foundation interaction during earthquakes which are of major significance to earthquake engineering.

The practical importance of these effects depends on the properties of the soil-structure system. in terms of the dynamic properties of the system, this dynamic coupling, or interaction between a building and the surrounding soil will generally have the effect of reducing the fundamental frequency of the system from that of the structure on a rigid base, and dissipating part of the vibrational energy of the building by wave radiation into the foundation medium. There will also be energy losses due to internal friction of the soil. Because of these effects, the response of a structure on a soft soil to a given earthquake excitation will, in general, be different from that of the same structure supported on a rigid ground. The influence of the flexibility of soil on the response of structures subjected to earthquake motion is also studied in this thesis.

1.4.1 Dynamic soil - structure interaction

When a structure founded on rock is subjected to ground motion, the extremely high stiffness of the rock constrains the rock motion and the structure response is considered to be that of a fixed base structure. The same structure would respond differently if supported on a soft soil. The motion of the base of the structure, deviate from the free field motion, due to the inability of the foundation to conform to the deformations of the free field. In addition, the dynamic response of the structure would induce deformation of the supporting soil. The response of the soil influences the motion of the structure and the response of the structure influences the motion of the soil. This process is generally referred to as dynamic soil-structure interaction.

The dynamic characteristics of a structural system changes when the supporting medium of soil is also considered as an integral part of the structure, when compared to those with the conventional, completely restrained supports. This is reflected in significant modification of stress components and deflections, in the structural system, from the expected behaviour of the system on a rigid supporting foundation.

More recent trends in earthquake engineering include analyzing the displacement that a structure undergoes during an earthquake, and considering the structural as well as nonstructural damage that it causes. Even though soil-structure interaction increases dampening, which is beneficial, it can also cause additional displacement to the overall structure. This demand on the structure can, in some cases, have detrimental effects. In structures founded on softer soil, the interaction can cause large increases in the natural period of the structure, leading to much larger relative displacements.

1.4.2 Methods of analysing soil-structure interaction

There are two commonly used methods of SSI analysis, multistep methods and direct methods. Multistep methods use the principle of superposition to combine the effects of kinematic and inertial interaction, and are limited to the analysis of linear or equivalent linear systems. Direct methods model the entire soil–foundation–structure system in a single step and are more robust than multistep methods, although they are computationally more demanding.

In the direct method, the entire soil-foundation-structure system is modeled and analysed in a single step. The free field motions are specified along the base and sides of the model and the resulting response of the interacting system is computed for a finite element model from the equations of motion.

In the multistep or substructure method, the soil–structure system is divided into two substructures, a structure, which may include a portion of non-linear soil or soil with an irregular boundary, and the unbounded soil. These substructures are connected by the general soil–structure interface. Usually, a dynamic soil–structure interaction analysis by the multistep method can be performed in three steps as follows:

i) Determination of seismic free-field input motion along the general soil–structure interface;

ii) Determination of the reaction of the unbounded soil on the general soil–structure interface in the form of a displacement–force relationship;

iii) Analysis of the bounded soil–structure system under the action of the externally applied transient loading and the ground interaction force determined by steps (i) and (ii).

1.5 DYNAMIC ANALYSIS PROCEDURES

Linear dynamic analysis procedures are presently used for earthquake resistant design and safety evaluation of structures. The linear dynamic analysis is performed using the response spectrum or the time-history modal superposition method. The primary feature of the modal analysis is that the total response of a structure is obtained by combining the response of its individual modes of vibration, calculated separately. The response spectrum analysis is adequate for structures whose responses to earthquakes are within the linear elastic range. But for structures in which the cracking strength of the concrete and yield strength of the reinforcing steels may exceed under a major earthquake, a linear time-history analysis provides additional information that is essential to approximating the damage or expected level of inelastic response behavior.

In the response spectrum analysis, the maximum response of the structure to earthquake excitation is evaluated by combining the maximum responses from individual modes and multi-component input. The accuracy of the results depends on the number of vibration modes considered and the methods of combination used for the modal and multi component earthquake responses. Each computed result represents the maximum magnitude for that response quantity.

Linear time-history analysis involves computation of the complete response history of the structure to earthquakes, and not just the maximum response values. The results of such analysis serve to demonstrate the general behavior of the seismic response. Combined with rational interpretation and judgment, this can provide a reasonable estimate of the expected inelastic behavior.

1.6 OBJECTIVES AND SCOPE OF THE WORK

The design of berthing structures has been largely based on static analysis of the structure under a variety of deck and environmental loading. The cyclic wave loading applied quasi- statically i.e. the wave force would be considered at which there is maximum base shear and overturning moment. These techniques are adequate when the natural frequency of vibration of the structure is well away from that of wave loading. Consequently as far as serviceability of structure is concerned, an estimate of dynamic effect is necessary. To check the functionality of structure it becomes essential to calculate the deck deflections and corresponding stresses with respect to dynamic forces. Berthing structures when subjected to heavy lateral loads have to be transferred from weak subsoil to firm strata. Soil behaviour is also important parameter for design of berthing structures.

Hence the objective of the present research is to perform the static and dynamic wave response analysis to know the static and dynamic displacements with and without including soil-structure interaction and to perform time history analysis for different earthquake motions.

In the present work, the study is carried out in two phases.

(i) Wave response analysis is carried out with and without considering soil flexibility on dynamic parameters of the models. Experimental investigation is also done on single cylinder and simple berthing structure models.

(ii) Time history analysis of berthing structure is carried out to study the static and dynamic soil-structure interaction effects and due to earthquake effects.

1.7 FORMAT OF PRESENTATION

The thesis is presented in seven chapters. **Chapter 1** introduces the basic concepts of berthing structures, wave response analysis, seismic response analysis, dynamic soil-structure interaction procedures with scope of the present work. **Chapter 2** gives a review of the literature of previous research and the formulation of the present research work. **Chapter 3** describes the general requirements of the berthing structure and the various loads and its contribution on the structure. **Chapter 4** discusses the experimental investigation on single cylinder and small scale berthing structure model. In **Chapter 5** wave response analysis is described with idealization of structure and the methodology followed. It also deals with the time history analysis of structures with and without considering different types of soil medium as support. Earthquake response analysis results are presented and discussed in **Chapter 6**. Time history analysis results are also presented by considering soil structure interaction effects. **Chapter 7** represents the summary of present study with the main conclusions listed and also the scope for future work.

Chapter 2

LITERATURE REVIEW

2.1 WAVE – STRUCTURE INTERACTION

The wave forces form a major contribution to the time-varying components of the external forces acting on a coastal or marine structure. Such a structure vibrating in the fluid interacts with the fluid surrounding it. The dynamic parameters such as mass and damping are modified due to the fluid- structure interaction. The force exerted on a fixed vertical cylindrical pile by surface waves was first considered by Morison et al. (1950) under the restriction that the diameter (d) of the pile is small in the comparison with the length (L) of the waves encountered ($d \leq 0.223$), so that the distortion of the waves due to pile is negligible. The total wave force per unit length acting on a fixed vertical pile for a single degree of freedom system is given by Morison's expression. It is most widely employed in engineering calculations.

McCamy and Fuchs (1954) however undertook a different approach to the problem, especially where cylinders having diameters comparable to the wavelengths are involved. Following the analysis of Havelock (1955), they introduced a diffraction theory in order to obtain a total velocity potential as a sum of incident and reflected potentials which satisfy the proper boundary conditions. The theory developed is based on linear wave and is applicable only to the waves of small steepness. Skjelbreia and Hendrickson (1960) have obtained the values of the coefficients involved in Stokes' fifth order gravity wave and tabulated them for quick reference.

The theory of MacCamy and Fuchs (1954) assumes that the wave acts on a cylinder which extends from the floor up to and through the free surface of the fluid (water). If the cylinder reaches only part way in the fluid, the reflection of incident waves would be less than that for a cylinder piercing through the free surface. The diffraction theory developed by MacCamy and Fuchs (1954) for a vertical cylinder is applied herein to Stokes' fifth order wave and by Skjelbreia and Hendrickson (1960) an expression for the total velocity potential is derived, which satisfies the proper boundary condition at the cylinder surface. This potential function is used to obtain the dynamic pressure due to the wave on the cylinder. The horizontal wave force is calculated by integrating this pressure over the submerged part of the curved surface of the cylinder. The expression for the effective inertia coefficient for each component of the water particle acceleration was obtained.

Chakrabarti (1972) investigated nonlinear wave forces on a vertical cylinder considering diffraction theory obtained solution in a closed form. An expression was derived which was used to calculate the total horizontal force on a vertical cylinder when the wave height is smaller in comparison with the water depth. The force has been written in an equivalent expression of the inertia part of Morison's equation where the effective inertial coefficients were found to be different for different harmonics.

In the study by Chakrabarti et al. (1976), a 76mm diameter vertical tube was tested in a wave tank in 3m of water. Total forces in the direction of the wave and in the transverse direction were measured. In addition, two 0.305m sections located 0.6m apart were instrumented to measure the in line and transverse loads on these sections. The values of C_M and C_D in the Morison equation were calculated from the in- line forces on the two 0.305m sections. The values of horizontal velocities and accelerations in the Morison formula were calculated at the center of the sections using linear wave theory. The total forces on the tube were reproduced theoretically based on the mean curves of C_M and C_D from the experimental data. These theoretical forces were compared with the measured total forces. The frequency and magnitude of the lift forces measured on the tube were also investigated. The lift frequencies and the lift coefficients were presented as functions of the period parameter. The C_M values showed little scatter. The scatter in the C_D values may be partly attributed to the relatively lower proportion of drag compared to the inertia force. It has been shown that the lift force on a small tubular member can be significant. For a period parameter of about 15, the resultant horizontal force on the pile is as much as 60% higher than the in line force. Thus from a design standpoint it is of utmost importance to consider the lift force. Since the lift force is irregular.

Molin R (1979), proposed a method which allows to compute second – order diffraction loads upon three dimensional bodies of any shape. Application has been made to horizontal forces upon axisymmetric bodies but the method can be easily generalized to wave forces in heave or pitch, and to the case of a moving body. It has been explained that the radiation condition does not account for second order locked waves, which are dominant in shallow water.

Graham et al. (1979) have shown that submerged cylinders with varying buoyancy can play as large as inertial forces. The interplay between inertia and buoyancy leads in certain situations to entirely negative heave forces which act at twice the wave frequency. Morison's equation has been adapted to predict the effects for regular waves by introducing a varying volume and a buoyancy term. For small values of wave amplitude and wave steepness the agreement with experiment is made fairly well by fitting the inertial coefficient to each of the experimental curves. Although there is a spread in the values of C_M obtained, it is possible to retain a good deal of accuracy using the theoretical value for a submerged value for a submerged cylinder, $C_M = 2$, as a design coefficient in all cases.

Dennis et al. (1984) conducted a wave tank test on fixed vertical and inclined cylinders in which forces on the cylinder in regular waves were measured. The hydrodynamic coefficients are computed based on the measured local forces on the cylinder, Morison's equation and Stream function theory. The inertia and drag forces were shown as functions of the Keulegan- Carpenter (KC) number. The lift coefficients and lift force frequencies were presented as functions of the Keulegan- Carpenter (KC) number.

Eatock Taylor et.al. (1987), presented analytical expressions for the second order force on a vertical surface, piercing cylinder. Except for the aforementioned contribution from the free surface integral, these expressions confirm those given by Molin and Marion. Numerical results agree, for the cases where they made comparisons, thereby providing independent confirmation of the evalution of the free surface integral (which is found to make a very significant contribution). Additionally, results for infinite water depth are also derived and compared. So this solution is accurate and economical of forces for general three dimensional bodies in regular waves.

Chen (1988), analysed random wave forces consisting of drag and inertia components. The forces were measured in a wave tank with circular cylinders at low Keulegan-Carpenter numbers. The values of the hydrodynamic coefficients were chosen from the corresponding tests in regular waves. The method used in choosing these values applicable to the case of random waves was discussed. The nonlinear drag force contribution was linearized so that the spectral approach may be applied directly. Good correlations between the measured force spectra and the computed spectra were obtained in several cases of random waves. The short-term distribution of the in-line and transverse forces is given. It is found that the in-line force amplitudes follow the Rayleigh distribution reasonably well. The relations of the transverse force to an exponential distribution are exhibited.

In the study by Malenica et al. (1995) the problem of a vertical cylinder subjected to both regular waves and moderate current in water of finite depth is considered. The cylinder was free to move in surge and sway at the frequency of the incoming waves. First-order and steady second-order forces were calculated. In the calculation of mean drift forces and wave drift damping coefficients, both near-field and far-field methods were used and compared with numerical results provided by Grue (1997) with those obtained by a simple formula for wave drift damping. The method used for the calculation of the potentials is semi-analytical and based on Eigen function expansions.

Jessica (2003) compared between the equivalent static load method prescribed by current bridge design specifications and the dynamic analysis techniques. Impact loads and structural deformations predicted by the static method and the dynamic analysis techniques were compared for barge collisions of varying kinetic energies. Results from such comparisons indicate that, for barge collision events associated with relatively low kinetic energy levels, dynamic analysis techniques are preferable.

In the study by Kim et al. (2006), the diffraction of highly nonlinear Stokes waves on vertical cylinders of circular cross section was numerically simulated in the time domain. A finite-element method, based on Hamilton's principle, was used to discretize the fluid domain. The Stokes waves, input at the numerical wave maker, were obtained numerically from the two-dimensional steady solution of the finite element model. A new matching scheme was developed to match the two-dimensional wave at the far field and the three-dimensional diffracted wave in the near field Numerical examples were presented for the diffraction of Stokes waves with various steepnesses by single and multiple circular cylinders.

In the study by Chakrabarti et al. (2007), the steady wave drift force on a submerged body was considered as second-order quantity. The steady drift forces on the following basic structures were computed; A vertical circular cylinder, a submerged horizontal cylinder, a bottom-seated horizontal half cylinder and a bottom-seated hemisphere. The results developed demonstrate the importance of various independent non-dimensional parameters. A numerical program based on linear diffraction theory was used to validate the closed form solution.

In the study by Lei Geng (2010), a time-domain numerical model on wave diffraction from large-scale structures is developed with a higher-order boundary element method (HOBEM). Then the velocity and acceleration of any water particle in the fluid domain can be produced from the solved diffraction potential. Results show that the incident wave force was even equal to diffraction wave force when the diameter of the large-scale cylinder is 8 times as great as the small-scale one; and the total force is different with the locations of the small-scale cylinder. The maximum total force was 1.57 times as great as only the incident wave force. Therefore, the wave force on the stake produced by the diffraction waves due to the existence of upper structures should be considered in the practical engineering.

2.2 EARTHQUAKE EFFECTS

Earthquakes are an open, direct threat to marine structures, when structures are located near the epicenter. The structure in this case will be exposed to the devastating shaking effect of the seismic action, and the result can be catastrophic. Earthquakes may also be a threat to marine structures in an indirect way, through the shaking of the supporting soil. The stability and integrity of structures will be at risk if the soil fails due to liquefaction as a result of the shaking of the soil.

Kenichiro et al (2002) studied the damage caused on piled berthing structure in port structures due to 1995 Hyogoken-Nambu (Kobe) Earthquake.

The result of damage investigation on an open-piled marginal wharf and makes clear the mechanism of the process of damage by comparing the results of numerical analysis and shake table tests with the observed results. It was made clear that the wharf has highly seismic endurance unless retaining walls move towards the wharf. Moreover, it was confirmed that liquefaction of the ground may cause buckling of piles due to large movement of wharf. The following conclusions were drawn regarding the damage of the open piled marginal wharf based on the results of numerical analysis and shake table tests:

1 Large seaward movement of the wharf was caused by inertia force applying to the deck and the seaward movement of the retaining wall during the earthquake. The movement of the retaining wall was derived from liquefaction of back filling ground behind the wall and the alluvial sand layer below the rubble
2 It is possible to simulate the process of local failure/damage during earthquakes with high intensity by the two-step nonlinear dynamic analysis. The analysis to the ground taking into account the effective stress and the analysis to a frame model taking into account the pile ground interactions.
3 The ratio of lateral earth pressure to effective surcharge was about 1.26 at the ultimate state, which was rather small compared to general design condition for steel pile foundations.
4 Shake table tests was effective to investigate the bending moment distribution of piles and movement of the wharf. However, the horizontal displacement of the wharf was smaller than the observed result because of the limitation of scale modeling to mechanical properties of piles and of dissipation of the excess pore water pressure.

Jaiswal et al. (2011), conducted study on building which was located in seismic zone IV. Static analysis on fixed base condition and dynamic analysis on base-isolated condition was performed. Also, response spectrum analysis and linear time history analysis were used on both of fixed base and base isolated buildings. Base isolation helps in reducing the design parameters i.e. base shear and bending moment in the structural members above the isolation interface by around 4-5 times. The absolute displacements increases but relative displacements are reduced thus reducing the damage to the structure when subjected to an earthquake. The shear and bending moments were reduced due to the higher time period of the base isolated structure which results in lower acceleration acting on the structure and also, due to the increased damping in the structure due to the base isolation devices. By the dynamic analysis it was found that the base shear reduced 55-60 % in response spectrum analysis, whereas time history analysis base shear reduces by 70-80%. Generally, the peak displacements obtained by the time history analysis were less than those of the response spectrum method of analysis. This is the case because damping due to the hysteretic effect is more than the equivalent damping considered in the response spectrum method of analysis.

2.3 SOIL- PILE STRUCTURE INTERACTION

The effect of local soil conditions on ground motions have been illustrated in earthquakes around the world, from many observations. The response to earthquake motion of a structure founded on a deformable soil will be different from the response of the structure that is supported on a rigid foundation. Significant progress has been made in the last three decades in developing methods to analyse the interaction between structure and its foundation medium.

In the recent years, a number of studies have been conducted in the area of soil–structure interaction, modeling the underlying soil in numerous sophisticated ways. The search for a physically close and mathematically simple model to represent the soil media in the soil–structure interaction problem, leads to two basic classical approaches, viz., Winkler approach and Continuum approach. Since the philosophy of foundation design is to spread the load of the structure on to the soil, ideal foundation modeling is that wherein the distribution of contact pressure is simulated in a more realistic manner. From this viewpoint, both these fundamental approaches have some characteristic limitations. The mechanical behavior of subsoil appears to be erratic and complex and it seems to be impossible to establish any mathematical law that would conform to actual observations. Simplicity of models that yield reasonable results becomes a prime consideration in this context. Many attempts have been made, to improve upon these models by some suitable modifications, to simulate the behavior of soil more closely from physical aspects.

Winkler's idealization represents the soil medium as a system of identical but mutually independent, closely spaced, discrete, linearly elastic springs. Hetenyi (1946), Dutta and Roy (2002), Brown et al. (1977) and Allam et al. (1991) conducted studies in the area of soil–structure interaction on the basis of Winkler hypothesis for its simplicity. This model necessitates the determination of the stiffness of elastic springs used to replace the soil below foundation. The subgrade stiffness is the only parameter in the Winkler model to idealize the physical behavior of the subgrade and representative values for the same were presented in the literature by Terzaghi (1955). The stiffness of the springs for arbitrary shaped footings resting on homogeneous elastic half-space was suggested by Gazetas (1991). An analytical method to estimate the stiffness of the foundations embedded into the stratum over rigid rock corresponding to different stress distribution below the foundation was presented by Baidya and Sridharan (1994). This study highlighted on the sensitivity of the stress distribution below the foundation in the estimation of the dynamic stiffness of the underlying soil media.

In the continuum approach the infinite soil medium is considered as a continuum. Soil mass basically constitutes of discrete particles compacted by some inter granular forces. The problems commonly dealt in soil mechanics involve boundary distances and loaded areas, which are very large compared to the size of the individual soil grains. In the continuum idealization, generally the soil is assumed to be semi-infinite and isotropic for the sake of simplicity. Boussineq analysed the problem of a semi-infinite, homogeneous, isotropic, linear elastic solid subjected to a concentrated force acting normal to the plane boundary, using the theory of elasticity effectively as a representation for the soil continuum media. This approach provides much more information on the stresses and deformations within the soil mass than the Winkler model.

It also has the important advantage of simplicity of the input parameters, modulus of elasticity and Poisson's ratio.

Since the scope of numerical methods is incomparably wider than that of analytical methods, the extensive use of general purpose finite element method (FEM) with the widespread availability of powerful computers is on the rise. The general principles and use of this method is well documented in the literature of Desai and Abel (1987), and Zienkiewicz et.al (1989). The finite element method treats a continuum as an assemblage of discrete elements whose boundaries were defined by nodal points and assumes that the response of the continuum can be described by the response of the nodal points. The nature of the soil can be considered as linear or nonlinear. Since its appearance, the finite element method has turned out to be one of the most important analytical tools in interaction investigation. However, the use of three dimensional elements to idealize the soil and footings of skeletal structure, or even two-dimensional finite elements in the case of nonlinear interaction, often appears uneconomical due to the large number of degrees of freedom necessary to describe the behavior of the soil-footing system. In order to overcome this drawback, researchers have experimented with several approaches and explored various techniques, using translational and rotational springs, fictitious members, and line and surface elements to simulate the soil-footing system. In many instances, the dynamic properties of the half space were further approximated by discrete springs and dash pots. However, it was agreed that at certain complex sites, finite element idealization of elastic half space denoting soil below foundation may prove useful (Kameswara Rao 1998).

Various analytical formulations have been developed to solve complex practical problems assuming linear soil-structure interaction. However, the effect of nonlinear behavior of the supporting soil on the seismic response of structures has not been fully addressed in the literature. Numerous studies were made on the effect of soil–structure interaction under static loading by Chamecki (1956), Morris (1966), Subba et al. (1985), and Noorzaei et al. (1993), considering the effect in a very simplified manner, demonstrating that the force quantities were revised due to such interaction. Dutta et al. (1999) summarised the effect of various framing action parameters like ratio of flexural stiffness of beam and column, number of storey, and number of bays on change in column moment due to soil-structure interaction. The researcher also formulated approximate method to predict the change in column moments from various parameters which may include the soil-structure interaction effects.

Poulos (1975), studied numerically the dynamic soil-reaction characteristics of axially loaded single piles. The proposed dynamic t-z and Q-z models were shown to yield reliable pile responses for a variety of soil-pile and loading conditions. One of the convenient features of the models was that they can be easily incorporated into both frequency and time-domain analysis. Therefore, the proposed models can be extended to problems with nonlinear soil-pile interaction effects, even though the models are based purely on linearly elastic soil-pile conditions. The proposed models may be utilized in a variety of problems. For static applications, the models can be used to improve the reliability of existing static t-z and Q-z relations at small pile settlements. For dynamic problems, the models can be used to predict the axial responses of dynamically loaded pile foundations.

In most of the studies the soil is idealized as a linear, homogeneous, isotropic, elastic half space. But for many open plane frames and particularly for the interior frames of long structures, which may at times rest on a raft foundation, a plane-strain approach is commonly adopted for representing the soil support in order to achieve savings in core memory requirement and computational effort. Srinivasa Rao (1995) compared the more realistic half space continuum and the plane strain approach to examine the approximation involved in the latter type of representation of soil, considering symmetrical R.C. open plane frames and concluded that if forces in the superstructure elements only are the focus of interest, representation of the soil support by the plane strain condition was a fair approximation of the actual situation. Onu (1996) described a simplified procedure for the linear and nonlinear two-dimensional analysis of the soil-footing-structure system. The footing-soil system was replaced by a number of beams interconnected with the discretized model of the superstructure, at the column-footing interface nodes. Kawano (1979) and Kimura (2003) applied the finite element method for the nonlinear dynamic analysis of soil-structure interaction systems.

Abul-Azm et.al (1988) presented a solution for the hydrodynamic loading on a stationary, truncated circular cylinder in water of arbitrary uniform depth. Two situations were considered here: the structure may either be completely submerged and resting on sea bed, or partially immersed in the free surface. The method employed to calculate the second order forces and moments due to the second-order velocity potential does not involve the explicit calculation of this potential. Instead, the hydrodynamic loading components due to this potential are found by applying Green's second identity and solving a series of linear radiation problems at the second-order wave frequency. Numerical results are presented for several example structures that illustrate the relative importance of the second-order effects over a range of wave periods. In general, the components that are explicitly due to the second-order potential are found to result in the dominant loading effects at that order.

Mamoon (1990) carried out a frequency domain dynamic analysis of piles and pile groups. A hybrid boundary element formulation is presented to evaluate the impedance and compliance functions of piles and inclined pile groups embedded in a homogeneous soil media. The piles are represented by compressible beam-column elements and the soil as a hysteretic viscoelastic half space. A new Green function for semi-infinite solids is used in the numerical formulation. A realistic lateral load calculation procedure for piles of circular and non circular cross sections is proposed. Particular attention is given to incorporate accurately the inertia effects. Extensive comparisons with the most accurate dynamic solutions available for vertical piles confirm the accuracy of the proposed formulation, at greatly reduced effort and cost. The procedure is general and may easily be extended to analyze any arbitrary pile group configuration that may be subjected to combined vertical, horizontal, and moment loading in homogeneous soil deposits.

Oliveto et.al (1995) checked certain approximations that were made in the engineering literature, in dealing with the soil-structure interaction problems. It was shown how the complex frequencies and modes of vibrations were derived. The analysis of the results showed that the damped frequency of the soil-structure interacting model was always smaller than the natural frequency of the same structure on a fixed base.

Investigations by Yang et al. (1996) demonstrated the efficiency of condensation technique, to formulate the soil-structure interaction problems in a rather straightforward manner. Condensation technique was used for calculating the equivalent seismic forces exerted by the far-field soil on the near-field soil, to analyze the behavior of the structure and the near-field soil.

Surya Rao et al. (1997) measured wave forces and moment on a vertical circular cylinder due to regular and random waves and compared the measured results with the predicted results from theoretical models. Waves were generated by an electronically controlled wave maker. The cylinder was mounted as a cantilever with two force transducers at the top and free to move at the bottom. The force transducers were mounted to give in-line forces and moments followed by transverse forces and moments. Wave forces and moments for regular waves were predicted using the charts based on the stream function theory presented in the Shore Protection Manual. For random waves, the force and moment spectra were predicted from the wave spectra using Borgman's methodology.

Kutanis et al. (2001) suggested an idealized two dimensional plane strain finite element analysis based on a substructure method for seismic soil-structure interaction by using original software developed by the researchers. To investigate the effects of soil-structure-interaction, computations were achieved for different peak accelerations 0.15g, 0.30g and 0.45g and for different soils. The study considered the material non linearity for both soil and the structure within the framework of plasticity theory. They concluded that the fixed base analysis resulted in greater displacements.

Christopher et al. (2002) proposed a time domain formulation for the transient analysis of three dimensional structures resting on a homogeneous elastic half space subjected to either external loads or seismic motions. The formulation was verified by applying it to the analysis of a railway track subjected to moving load. Dynamic soil-structure interaction is a complex phenomenon with significant uncertainties associated with the input motions and analytical models used for both the interaction and the dynamic properties of the materials.

Roger et al. (2002) presented the probabilistic analysis of the seismic soil-structure interaction problem with a procedure which accounts for uncertainty in both the free-field input motion as well as in local site conditions, and structural parameters. The dynamic equations of a porous medium and finite element formulations based on the variational principles were given by Zhang (1995). The foundation soil of a structure was simulated as a two-phase saturated porous medium and a corresponding explicit-implicit algorithm on the substructure level for dynamic analysis of the semi-infinite body was given, to illustrate the influence of pore water in the soil on structural responses.

Dutta et.al (2002) suggested that the effect of soil–structure interaction on dynamic behaviour of structure may conveniently be analyzed using lumped parameter approach. Modeling the system through discretization into a number of elements and assembling the same using the concept of finite element method was proved to be a very useful method, which should be employed for studying the effect of soil-structure interaction.

Jen-Cheng et.al (2003) carried out an analytical model for deflection of laterally loaded Piles They proposed an efficient analytical model based on energy conservation of pile-soil system. They developed method for analyzing deformation data of lateral loaded piles, an analytical model is proposed based on energy conservation of pile-soil system. The proposed analytical model not only provides a direct solution of the pile deflection function, but also has less need of complicated subsurface soil properties. This derived deflection function can provide a realistic description of relation between soil and piles.

Banerjee (2003) analysed inelastic pile soil structure interaction by using a hybrid type of numerical method. Piles and structural elements were modeled as linear finite elements and soil half space was modeled by using boundary elements. It was found that the analysis described was not only capable of predicting the general trend of pile group behavior but it also capable of predicting the general trend of pile settlement, which is of prime importance in the design of pile foundations.

A full three-dimensional dynamic soil–foundation–structure interaction analysis of a famous landmark in Luxor, Egypt, the South Memnon Colossus, was performed by Casciati and Borja (2004) to investigate the response of this historical monument to seismic excitation. The analysis was carried out using the finite element method in time domain for artificially generated earthquakes of different return periods. Finite element models of the foundation and the surrounding soil deposit were constructed and coupled with the statue model to analyze the seismic response of the entire system incorporating dynamic soil-foundation-structure interaction effects. The structure was mounted on a 6m thick flat soil region and supported by the limestone layer, considered as the bedrock. Vertical walls on the sides of the soil region form an elliptical boundary around the structure. The soil region was discretized in horizontal sub layers with three dimensional brick finite elements. This study was conducted for future conservation efforts of this historical landmark, and more specifically for designing possible retrofit measures for the fractured base to prevent potential collapse of the monument from overturning during an earthquake.

Mehrdad et al. (2004) concluded that the nonlinear response of piles was the most important source of potentially nonlinear behavior of offshore platforms due to earthquake excitations. It is often necessary to perform dynamic analysis of offshore platforms that accounts for soil non linearity, discontinuity condition at pile soil interfaces, energy dissipation through soil radiation damping and structural nonlinear behaviors of the piles. In this paper, an attempt was made to introduce a practical BNWF (Beam on Nonlinear Winkler Foundation) model for estimating the lateral response of flexible piles embedded in layered soil deposits subjected to seismic loading. This model was incorporated into a Finite Element program (ANSYS) which was used to compute the response of laterally excited piles. Equivalent linear earthquake site response approach was used for seismic free field ground motion analysis. Quantitative and qualitative findings and conclusions, which are needed for the design of offshore piles, are discussed and addressed

Chandrashekar (2005) analysed statically and dynamically a three dimensional soil- raft-super structure systems. The response of the structure differs due to consideration of foundation and soil system together with the structure in analysis, from the fixed base assumption. Various buildings are modeled and analysed using ANSYS with two approaches – frame with fixed base condition and frames with raft foundation and supporting soil system. Under static analysis, variation of shear force, bending moment in beams, moment and axial force in columns due to soil interaction were studied. Also effect of soil structure interaction on natural period, the primary dynamic characteristic of the structure was studied for various building models.

Yingcai et al (2009) concluded that the seismic response of structures supported on a pile foundation is extremely complex, since the soil behavior is non-linear and liquefaction may occur during earthquakes. The soil-pile-structure interaction becomes extremely important for seismic analysis and design; two commercial software packages were used for considering the nonlinear soil-pile-structure interaction. Stiffness and damping of the pile foundation are generated from a computer program DYNAN and then input into a finite element model by SAP2000 program. The seismic response of a vacuum tower structure supported on pile foundation was examined in a high seismic zone, including response spectrum analysis and time history analysis. The vacuum tower with weight of 5,600 kN and height of 35 m set on a steel frame. To illustrate the effects of soil-pile-structure interaction on the seismic response of structure, three different base conditions were considered, rigid base, i.e. no deformation of the foundation; linear soil-pile system; and nonlinear soil-pile system. The case of pile foundation with liquefaction of sand layer was also discussed. The method and procedure introduced can be applied to the design of tall buildings, bridges, industrial structures and offshore platforms with soil-pile-structure interaction under seismic, blast, sea wave and other dynamic loads.

2.4 EXPERIMENTAL TECHNIQUES

The local pressures on a vertical submerged cylinder were obtained experimentally by Lappo and Kaplun (1975). They used pressure transducers to measure the dynamic part of the wave pressure on the vertical cylinder. Their experimental values were compared with the theory developed by Chakrabarti and a good correlation was observed.

Harihara Raman et al. (1976) presented a nonlinear diffraction theory due to Stokes' second-order wave for computing the wave forces on a vertical cylinder. The results obtained from this theory are compared with the linear theory and also with experimental results. Here, a rigid vertical cylinder is acted upon by a train of regular surface waves. Its motion is explained by a set of equations that depends on velocity potential function. These equations are solved by perturbation method. The first and second order problems for the scattered potential are obtained. They are solved using the method of variation of parameters. The experiments to measure the force were carried out in a 0.9m x 0.9m x 31m long wave flume. Waves were generated by a plunger type wave machine. The model was of hard polyvinyl chloride pipe of 0.2m outer diameter and 0.8m length and was equipped with strain gauges. A wave probe was located away from the model on the upstream side to measure the incident wave height. The time relation of maximum force to wave crest was determined by mounting a second wave probe adjacent to the model. The water depth was 0.5m. Comparisons of the numerical method were made with previous analytical, numerical, and experimental results and good agreement was obtained in all cases. As examples of the program's application, results were presented for the forces on an isolated circular cylinder, neighboring circular cylinders, and a square caisson at arbitrary orientation to the incident wave direction.

Milos Novak et al. (1984) carried out dynamic experiments with a large group of 102 piles with closely spaced in the field. The results of the experiments were compared with theoretical predictions made for vertical and horizontal response on the bases of different approaches accounting for the group effects, i.e., pile-soil-pile interaction. The theoretical approaches employed included static interaction factors, dynamic interaction factors, and complete dynamic analysis, as well as direct static analysis and the equivalent pier. Dynamic analytical techniques were able to predict the main features of dynamic behavior of pile groups for small strain. Static interaction factors provide an approximate estimate of dynamic group stiffness for smaller groups and low frequencies but may underestimate the stiffness of large groups and do not yield any estimate of group geometric damping. The increase in group damping due to interaction suggested by dynamic analysis seems to be confirmed by experiments. Dynamic analysis of pile groups tends to overestimate damping. The inclusion a weak zone around the piles may alleviate this overestimation. The comparison with the experiments indicates that dynamic analytical techniques are able to predict the main features of dynamic behavior of pile groups for small strains and that the interaction effects are very significant. The omission of pile-soil-pile interaction can make the analysis of pile groups quite unrealistic. Further theoretical and experimental research into dynamic behavior of pile groups is needed.

Dynamic centrifuge model tests were performed to explore the effect of hidden localised soft soil stratification on the transmission characteristics of the ground motion shaking by Ghosh et.al (2003) effectively. It was noticed that the structural response is greatly influenced by the layering of soil and a localised hidden soft patch in a dense soil could be dangerous if not detected in routine site tests. It was seen that the shear force attracted by the building base could be 1.5 times more than what it would have been designed for.
Gudehus et al. (2004) conducted experiments with a novel laminar shake box and real seismic records from well-documented sites during strong earthquakes were used to verify the adequacy of the hypoplasticity-based numerical model for the prediction of soil response during strong earthquakes. Shintaro et al. (2004) conducted shaking table tests by means of a large scale laminar box in order to investigate behavior of a soil-pile-

superstructure system in liquefiable ground. A model two- storey structure, supported by a pile group, was set in a saturated sand deposit, and subjected to a sinusoidal base motion of increasing amplitude. The transient state prior to soil liquefaction was shown to be important in the design of a pile because dynamic earth pressure shows peak response in this state. The reduction of the stiffness due to excess pore water generation and strain dependent nonlinear behavior was evaluated.

Effects of inertial and kinematic forces on pile stresses were studied by Kohji et al. (2004) based on large shaking table tests on pile-structure models with a foundation embedded in dry and liquefiable sand deposits. Pseudo-static analysis was conducted to estimate maximum moment distribution in pile incorporating the effects of earth pressures on the embedded foundation and pile. It was assumed that the maximum moment was equal to the sum of the two stresses caused by the inertial and kinematic interaction. A detailed strong motion observation was conducted in buildings and in the surrounding grounds at Building Research Institute, Tsukuba, Japan (Toshihide 2004). This report discussed about the dynamic characteristics of the building and effect of the surface geology by means of analysis of strong motion records. The amplification effects of the surface soil layers, dynamic characteristics of an eight- storey building and soil-structure interaction phenomena were investigated.

Hosseinzadeh et.al (2004) evaluated the soil-structure interaction effects in dynamic response of single and adjacent building structures from experimental testing carried out using a ground model specimen made of relatively soft soil and four steel building models of 5, 10, 15, and 20 storey's. The combined system of soil-foundation-structure models were subjected to horizontal component of two real earthquake records generated by shaking table. It was concluded that the effect of kinematic interaction was negligible in comparison with inertial interaction and in lower buildings; the horizontal and rocking motions of foundations were the main causes of soil-structure interaction.

A series of two-dimensional shake table tests on soil-pile-structure models were performed in National Research Institute for Rural Engineering in Japan by Takahito et al. (2004). A cylindrical laminar box with 1.8 m diameter and 1.5 m height was employed and a model of saturated sand to depth of 1.35 m, a group of four steel piles and a superstructure was subjected to two horizontal components of input acceleration. The computed response by three dimensional nonlinear effective stress analysis methods was in general agreement with that measured in the experiment. The acceleration responses of soil-pile-structure system were strongly affected by the pore pressure built-up. The results from a series of dynamic centrifuge tests were reported by Ghosh et al. (2006). These tests were performed on different types of soil stratifications supporting a rigid containment structure. Test results indicate that accelerations transmitted to the structure's base were dependent on the stiffness degradation in the supporting soil.

2.5 SUMMARY OF LITERATURE REVIEW AND PROBLEM FORMULATION

The design of berthing structures has in the past been largely based on static analysis of the structure under a variety of loading. The cyclic wave loading were applied quasi-statically i.e. the waveform would be considered at which there is maximum base shear or overturning moment (Cronin & Galgoul,1981) This type of analysis was used to design the structure to resist the extreme storm conditions.

Deep foundations consisting of driven or drilled-in piles and piers are routinely employed to transfer axial structural loads through soft soils to stronger bearing strata at greater depth. These soil layers may also be subject to transient or cyclic lateral loads arising from earthquake, wind, wave, blast, impact, or machine loading. The coincidence of major pile-supported structures sited on soft soils in coastal areas of earthquake hazard results in significant demands on these deep foundations. Possible resonance effects between longer period soft soil sites, which may amplify ground motions and large structures, can exacerbate the problem. Liquefaction and/or strain-softening potential in these soft soils can impose additional demands on pile foundation systems. Historically, it has been common seismic design practice to ignore or simplify the influence of pile foundations on the ground motions applied to the structure. This is generally accepted as a conservative design assumption for a spectral analysis approach.

However, in observations of pile performance of berthing structure during earthquakes, two principal facts emerge: pile foundations do affect the ground motions the superstructure experiences, and piles can suffer extreme damage and failure under wave and earthquake loading. The purpose of this research is to examine these two facets of this complex soil-structure interaction problem when wave loading and earthquake loading exists.

Centrifuge and shaking table model tests have been used by many people to augment the field case histories with laboratory data obtained under controlled conditions. The vast majority of centrifuge and shaking table model tests have studied soil-pile seismic response in cohesion less soils with liquefaction potential. But many pile foundations supporting critical structures are sited on soft clays, which have the potential for cyclic strength degradation during seismic loading, Finite element analysis of the integral system of soil, pile and structure as a whole, are carried out to examine the soil-structure interaction effect on the behavior of structures under static and dynamic loading.

In the present study the size of the proposed berthing jetty considered was 350m x 40m, having 6 units of 50m x 40m each with an expansion gap of 20mm. The modeling of the berthing structure for wave response analysis without including soil-structure interaction and including soil-structure interaction was done using software StruCAD .Time- history analysis is done using SAP 2000. The 3D structural models were developed using both the software. Airy's wave theory was adopted in the software for calculation of wave forces. The seismic input for structure was defined in terms of an accelerogram applied at the base of the structure. Classical El Centro earthquake recorded in Imperial valley, California, on 15 October 1940, Kobe earthquake of 1995 and Northridge earthquake of 1994. All the accelerations were presented as a fraction of g (the gravitational acceleration). The total duration of El Centro earthquake was 18sec and that of Kobe and North Ridge earthquake was 25.64sec and 39.98sec respectively. The responses of the structure were compared with soil structure and with no soil structure interaction.

CHAPTER 3

GENERAL REQUIREMENT OF BERTHING STRUCTURES

3.1 GENERAL

The conceptual layout should include the length, width, dredge level, top level, location of beams for handling equipment, spacing of bollards and fenders and configuration of the structure. The configuration includes the arrangement of piles, deck system and dimensions of various structural members. The size of berthing structure depends on the principal dimension of the large vessel to be handled, area required for transit shed, number of rail tracks, truck lines, crane rail and width of apron required to accommodate mooring facilities and utility service.

3.1.1 Length of berthing structure

Vessel overall length is designated as LOA, which governs the length of berth. The minimum length should be sufficient for mooring the largest ship expected to arrive. The length of pier or wharf for single vessel is 50 to 60 m larger than the overall length of the design ship. Where more than one vessel is to be accommodated, the length of berth should not be less than the overall length of design vessel plus 10% more subject to a minimum of 15 m. It may be increased to 20%, if the berth is exposed to strong wind and tide conditions (IS 4651-Part V, 1980).

3.1.2 Width of berthing structure

The width of a quay or a wharf depends upon the commodity that is to be handled. It is advisable to reserve 20 m for crane equipment in the berthing quay front. About 40m inside this area up to the limit of storage area of transit shed is reserved for traffic in general cargo or container berth. Bulk terminals for mass transport such as coal or oil differ greatly from each other depending upon the character of the loading and unloading equipment. For the container berth, the apron width is governed by the distance between the legs of the gantry crane which is typically 20 to 30 m. The minimum width of apron is given in (IS 4651-Part V, 1980).

3.1.3 Dredged area

The length of dredged area of the berth alongside of pier or wharf should be minimum of 1.25 times LOA where tug assistance is provided and up to 1.5 times LOA where tug assistance is not provided. The width of dredge area should be at least 1.25 times the beam of the vessel. The dredge depth should be greater than or the sum total of actual draft, vertical ship motions, keel clearance, dredging tolerance and depth sounding accuracy. As a thumb rule, the keel clearance is usually 0.3 to 0.5 m, dredging tolerance is 0.1 to 0.5 m and sounding accuracy is normally about 0.2 m. The minimum dredge depth should not be less than loaded draft of the largest vessel plus an allowance of 0.6 to 0.75 m. When the harbour bottom is hard, the allowance should be increased to 1 m (IS 4651-Part V, 1980).

3.1.4 Deck level of berthing structure

The minimum deck level of berthing structure should correspond to a combination of high water level and wave action.

For operational reasons, a higher deck elevation of around 2m above the mean sea level is preferred. The deck elevation should normally be at or above highest high water spring plus half of an incident wave at the berth location plus a clearance of 1 m (IS 4651-Part V, 1980).

3.1.5 Fenders and bollards

The important function of fender is to absorb the kinetic energy from the impact of berthing vessel and also to avoid damage to the vessel and to the structure.

Functionally, fenders shall accomplish the following purpose:

1. Absorb the berthing energy or impact of vessel and transmit a desired or calculated thrust to the structure.
2. Hold the vessel off the face of the structure and avoid rubbing of the vessel against structure and consequent damages to the vessel and the structure.
3. Impart the thrust from berthing loads to the structure at predetermined or design points.

The fenders with low reaction per absorbed unit of energy can be preferred. The berthing force depends on the load deflection characteristic of the fender system. A factor of safety of 1.4 should be applied over the ultimate energy absorption capacity of fenders (IS 4561-Part IV, 1989). The absorption of kinetic energy depends on type of fender and its material property. The fenders are classified as

i) Standard pile fenders
ii) Rubber fenders
iii) Pneumatic fenders
iv) Gravity type fenders

3.1.6 Expansion joint

Expansion joint shall be provided depending upon type of the structure, sub-soil and atmospheric conditions in order to accommodate moments arising from shrinkage, temperature changes and yielding of foundation.

As a general rule, a length of 39 m between the expansions joint is recommended for structures such as solid quay walls or walls resting on piles. An expansion joint gap of 20 mm is generally provided. A spacing of 60 m for expansion joint is provided for better stiffness (IS 4651-part IV, 1989).

3.2 FORCES ON BERTHING STRUCTURE

The berthing structure is subjected to dead load, live load, berthing force, mooring force, earthquake load and other environmental loading due to wind, waves and currents.

3.2.1 Live Loads

Vertical live loads: Surcharges due to stored and stacked material such as general cargo, bulk cargo, containers handling equipment and construction plant, constitute vertical live loads.

i) Crane Loads: Concentrated wheel loads from crane wheels and other specialized mechanical handling equipment should be considered.

An impact of 25 percent shall be added to wheel loads in the normal design of deck and stringers, 15 percent where two or more cranes act together and 15 percent in the design of pile caps and secondary framing members.

ii) Railway Loads: Concentrated wheel loads due to locomotive wheels and wagon wheels in accordance with the specification of Indian Railways for the type of gauge and service at the locality in question. For impact due to truck and Railways one third of the impact factors specified in the relevant codes may be adopted.

iii) Truck loading and uniform loading: The berth shall be generally designed for the truck loading and uniform loading as given in Table 3.1 (IS 4651: Part III, 1974).

Table 3.1 Truck loading and Uniform loading for different type of berths.

Type of Berth	**Truck Loading (IRC Class)**	**Uniform Vertical live load (t/m^2)**
Passenger Berth	B	1.0
Bulk unloading and loading berth	A	1.0 to 1.5
Container berth	A or AA or 70 R	3.0 to 5.0
Cargo berth	A or AA or 70 R	2.5 to 3.5
Heavy cargo berth	A or AA or 70 R	5 or more
Small boat berth	B	0.5
Fishing berth	B	1.0

iv) Special Loads*:* Special loads like pipelining loads or conveyor loads or exceptional loads such as surcharges due to ore stacks, transferring towers, heavy machinery or any other type of lifts should be individually considered.

3.2.2 Berthing Load

When an approaching vessel strikes a berth, horizontal forces acts on the berth. The magnitude of this force depends on the kinetic energy that can be absorbed by the fendering system. The reaction forces for which the berth is to be designed can be obtained from the deflection reaction diagrams of the fendering system. As per the IS 4651 part III: 1974 the kinetic energy (E), imparted to a fendering system, by a vessel moving with velocity V is given by

$$E = \frac{W_D \times V^2}{2g} \times C_m \times Ce \times Cs \text{ --- } (3.1)$$

Where, E = berthing energy in t-m,

W_D = Displacement tonnage in t.

V = Berthing velocity in m/s,

Cm = mass coefficient,

Ce = Eccentricity coefficient,

Cs= Softness coefficient and

g = Acceleration due to gravity in m/s^2

Mass Coefficient (Cm) is calculated using following equation,

$$C_m = 1 + \frac{2D}{B} \text{ -- } (3.2)$$

Where D = Draught of vessel in m, and

B = beam of vessel in m.

Alternative to the above in case of a vessel which has a length much greater than its beam or draught or generally for vessels with displacement tonnage(W_D) greater than 20,000, the addition weight may be approximated to the weight of cylindrical column of water of height equal to the length of vessel and diameter equal to the draught of vessel, then,

$$C_m = 1 + \frac{\pi/4\, D^2 LW}{W_D} \text{ --- } (3.3)$$

Where, D = draught of the vessel in m,

L = length of the vessel in m,

W = unit weight of se a water (1.025 t/m3) and

W_D = displacement tonnage of the vessel in t.

Eccentricity coefficient (Ce): A vessel generally approaches a berth at an angle, denoted by θ and touches it at a point either near to the bow or stern of the vessel. In such eccentric cases the vessel imparts a rotational force at the time of contact, and the kinetic energy of the vessel is partially expanded in its rotational motion. The eccentricity coefficient (Ce) may then derive as follows:

$$C_e = \frac{1+\left(\frac{1}{r}\right)^2 Sin^2\theta}{1+\left(\frac{1}{r}\right)^2} \quad \text{---} (3.4)$$

Where, l = Distance from the center of gravity of the vessel to the point of contact projected along the water line of the berth in m, and

r = Radius of gyration on the plane of the vessel from its center of gravity in m.

The approach angle θ unless otherwise known with accuracy should be taken as 10°. For smaller vessels approaching wharf structure, the approach angle should be taken as 20°. The rotational radius of a vessel may be approximated to L/4 and in normal case the point of contact of the berthing vessel with the structure is at a point about L/4 from the bow or stern of the vessel, which is known as quarter point contact. If the approach angle θ is nearly 0° and r = 0.25L, then Ce=0.5.

Softness Coefficient (Cs)*:* This coefficient indicates the relation between the rigidity of vessel and that of fender, and also the relation between the energy absorbed by the vessel and the fender. Since the ship is relatively rigid compared with the fendering system, a value of 0.9 is generally used for this factor.

Quinn (1961) has suggested a suitable formula for calculating berthing energy assuming the 50% of the total energy of the berthing vessel to be absorbed by the fenders.

$$E = \frac{1}{2}\left[\frac{1}{2}mv^2\right] \quad \text{---} (3.5)$$

Where, m is added mass and mass of vessel and v is the berthing velocity.

3.2.3 Mooring Loads

The mooring loads are the lateral loads caused by the mooring lines when they pull the ship into or along the deck or hold it against the forces of wind or current. The maximum mooring loads are due to wind on the exposed area on the broad side of the ship in light condition:

$$F = C_W A_W P \text{ -- } (3.6)$$

Where, F = force due to wind in kgf,

Cw = shape factor (1.3 to 1.6),

Aw = windage area in m²,

P = wind pressure in kg/m² to be taken in accordance with IS: 875-1964.

The windage area (Aw) can be estimated as follows,

$$A_W = 1.175 L_V \left(D_M - D_L\right) \text{ -- } (3.7)$$

Where, Lv = Length between perpendicular in m,

D_M = Moulded depth in m,

D_L = Average light draft in m.

3.2.4 Force due to wind

The maximum mooring loads are due to the wind forces on exposed area on the broad side of the ship in light condition. Wind forces and windage area are calculated as per equation 3.6 and 3.7. When the ships are berthed on both side of jetty, the total wind forces acting on the jetty should be increased by 50 percent to allow for wind against the second ship. The design wind pressure at height above mean ground level depends on wind speed and is calculated using the following equation as per the IS 875-Part III, 1989.

$$P_Z = 0.6 (V_Z)^2 \text{ ———————————————— } \mathit{(3.8)}$$

-------------- *(3.8)*

Where, Pz = design wind pressure in N/m² at height Z, Vz = design wind velocity in m/s at height Z.

3.2.5 Wave forces on structural elements

The various wave force regimes resulting from the interaction between the structure and the waves are given below:

• For D/L > 1 Condition approximate to pure Reflection.

• For D/L > 0.2, Diffraction is increasingly important.

• For D/L < 0.2, Diffraction is negligible.

• For $D/W_0 > 0.2$ Inertia is dominant.

• For $D/W_0 < 0.2$, Drag becomes more important.

Where D= diameter or characteristic dimension,

L= wave length

W_o=orbit width parameter equal to wavelength in deep Water.(Brebbia et.al 1979)

Based on the type and size of the members wave forces on structure (marine/offshore) are calculated in three different ways,

• Morison equation

• Froude-Krylov theory

• Diffraction theory

In the Morison equation (Morison et al, 1950) the force composed of inertia and drag forces linearly added together.

[illegible] -------------------------------- (3.9)

ρ = density of the fluid. (1.025 kN/m³),

D = diameter of the pile,

u = horizontal water particle velocity at the axis of the pile (calculated as if the pile were not there), du/dt = total horizontal water particle acceleration at the axis of the pile (calculated as if the pile where not there),

C_D= hydrodynamic force coefficient, the "drag" coefficient,

C_M = hydrodynamic force coefficient, the "inertia" or "mass" coefficient. In a uniformly acceleration flow, the inertia coefficient may be shown to be equal to 2.

The components involve an inertia (or mass) coefficient, which must be determined experimentally. The Morison equation is applicable when drag force is significant. This is usually the case when a structure is small compared to wavelength.

When the drag force is small and the inertia force is predominant, and the structure is relatively small, the Froude-Krylov theory is used. It utilizes the incident wave pressure and the pressure area method on the surface of the structure to compute the structure forces. The advantage of this method is that for certain symmetric objects the force may be obtained in a closed form and the force coefficients are generally easy to determine. When the size of structure is comparable to wavelength, the presence of the structure is expected to alter the wave field in the vicinity of the structure. In this case the diffraction of the wave from the surface of the structure is taken into account for evaluation of wave forces. It is generally known as diffraction theory. For this the Morison theory is not applicable.

In order to determine the applicability of the above three methods, simpler dimension analysis is performed first. The force, f due to wave on the structure is defined by characteristic dimension, D (e.g. the diameter of vertical pile) as following function.

$$f = \psi(t, T, D, L, u_O, \rho, v) \quad \text{--} \ (3.10)$$

Where, t=time, T= wave period,

L=wave length,

u_0=maximum horizontal water particle velocity,

ρ=mass density,

υ =kinematic viscosity.

Water particle accelerations are known from velocity. The dimensionless forces can be expressed as a function of four non-dimensional quantities.

$$\frac{f}{\rho u_0^2 D} = \varphi\left(\frac{t}{T}, \frac{u_0 T}{D}, \frac{u_0 D}{v}, \frac{\pi D}{L}\right) \quad \text{-- (3.11)}$$

Where t/T = dimensionless time,

u_0T/D = Keulegan-Carpenter parameter (KC),

U_oD/υ = Reynolds number, and

πD/L = diffraction-n parameter.

The KC number is a measure of the importance of the diffraction effect. When the KC number is large, the diffraction is small & vice versa (Chakravarti 1987). Thus large diffraction effect necessarily means small drag effect and, inversely when drag effect is large, the diffraction is negligible.

3.3 COMPUTATION OF WAVE FORCE ON A VERTICAL PILE

Morison, et al.(1950) proposed that the force exerted by unbroken surface waves on a vertical pile, which extend from the bottom through the free surface is composed by two components, inertia and drag. Definition sketch is as shown in (Fig.3.1)

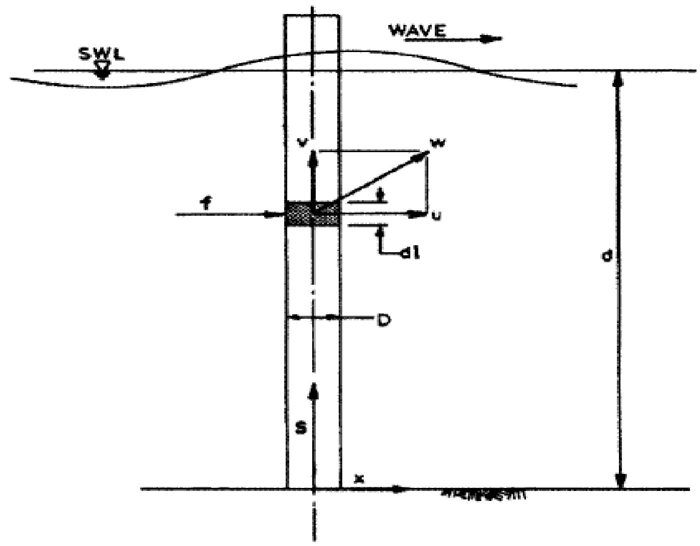


Fig.3.1 Definition sketch for wave forces on small diameter piles

According to Morison et al (1950), the horizontal force per unit length of a vertical pile $f = f_i + f_D$ --- (3.11)

Where, f_i = inertial force per unit length of the pile,

f_D = drag force per unit length of the pile,

3.4 CURRENT FORCE

Force due to current will be applied to the area of vessel below the water line when fully loaded. It is approximately equal to $wv^2/2g$ per m^2 area, where, v is velocity in m/s and w unit weight of water in t/m^3. The ship is generally berthed parallel to the current; the likely force should be calculated by any recognized method and taken into account.

3.5 SEISMIC FORCE

The horizontal force caused by the earthquake is called seismic force. Earthquake causes random motion of ground causing the structure to vibrate. The characteristics of the seismic vibrations expected at any location depend upon the magnitude of earthquake, the depth of focus, distance from the epicenter and the strata on which the structure stands. The random earthquake ground motions, which cause the structure to vibrate, can be resolved in any three mutually perpendicular directions. The predominant direction of vibration is horizontal. The response of the structure to the ground vibration is a function of the nature of foundation soil, material, form, size and mode of construction of the structure, and the duration and intensity of ground motion. Currently there is no uniform seismic code for the design and construction of berthing structures. Existing building code and seismic design criteria are commonly used. The horizontal forces caused by the ground motion are calculated by using IS 1893 , 2000

3.6 SUMMARY OF DESIGN LOADS

So far general requirement for the berthing structure was studied and load calculation was done, based on the data available from New Mangalore Port Trust. The port of New Mangalore is located on the Latitude 12˚ 55’ 2” N and longitude 740˚ 46’ 17.6” E on the west coast of India, midway between Cochin and Mormugoa. The berth is L shaped and its main portion having a size of 350 m x 40 m and return portion of the birth is having a size of 60 m x 40 m.Total estimated cost of project was 42 crore.

This multipurpose berth comprising of RCC bored piles of 1.0 m diameter, 284 numbers and 1.2 m diameter, 32 numbers. The deck was made of precast beams and slabs connected by in-situ topping.It also consist of 1100mm thick diaphragm wall of length

390m with prestressed inclined rock anchors. The climate at Mangalore is tropical with high humidity and a maximum temperature of 36˚C. The average annual rainfall is 3330mm.The tidal particulars at New Mangalore Port are as follows

Higher high water springs	: +1.68m
Mean higher high water	: +1.48m
Mean lower high water	: +1.26m
Mean sea level	: +0.95m
Mean lower low water	: +0.26m
Chart datum	: 0.00m

The deck elevation should normally be at or above highest high water spring plus half of an incident wave at the berth location plus a clearance of 1m.The deck level of the jetty was fixed at +5m above the datum. An expansion joint gap of 20 mm was provided. A spacing of 60 m for expansion joint was provided for better stiffness.

3.6.1 Design Dead load

Dead loads include all the fixed items in the structures. It includes all primary structural members, secondary structural items such as Wearing coat, Deck slab, Pile cap, Transverse beam, Longitudinal beam, Fender beam, Pile etc.

3.6.2 Design Live load

Live loads are defined as movable loads and will be temporary in nature. Live loads will be applied only on areas designated for the purpose of storage either temporary or long term. Further, the areas designed for lay down during transfer of materials from boat shall also be considered as live loads. Other live loads include open areas such as walkways, access platforms etc. These loads shall be applied in accordance with the requirement from the operator of the platform. This load varies in nature from owner to owner. Live load of 50 kN/m^2 was considered for the present study

3.6.3 Design Berthing Force

Berthing Forces are determined in accordance with IS: 4651(Part-III) - 1989. Following shows the calculation of Berthing Energy for governing vessel size and its parameters.

Type of ship =Bulk carriers.

Design Vessel Size = 200000 DWT

Overall Length, L = 350 m

Draft, D = 18.2 m

Unit Weight of Sea Water, $\gamma_{seawater} =$ 1.03 kN/m^3

Angle of Approach (From Cl.5.2.1.3.b of IS: 4651 (III)) = 10 degrees

Velocity of Approach (Sheltered & Difficult), (Table 2 of IS: 4651 (III))V = 0.1 m/s

Factor of Safety (Cl.9.3.e of IS: 4651 (IV)) = 1.40

DT/DWT Factor (From Fentak Manual) = 1.24

Displacement Tonnage (DT), W_D = 248000 t

Mass Coefficient,(Cl.5.2.1.2 of IS: 4651 (III)) $C_m = 1 + \pi D^2 L \gamma_{seawater} / 4W_D = 1.38$

Eccentricity Coefficient, (Table 3 of IS: 4651 (III)) $C_e = 0.51$

Softness Coefficient, (Cl.5.2.1.4 of IS: 4651 (III)) C_s=0.90

Berthing Energy, (Cl.5.2.1 of IS: 4651 (III)) $E = W_D V^2 C_m C_e C_s / 2g$ = 79.96 t-m

Design Fender Energy =1.4*79.96 = 111.94 t-m

3.6.4 Mooring Forces

For 2, 00,000 DWT vessels, as per IS 4651 Part III (1974), the mooring force on each end is 2000 kN. 4 Nos. of 600 kN capacity bollards are to be provided along the berthing side (front) of the jetty.

3.6.5 Wind load

The wind in the monsoon months (June-September) are predominantly from SW to W, with a maximum intensity of 20 to 60 m/s. The winds in the remaining months of the year are predominantly from NW with a maximum intensity of 20 to 60 m/s. There is minor seasonal variation of the wind speed. Around Mangalore, 92% of all winds have speeds less than 19m/s, and the average wind speed considered was 35 m/s.

3.6.6 Wave force

The predominant direction of waves in the vicinity of New Mangalore Port during monsoon months (June-September) is West and South West, whereas the predominant direction during the fair months is North –West. High waves are experienced only during the monsoon months. From observations carried out in 1974 and 1975 as mentioned above, the following information gives the range of variation of wave heights and periods:

Table 3.1 Wave data collected in NMPT during the year 1974-1975 by the CWPRS

Month	Range of wave heights Hs (m)	Range of wave periods Ts (sec)	H_{max} (m)	Corresponding period (sec)
July 1974	1.107-3.21	6.5-13.2	5.0	12
Aug. 1974	1.04-2.74	6.3-10.8	5.2	9
July 1975	0.86-1.96	6.3-11.9	3.2	10
Aug.1975	0.92-3.33	6.3-13.4	5.5	11
Sept.1975	0.34-1.19	4.6-11.0	3.1	9
Oct.1975	0.34-1.19	4.6-11.0	2.3	8
Nov.1975	0.40-1.06	5.6-8.2	1.8	7

The analysis of 15 minutes record of 1974 indicated that the maximum significant wave height was 3.21m and the largest single wave in that wave train was 4.70m. However, in another wave train, the largest single wave height was 5.20m, in which the corresponding significant wave height was 2.55m. The analysis of the 15 minutes record 1975 indicated that the maximum significant wave height was 3.30m and the largest single wave height in that wave train was 5.50m. For the present study ,maximum wave height of 5.5m for period 11sec was considered.

3.6.7 Current

The current along the coast during the SW monsoon (from June to September) is generally towards the South (from 160^0 to 200^0) with strength of 0.22 to 0.80 knots. During the NE monsoon (from November to January), the current is generally towards the North (from 0^0 to 40^0 and 320^0 to 360^0 bearing) with a velocity of 0.22 to 0.60 knots. In the port entrance channel protected by breakwater, the current direction lags 6^0 to 8^0 behind the coastal current.

The current in the lagoon area further lags behind the approach channel on an average by 10^0 to 15^0. The magnitude of the current outside the lagoon area during the monsoon as experienced by pilots is about 1 to 1.5 knots. For analysis current force considered is 1.5 knots. The modeling and analysis was done using StruCAD and SAP 2000 software.

CHAPTER 4

VALIDATION OF EXPERIMENTAL INVESTIGATION

4.1 CANTILEVER BEAM

An experiment on simple cantilever was done to find the natural frequency and the mode shapes. The experiment was conducted using a steel scale of length 250 mm, width 25 mm and thickness of 1mm. One end of the cantilever was fixed on a shaker and the other end was free is as shown in Plate 4.1

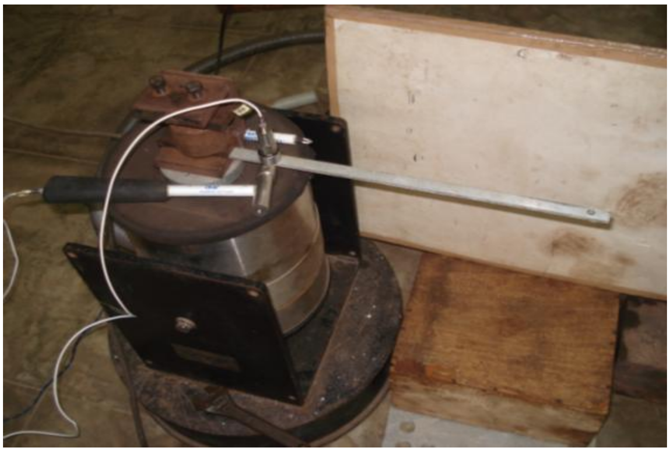


Plate 4.1 Cantilever fixed on shaker

Accelerometer was placed on cantilever which was connected to USB data acquiring unit. The specification of USB data acquiring unit is shown in the table 4.1. The steel scale was made to vibrate by hand flick and by force hammer, then the vibrations were captured by accelerometer. Lab VIEW was used as a GUI software to read data from data acquisition unit and to present the data in the form of graphs.Fig 4.1 shows a typical plot of Time Vs Amplitude graph and plot of frequency Vs amplitude graph is as shown in Fig 4.2.

Table 4.1 Specifications of USB data acquiring unit

USB type data acquisition unit	Specifications
Maximum sampling rate	51.2 kS/s per-channel
Input	±5 V
Resolution	24-bit
Dynamic range	102 dB

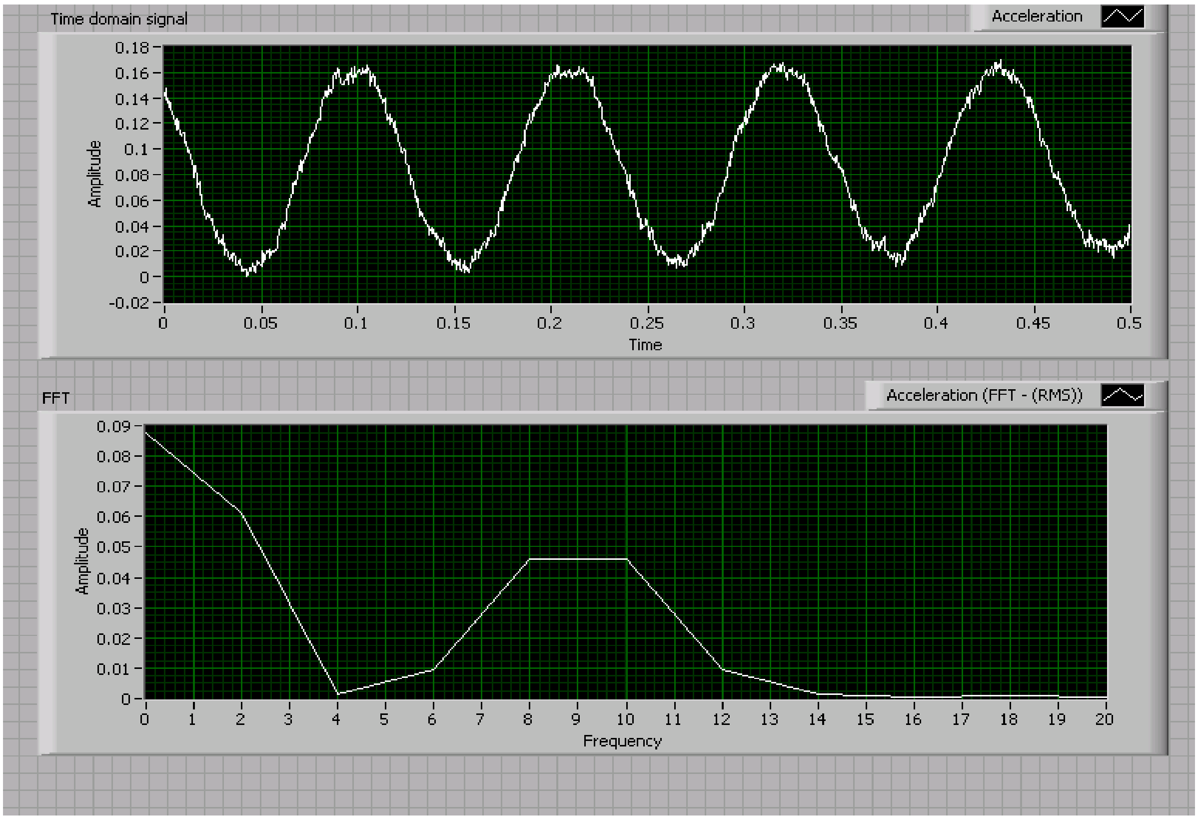


Fig 4.1 Amplitude v/s Time and Fig 4.2 Amplitude v/s Frequency graphs

An electromagnetic shaker was also used to induce vibrations on cantilever as shown in Plate .4.3. The steel scale began to vibrate because of the discontinuous magnetic field produced by the electromagnetic shaker.

Plate 4.2 Magnetic shaker below the cantilever

4.2 TEST ON SINGLE CYLINDER

The study was done experimentally to measure wave forces and moments on a vertical circular cylinder due to regular waves in a wave flume and the predicted results were compared with theoretical solution. In Marine structures laboratory of Department of Applied Mechanics and Hydraulics of National Institute of Technology, Karnataka, Surathkal, India, with existing facilities of two dimensional wave flume, regular waves of heights ranging from 0.08 m to 0.24 m and period ranging from 0.8 sec to 4.0 sec can be produced. The cylinder was mounted as a cantilever with two force transducers at the top and free to move at the bottom. The force transducers were mounted to give in - line forces. Wave forces and moments were predicted using the linear wave theory. The experiments were conducted in a glass walled wave flume 1m wide, 1m deep and 50 m long All the experiments were conducted at a still water depth of 0.5 m. The waves were regular and nonbreaking.

The waves were generated by a electronically controlled wave maker consisting of a wave paddle hinged at the bottom and installed at the upstream end of the flume and controlled by electronic inverter.

The diameter (D) of cylinder is 42 mm and length (l) is 71 mm. Two force transducers (piezoelectric type, Kistler) are used and placed inline on either side of the cylinder as shown in Plate 4.3. National Instrument's Data acquiring unit was used. Computer connectivity is done using Lab View software. The sample was done at a rate of 1000 samples/sec.

Wave height was measured using wave probe. Wave probe used was of capacitance type and it had been interfaced with a computer to find wave height and wave period. Twelve experiments were conducted with regular waves with wave heights as shown in Table 2. Each run was repeated for 25 sets and average value was taken. The calibration of the test pile was done by applying the known loads horizontally in the longitudinal direction. The known loads were applied by means of static load cells. The cylinder was hung from the plate which acted like a cantilever bending about vertical axis. Perfect fixity was ensured at the top of the cylinder.

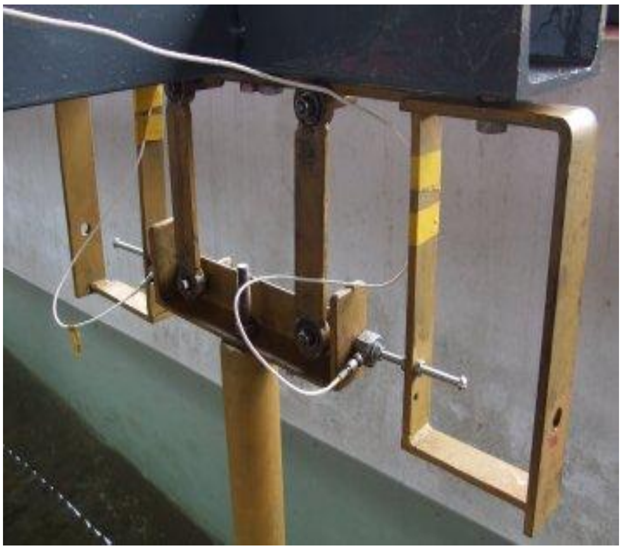


Plate 4.3 Experimental setup

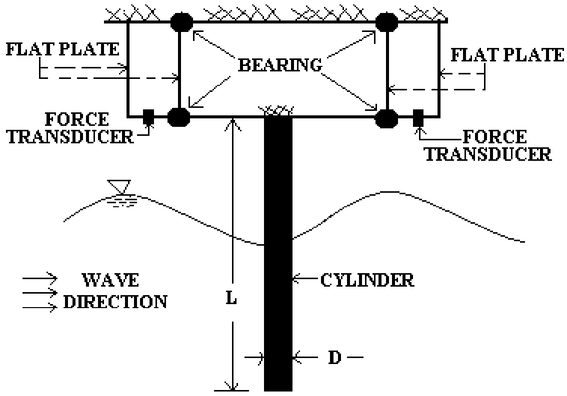


Fig 4.3 Schematic diagram of experimental setup

Table 4.2 Experimental and predicted in-line force and moment

Expt. No	Measured wave height at SWL cm	Wave period T sec	Max.in-line force		Max. moment in–line direction	
			Experiment N	Predicted N	Experiment (Nm)	Predicted (Nm)
1	6.0	4.0	0.515	0.512	0.110	0.132
2	7.0	3.5	0.702	0.698	0.122	0.141
3	8.0	3.5	0.733	0.721	0.156	0.167
4	10.0	3.0	0.742	0.745	0.180	0.199
5	10.0	3.0	0.740	0.765	0.192	0.204
6	10.0	2.5	0.774	0.798	0.210	0.213
7	10.5	2.5	0.831	0.842	0.220	0.235
8	12.0	2.5	1.176	1.156	0.289	0.295
9	14.5	2.5	1.499	1.550	0.387	0.395
10	15.0	2.4	1.631	1.624	0.413	0.426
11	15.0	2.2	1.733	1.758	0.450	0.548
12	19.5	2.2	2.680	2.702	0.679	0.699

4.3 EXPERIMENTAL SETUP FOR SOIL STRUCTURE INTERACTION

The study of vertical cylinder such as piles is very important for the design of marine structures since the piles used for these structures are vertical and circular in shape having greater length compared to its diameter. The piles are made up of either concrete or steel. The experiments were done by using hollow PVC cylinders of 42mm external diameter.

The tank of length 610 mm, width 410 mm and height 315 mm which rested on a platform of length 665 mm and width 460 mm having roller support at the bottom as shown in Plate 4.4. This wave tank was connected to the flange fixed to the shaft of a 28 ampere DC electric motor by a crank of length 700 mm. An autotransformer was used to vary the voltage of the motor which changed the frequency in which the tank vibrated. The minimum limit of the shaft was 25 RPM. As the flange rotated with the shaft, the wave tank moves horizontally to and fro. Two holes at 5 mm and 10 mm distance from centre of the shaft are provided on the shaft.flange. When the crank was connected to 5 mm and 10 mm distance away from centre of the shaft, the stroke produced for the wave tank was 10 mm and 20 mm respectively.

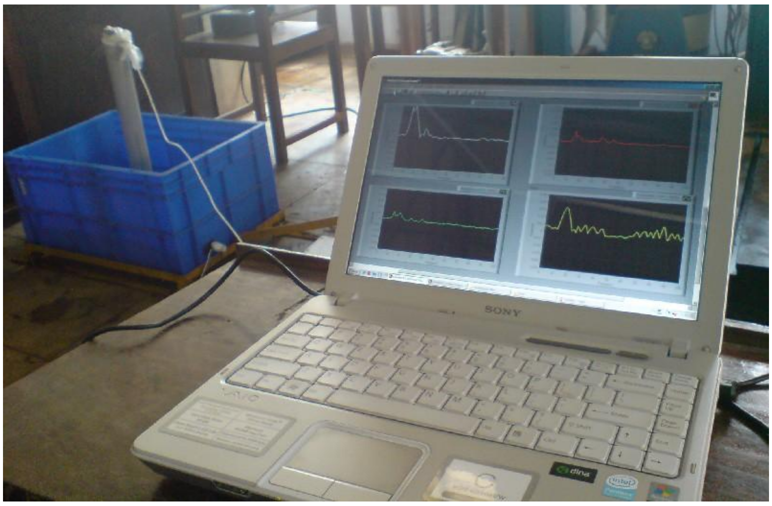


Plate 4.4 Experimental set up for the vertical cylinder

The hollow PVC cylinder of 42 mm diameter and of height 600 mm was fixed at the centre of the tank. A plastic washer was fixed beneath the cylinder to ensure that no water could go inside the hollow cylinder. One tri-axial accelerometer was mounted on the top of the cylinder to measure the movement of cylinder in X, Y and Z directions. The tri-axial accelerometer has sensitivity 198mV/g in X, Y and Z directions. Also an uni-axial accelerometer was connected to the bottom of the wave tank to measure the movement of tank, i.e. to measure the input frequency of the motor. The motor was rotated with different speeds with ranges of 25-60 RPM.

Natural frequency was obtained by giving free vibration to the structure by hand flick method. Then the tank was filled with water at different levels of 100mm, 150mm and 200mm heights. Forced vibrations were given by rotating the motor for 2,4,6,8 and 10 seconds. Both forced and free vibrations were taken for all these heights. Readings were taken by filling the tank with marine soil of height 100mm with water up to 200mm height.

4.4 SIMPLIFIED BERTHING STRUCTURE MODEL

The experiment was also done on a simplified berthing structure model which has four steel vertical members and an aluminum plate fixed on the top which resembles prototype berthing structure as shown in Plate 4.5. Height of the structure was 596mm and each column was 10mm diameter. The thickness of plate was 10mm and its size was 226mm x 226mm. The structure was fixed to the bottom of tank by fixing another plate of same size. Experiments were repeated as in the case of vertical cylinders.

Plate 4.5 Experimental set up for simplified berthing structure model

4.5 SENSORS USED

4.5.1 Accelerometer

A piezoelectric accelerometer is an electro mechanical transducer, which at its output terminal, gives an electrical signal proportional to the acceleration to which it is subjected, due to the fact that the deformation of a piezoelectric sensing element used therein by an applied force will produce an electrical signal proportional to that force. There are many different types of transducers which can be used to measure vibration. In practice, velocity and displacement can be easily obtained from acceleration by use of an integrating circuit. It turns out that the piezoelectric accelerometer is nearly always the best transducer as it is self generating,has large dynamic range, wide frequency range, orientation to measure acceleration along one axis, very reliable with long term stability.It has no moving parts and it is robust, compact and relatively cheap. Uni-axial accelerometer and tri-axial accelerometer were used for the experiments. Uni-axial accelerometer measures acceleration in only one direction and tri-axial accelerometer measures the acceleration in X, Y and Z directions. The sensitivity of uni-axial accelerometer used was 10mV/g. The tri-axial accelerometer which was used for the experiments on vertical cylinder and simplified offshore structure, has sensitivity 198mV/g in X, Y and Z directions.

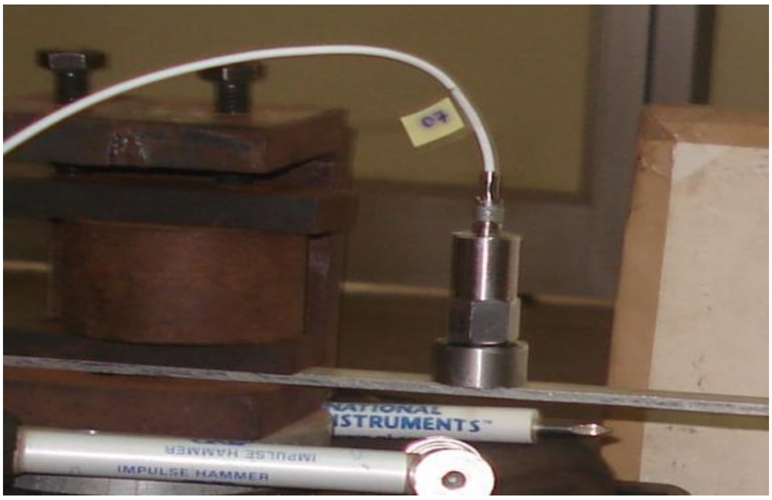


Plate 4.6 Uni-axial accelerometer connected to setup

4.5.2 Strain Gauge

In order to measure strain on the material the strain gauges were used as shown in Plate 4.7. Strain gauges were connected to an electric circuit that was capable of measuring the minute changes in resistance corresponding to strain as the material in the strain gauges stretches with the change in material under study when under strain. They operate on the principle that when the foil is subjected to stress, the resistance of the foil changes in a defined way. The majority of strain gauges are foil types, available in a wide choice of shapes and sizes to suit a variety of applications. They consist of a pattern of resistive foil which is mounted on a backing material.

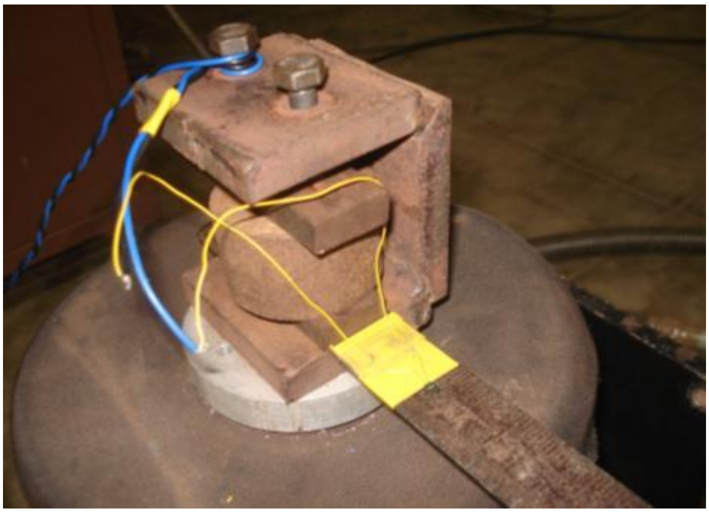


Plate 4.7 Strain gauge connected to experimental setup

4.6 RESULTS & DISCUSSION

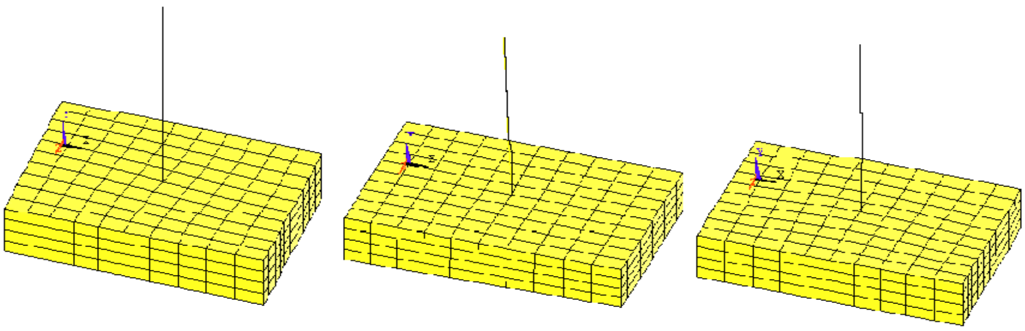
The variation in natural period of different structures was studied with experiments. A small experiment on steel scale was done to get natural frequency of a cantilever beam. The natural frequency obtained was 9.1Hz by using accelerometer and the same result was got by using strain gauge also.

The natural period of simplified berthing structure model was found out using experimental setup and was compared with results obtained by available FEM software ANSYS. Natural frequencies of different models were presented in Table 4.3.

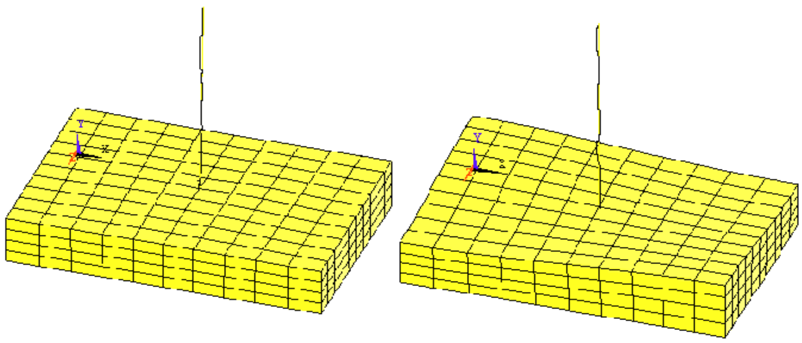
Table 4.3 Natural frequency of different model at different experimental conditions

Types of structures with experimental conditions	Natural frequency ω obtained from experiment (Hz)
32mm diameter hollow cylinder	0.312
hollow cylinder + 100mm soil	0.40
hollow cylinder +100mm water	0.41
hollow cylinder + 200mm water	2.27
hollow cylinder + 100mm soil and water	6.67
soil filled cylinder	1
soil filled cylinder +100mm soil	1.25
soil filled cylinder +100mm water	1.43
soil filled cylinder +200mm water	3.6
soil filled cylinder +100mm soil and water	8.6
berthing structure without water	0.434
berthing structure + 100mm soil	1.25
berthing structure + 100mm water	0.769
berthing structure + 200mm water	0.91
berthing structure + 100mm soil and water	1.67

It was observed that the presence of water and soil tends to increase the natural frequency. With the water natural frequency of the hollow cylinder was increased by 32%. For soil filled cylinder it was increased by 25%. When compared to the natural frequencies of hollow and solid vertical member it was observed that natural frequency tends to increase. The results obtained were compared with ANSYS model and results were tabulated in table 4.3. It was observed that the experimental results are closer to the ANSYS results. The different mode shapes of the 42mm cylinder and the simplified berthing structure in soil is as shown in Fig.4.4 and 4.5.

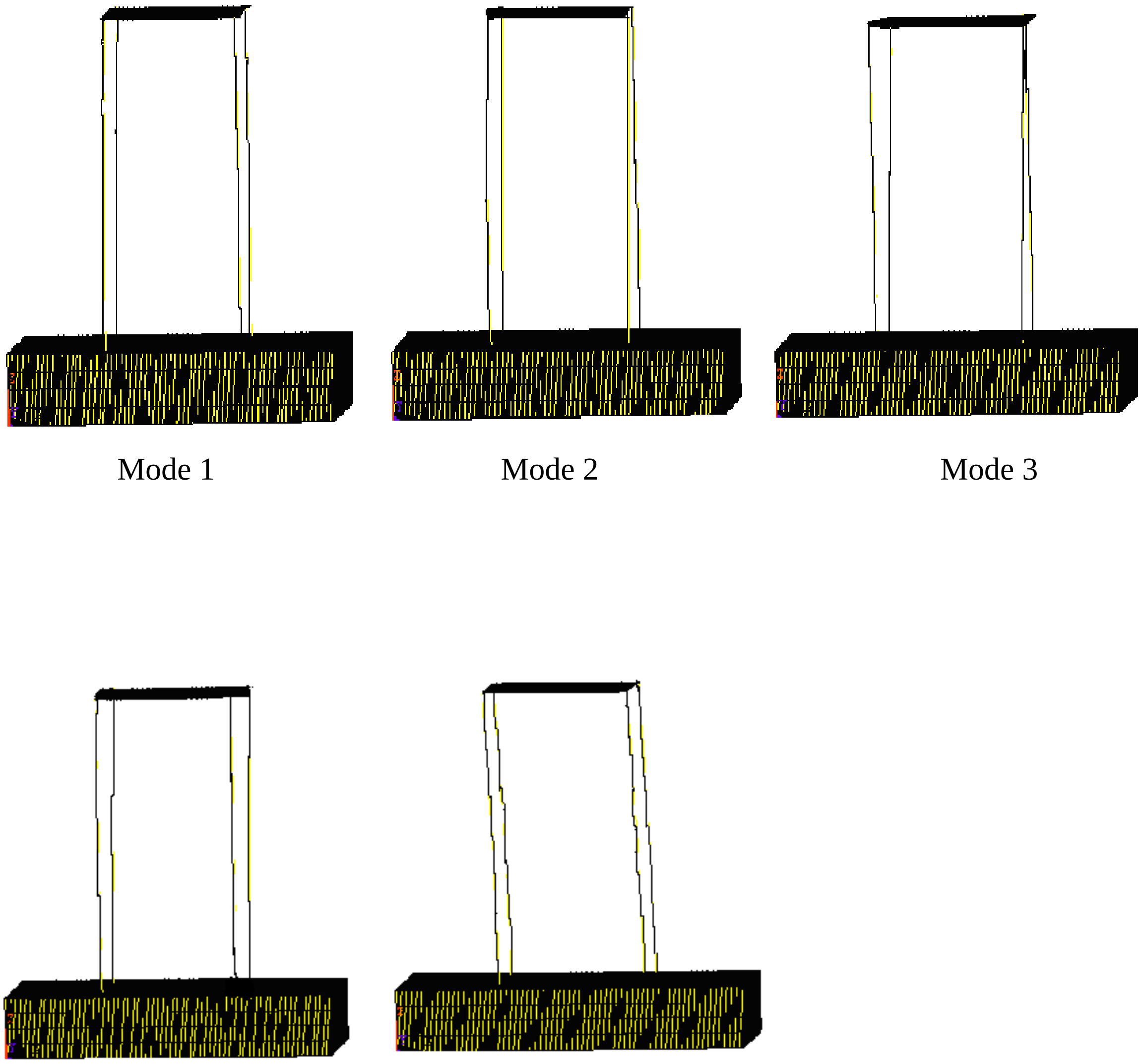


Mode 1 Mode 2 Mode 3



Mode 4 Mode 5

Fig 4.4 Mode shapes of 42mm diameter cylinder with soil



Mode 1 Mode 2 Mode 3

Mode 4 Mode 5

Fig 4.5 Mode shapes of simplified berthing structure with soil mass

Response of various model due to forced vibrations for 2 sec were shown in Fig 4.6 to Fig 4.9.

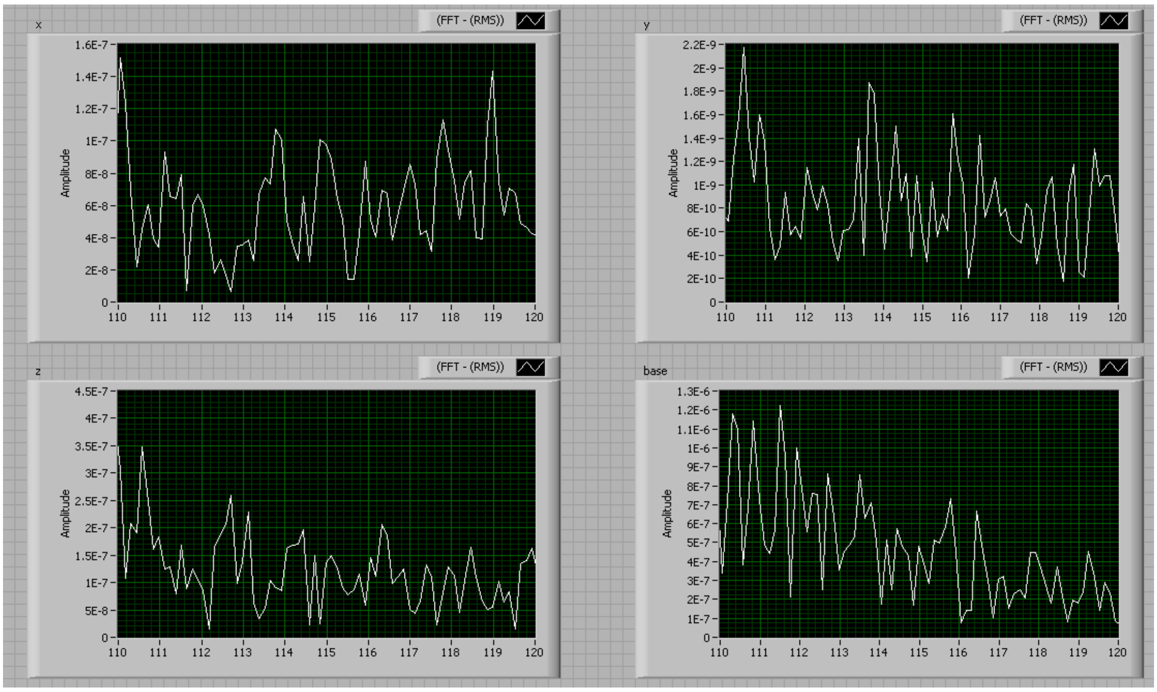


Fig 4.6 The Amplitude Vs Frequency graphs showing X,Y and Z directional variation of the 42mm hollow cylinder and the input frequency

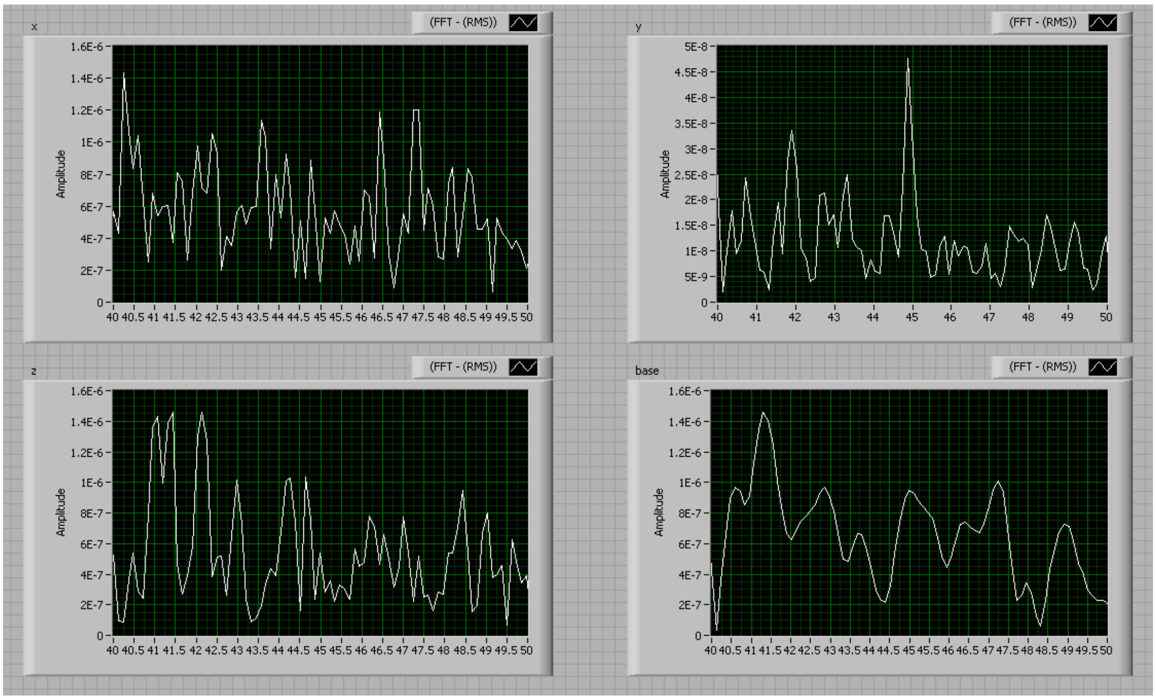


Fig 4.7 Amplitude Vs Frequency graphs for 42mm hollow cylinder in 100 mm soil and the input frequency

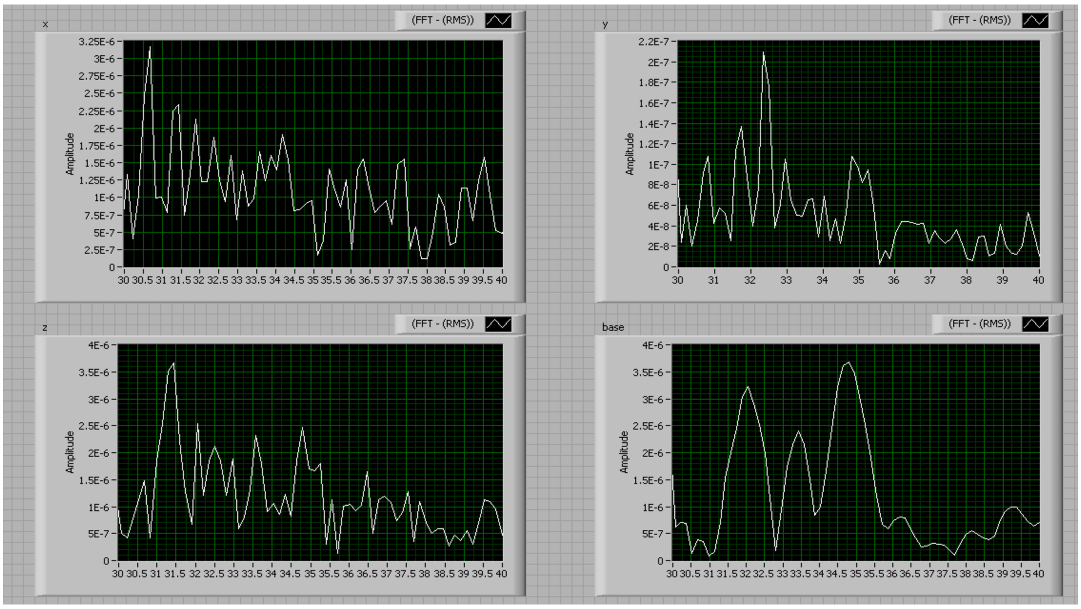


Fig 4.8 Amplitude Vs Frequency graphs of 42mm hollow cylinder in 200mm water and the input frequency

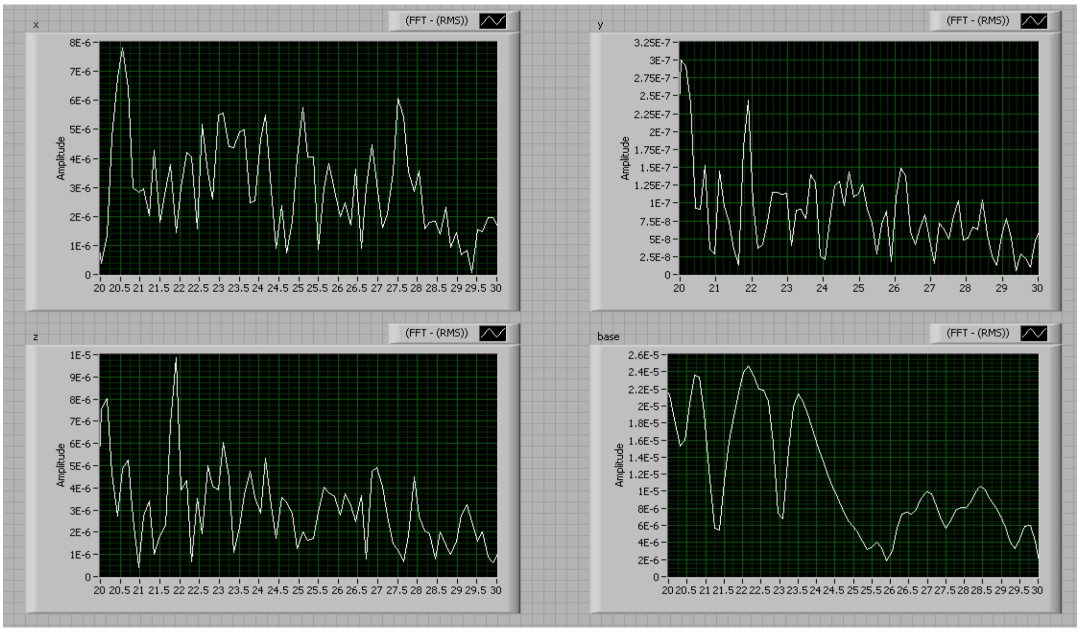


Fig 4.9 Amplitude Vs Frequency graphs of 42mm hollow cylinder in 100mm soil + water and the input frequency

It was observed that response at the top of the vertical member was slightly lesser than the applied frequency at the base. This may be due to damping effect of the structure. Further inclusion of soil and water along with vertical member tends to reduce the natural period. The damping ratio calculated for 42mm hollow cylinder is shown in table.4.4.

Table 4.4 Damping ratio of 42mm cylinder at different experimental conditions

Experimental conditions	Damping ratio
42mm hollow cylinder	0.023
Hollow cylinder +100mm water	0.035
Hollow cylinder + 200mm water	0.098
Hollow cylinder + 100mm soil and water	0.135

The damping ratio increases when the structure in water and soil.

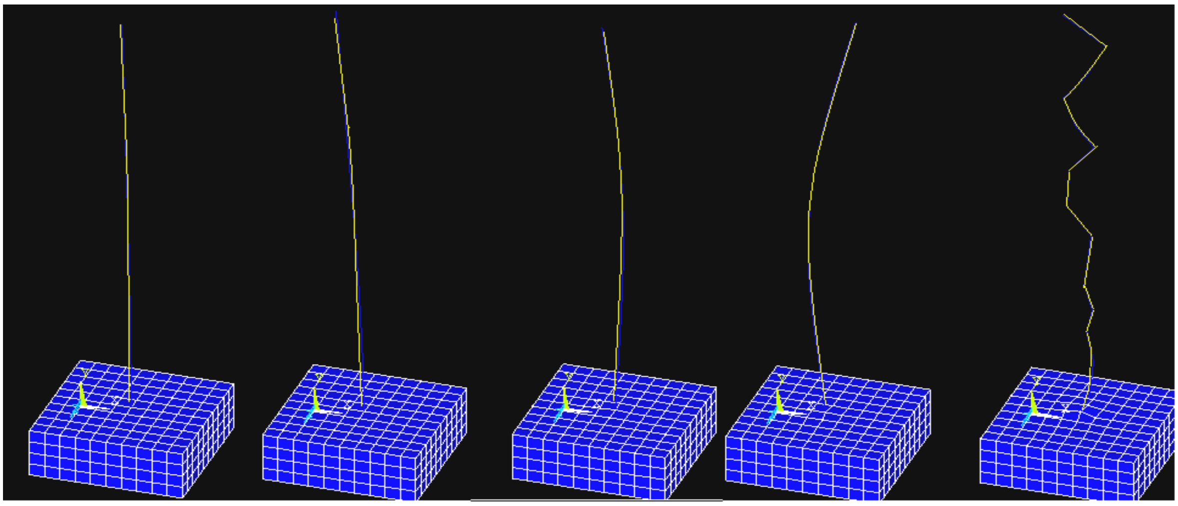
4.7 MODAL ANALYSIS – NATURAL FREQUENCIES

Different modes of vibration of the structure were obtained by performing the modal analysis. The accuracy of the PIPE59 element used in 2D structure was checked with the results of Veeraraja (2001) and a good correlation was found among the results. The damping and the hydrodynamic coefficients greatly influence the natural frequency of the structural system. In the present work the effect of the damping is ignored. The added mass, which is related to the coefficient of inertia, C_m, has its influence on the free vibration mode shapes and frequencies of the structure. As the mass of the structure increases its fundamental frequency is 'depressed' and this frequency may fall in line with the driving frequency. Vinod (2001) checked that with added mass (varying C_m) and found that it has very little effect on the free vibration modes. The natural frequency obtained in ANSYS is 0.248 Hz.

The different modes of vibration of structures were presented in the Fig 4.10. The time period of ocean waves is usually between 8–12 seconds along the Mangalore coast. In the present investigation, it was found that the lowest frequency for the type of cylinder considered was about 21.931 Hz, corresponding to a period of 0.0456 seconds and is far from the driving frequency i.e. the wave frequency. Higher frequencies correspond to still lower periods and hence there are very limited chances of the structure being set to resonance because of wave loads.

Table 4.5 Modal analysis – Comparison of results

MODE	Frequency without soil mass (Hz)	Frequency with soil mass (Hz)
Mode 1	21.931	21.97
Mode 2	21.931	21.97
Mode 3	135.561	137.153
Mode 4	135.561	137.153
Mode 5	371.771	380.504



Mode 1 Mode 2 Mode 3 Mode 4 Mode 5

Fig 4.10: Modes of vibration of 2D cylinder with soil mass

4.8 TRANSIENT ANALYSIS

Quite often the loads coming on a structure are time dependent only over a small interval of time and become either constant or decay with time. Many times the structure experiences maximum deflections and stresses due to such transient loads. Structures are evaluated with reference to transient loads taken in the form of suddenly applied loads with a step/ramped variation. For a pile structure this may simulate the condition of wave load acting on the structure. In the present investigation the transient dynamic analysis of the vertical cylinder, subjected to transient wave loads is attempted using ANSYS software.

The loads considered are taken in the form of ramped loading. The reduced solution method is used for the response analysis in each time step. A time step of 2.5 sec is chosen in the present investigation. Since the wave period in the Mangalore Coast varies from 8-12 sec, the time response history is found for wave periods of 8- 12 seconds interval of time. The wave forces were calculated from Morison's equation substituting suitable wave data and are applied with a time step of 2.5 sec.

After giving suitable constraints and degrees of freedom at the nodes, the time varying forces were applied to the structure. Then when the forces were applied, as results, the time history response of the structure was obtained. The deflection of the structure under the time varying force was one of the responses to be found.

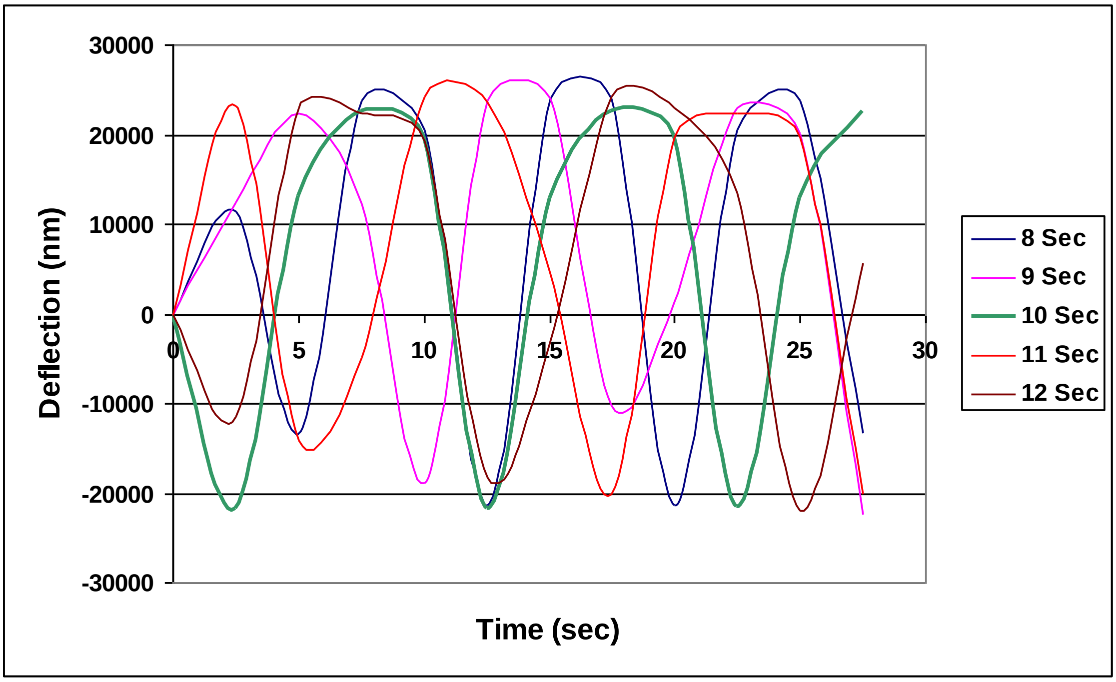


Fig.4.11 Transient response, Ux, lateral deflection of 2D cylinder at different wave periods

The deflections of the structure, due to transient loading for the 2D cylinder is as shown in Fig 4.11, the deflections got for the different wave periods ranging 8–12 sec, vary from a minimum of -23.2 mm to a maximum of 25.03 mm. For 8sec period, the maximum deflection is 24.87 mm; for 9 sec period, maximum deflection is 24.59 mm; for 10 sec, maximum is 23.34 mm; for 11 sec, the maximum is 25.0 mm and for 12 sec, maximum is 25.03 mm.

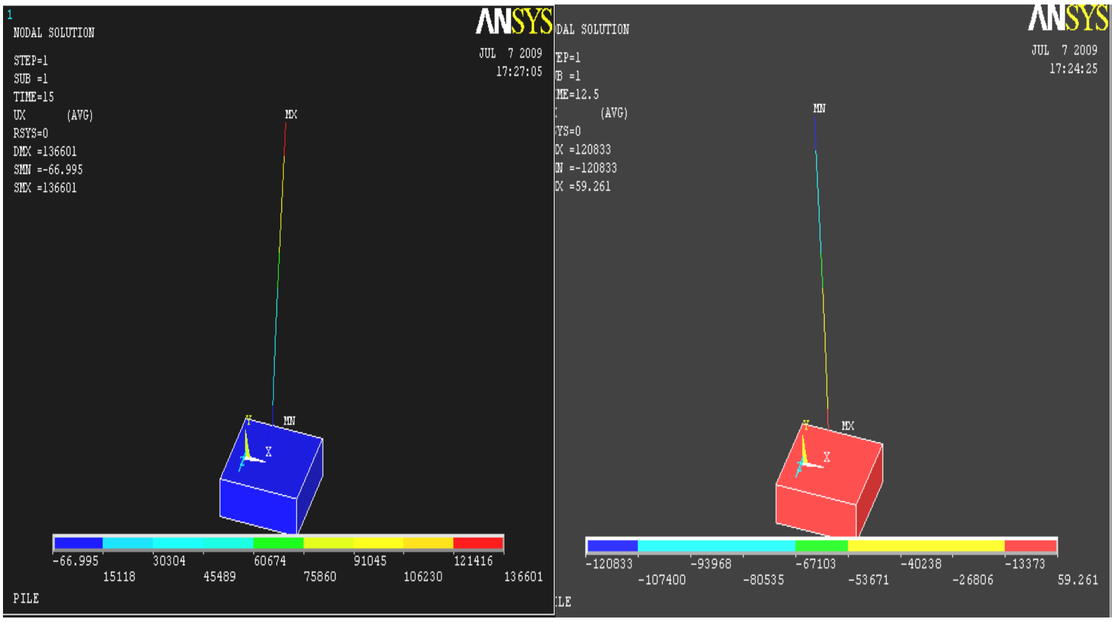


Fig. 4.12 & Fig. 4.13 The deflections of the cylinder in positive and negative direction

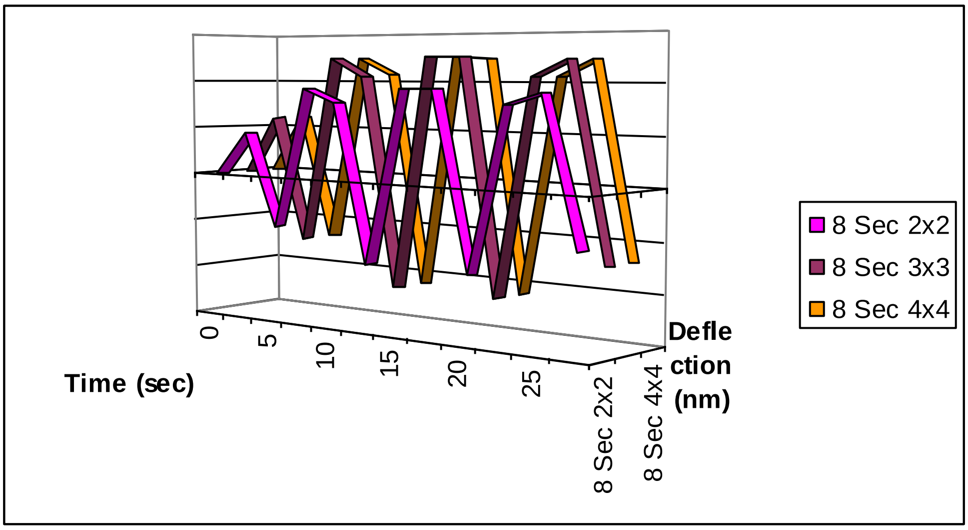


Fig 4.14: Transient response, Ux, lateral deflection for different soil mass on 2D cylinder

Then considering the soil- structure interaction, the stress in the soil was also analysed.

Considering Fig. 4.14 the graph shows the deflection of cylinder with different soil masses. The area of soil mass around the cylinder at the bottom was considered as 2 times the diameter, 3 times the diameter and 4 times the diameter. This was done to find the optimum area of soil required for modeling around a pile. Results show that the deflection of cylinder in 3m x 3m and 4m x 4m soil mass are similar. Further increase in area would not affect the deflections of the cylinder. So a soil mass of area 3 x diameter of pile is considered as the optimum area for the modeling of soil mass. Also the maximum stress in the soil was found to be approximately 5 Pa which is negligibly small and much below the permissible stress. Based on this study berthing structure model of size 350m x 40m was analysed taking soil mass of 3D,4D,5D and keeping depth same for all the cases.

4.9 CONCLUSIONS

1. The natural frequency of the experimental model fairly agreed with the FEM model.. Inclusion of water and the soil tend to increase the natural frequency of the structure.
2. The response at the top of the vertical cylinder is slightly less than the applied frequency at the base. This may be due to damping effect of the structure. Further inclusion of soil and water along with vertical member tends to reduce the natural period.
3. When structure is in water and soil, the damping ratios of vertical member at different experimental conditions increases.
4. Detailed modal and transient dynamic analysis have been conducted on the structure and based on this study, the natural frequency of the structure obtained is 0.247Hz which shows that the natural time period of vibration of the structure is nowhere near the actual field loading conditions where waves are of 8-12 sec period. So there are little chances of resonance due to wave loading.
5. Modal analysis was done for a 2D structure with and without soil mass at bottom and their frequencies were found to be almost same.

6. Performed time-domain transient analysis, and found that, the maximum deflection of the structure under wave action that was obtained in the analysis was 25 mm, which is a reasonable value as in real conditions.
7. Transient response on a 2D cylinder with varying soil mass was obtained. The soil mass considered was 2mx2m, 3mx3m and 4mx4m square mass. From the results, it can be seen that both 3mx3m and 4mx4m soil masses have almost same deflection patterns. i.e. 3mx3m is the optimum soil mass that can be used for this model study.
8. The Maximum stress in the soil due to transient loading in the structure was obtained as 5 Pa which is also negligibly small and much below the permissible stress in the soil.

Chapter 5

WAVE RESPONSE ANALYSIS

5.1 GENERAL

The StruCAD*3D software is used to find the response due to the dynamic effects of wave loadings on berthing structures. The member end forces calculated by the wave response module can be used directly by the StruCAD*3D detailed fatigue module to estimate the fatigue damage of tubular connections. The structure is subjected to a single repeatable wave and the steady state response is calculated using the Fourier series. The basic assumption behind this approach is that the same repeatable wave is sustained long enough to establish a steady state response. The theoretical approach is as follows:

1 A full cycle of wave is applied to the structure. The hydrodynamic forces are calculated using Morison's equation, and saved for each time step for each member. These distributed member forces are then converted to equivalent joint loads using static equilibrium.

2 For each wave time step, the joint load vector created above is multiplied by the mode shape deflections to calculate the generalized forces for each mode.

3 Working with a mode at a time, and considering the fact that the generalized forces calculated for different time steps represent a periodic function, a Fourier transformation can be applied to generate a series of sinusoidal forcing functions with different frequencies and amplitudes, which would represent the same periodic function if they are superimposed at any given time during the wave period.

4 Considering the fact that each structural mode represents a single degree of freedom system with appropriate stiffness, mass, and damping properties, the steady state response of each mode due to any of the sinusoidal Fourier components can be easily calculated. The total response of each mode can be obtained by linear superposition of responses due to each Fourier component.

5 The responses calculated above for each individual mode can be linearly summed to derive the overall response of the structure.

6 After completion of the process, the program will report the time history of the mud line forces and moments for both modal static and modal dynamic cases. The comparison between the results of these two reports can be used to estimate the dynamic amplification factors.

5.2 BERTHING JETTY

The size of the proposed berthing structure is 350m x40m. The jetty was divided into 6 units of 60m x 40m each with an expansion gap of 20mm, and is symmetrical. The water depth at the berthing jetty location is 20m. Two mooring Dolphins were provided for berthing the proposed 303 m long ship. The deck level of the jetty was fixed at + 5 m above the datum. One unit is shown in Fig 5.3. 1m diameter bored cast-in-situ RCC Piles are provided. The spacing in the transverse direction is 5m c/c. Pile caps of 1.85m x 1.40m x 0.40m were provided on the berthing side and 1.40m x 1.40m x 0.4m are provided on the remaining piles. Above the pile caps were the transverse beams. The transverse beams were 1.80m x 0.70m, The longitudinal beams were precast T-beams with flange width of 2m, for 2 m c/c spacing, flange thickness of .250 mm and depth of 1.40m and it run over transverse beams. On the berthing side of the jetty, a fender beam of size 2.40m x 1.20m was provided and it rests on the piles. RCC deck slab of 350 mm is provided with 100mm thick wearing coat. Considering the tidal variation the deck level of the jetty was fixed at + 5.0 above the chart datum.

5.2.1 Approach jetty

The length of the approach jetty is 350 m and the width is 40m. Approach jetty is connected to the main jetty with an expansion gap of 20mm. Approach jetty consists of 1.0m diameter bored cast in-situ RCC piles provided at 10m c/c in longitudinal direction and 5 m in the transverse direction. Fig 5.1 shows the plan of berthing structure and all dimensions are in meters.

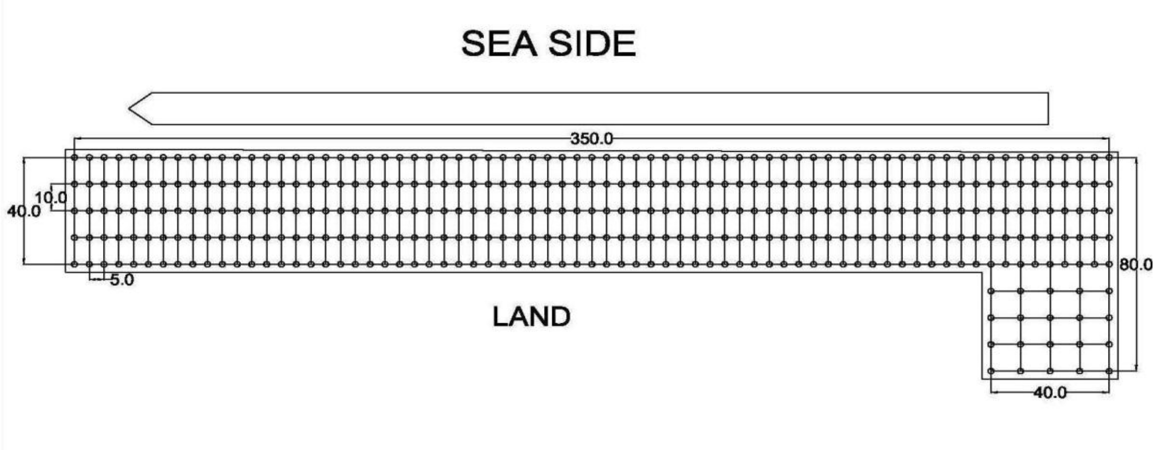


Fig 5.1 plan of berthing structure

5.2.1 Structural model

The modeling of the berthing structure is done using the software StruCAD. The 3D structural model developed is shown in the Fig 5.2

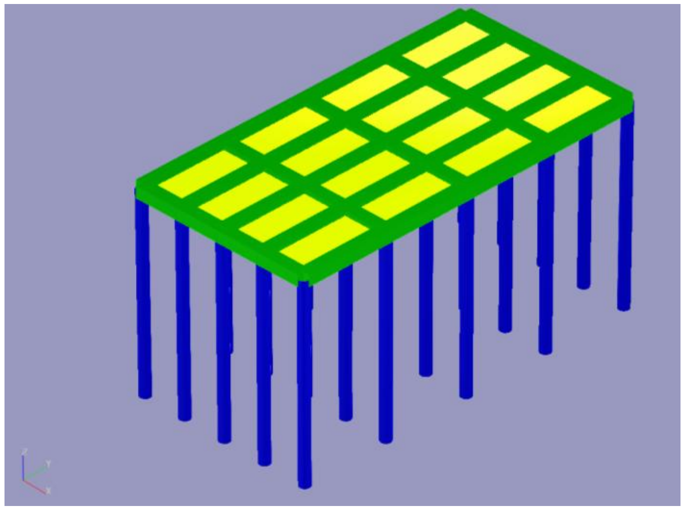


Fig 5.2 Model of Berthing Structure

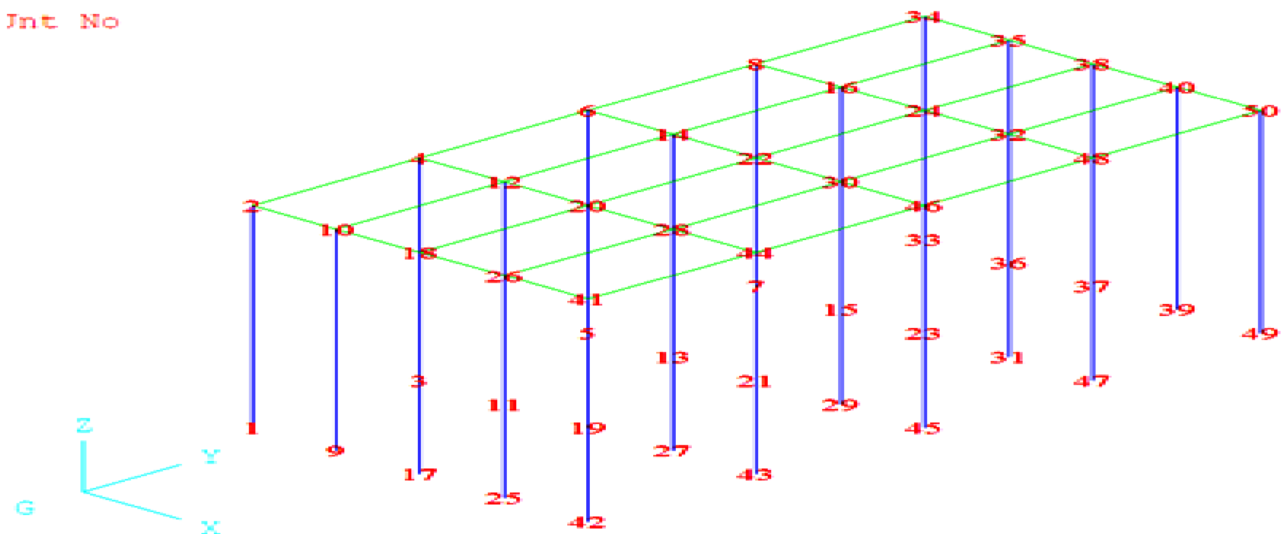


Fig 5.3 Node numbering in StruCAD

5.3 LOAD COMBINATION

Structure shall be designed to sustain safely the effect of the combination of various loads, forces and stresses that can possibly co-exist. The following load combinations were considered (IS 4651: part IV, 1979).

1. Dead Load + Live Load + Wave Load + Current Load+ Berthing load
2. Dead Load + Live Load + Wind Load+ Mooring Load
3. Dead Load + Live Load + Wave Load + Current Load

5.4 STATIC ANALYSIS

Static analysis is carried out for 2, 00,000 DWT vessel using StruCAD software. Tables 5.1 and 5.2 shows the analysis result for different load combination for 1m and 1.2m diameter piles respectively.

Table 5.1 force, moment and deflection for different load combination for 1m diameter pile

Load combinations	Maximum Force kN	Maximum Moment kN-m	Deflection at top of the structure (mm)
Dead Load + Live Load + Wave Load + Current Load+ Berthing load	138.46	1726.59	136.5
Dead Load + Live Load + Wind Load+ Mooring Load	111.18	1437.81	121.9
Dead Load + Live Load + Wave Load + Current Load	37.69	381.13	22.8

Table 5.2 force, moment and deflection for different load combination for 1.2mdiameter pile

Load combinations	Maximum Force kN	Maximum Moment kN-m	Deflection at top of the structure (mm)
Dead Load + Live Load + Wave Load + Current Load+ Berthing load	148.23	1817.76	66.9
Dead Load + Live Load + Wind Load+ Mooring Load	125.33	1564.45	59.7
Dead Load + Live Load + Wave Load + Current Load	69.39	622.56	14.2

The total wave force is given by the summation of drag component and inertia component; both these forces depend on diameter of the pile. Hence from the Tables 5.1 and 5.2 we can observe that the forces and moment are maximum for larger diameter pile compared to smaller diameter pile i.e., there is an increase of 6 % for larger diameter piles. Also it was observed that top deflection of the structure for smaller diameter is more compared to the larger diameter i.e., there is an increase of 50 % for smaller diameter piles due to less stiffness. The significant wave height of 3.2m is considered having frequency of 0.10 Hz.

5.4.1 Deflection, for significant wave height of 3.2m.

The wave directions considered for the present analyses are 45°, 90° and 135° with respect to the structural alignment. The values of deflections for different diameters of piles and for different wave direction are presented in Table 5.3. From the results it is seen that deflection varies with respect to wave direction and the structural response is maximum for wave direction perpendicular to the structural alignment. Thus it is important to consider the perpendicular direction for analysis since it is a critical condition

Table 5.3 Deflection of pile top for different wave directions and different pile diameters

Joint id	Wave direction (degree)	1000mm dia Deflection (mm)		1100mmdia Deflection (mm)		1200mm dia Deflection (mm)	
		X	Y	X	y	X	y
1	45	-133.68	-1.376	-94.79	-09.58	-71.018	-07.42
	90	-153.33	-1.388	-109.19	-09.60	-81.55	-06.88
	135	-117.23	-0.893	-84.73	-08.35	-62.56	-06.01
2	45	-110.3	-1.375	-78.237	-09.574	-58.71	-07.41
	90	-128.19	-1.386	-91.27	-09.54	-68.24	-06.87
	135	-97.06	-0.892	-70.32	-08.33	-51.91	-05.99
3	45	-86.77	-1.370	-61.41	-09.55	-46.13	-07.37
	90	-102.74	-1.382	-73.04	-09.54	-54.62	-06.82
	135	-76.65	-1.199	-55.65	-08.29	-41.02	-05.95
4	45	-63.64	-1.363	-45.07	-09.48	-34.02	-07.29
	90	-77.75	-1.375	-55.26	-09.47	-54.62	-06.82
	135	-5666	-1.192	-41.40	-08.22	-30.53	-05.88
5	45	-40.95	-1.355	-29.16	-09.405	-22.32	-07.22
	90	-53.18	-1.367	-37.90	-09.39	-28.69	-06.68
	135	-37.04	-1.185	-27.52	-08.15	-20.41	-05.81
6	45	-18.57	-1.349	-13.57	-09.34	-10.93	-07.16
	90	-28.91	-1.361	-20.84	-09.33	-16.24	-06.61
	135	-17.71	-1.178	-13.91	-08.15	-10.55	-05.81
7	45	08.76	-1.345	05.86	-09.30	04.04	-07.12
	90	-04.79	-1.357	-03.93	-09.29	-03.93	-06.58
	135	08.94	-1.175	06.03	-08.05	04.20	-05.71
8	45	31.65	-1.344	21.84	-09.29	15.6	-07.11
	90	28.74	-1.356	19.74	-09.28	08.98	-06.56
	135	29.25	-1.173	20.20	-08.03	14.45	-05.70

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5.5 WAVE FORCE ANALYSIS

The design of the berthing structure is to be done considering the maximum wave expected to occur during the life of the structure. Considering the wave data collected at the site, the maximum wave height was 5.5m with a period of 11sec.

Table 5.4 Forces and Moment during a wave cycle for wave height 5.5 m and period 11 Secs for 1 m diameter pile

Crest Position of wave	Phase Angle	Resultant	
		Force	Moment
(m)	(Deg)	kN	kN-m
0	0	321.91	5026.42
7.61	20	392.64	4907.40
15.21	40	525.73	5930.91
22.82	60	640.37	6984.68
30.42	80	651.59	6969.34
38.03	100	596.43	6437.08
45.64	120	510.41	5705.03
53.24	140	372.16	4446.96
60.85	160	255.47	3437.21
68.45	180	285.11	3775.39
76.06	200	429.38	5317.00
83.67	220	612.38	7564.83
91.27	240	769.23	9744.63
98.88	260	851.86	11170.19
106.48	280	853.02	11485.81
114.09	300	743.63	10604.85
121.69	320	612.65	9121.50
129.3	340	430.79	6866.89
136.91	360	321.91	5026.42

The analysis of the structure for the maximum wave height of 5.5m was carried out for a full cycle wave approaching perpendicular to the alignment of structure. The wave crest position and phase angle of one wave cycle with respect to structure was shown.

Table 5.5 Forces and Moment during a wave cycle for wave height 5.5 m and period 11 sec for 1.2 m dia pile

Crest Position of wave	Phase Angle	Resultant	
		Force	Moment
(m)	(Deg)	kN	kN-m
0	0	411.61	6119.12
7.61	20	534.89	6310.63
15.21	40	727.2	7961.81
22.82	60	861.25	9242.98
30.42	80	873.88	9282.53
38.03	100	787.32	8448.11
45.64	120	615.19	6801.72
53.24	140	443.4	5348.06
60.85	160	310.21	4207.39
68.45	180	384.51	4905.21
76.06	200	610.49	7322.61
83.67	220	863.59	10445.97
91.27	240	1040.47	12977.14
98.88	260	1124.67	14562.86
106.48	280	1184.95	14646.55
114.09	300	971.93	13592.52
121.69	320	706	10706.69
129.3	340	500.2	8088.36
136.91	360	411.61	6119.12

Table 5.4 and Table 5.5 shows the wave crest position and phase angle of one wave cycle with respect to structure. It also shows the force and moment for the wave position and phase angle.

From Table 5.4 and Table 5.5 it is observed that maximum force and maximum moment for 1m diameter pile is 853.02 kN and 11485.81 kN-m and for 1.2 m diameter pile is 1184.95 kN and 14646.55 kN-m at a phase angle of 280^0 because the wave force is applied at 270^0 which is perpendicular to the alignment of structure.

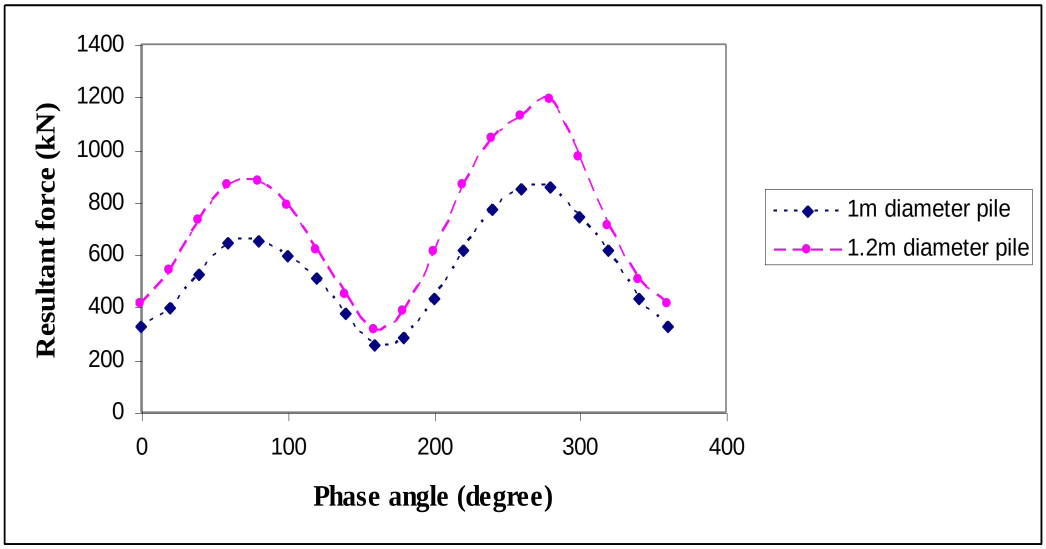

Fig 5.4 Variation of force during a wave cycle

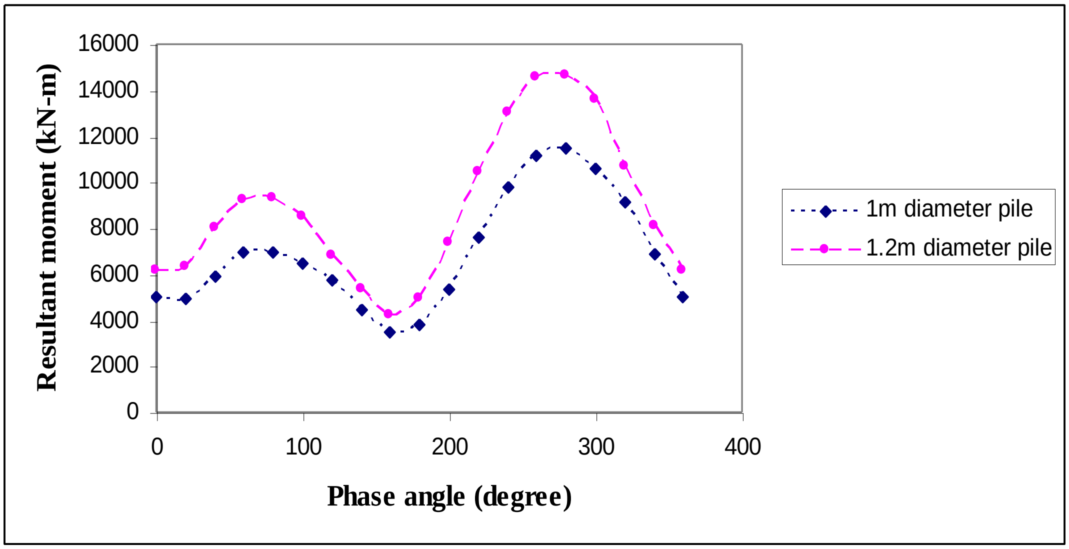

Fig 5.5 Variation of Moment during a wave cycle

A full wave cycle was considered for plotting the results. The variation of forces and moment on the structure element were presented for one wave cycle. Fig 5.4 shows the resultant forces for 1m and 1.2m diameter pile and Fig 5.5 shows the resultant moment for 1.0m and 1.2m diameter pile respectively. It was also observed that force and moment are both harmonic in nature. The force and moment for 1.2m diameter pile was 23 percent more than the 1.0m diameter pile. This is due to increase in drag and inertia force, which is directly proportional to diameter of pile.

5.5.1 Joint deflection

The values of deflections for different diameters of pile for perpendicular wave direction are summarized and presented in Tables 5.6 and 5.7.

From Tables 5.6 and 5.7 it is observed that top node deflection is maximum in front row side of the pile and it decreases as it moves back side pile for both 1m and 1.2m diameter pile because when wave hits the front side pile its energy gets dissipated and the piles which are in-line at back side will get less wave energy.

Table 5.6 Maximum joint deflection for 1 m diameter pile

Pile Joint No.	**Max. Top Node Deflection (cm)**
34,35,38,40,50	1.84
6,14,22,30,46	1.73
2,10,18,26,41	1.61

Table 5.7 Maximum joint deflection for 1.2 m diameter pile

Pile Joint No.	**Max. Top Node Deflection (cm)**
34,35,38,40,50	1.11
6,14,22,30,46	1.05
2,10,18,26,41	0.99

It decreases from 10% to 12% for both diameter piles. The maximum top joint deflection is 1.84 cm for 1m diameter pile. It is observed that deflection decreases as the diameter increases and it decreases about 39.67 % when compared to the smaller diameter as it depends on stiffness of the structure. One typical variation of joint No 18 is shown in Fig 5.6

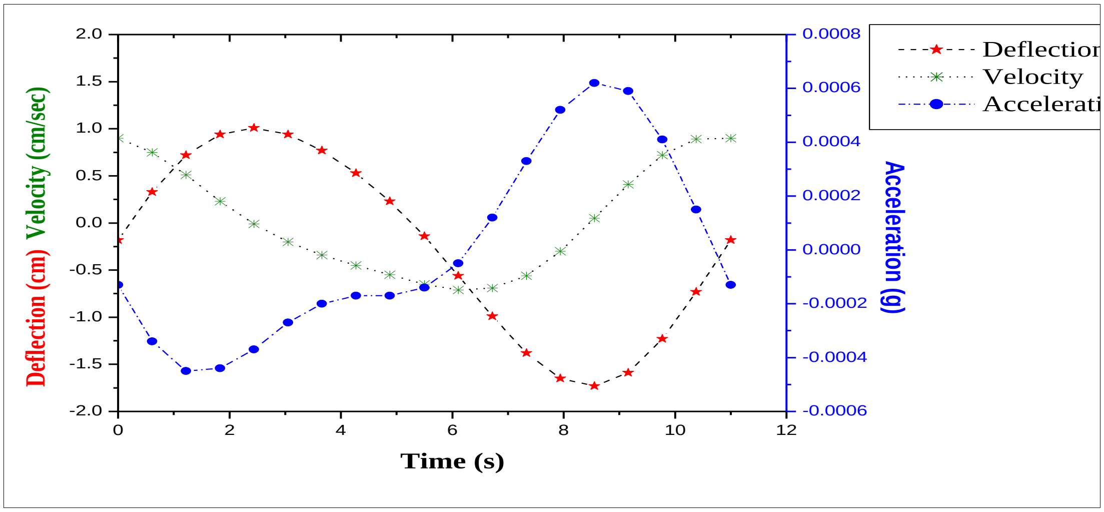

Fig 5.6 displacement, velocity and acceleration for joint node no.18 for 1m diameter pile

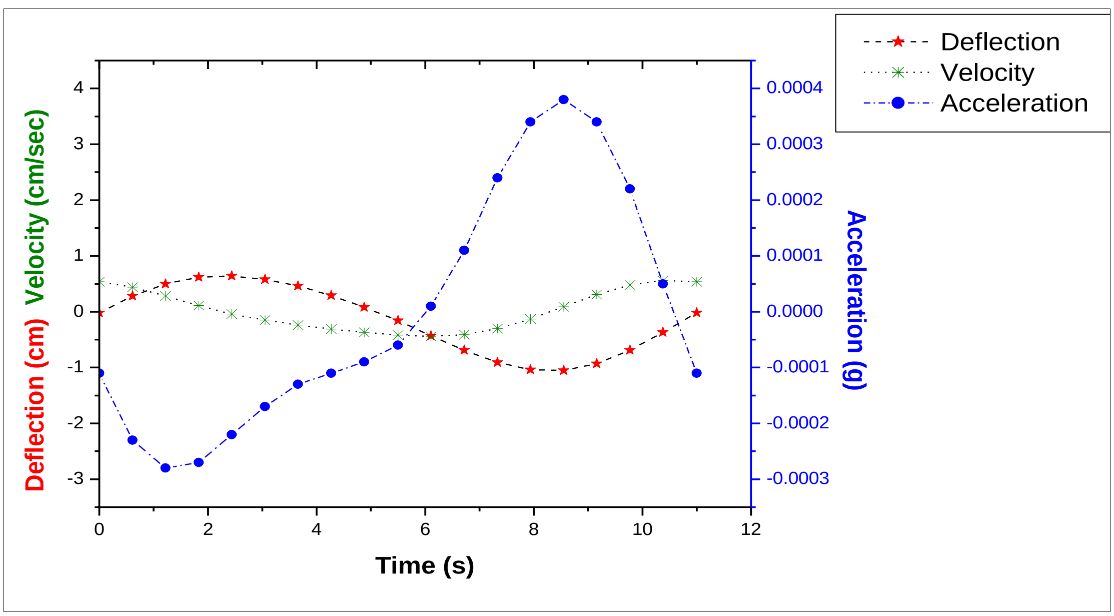

Fig 5.7 displacement, velocity and acceleration for 1m diameter pile

Fig 5.6 and Fig 5.7 gives the dynamic response of joint no. 18 for a wave height 5.5 m for first mode. The fundamental time period of structure is 2.7237 Sec and 2.0094 Sec for 1m and 1.2m diameter pile. So there is no chance of resonance. We can observe that there is a phase difference of 90^0 between displacement and velocity and 180^o between structural displacement and acceleration.

5.6 Static and dynamic response for wave loading

Table 5.8 to Table 5.11 shows the forces and moments during the wave cycle for maximum wave height. Peak dynamic response for each loading is compared to its static response. The ratio of dynamic to static response is called Dynamic Amplification Factor (DAF). This factor is calculated by comparing static and dynamic modal analysis results of wave action.

Table 5.8 Modal static response for maximum wave height 5.5 m and period 11 sec for 1 m dia pile.

Time Step	FORCE			MOMENT			RESULTANT	
	X	Y	Z	Mx	My	Mz	Shear Force	Overturning Moment
(Sec)	/--------------kN-------------------/			/----------------kN-m--------------/			(kN)	(kN-m)
0	211.09	-34.45	-61.17	-437.87	5879.62	-3362.88	213.88	5895.9
0.6111	183.36	78.75	-45.67	-2939.59	5034.01	-1887.03	199.55	5829.45
1.2222	167.06	165.07	-27.95	-4721.88	4452.53	-1135.13	234.86	6490.08
1.8333	149.85	216.25	-12.97	-5683.5	3875.12	-586.95	263.09	6878.86
2.4444	133.13	231.74	-0.65	-5809.36	3336.06	-540.55	267.26	6699.1
3.0556	129.87	216.21	11.73	-5157.78	3132.87	-637.07	252.21	6034.7
3.6667	135.52	175.85	25.39	-3858.43	3140.31	-919.19	222.01	4974.84
4.2778	136.93	115.67	38.28	-2079.53	3049.06	-1468.99	179.25	3690.69
4.8889	137.2	38.79	47.1	30.61	2969.36	-2208.82	142.58	2969.51
5.5	144.8	-52	50.33	2369.74	3127.64	-3472.25	153.85	3924
6.1111	157.8	-151.51	48.97	4829.54	3466.21	-5132.51	218.77	5944.66
6.7222	175.58	-250.13	44.21	7194.6	3957.54	-6735.72	305.6	8211.24
7.3333	202.15	-333.82	34.96	9091.03	4712.92	-8377.32	390.26	10240.04
7.9444	231.66	-386.84	18.46	10066.3	5612.95	-9474.45	450.9	11525.44
8.5556	252.94	-396.28	-6.07	9781.29	6385.8	-9646.01	470.13	11681.27
9.1667	264.58	-356.67	-34.15	8194.16	6952.43	-8999.03	444.09	10746.19
9.7778	264.9	-272.65	-56.99	5607.75	7184.41	-7652.77	380.14	9113.87
10.3889	245.2	-158.49	-66.62	2548.26	6786.4	-5646.78	291.97	7249.05
11	211.09	-34.45	-61.17	-437.87	5879.62	-3362.87	213.88	5895.9

Table 5.9 Modal dynamic response for maximum wave height 5.5 m and period 11 sec for 1 m dia pile.

Time Step	FORCE			MOMENT			RESULTANT	
	X	Y	Z	Mx	My	Mz	Shear Force	Overturning Moment
Sec	/--------------kN-------------------/			/----------------kN-m--------------/			kN	kN-m
0	221.21	-43.94	-61.22	-201.82	6133.36	-3673.8	225.54	6136.68
0.6111	173.66	81.55	-45.75	-3011.3	4792.19	-1272.6	191.85	5659.75
1.2222	152.13	176.85	-28.01	-5018	4079.59	-610.67	233.28	6467.11
1.8333	157.61	232.43	-13.03	-6089.4	4069.61	-897.76	280.82	7324.06
2.4444	149.24	248.49	-0.7	-6229.3	3739.33	-550.43	289.86	7265.48
3.0556	119.16	231.79	11.67	-5548.7	2865.76	-274.63	260.62	6245.03
3.6667	113.54	190.7	25.33	-4231.1	2591.15	-600.27	221.94	4961.44
4.2778	142.88	131.26	38.23	-2470.3	3198.38	-1295.9	194.02	4041.3
4.8889	156.59	55.92	47.08	-398.22	3454.51	-2237	166.28	3477.39
5.5	138.37	-34.55	50.33	1933.3	2966.78	-3170.5	142.62	3541.11
6.1111	135.09	-137.23	48.99	4472.57	2897.91	-4507.5	192.56	5329.33
6.7222	169.17	-243.7	44.24	7034.4	3796.93	-6569	296.66	7993.72
7.3333	208.36	-339.18	35.02	9226.35	4867.6	-8526.7	398.07	10431.63
7.9444	231.22	-405.23	18.55	10528.2	5600.78	-9796.7	466.55	11925.23
8.5556	249.68	-425.19	-5.95	10506.9	6302.95	-10085	493.08	12252.44
9.1667	268.64	-390.3	-34.03	9037.69	7052.69	-9410.5	473.81	11463.87
9.7778	276	-303.68	-56.92	6385.31	7461.39	-8135.9	410.36	9820.62
10.3889	261.11	-180.51	-66.63	3098.73	7184.31	-6268.7	317.43	7824.09
11	221.21	-43.94	-61.22	-201.83	6133.36	-3673.8	225.53	6136.68

Table 5.10 Modal static response for maximum wave height 5.5 m and period 11 sec for 1.2 m dia pile.

Time Step	FORCE			MOMENT			RESULTANT	
	X	Y	Z	Mx	My	Mz	Shear Force	Overturning Moment
Sec	/--------------kN---------------/			/---------------kN-m--------------/			kN	kN-m
0.0000	257.50	-3.85	-76.38	-1529.04	7190.46	-3696.19	257.52	7351.23
0.6111	216.15	139.85	-54.04	-4647.88	5937.08	-1453.23	257.45	7540.01
1.2222	186.07	244.64	-30.02	-6757.81	4948.93	-381.26	307.36	8376.15
1.8333	172.87	301.43	-10.74	-7768.06	4429.69	-126.23	347.49	8942.3
2.4444	165.01	311.13	4.34	-7689.77	4085.07	-431.17	352.18	8707.48
3.0556	157.26	280.88	19.26	-6615.59	3744.82	-693.06	321.91	7601.95
3.6667	157.75	219.35	35.96	-4721.34	3593.29	-1116.17	270.18	5933.19
4.2778	165.43	133.3	51.8	-2231.83	3630.11	-1853.11	212.45	4261.31
4.8889	170.06	27.09	62.28	647.7	3643.05	-2952.32	172.2	3700.18
5.5	175.67	-95.01	65.19	3763.67	3754.8	-4536.01	199.71	5316.37
6.1111	193.96	-225.38	61.72	6950.68	4246.43	-6710.8	297.35	8145.19
6.7222	221.69	-350.6	53.6	9909.84	5019.97	-8904.55	414.81	11108.78
7.3333	250.32	-452.03	39.79	12153.04	5871.36	-10755.7	516.71	13497.01
7.9444	282.21	-509.68	17.22	13114.5	6890.58	-11936.8	582.59	14814.53
8.5556	314.89	-508.26	-14.68	12400.36	8021.11	-12154.2	597.9	14768.45
9.1667	331.51	-442.95	-49.87	10017.97	8782.04	-11327.5	553.27	13322.31
9.7778	322.92	-322.28	-77.08	6420.56	8834.08	-9021.38	456.23	10920.83
10.3889	295.61	-166.81	-86.49	2331.73	8243.06	-6532.87	339.42	8566.5
11	257.5	-3.85	-76.38	-1529.05	7190.45	-3696.19	257.52	7351.23

Table 5.11 Modal dynamic response for maximum wave height 5.5 m and period 11 sec for 1.2 m dia pile.

Time Step Sec	FORCE			MOMENT			RESULTANT	
	X	Y	Z	Mx	My	Mz	Shear Force	Overturning Moment
	/--------------kN----------------/			/----------------kN-m---------------/			kN	kN-m
0.00	264.35	-12.35	-76.45	-1318.07	7362.66	-4156.67	264.64	7479.71
0.6111	220.32	140.05	-54.13	-4654.87	6042.19	-1333.29	261.06	7627.31
1.2222	178.19	251.57	-30.1	-6932.96	4752.76	-541.72	308.29	8405.63
1.8333	168.82	312.25	-10.8	-8039.86	4329.02	-185.11	354.96	9131.25
2.4444	172.12	323.37	4.28	-7997.05	4263.59	-224.92	366.32	9062.61
3.0556	156.82	293.32	19.2	-6928.02	3734.53	-755.42	332.61	7870.47
3.6667	148.58	232.03	35.89	-5039.78	3364.64	-877.93	275.52	6059.71
4.2778	166.61	146.79	51.75	-2570.33	3660.07	-1599.5	222.05	4472.44
4.8889	176.47	41.46	62.26	287.92	3803.56	-2992.5	181.27	3814.45
5.5	168.86	-81.02	65.2	3413.98	3584.53	-4149.42	187.29	4950.16
6.1111	183.58	-214.41	61.75	6676.9	3986.67	-6310.55	282.27	7776.54
6.7222	223.14	-345.91	53.64	9793.54	5055.71	-9010.46	411.64	11021.5
7.3333	250.82	-456.21	39.86	12259.08	5883.35	-10720.4	520.61	13597.75
7.9444	272.98	-523.37	17.33	13459.28	6658.62	-12007.5	590.28	15016.31
8.5556	314.09	-529.56	-14.55	12936	7999.66	-12473.3	615.7	15209.69
9.1667	344.04	-467.75	-49.75	10640.89	9094.21	-11753.5	580.65	13997.62
9.7778	330.19	-345.48	-77.02	7002.34	9015.13	-9672.74	477.89	11415.13
10.3889	296.88	-183.93	-86.5	2759.77	8275.03	-6413.92	349.24	8723.1
11	264.35	-12.35	-76.45	-1318.07	7362.66	-4156.66	264.64	7479.71

We can calculate the Dynamic Amplification Factor (DAF) (STRUCAD-3D Reference Manual 2001) for 1.0m and 1.2 diameter piles. The DAF for 1.0m diameter pile varies from 1.02 to 1.43. The DAF for 1.2 m diameter pile varies from 1.03 to 1.53. From the results of dynamic amplification factor it can be observed that there is an increase in response of 43% and 53% for 1.0m and 1.2m diameter pile respectively due to wave loading compared to static loading. The critical wave position was determined by the maximum moment and the critical wave position is at time step 8.56sec..

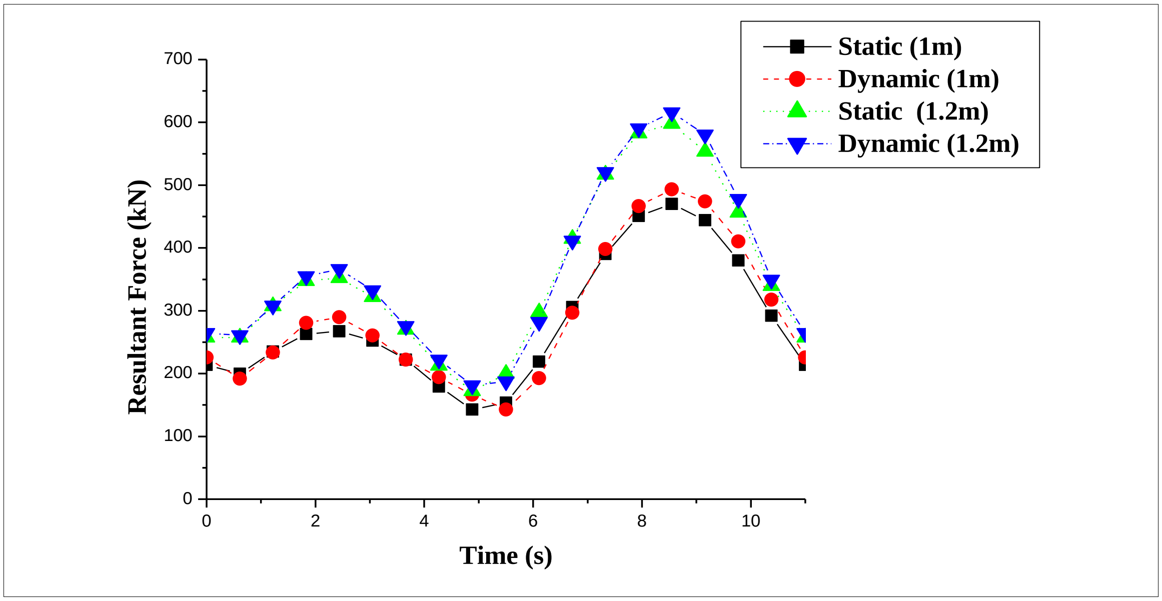

Fig 5.8 Variation of resultant forces with time

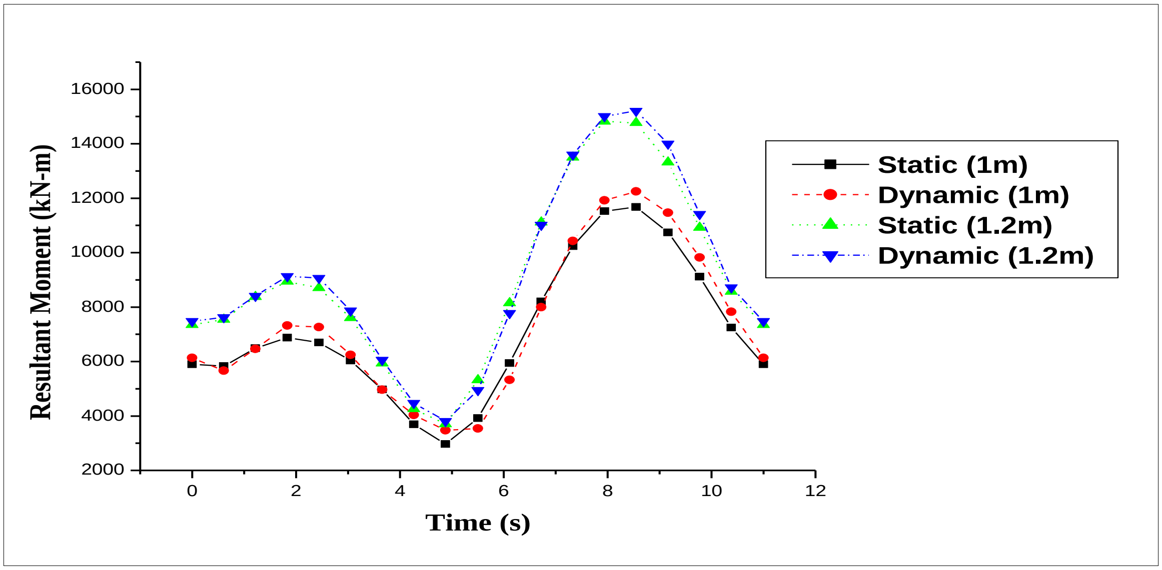

Fig 5.9 Variation of resultant moment with time

Fig. 5.8 and Fig 5.9 present the variation of forces and moments for different time steps for static and dynamic analysis. It was observed that the values of forces and moment calculated by dynamic analysis were more than that by static analysis. Static results are 2 00o 14 % and 1 to 6 % less than dynamic analysis for 1000mm diameter and 1200mm diameter piles respectively.

5.7 SOIL STRUCTURE INTERACTION USING SAP 2000 SOFTWARE

SAP2000 is a full-featured program that can be used for the simplest problems or the most complex projects. SAP2000 is a linear and non-linear, static and dynamic, analysis and design of three dimensional structures package. 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 today. The program is composed of several modules, which are easily accessible and straight forward to use.

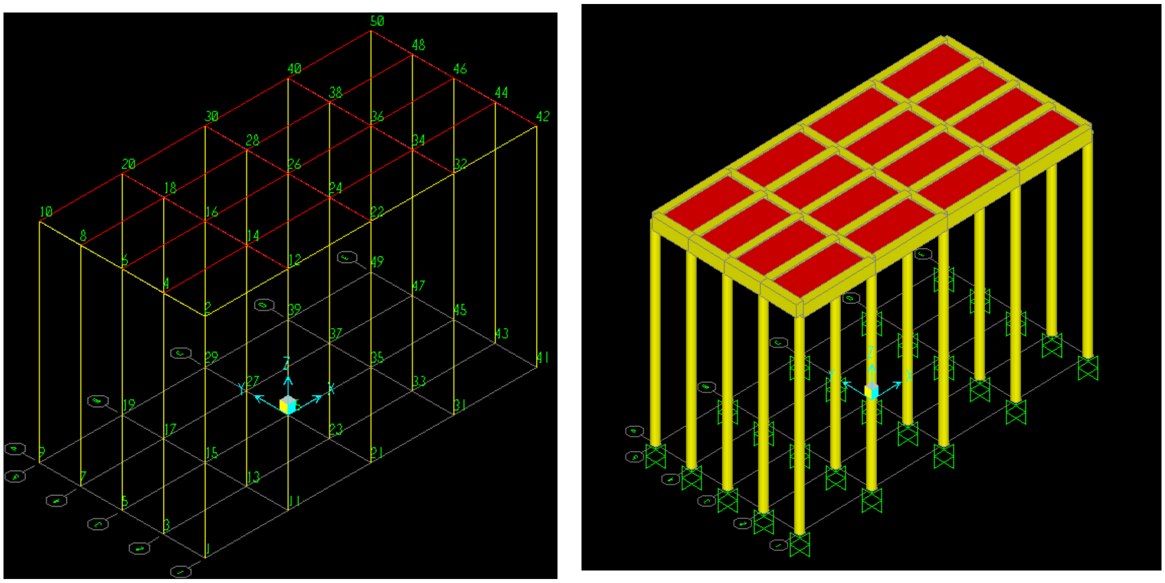

Fig 5.10 Node numbering and model in SAP without soil mass

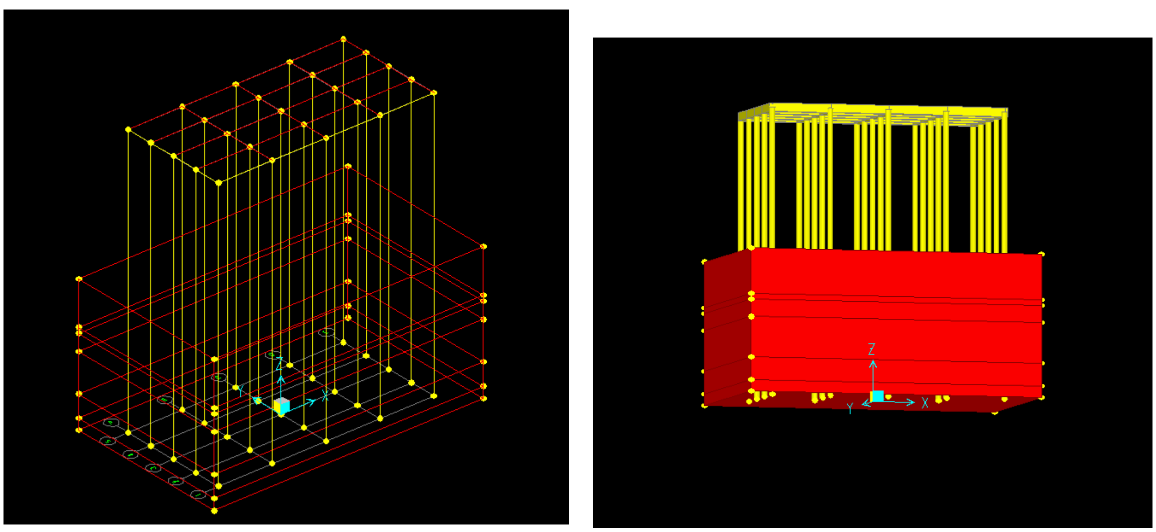

Fig 5.11 Node numbering and model of berthing structure with soil mass using SAP

5.7.1 METHODOLOGY FOR STATIC ANALYSIS

1 Number of bays and bay width in x and y direction and total pile length are specified.
2 Slab is modeled as 4-noded thin plate element by specifying the thickness. The plate element has 6 degrees of freedom at each node (3 translations and 3 rotations).
3 Pile caps are modeled as plate bending element with 3 translatory and 2 rotational degrees of freedom.
4 Piles are modeled as 2 nodded beam elements with 6 degrees of freedom at each node (3 translations and 3 rotations).
5 Soil mass is modeled as 3D solid 8-noded brick element with 6 degrees of freedom at each node (3 translations and 3 rotations).
6 All the loads such as dead, live, berthing, wave, mooring, current and wind forces are calculated manually and applied on the berthing structure and analysed.
7 Maximum force, moment and top node deflections were obtained for different load combinations on the structure.

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5.8 CONSIDERING SOIL MASS OF SIZE 3D

Soil Mass of size 3 times the width from the centre of the structure is taken and the depth is kept equal to 25m for all cases. Static and wave response analysis is carried out.

5.8.1 Static analysis

From static analysis including soil structure interaction forces, moments and deflections were found for both 1000mm and 1200mm diameter piles. 4.3. Fig 6.13 shows the bore log details considered for soil-structure interaction. Tables 6.11 and 6.12 shows the analysis result for different load combination for 1m and 1.2m diameter piles respectively.

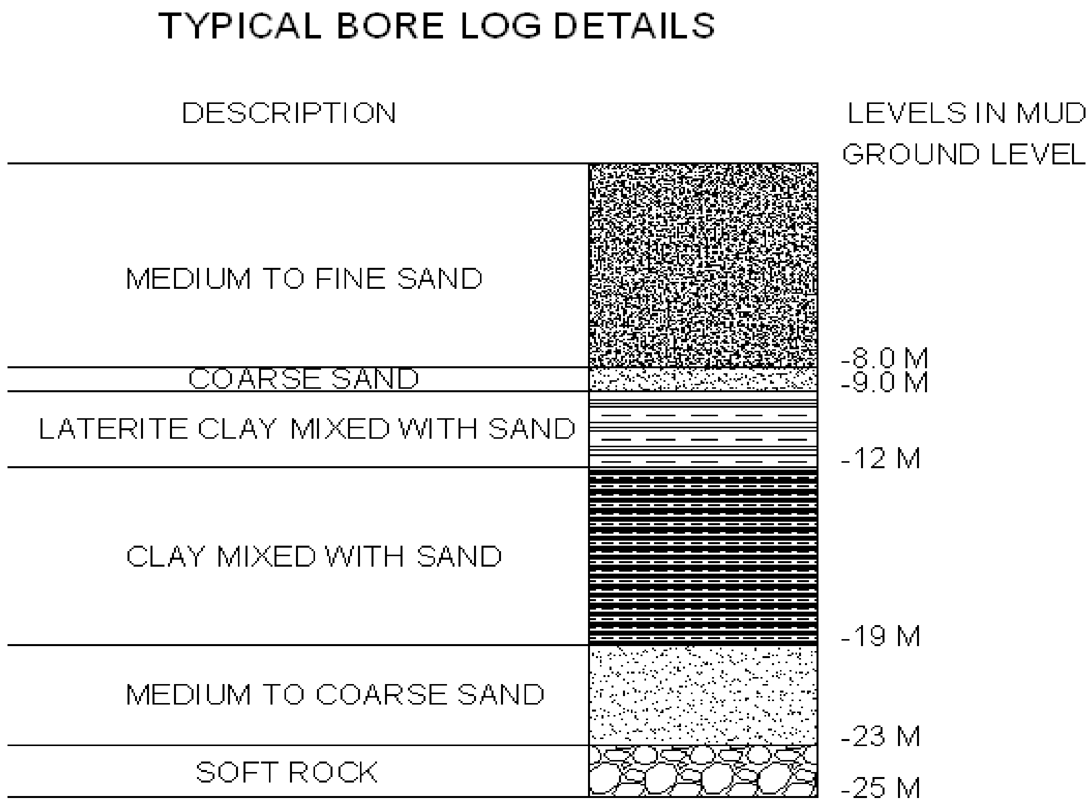


Fig 5.12 Bore log details of NMPT.

Table 5.12 force, moment and deflection for different load combination for 1m dia pile

Load combinations	Maximum Force kN	Maximum Moment kN-m	Deflection at top of the structure (mm)
Dead Load + Live Load + Wave Load + Current Load+ Berthing load	856.772	6690.746	-177.33
Dead Load + Live Load + Wind Load+ Mooring Load	476.076	3085.772	81.22
Dead Load + Live Load + Wave Load + Current Load	51.623	1406.16	-28.67

Table 5.13 force, moment and deflection for different load combination for 1.2m diameter pile

Load combinations	Maximum Force kN	Maximum Moment kN-m	Deflection at top of the structure (mm)
Dead Load + Live Load + Wave Load + Current Load+ Berthing load	855.319	6924.054	-88.47
Dead Load + Live Load + Wind Load+ Mooring Load	475.277	3119.698	39.58
Dead Load + Live Load + Wave Load + Current Load	60.901	1350.69	-16.02

The total wave force is given by the summation of drag component and inertia component. Both these forces depend on diameter of the pile. Hence from the Tables 5.12 and 5.13 we can observe that the forces and moment are maximum for larger diameter pile compared to smaller diameter pile. It was also observed that top deflection of the structure for smaller diameter is more compared to the larger diameter i.e., there is an increase of 50 % for smaller diameter piles due to less stiffness.

The maximum top joint deflection is 28.68 mm for 1.0m diameter pile and that of 1.2m diameter pile was found to be 16.08mm. It was observed that deflection decreases as the diameter increases and it decreases about 44.10 % when compared to the smaller diameter as it depends on stiffness of the structure.

5.9 CONSIDERING SOIL MASS OF SIZE 4D

Tables 5.14 and 5.15 shows the Static analysis result for different load combination for 1.0m and 1.2m diameter piles respectively.

Table 5.14 force, moment and deflection for different load combination for 1m dia pile

Load combinations	Maximum Force kN	Maximum Moment kN-m	Deflection at top of the structure (mm)
Dead Load + Live Load + Wave Load + Current Load+ Berthing load	1672.661	22998.62	-46.45
Dead Load + Live Load + Wind Load+ Mooring Load	1128.269	14886.53	21.61
Dead Load + Live Load + Wave Load + Current Load	1367.383	16961.33	-6.90

Table 5.15 force, moment and deflection for different load combination for 1.2m dia pile

Load combinations	Maximum Force kN	Maximum Moment kN-m	Deflection at top of the structure (mm)
Dead Load + Live Load + Wave Load + Current Load+ Berthing load	1766.32	22345.27	-25.20
Dead Load + Live Load + Wind Load+ Mooring Load	1127.25	13672.23	11.46
Dead Load + Live Load + Wave Load + Current Load	1443.46	15464.89	-4.21

5.9.1 Joint deflection from wave response analysis

The values of deflections for different diameters of pile for perpendicular wave direction The maximum top joint deflection is 6.91 cm for 1.0m diameter pile and 4.02mm for 1.2m diameter piles.. It is observed that deflection decreases as the diameter increases and it decreases about 38.92 % when compared to the smaller diameter as it depends on stiffness of the structure. By taking 5D there is no difference in responses .So the optimum soil mass to resist wave forces will be 3D

5.10 SUMMARY AND CONCLUSIONS

Berthing structures are the facilities constructed in ports for berthing and mooring of vessels, for loading and unloading of cargo and for embarkment and disembarkment of passengers or vehicles. The berthing structures are designed for dead load, live load, berthing force, mooring force, earthquake load and other environmental loading due to winds, waves, currents etc,. In the present study layout of jetty for berthing 200000 DWT ship at NMPT is modeled using the ship dimensions from IS code and analyzed for the available environmental data from NMPT using StruCAD*3D software. The detailed analysis of the berthing structure for the maximum wave height of 5.5m for a period 11 secs was carried out for a full cycle of wave and the variation of deflection, forces and moments for perpendicular wave directions and different pile diameters is done by static and dynamic methods. Dynamic Amplification Factor was calculated by comparing static and dynamic analysis results.

The following conclusions were drawn.

1. The wave force acting on a berthing structure depends on the diameter of piles. The total wave force is given by the summation of drag component and inertia component. The drag force depends on the square of the fluid particle velocity and introduces nonlinearity to the excitation force. The higher order terms in the dynamic analysis occur not only because of the velocity-squared term but also due to higher order wave theory, variable submergence of elements near the water surface, etc.
2. It is seen that maximum force and maximum moment for 1m diameter pile is 853.02 kN and 11485.81 kN-m and for 1.2 m diameter pile is 1184.95 kN and 14646.55 kN-m at a phase angle of 280^0 i.e., perpendicular to the structure. It is seen that the force and moment for 1.2m diameter pile is 23 percent more than the 1m diameter pile.
3. It is seen that top joint deflection is maximum in front row side of the pile and it decreases as it moves back side pile for both 1m and 1.2m diameter pile. And it decreases from 10% to 12% for both diameter piles. The maximum top joint deflection is 1.84 cm for 1m diameter pile. It is observed that deflection decreases

as the diameter increases and it decreases about 39.67 % when compared to the smaller diameter.

4. It is seen that the values of forces and moment calculated by dynamic analysis are more than that by static analysis. Static results are 2 to 14 % and 1 to 6 % less than dynamic analysis method for 1m diameter and 1.2m diameter piles respectively.
5. The Dynamic Amplification Factor (DAF) is calculated from static and dynamic results. The DAF for 1m diameter pile varies from 1.02 to 1.43 and for 1.2 m diameter pile varies from 1.03 to 1.53. From Dynamic Amplification Factor it is seen that there is increase in response of 43% and 53% for 1m and 1.2m diameter pile respectively due to wave loading compared to static loading.

CHAPTER 6

EARTHQUAKE RESPONSE ANALYSIS

6.1 GENERAL

Recent earthquake disasters have revealed the importance of countermeasures against soil liquefaction in seismic design. In particular, even the backfill soil with gravel that does not supposed to be liquefied at the 1995 Hyogoken Nanbu earthquake (Kobe earthquake). This new type of liquefaction gave great impact in the seismic design. Many researchers have elucidated the behavior of soil liquefaction and developed the simulation method. As a result, some computer programs evaluate the behavior. In order to estimate accurately the dynamic behavior of pile supported structure, not only the super structure but also piles and soil nonlinearity must be considered.

The first step in any seismic analysis is the definition of the seismic input. In this study, the seismic input for structure was defined in terms of an accelerogram applied at the base of the structures. An accelerogram basically the time history of the acceleration experienced by the ground in a given direction during a seismic event. There are varies methods of scaling accelerograms but the scaling procedure which is commonly in use is scaling in terms of the amplitude. This is found to be a, good practice as the frequency content of the time history is not altered. We need to input the accelerogram as a generic function defined in SAP starting from a TXT file. Once the structural model under investigation has been created, selection from the main bar for time histories was done by clicking on display graph, we can visualize the wave form. The seismic input is the classical El Centro earthquake recorded in Imperial valley, California, on 15 October 1940, Kobe earthquake of 1995 and North ridge earthquake of 1994.

Fig.6.1 shows a plot of the acceleration time-history of the El Centro earthquake. Fig .6.2 represents the plot of the acceleration time-history of Kobe earthquake and Fig 6.3 represents the plot of the acceleration time-history of Northridge earthquake. All the accelerations are presented as a fraction of g (the gravitational acceleration). The total duration of El Centro earthquake is 18sec and that of Kobe and north ridge earthquake is 25.64sec and 39.98sec respectively. El Centro accelerogram is the standard input in most earthquake engineering studies and also it represents a typical strong, broad-band motion.

These data are downloaded from the National Information Service Center for Earthquake Engineering at the FTP site: “nisee.ce.berkeley.edu”.

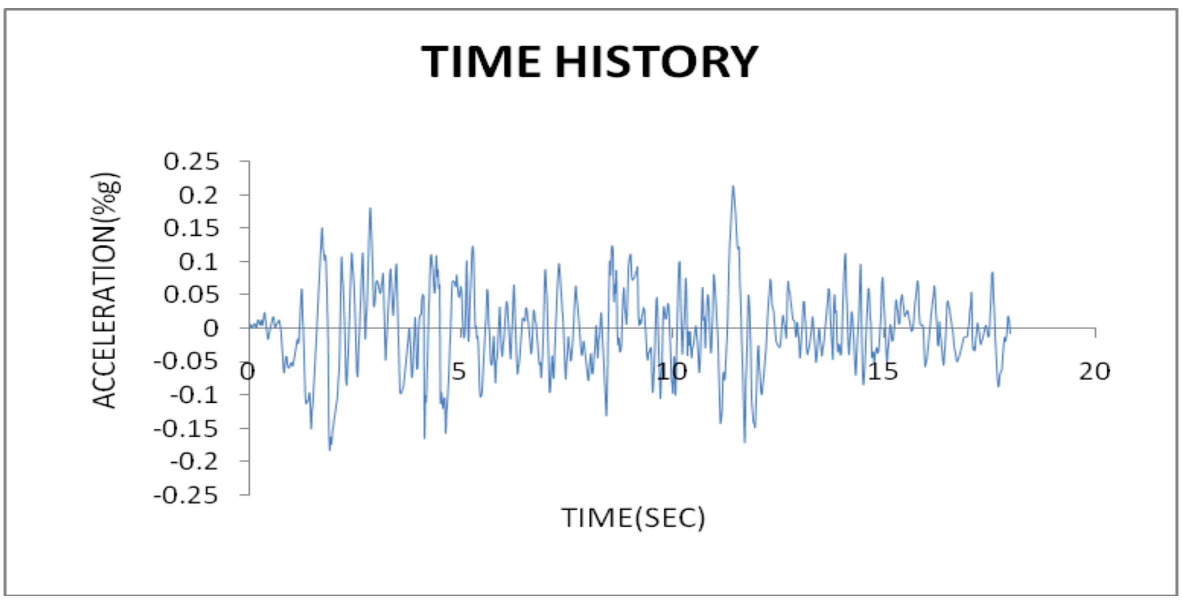


Fig. 6.1 Acceleration time-history of El Centro Earthquake (East-west component)

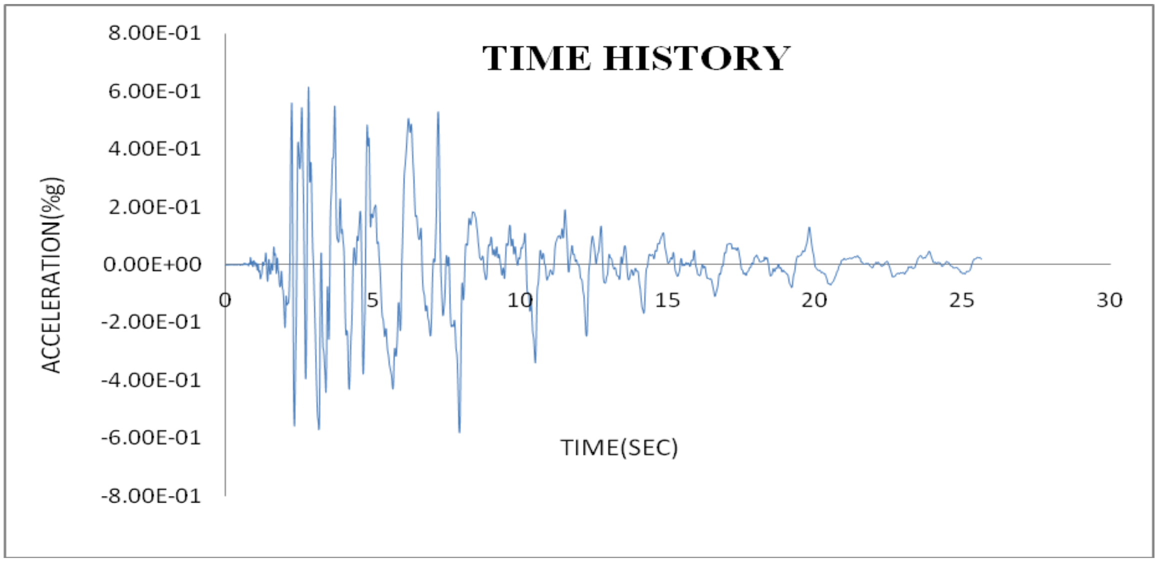


Fig. 6.2 Acceleration time-history of Kobe earthquake.

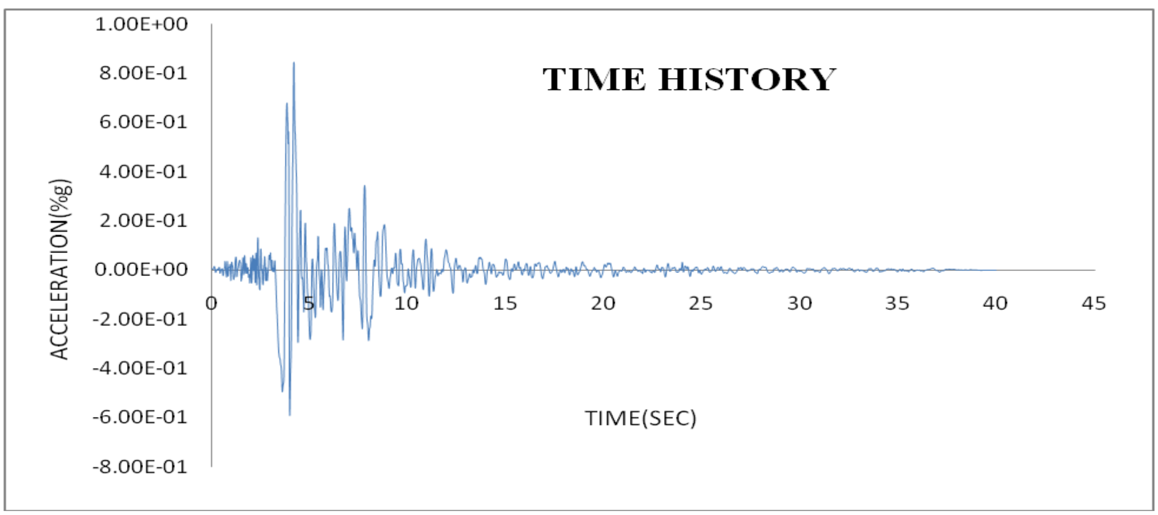
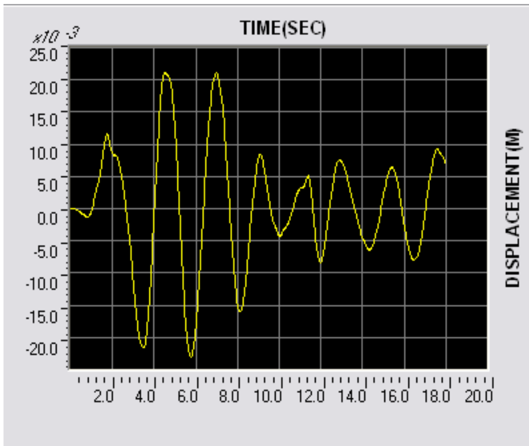


Fig.6.3 Acceleration time-history of Northridge earthquake.

6.2 RESPONSE OF THE STRUCTURE FOR EARTHQUAKE WITHOUT SOIL -STRUCTURE INTERACTION

The displacement response of a structure to a particular earthquake motion depends on damping and natural periods. In the analysis assumed a constant damping of 5% (IS 1893 (Part1):2002).Wave forms generated in SAP software were shown in Fig 6.4 for 1.0m diameter pile and for pile diameter 1.2m displacement wave forms for El centro earthquake is shown in Fig 6.5. It was observed that the maximum deflection of 21.1mm and 13.4mm occurred for El Centro earthquake at a time step of 6.98sec and 10.18sec for 1.0m and 1.2m diameter pile respectively. We can observe from Fig. 6.6 and Fig. 6.7 the maximum deflection of 87.77mm and 61.9mm occurred for Kobe earthquake at a time step of 10.96sec. The maximum deflection of 76.1mm and 58.7mm occurred for North ridge earthquake at a time step of 8.66 sec and 5.60 sec for 1.0m and 1.2m diameter pile respectively, which can be seen from Fig.6.8 and Fig 6.9.



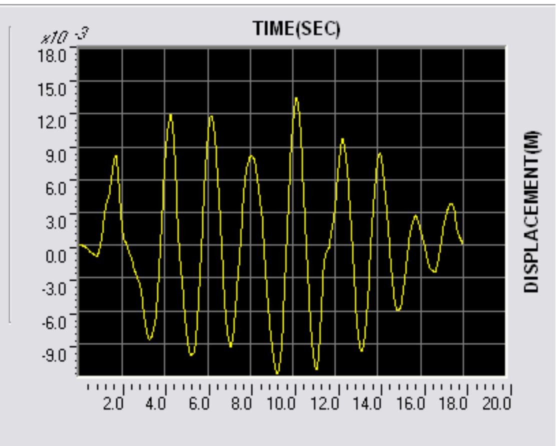
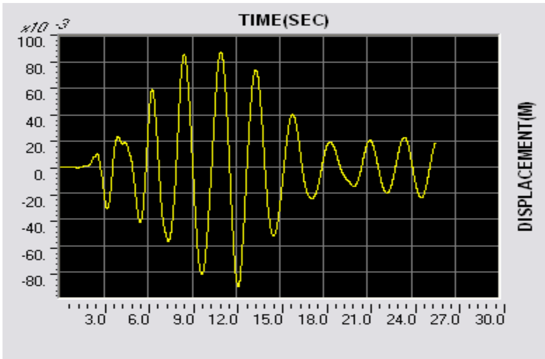


Fig. 6.4 Displacement of 1.0 m dia pile. Fig. 6.5 Displacement of 1.20 m dia pile for El Centro



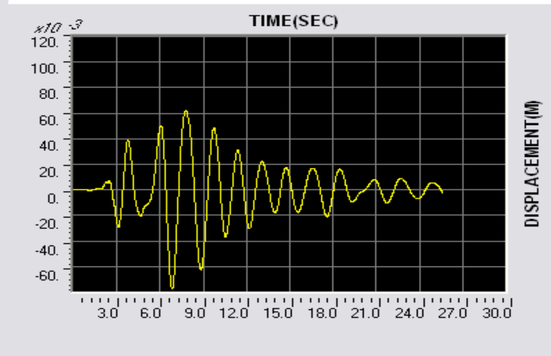
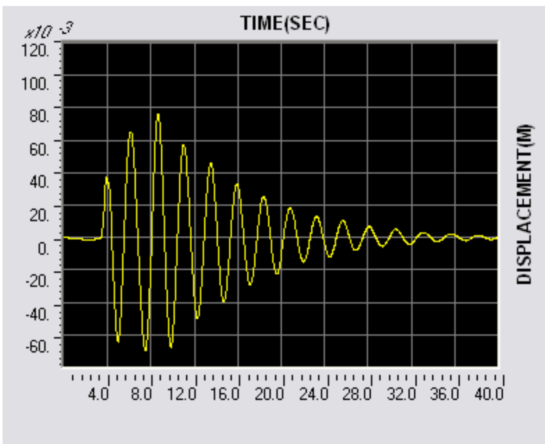


Fig. 6.6 displacement of 1.0 m dia pile. Fig. 6.7 displacement of 1.20 m dia pile for kobe earthquake



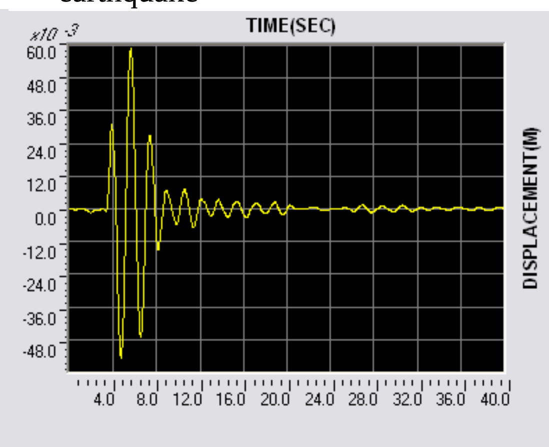


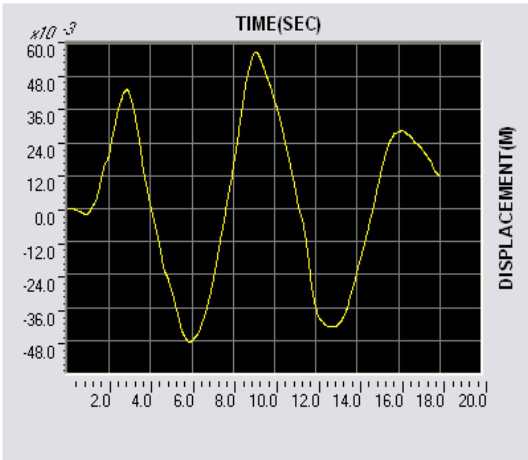
Fig. 6.8 displacement of 1.0 m dia pile. Fig. 6.9 displacement of 1.20 m dia pile for Northridge earthquake

The difference in the displacement for 1.0m and 1.2m pile was due to varying natural periods. Also from the above results we can conclude that longer the natural period greater is the peak deformation. The maximum deformation for 1.2m pile is delayed compared to 1m pile due to larger stiffness and larger mass.

6.3 RESPONSE OF THE STRUCTURE WITH SOIL-STRUCTURE INTERACTION FOR WIDTH OF 3D

In this study, the seismic input for structure was defined in terms of an accelerogram applied at the base of the structure.

The seismic input is the classical El Centro earthquake recorded in Imperial valley, California, on 15 October 1940, Kobe earthquake of 1995 and North ridge earthquake of 1994. All the accelerations were presented as a fraction of g (the gravitational acceleration).



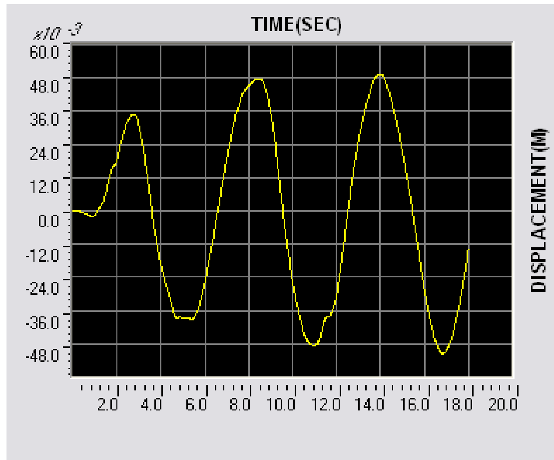
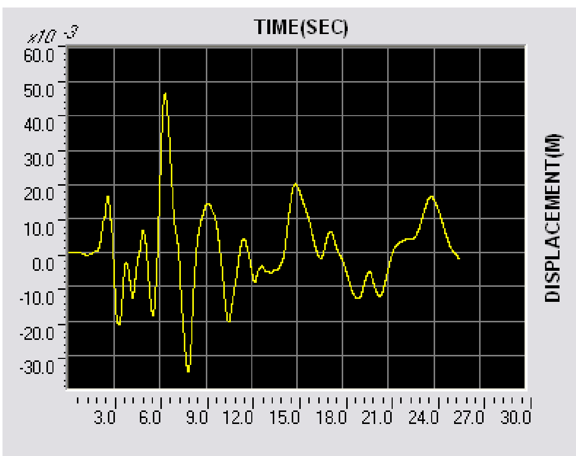


Fig 6.10 Deflection time-history for 1.0 m. Fig 6.11 Deflection time-history for 1.2m for El Centro Earthquake



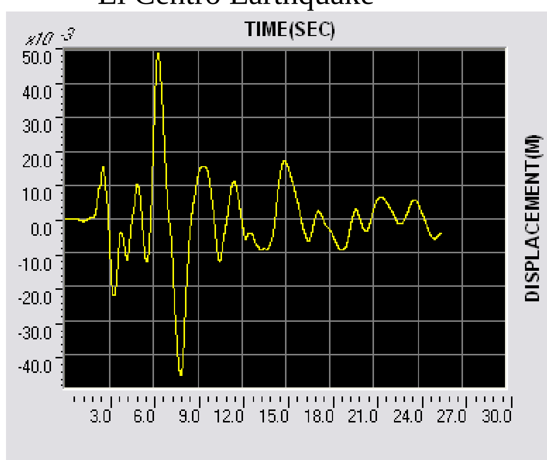
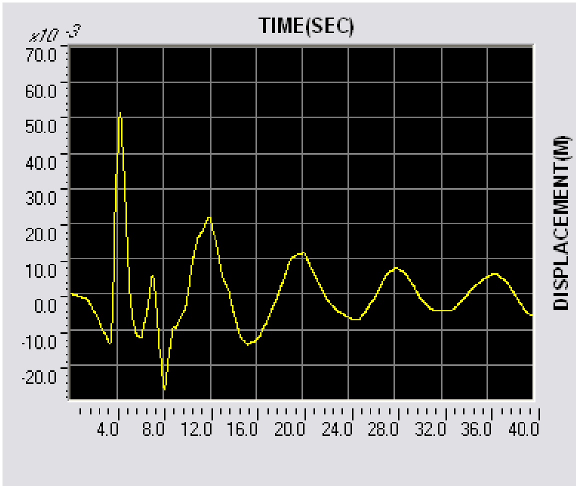


Fig. 6.12 Displacement of 1.0 m dia pile Fig 6.13 Displacement of 1.2 m pile for Kobe Earthquake



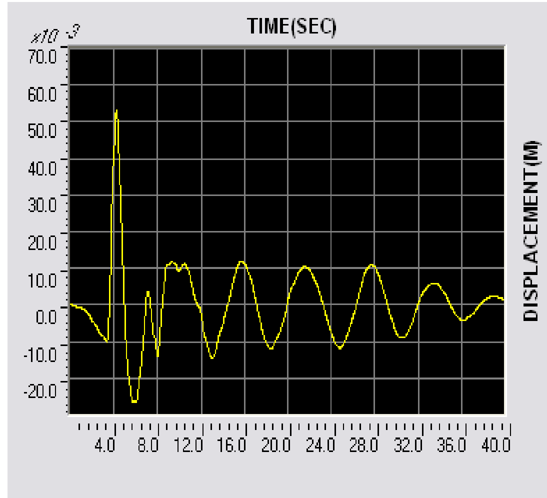
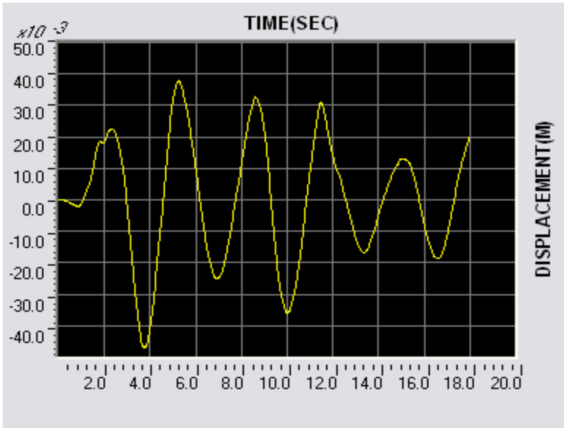


Fig. 6.14 Displacement of 1.0 m dia pile Fig 6.15 Displacement of 1.2 m pile for Northridge Earthquake

It was observed from Fig.6.10 toFig.6.15 that the maximum deflection of 57.0mm and 49.2mm occurred for El Centro earthquake at a time step of 9.10sec and 13.98sec for 1.0m and 1.2m diameter pile respectively. The maximum deflection of 46.5mm and 49.2mm occurred for Kobe earthquake at a time step of 6.31sec and 6.30sec for 1.0m and 1.2m diameter pile respectively. Deflection of 51.4mm and 53.2mm occurred for North ridge earthquake at a time step of 4.22sec and 4.21sec for 1.0m and 1.2m diameter pile respectively.

6.4 RESPONSE OF THE STRUCTURE WITH SOIL-STRUCTURE INTERACTION FOR WIDTH OF 4D



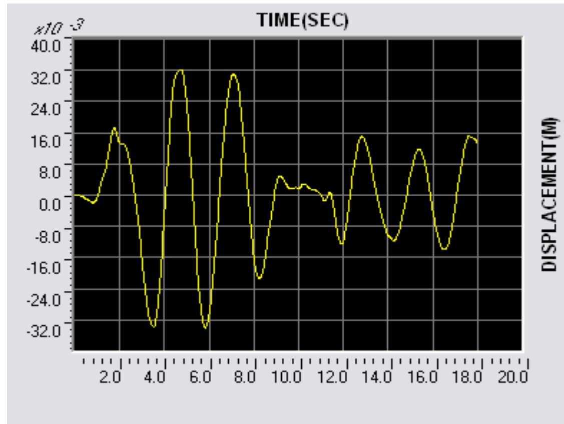
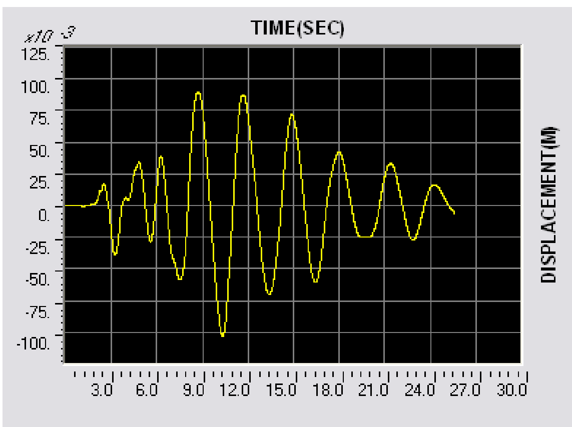


Fig. 6.16 Displacement of 1.0 m dia pile

Fig 6.17 Displacement of 1.2 m pile for El centro Earthquake



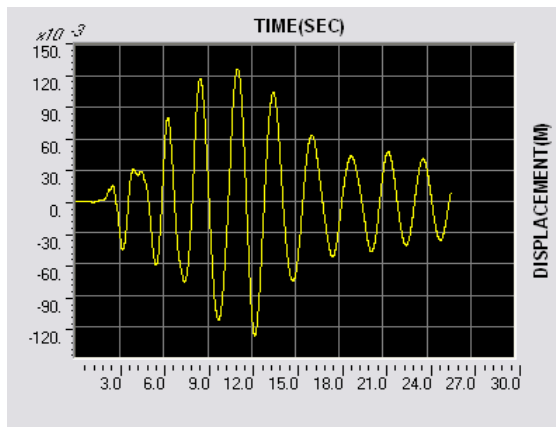
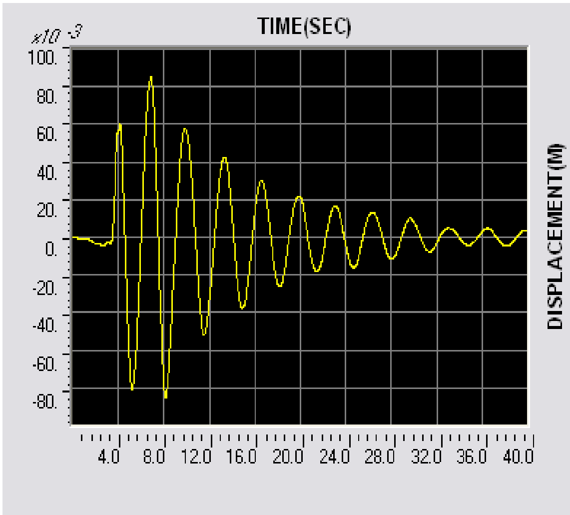


Fig. 6.18 Displacement of 1.0 m dia pile

Fig 6.19 Displacement of 1.2 m pile for Kobe Earthquake



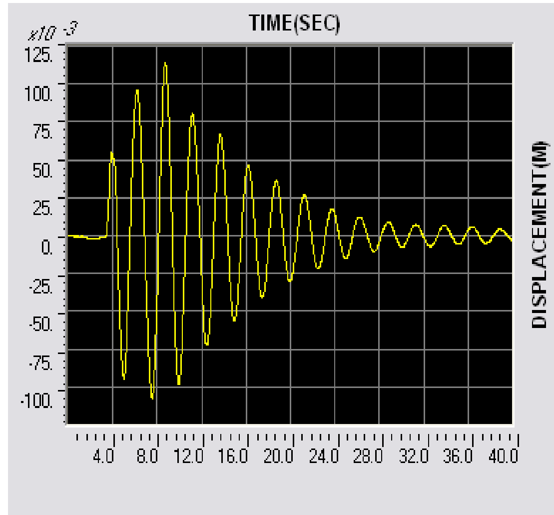


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Fig 6.20 Displacement of 1.0 m dia pile.

Fig 6.21 Displacement of 1.2 m pile for Northridge earthquake

The deflection response of a structure to a particular earthquake motion depends on damping and natural periods. From Fig.6.16 to Fig.6.21 it is observed that the maximum deflection of 37.7mm and 32.1 mm occurred for El Centro earthquake at a time step of 5.26sec and 4.74sec for 1.0m and 1.2m diameter pile respectively. The maximum deflection of 89.8mm and 12.73mm occurred for Kobe earthquake at a time step of 8.70sec and 11.05sec for 1.0m and 1.2m diameter pile respectively. It is also observed that the maximum deflection of 85.3mm and 11.43 mm occurred for Northridge earthquake at a time step of 6.84 sec and 8.74 sec for 1.0m and 1.2m diameter pile respectively. We can notice that the difference in the deformation was due to varying natural periods. Also from the above results we can conclude that longer the natural period greater is the peak deformation.

6.5 CONCLUSIONS

Static analysis is needed before performing dynamic analysis to define deformations generated by time history. From time history analysis it is seen that the maximum deflection of 34.427 cm and 20.974 cm occurred at a time step of 7.04 sec and 10.1sec for 1 m and 1.2 m diameter pile respectively. We can notice that the larger difference in the deformation for 1m and 1.2m pile is due to varying natural time periods.

Longer the natural period greater is the peak deformation. The maximum deformation for 1.2m pile is delayed compared to 1m pile due to larger stiffness and larger mass.

At the beginning of the earthquake with greater acceleration applied recorded deformation has smaller values shows the energy absorbance and damping. After that recorded deformations illustrates acceleration amplification in soil.

Even though the soil structure interaction increases damping, this causes the additional displacement to the overall structure. It was observed that from the displacement wave forms for El Centro earthquake, maximum deflection of 21.1mm and 13.4mm occurred for at a time step of 6.98sec and 10.18sec for 1.0m and 1.2m diameter pile without soil structure interaction. Taking soil structure interaction into account it was observed the maximum deflection of 57.0mm and 49.2mm occurred for El Centro earthquake at a time step of 9.10sec and 13.98sec for 1.0m and 1.2m diameter pile respectively.

CHAPTER 7

SUMMARY & CONCLUSIONS

7.1 SUMMARY

Berthing structures are the facilities constructed in ports for berthing and mooring of vessels, for loading and unloading of cargo and for embarkment and disembarkment of passengers or vehicles. The berthing structures are designed for dead load, live load, berthing force, mooring force, earthquake load and other environmental loading due to wind, waves, currents etc. In the present study layout of jetty for berthing 200000 DWT ship at NMPT was modeled using the software SAP 2000 and StruCAD*3D. Simple experiment was conducted on a single cylinder to know the forces and moments. These forces and moments were compared with the theoretical model of Morrison's equation. Experiment was conducted on a small scale model of berthing structure and the detailed analysis of the berthing structure was done without considering soil structure interaction and with considering soil structure interaction. The maximum wave height of 5.5 m for a period 11 sec was taken for the analysis. For a full cycle of wave the variation of deflection, forces and moments for perpendicular wave directions and different pile diameters was done by Static and Dynamic methods. Dynamic Amplification Factor was calculated by comparing static and dynamic analysis results. Time history analysis was also done for three different earthquake motions with the combination of the wave loading and deflection of the structure was found out.

The wave force acting on a berthing structure depends on the diameter of piles. The total wave force is given by the summation of drag component and inertia component. The drag force depends on the square of the fluid particle velocity and introduces nonlinearity to the excitation force. The higher order terms in the dynamic analysis occur not only because of the velocity-squared term but also due to higher order wave theory, variable submergence of elements near the water surface, etc.

7.2 CONCLUSIONS

- It is seen that maximum force and maximum moment for 1m diameter pile are 853.02 kN and 11485.81 kN-m respectively and for 1.2 m diameter pile are 1184.95 kN and 14646.55 kN-m respectively at a phase angle of 280^0 i.e., perpendicular to the structure. It is seen that the force and moment for 1.2m diameter pile are 23 percent more than the 1m diameter pile without considering soil structure interaction.
- It is seen that top joint deflection is maximum in front row side of the pile and it decreases as it moves back side pile for both 1m and 1.2m diameter pile. And it decreases from 10% to 12% for both diameter piles. The maximum top joint deflection is 1.84 cm for 1m diameter pile. It is observed that deflection decreases as the diameter increases and it decreases about 39.67 % when compared to the smaller diameter without considering soil structure interaction
- It is observed that the values of forces and moments calculated without considering SSI by dynamic analysis are more than that by static analysis. Static results are 2 to 14 % and 1 to 6 % less than dynamic analysis method for 1m diameter and 1.2m diameter piles respectively.
- The Dynamic Amplification Factor (DAF) is calculated from static and dynamic results. The DAF for 1m diameter pile varies from 1.02 to 1.43 and for 1.2 m diameter pile. DAF varies from 1.03 to 1.53. From Dynamic Amplification Factor it is seen that there is increase in response of 43% and 53% for 1m and 1.2m diameter pile respectively due to wave loading compared to static loading.
- From time-history analysis, it is observed that the maximum deflection of 2.11 cm occurred for El Centro earthquake at a time step of 6.98 sec for 1m diameter pile. Maximum deflection of 8.77 cm occurred for Kobe earthquake at a time step of 10.96 sec for 1 m diameter pile and 7.61 cm occurred for

North ridge earthquake at a time step of 8.66 sec for 1 m diameter pile without considering SSI.

- From time-history analysis for soil mass of size 3D, it is observed that the maximum deflection of 57mm occurred for El Centro earthquake at a time step of 9.10sec for 1 m diameter pile. Maximum deflection of 49.2mm occurred for Kobe earthquake at a time step of 6.30 sec for 1.2 m diameter pile and 53.2mm occurred for Northridge earthquake at a time step of 4.22 sec for 1.2 m diameter pile.

- From time-history analysis for soil mass of size 4D and 5D it is observed that the maximum deflection was almost same and there is no much variation in the deflection criteria .
- When soil-structure interaction effects are included, the natural period of the structure will increase.
- As we increase the width and length of the soil mass the deflection of the structure decreases.
- Deflection of the structure is less when analysed without considering soil mass and it is more when analysed by considering soil structure interaction. Therefore it is very important to consider soil-structure interaction in dynamic analysis.
- The natural frequency of the experimental model fairly agreed with the FEM model.
- Inclusion of water and the soil tend to increase the natural frequency of the structure.
- The response at the top of the vertical cylinder is slightly lesser than the applied frequency at the base. This may be due to damping effect of the structure. Further inclusion of soil and water along with vertical member tends to reduce the natural period.
- The damping ratios of vertical member at different experimental conditions are found that the damping ratio increases when structure is in water and soil.

7.3 SCOPE FOR FUTURE WORK

- Carryout the similar studies with breaking wave interaction
- Investigate with different pile sizes and for different shapes
- Testing of full size models with varying soil and water depth.
- In the present analysis the effect of damping provided by the fluid medium is not considered. Taking this factor into account more rigorous analysis can be performed.

APPENDIX I

REFERENCES

Abul-Azm A. G. and Williams A. N. (1988), “Second order diffraction loads on truncated cylinders”, *Journal of Waterways, Harbours and Coastal Engineering Division,* ASCE, vol.114, No.4, July.

Allam, M.M, Subba Rao, K.S., Subramanya, I., (1991).“Frame soil interaction and winkler model”, *Proceedings of Institution of Civil Engineers,* Part 2. pp. 477–494.

Baidya, D.K., Sridharan, A. (1994). “Stiffnesses of the foundations embedded into elastic stratum”, *Indian Geotech Journal*, 24(4), 353–367.

Bao-lei Geng (2010), “A time domain analysis of wave force on small scale cylinders of offshore structures”, *Journal of Marine science and technology,* Vol 18, No 6,pp 875-882

Banerjee (2003). “Effect of soil– structure interaction on building frames”, *Proc Indian Geotechnical Conference,* 1, 123–136.

Bernard Molin, (1979), “Second – order diffraction loads upon three dimensional bodies”. Applied ocean Res., 1, 197-202

Bowles, J.E. (1997). “Foundation Analysis and Design”, McGraw Hill, New York.

Brown,C.B., Laurent, J.M.,Tilton,J.R., (1977). “Beam-plate system on winkler foundation”, Journal *of Engineering Mechanics,* 103(4), 589–600.

Casciati, S. and Ronaldo, I., Borja (2004). “Dynamic FE analysis of South Memnon Colossus including 3D soil–foundation–structure interaction”, *Computers and Structures,* 82, 1719–1736

Chakrabarti S. K. (1972), "Nonlinear Wave Forces on a Vertical Cylinder " *Journal of Hydraulics Division* , ASCE, Vol.98.

Chakrabarti S. K., Allan L. Wolbert, William A. Tam (1976), "Wave forces on vertical circular cylinder", *Journal of the Waterways, Harbours and Coastal Engineering Division ASCE* ,vol 119, No 3, July

Chakrabarthi, S, (2005). "Handbook of Offshore Engineering", volume I&II, ELSEVIER Publication.

Chakrabarti S. and Gupta A. (2007), "Steady Wave Drift Force on Basic Objects of Symmetry", Vol.92, WIT Press.

Chamecki,C.(1956). "Structural rigidity in calculating settlements", Journal of *Soil Mechanics and Foundation Engineering* ,82(1), 1–19.

Chandrashekhar, A., Jayalekshmi, B.R. and Venkataramana, K. (2005). "Dynamic soil structure interaction effects on multistoreyed RCC frame" Proceedings of the International conference on Advances in structural dynamics and is applications, ICASDA, 454-467.

Chen H. L., Shah S. P, and Keer L. M. (1988). "Dynamic Response Of Shallow-buried Cylindrical Structures", ASCE, Vol. 116, No.1, Paper No. 24266.

Christina J. Curras, Ross W. Boulanger, Bruce L. Kutter, and Daniel W. Wilson (2001). "Dynamic experiments and analyses of a Pile-group-supported structure", ASCE, Vol. 127, No. 7, Paper No. 22310

Chopra, Anil K. (2001), "Dynamics of structure". Pearson Education (Singapore) Pte.Ltd, Indian Branch, 482, F.I.E, Patparganji.

Christopher, B. (2002). "Soil structure interaction in the time domain using half space Green's function", *Soil Dynamics and Earthquake Engineering*, 22, 283-295

.

Cronin Kand Galgoul Roesset J M (1981). "Effect of Nonlinear Soil Behavior on Inelastic Seismic Response of a Structure", *International journal of Geomechanics,* 36, 104-114.

Dan M.G and Roger G.G (2002). "Stochastic Finite-Element analysis of seismic Soil–Structure Interaction", *Journal of Engineering Mechanics,* 128(1), 66-77.

Dennis, R.W.and Chen (1984). "Geotechnical Earthquake Engineering Handbook", *Mc Graw Hill,* NewYork.

Desai, C.S., Abel, J.F. (1987). "Introduction to the finite element method: a numerical method for engineering analysis", CBS Publishers, New Delhi.

Dawson, T.H. (1989), "Offshore Structural Engineering". Prentice Hall, New Jersey.

Dutta, S.C. and Rana Roy (1999). "A critical review on idealization and modeling for interaction among soil–foundation–structure system". *Computers and Structures*, 80, 1579-1594.

Dutta, S.C. and Rana Roy (2002). " Dynamic analysis of soil–foundation–structure system". *Computers and Structures*, 80, 1579-1594

Eatock Taylor R and S M Hung (1987), "Second-order diffraction forces on a vertical cylinder in regular waves", Applied Ocean Res., 9, 19-30

Gazetas,. (1991). "Formulas and charts for impedances of surface and embedded foundations", *Journal of Geotechnical Engineering,* 117(9), 1363–81

Ghosh B and Madabhushi S.P.G. (2006). "Centrifuge modeling of seismic soil structure interaction effects" *Nuclear Engineering and Design*.

GovindaRaju,L.,Ramana,G.V.,Hanumantharao.C.,Sitharam.T.G.,(2004)."Site-specific ground response analysis", *Current Science, 87*, 1354-1362.

Graham A., Clive A. Greated and Stephen H. Salter Dixon (1979), “Wave Forces on Partially Submerged Cylinders”, *Journal of the Waterways, Harbours and Coastal Engineering Division, ASCE* vol 118, No.2, Paper No. 25436

Ghosh, B., Madabhushi, S.P.G., (2003). “Effects of localized soil in homogeneity in modifying Seismic Soil Structure Interaction”, *Paper for ASCE 16th Engineering Mechanics Conference.*

Gudehus G, R.O. Cudmani, A.B. Libreros-Bertini and M.M. Buhler (2004). “In-plane and anti-plane strong shaking of soil systems and structures” *Soil Dynamics and Earthquake Engineering*, 24, 319–342.

Grue (1997), “Studies on pressure distribution around cylindrical structures”, Journal of marine science ,pdf , pp 456-472.

Harihara Raman and Paruchuri Venkatanarasaiah (1976), “Forces due to Nomlinear Waves on Vertical Cylinders”, *Journal of the Waterways, Harbours and Coastal Engineering Division, ASCE, vol* 206, No.1, paper No. 31345.

Havelock (1955) , “Wave forces on piles-A diffraction approach”,Proceedings of the royal society, A mathematical,physical and Engineering science.

Hetenyi, M. (1946).“Beams on elastic foundations”, University of Michigan Press, Ann Arbor.

Hosseinzadeh, N.A. and Nateghi, F. (2004). “Shake table study of soil structure interaction effects on seismic response of single and adjacent buildings” *13th World Conference on Earthquake Engineering Vancouver, B.C., Canada* Paper No. 1918, 165–186.

(http://peer.berkeley.edu/nga/flatfile.html) (Last accessed in 15/02/2012).

Huaren Dou and Peter M.Byrne (1996). "Dynamic Response of Single Piles and Soil-Pile Interaction", *Geotechnical journal* , No.33, pp 80-96.

IS 4651-(part II-1969, part III-1974, part IV-1989, Part V-1980), Indian Standard Code of Practices for the Design of Port and Harbour Structures, BIS New Delhi.

IS 875 Indian Standard (1987) Code of Practice for Wind Loads, BIS New Delhi

IS 1893 Indian Standard (Draft Code) (2000) Criteria for Earthquake Design of .Structures, BIS New Delhi.

Jaiswal (2011), " Comparative study of fixed base and base isolated buildings using seismic analysis" International journal of Earth science and engineering ,Vol 04, No 06spl, oct 2011, pp 520-525.

Jen-Cheng, Spyrakos, C.C. and Xu, C. (2003). "Dynamic analysis of flexible massive strip–foundations embedded in layered soils by hybrid BEM–FEM", *Computers and Structures*, 82, 2541–2550.

Jessica Laine handrix (2003), "Dynamic analysis techniques for quantifying bridge pier response to barge impact loads", M.Tech Thesis, University of Florida

John W. Gaythwaite (1981) "Hydrodynamic of Offshore Structure", Computational Mechanics Publication.

Kameswara Rao, N.S. (1998). "Vibration analysis and foundation dynamics", Wheeler Publishers, Mumbai

Kawano,K.,(1979). "Application of the finite element method for dynamic analysis of soil structure interaction", *PhD Thesis*, Kyoto University, Japan

Kenichiro Minami,Hiroshi Yokata,Tokahiro Sugano (2002), "Damage investigation and structural analysis of piled berthing structures during 1995 Hyogoken-Nambu earthquake", *Proceedings of the twelth international offshore and polar Engineering conference ,Kitakyushu,Japan,* May 26-31, 2002, pp 507-513.

Kim and St. Issacson (2006), "Wave Runup around Large Circular Cylinder", Journal *of the Waterways, Harbours and Coastal Engineering Division ASCE*.

Kimura, Y., (2003). "Seismic Performance evaluation on Nonlinear Dynamic soil structure interaction systems", *Ph.D Thesis,* Kagoshima University, Japan.

Kohji Tokimatsu, Hiroko Suzuki and Masayoshi Sato (2004). "Effects of inertial and kinematic interaction on seismic behavior of pile with embedded foundation" *Soil Dynamics and Earthquake Engineering,* 25, 753–762.

Kramer, S.L. (2003). "Geotechnical and Earthquake Engineering", Pearson Education

Kutanis, M. and Elmas, M., (2001). "Non-Linear Seismic Soil-Structure Interaction Analysis Based on the Substructure Method in the Time Domain", *Turk Journal of Engineering and Environmental Science,* 25, 617 -626.

Lappo Raymond. and keplun L.F. (1975). "Empirical evaluation of inertial soil-structure interaction effects", *Report No. PEER-98/07*.

Lei, I.K., Geng P.T. (2010). "Structures–foundation interaction analysis", *Journal of Structural Engineering ASCE,* (11), 2413–31

Malenica S. Clark P.J.,Molin B., (1995), "Wave and Current Forces on Vertical Cylinder Free to Surge and Sway", *Applied Ocean Research* 17, pp 79-90.

Mehrdad (2004). "Interaction of continuous frames and soil media", *J Structural Engineering ASCE*, (5), 13–43.

Mamoon S. M., Kaynia A. M. and Banerjee P. K., (1990) "Frequency Domain Dynamic Analysis of Piles and Pile Groups" *Journal of Engineering Mechanics*, Vol. 116, No. 10., ASCE, ISSN 0733-9399/90/0010-2237 Paper No. 25114.

Mac Murdo and Madheswaran C.K. (1989). Wave Forces on Single and Group of Cylinder in Response to Two Wave Spectra" Third National Conference on Docks and Harbour Engineering, 6-9 Surathkal, pp 129-139.

McCamy and Fuchs, (1954), "Wave loading on structures", *Journal of marine science,* PDF, pp 234-246.

Mehrdad H.and ,Fuladgar A. (2004). "Dynamic soil-structure interaction effects on the seismic response of asymmetric buildings", *Soil Dynamics and Earthquake Engineering*, 24, 379-388.

Milos Novak and Bahaa El Shamouby, (1984) "Evaluation of Dynamic Experiments on Pile Group", Vol. 110, No. 6 ASCE, Paper No. 18931.

Micheal de St. Issacson (1978), "Vertical Cylinders of Arbitrary Section in Waves", *Journal of the Waterways, Harbours and Coastal Engineering Division ASCE*

Micheal de St. Issacson (1978), "Wave Runup Around Large Circular Cylinder", *Journal of the Waterways, Harbours and Coastal Engineering Division ASCE.*

Morris, D. (1966). "Interaction of continuous frames and soil media", *J Structural Engineering ASCE*, (5), 13–43.

Morrison, J.R., O'Brien, M.P, Johnson, J.W., Schaaf, S.A. (1950). "The Force Exerted by Surface Waves on Piles". *Petroleum Transaction*. Vol. (189), pp149-157.

Noorzaei,J., Godbole, P.N., Viladkar, M.N. (1993). "Nonlinear soil– structure interactionof plane frames—a parametric study", *Computers and Structures*, 49(3), 561–569.

Oliveto, G. A. and Santini (1995). "Time Domain Response of a one dimensional soil-structure interacting model via complex analysis", *Engineering Structures,* 18, 425-436.

Onu (1996). "Equivalences in the Soil-Structure Interaction", *Computers & Structures,* 58. No. 2, pp. 367-380.

Poluos (1975). "Dynamic Interaction Analysis of Soil and End Bearing Pile sing Boundary Element Model Coupled with Finite Element Model", *International Conference on Computational Intelligence, Communication Systems and Network.* pp 234-244.

Per Brunn, "Port Engineering", (1993). Gulf Publishing Company, Hudson, Texas.

Quinn A.D. (1968). "Design and Construction of Ports and Marine Structures", McGraw Hill, New York

Rajaram T.V. and Sassi Kumar P.K. (1985). Design and Construction of Cochin Oil Terminal, first national conference on docks & harbour engineering, 27-29 December Mumbai, India, pp 47-57.

Roberto Cairo, Enrico Conte and Giovanni Dente, (2005). "Interaction Factors for the Analysis of Pile Groups in Layered Soils" 10.1061 ASCE 1090 131:4(525)

Roger G.G and Dan M.G (2002). "Stochastic Finite-Element analysis of seismic Soil–Structure Interaction", *Journal of Engineering Mechanics,* 128(1), 66-77.

Roy, R., Dutta, S.C. (2001). "Differential settlement among isolated footings of building frames: the problem, its estimation and possible measures", *International Journal of Applied Mechanics and Engineering* , 6(1),

Satish S.Jadhav (2004). "Software Approach for Static and Dynamic Analysis of Template Structure using FEM." M.Tech thesis, National Institute of Technology Karnataka, Department of Applied Mechanics and Hydraulics, Surathkal.

Shintaro, Y., Koichi, K., Nozomu, Y., and Hiroshi, M., (2004). "Interactive behavior of soil–pile-superstructure system in transient state to liquefaction by means of large shake table tests" *Soil Dynamics and Earthquake Engineering*, 24, 397–404

Shore Protection Manual (1984) (SPM), US Army Corps of Engineers, CERC, Washington, D.C.

Skjelbreia and Hendrickson (1960), "Experience in computing wave loads on large bodies", *Journal of Waterways, Port, Coastal and Coastal Engineering Division,* ASCE,vol 104, No ww2, pp147-161.

Shintaro, Y., Koichi, K., Nozomu, Y., and Hiroshi, M., (2004). "Interactive behavior of soil–pile-superstructure system in transient state to liquefaction by means of large shake table tests" *Soil Dynamics and Earthquake Engineering*, 24, 397–404

Srinivasa Rao, P., Rambabu, K.V. and Allam, M.M. (1995). "Representation of soil support in analysis of open plane frames", *Computers and Structures,* 56, 917-925.

Subba, K.S., Rao, H., Sharada, B.V. (1985). "Interaction analysis of frames with beam footing". *Proc Indian Geotech Conference,* Roorkee, 389–95.

STRUCAD-3D Reference Manual (2001). Research Engineering Work Ver. 4.2.

Surya Rao S.,Muralidhar K. and Bijay Kumar Pandey (1997), "Forces on Vertical Circular Cylinder due to Regular and Random Waves", *Second Indian National Conference on Harbour and Ocean Engg., INCHOES* 97, December 7-10

Takahito Inoue Masayoshi Sato, Masoud Mohajeri, Yoshiyuki Mohri and Kazunobu Izumi (2004). "Three dimensional analysis of soil-pile-structure model in a shake table test", 13th *World Conference on Earthquake Engineering Vancouver, B.C., Canada* Paper No. 2518

Terzaghi, K.V., (1955). "Evaluation of coefficients of subgrade reaction", *Geotechnique*, 5(4), 297–326.

Tull (1997), "Fluid Forces on Oscillating Cylinders", *Journal of Waterways, Port, Coastal and Coastal Engineering Division* ASCE, vol.104, No.12389

Yang, Y.B., Kuo, S.R. and Liang, M. (1996). "A simplified procedure for formulation of Soil-structure interaction problems", *Computers & Structures*, 60, No. 4, pp. 511-520.

Yingcai Mengi and Salih Kirkgoz M, (2009) "Dynamic Response of Caisson Plate to Wave Impact", Vol. 112, No. 2 ASCE, Paper No. 20473.

Zhang Hong-Wu (1995)."Dynamic finite element analysis for interaction between a structure and two-phase saturated soil foundation", *Computers & Structures*, 56, No. 1, pp. 49-58.

Zienkiewicz, O.C., Taylor, R.L. (1989). "The finite element method", McGraw-Hill, London

APPENDIX II

PUBLICATIONS BASED ON THE PRESENT RESEARCH WORK

International Journal

1. **Shanthala** ,Subba Rao, Katta Venkatramana",Time history analysis of berthing structures including soil structure interaction" International **J. of Advances in Science and Technology (IJAST), Silicon** Valley Publishers,UK,Vol 3,No 02,August 2011, pp 77-85
2. **Shanthala** ,Subba Rao, Katta Venkatramana,"Analysis of Berthing structures for wave induced forces"**, International J. of Earth science and Engineering, Elsevier Science** Vol 04 No 01 February 2011, pp 112-121.

National Conferences

1. **Shanthala** ,Subba Rao, Katta Venkatramana,Pruthviraj U,"Experimental investigation of wave forces on vertical cylinders", Fourth Indian National Conference on Harbour and Ocean Engineering **(INCHOE-07)** held at N.I.T.K., Surathkal during 12th to 14th December, 2007, Vol. 2 pp 639- 642

2. **Shanthala** ,Subba Rao, Katta Venkatramana ,"Investigation on Non breaking wave forces on slender vertical cylinder"National Conference on advances on Civil engineering. **(ACE 2008)** ,held at AEC Bhatkal during 9th &10th March 2008 ,pp 54-58

3. **Shanthala** ,Subba Rao, Katta Venkatramana ,"Dynamic Analysis of Berthing Structures For Wave Induced Forces", National conference on 'Sustainable Water Resources Management' **SWaRM 2010** held at NITK, Surathkal, during 7th -9th January 2010

4. **Shanthala** ,Subba Rao, Katta Venkatramana,Harish,"Wave response Analysis on Berthing Structure for Maximum Wave", Recent Development in Civil Engineering **(RDCE-2011),**held at MIT Manipal during 13-14 January 2011