DESIGN OF PILE FOUNDATIONS FOR EARTHQUAKE LOADS

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With specialization in Soil Dynamics)

by

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JUNE, 2016

CANDIDATE'S DECLARATION

I hereby declare that the work which is being presented in this dissertation entitled, "**DESIGN OF PILE FOUNDATIONS FOR EARTHQUAKE LOADS**", in the partial fulfilment of the requirements for the award of the degree of **Master of Technology** in Earthquake Engineering, with specialization in Soil dynamics, submitted in the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out for a period from July 2015 to May 2016 under the supervision of **Dr. B. K. Maheshwari**, Professor, Department of Earthquake Engineering, Indian Institute of Technology Roorkee.

Date:

Place: Roorkee **(Devendra Shrimal)**

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

The response of pile foundation is complex under real earthquake motion. In analysis of design forces for pile foundation, applied motion parameter play significant role. Effect of PGA and predominant frequency plays important role in design parameters.

A three-dimensional soil-pile system is numerically formulated in finite difference based software FLAC^{3D}. For nonlinear behaviour of soil, Mohr-Coulomb model is assigned to soil media. Under dynamic loading free field boundary condition is applied to soil-pile system to absorb the reflecting waves.

Effect of kinematic interaction analyzed for single pile under Bhuj (2001) earthquake motion. PGA of the applied motion significantly changes behaviour of soil –pile system due to nonlinear soil model. Spectrum compatible ground motions of Bhuj earthquake , scaled for different earthquake zones of India (viz., 0.10g, 0.16g, 0.24g and 0.36g) are used for effect of PGA. Response of pile foundation is significantly increased if the predominant frequency of exciting motion is near to fundamental frequency of soil-pile system.

Pile group response is examined under increasing PGA. Behaviour is compared with single pile and found displacement slightly high in single pile and lateral shear significantly high for pile group near pilecap. Response of pile is get amplified due to consideration of superstructure.

ACKNOWLEDGEMENT

I wish to express my deep sense of gratitude to **Dr. B. K. MAHESHWARI**, Professor, Department of Earthquake Engineering, Indian Institute of Technology, Roorkee, for his timely suggestions, advice, inspiration and constant encouragement throughout the course of this literature review work. His effort in thoroughly reading the manuscript and invaluable guidance are greatly acknowledged.

I would sincerely thank all of my fellow friends and the staffs of Department of Earthquake Engineering, IIT Roorkee, who have helped me in every possible ways not only in the successful completion of my dissertation work but also in making the time spent in these one year a memorable one.

Finally, I owe a lot to my beloved parents and other family members for their constant support and enthusiasm at every stage of my life to make this possible.

Place: Roorkee

Date: Devendra Shrimal

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Chapter 1

1 INTRODUCTION

1.1 GENERAL

Pile foundation is an important structure to transfer the heavy load under loose soil condition. Pile foundations perform well under earthquake load so they are preferred in seismically active region. Generally it is used for complex geologic settings and all kinds of load conditions, especially for loose soil foundation. Thus pile foundation is a commonly applied deep foundation for civil structure.

The analysis of such a problem is complicated because of the complex stress-strain behaviour of the soil surrounding the piles.

- The behaviour of pile foundations in non-liquefiable soil under earthquake motion is significantly influenced by the changeability in the soil and seismic design factors. Displacement in pile foundation in soil is significant by inertial force from superstructure.
- Tall buildings and bridges on loose to medium dense sands are normally construct on piles to minimize settlements because the soil surface ground layers are not stiff enough to support the heavy structures. In an earthquake if these loose sands are saturated, they lose strength as excess pore water pressure is generated and the soil tends to liquefy.

1.2 PILE FAILURE

Observations from the recent earthquakes show that lateral earthquake induced forces are high and result in substantial damages for pile foundations. The loose soil (e.g., with a relative density of 40% or less) under un-drained condition if excited with dynamic loading, may liquefy, whereas the soil with very high relative density (90% or high) or cohesive soil in unsaturated condition is less susceptible to liquefaction.

Recent earthquake shows that lateral seismic forces are significant and result in considerable damage in pile foundation. The magnitude 7.4 Off-Miyagi Prefecture Earthquake showed in many of cases, damage to pre-stressed concrete piles, that were mainly caused by earthquake prompted vibration of the superstructure, according to Sugimura (1981). In two instances, total structural collapse followed, and in others, minor to moderate structural damage was sustained. The soil conditions where pile damage occurred ranged from sands to silt to clays to peat, but liquefaction was not considered to be a contributing factor to these cases. The failure modes included bending-shear failure at the pile head, and complete crushing at the pile head. Notably, the most heavily damaged piles have been found at the structure's perimeters, suggesting that rocking due to inertial loads from the structure overstressed the piles.

Fig.1.1 Damage of pile due to ground displacement Niigata 1964 [Finn and Fujita (2002)]

Damage to a pile foundation under a building in Niigata earthquake is caused by about 2 m of ground movement is shown in Fig.1.1. During the Kobe Earthquake seaward movement of quay wall approximately 1 m towards sea accompanied by lateral spreading of the backfill soils resulting in a number of cracks on the ground inland from the waterfront.

1.3 SCOPE OF THE STUDY

The pile foundation is important under earthquake loading. Thus neglecting the effect of different parameter (viz. PGA of earthquake loading, predominant frequency of earthquake and effect of superstructure) can lead to significant damage to pile foundation. Failure of structures is due to inaccurate estimation of forces on pile foundation. Therefore in this study, effect of above parameters are analysed on single and pile group.

1.4 OBJECTIVE OF THE STUDY

- To review the various design approaches for pile foundation under seismic loading.
- To investigate the effect of increasing PGA and predominant frequency of different earthquake on the response of a single pile.
- To examine the effect of increasing PGA of earthquake motion on a 2×2 pile group .
- To examine the effect of superstructure on response of pile group under earthquake loading.
- To analyze the behavior of pile group with respect to single pile under earthquake loading.

1.5 ORGANIZATION OF THE REPORT

The report has been documented in the following manner:

- Chapter 1.Discussion on the importance of pile foundation under seismic load and failure of pile foundation during the past earthquakes.
- Chapter 2. Literature review was performed related to design approach of pile foundation under seismic loading.

- Chapter 3. Modelling of soil-pile system and analysis methodology is discussed in detail using FLAC^{3D}.
- Chapter 4. Validation of static and dynamic analysis of the model developed in FLAC^{3D}.
- Chapter 5. To study the effect of PGA and predominant frequency of single pile under real earthquake loading.
- Chapter 6. To study the effect of PGA and superstructure on response of pile group.
- Chapter 7. Conclusion from the present study and scope for future work outlined.

Chapter 2

2 LITERATURE REVIEW

Behaviour of pile foundation is critical under earthquake load. Extensive work is done on pile foundation under seismic load. Various methods have been employed to analyze the pile foundation such as simplified linear elastic method, pseudo static approach, three-dimensional rigorous analysis and continuum approach.

2.1 SIMPLIFIED METHODS

2.1.1 Novak Model (1974) for Linear Analysis

The Assumption of this model is that Pile material linear elastic and pile is perfectly connected soil. For single pile stiffness and damping in sliding, rocking and cross rocking calculated using Novak's coefficient.
In linear analysis calculating stiffness and damping for group using group interaction factor and formed equation of motion by solving displacement then strain in horizontal, rocking and cross rocking.

2.1.2 Chandrasekaran Approach (1974)

In this approach pile is considered in discrete mass. The interaction effect of super structure is not considered rather than structure considered as a concentrated mass. Soil is assumed to linear Winkler's spring. The soil reaction is separated into discrete parts at the center point of the masses and soil modulus variation is considered both constant with depth and linearly varying depth.

For calculating fundamental time period non dimensional frequency factors used. In sand stiffness is linearly with depth and in clay soil stiffness constant withy depth. Relative stiffness factor calculated and from graph frequency factors obtained.

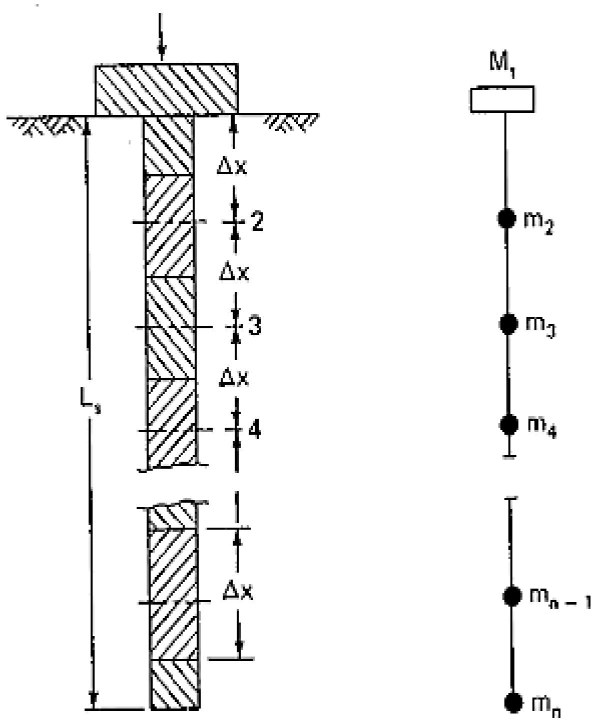


Fig.2.1: Pile Structure Idealization (Chandrasekaran 1974)

2.1.3 Seismic Pile Group Structure Interaction (Gazetas et al. 1992)

Soil-foundation-structure interaction analyses under earthquake motion can be done in three steps:

Apply earthquake excitation at bed rock of soil-foundation and obtained surface motion. This is called ‘foundation input motion’ includes translational as well as rotational component. For piles in group interaction factor for seismic loading is used.
Determine dynamic impedance (springs and dashpots) in sliding (Kx, Ky), rocking (Krx, Kry) and cross-sliding-rocking (Kx-ry, Ky-rx). For piles in group dynamic interaction factor is used.

Seismic response of the superstructure supported on springs and dashpots and subject to ‘foundation input motion’

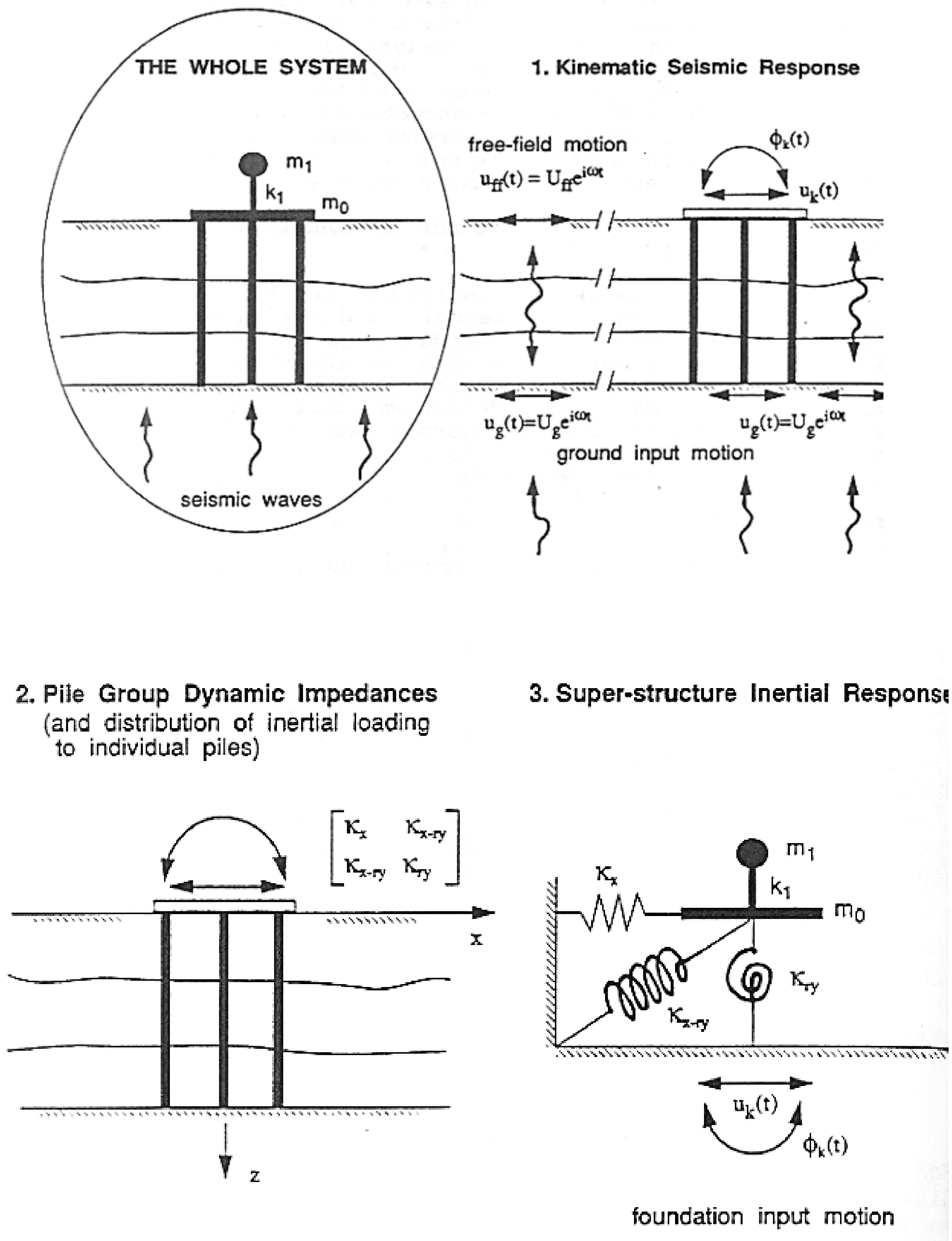


Fig.2.2 : General Procedure for Seismic Soil-Pile Interaction (Gazetas et al. (1992))

2.1.4 Characteristics Load Method (Duncan and Brettmann, 1996)

This method was developed by performing nonlinear P-Y analyses for a wide range of free-head and fixed-head piles in sand and clay. Representing the results in non-dimensional relationships between load-deflection equation, moment-deflection equation and load-moment equation are given.

$$\left(\frac{y_t}{D}\right) = a\left(\frac{P_t}{P_c}\right)^b \tag{2.1}$$

$$\left(\frac{y_t}{D}\right) = a\left(\frac{M_t}{M_c}\right)^b \tag{2.2}$$

$$\left(\frac{P_t}{P_c}\right) = a\left(\frac{M_{max}}{M_c}\right)^b \tag{2.3}$$

For these equations different values of constants 'a' and 'b' for each equation are given. The input parameters are Pt and Mt loading condition. Bhattacharya et al. (2012) used this method to check against lateral deflection and limiting moment capacity of pile under liquefiable soil.

2.1.5 Pseudo Static Approach (Tabesh and Polous, 2001)

The method involves two steps:

1. A free field site response is to obtained time history of the surface motion and displacement of the soil mass along the pile length. This was done using SHAKE program. The displacement taken as static soil movement along the depth profile and from surface motion the spectral acceleration is calculated.
2. Pile is exposed to simultaneous application of a lateral force (mass*spectral acceleration) at its head and soil movement profile, and maximum pile moment and shear obtained.

Time period of the structure calculated by reducing superstructure as a cap-mass and obtain spectral acceleration correspond to time period.

$$T = 2\pi\left(\frac{cap-mass}{K}\right)^{0.5} \tag{2.4}$$

When cap-mass large the use of maximum spectral acceleration gives conservative result. When cap-mass small the natural period small and not under the range of dominant period of surface motion. Simple gross assumption of structure mass as pile cap-mass in this method needs cautious observation as the eccentric superstructure weight has distinct effect on the response of pile.

2.2 FINITE ELEMENT METHODS

There are plenty of studies has been done three-dimensional simulation of soil-pile system.

2.2.1 Haldar and Babu (2010)

In their work they simulated pile as a structural element and soil as a continuous media. The model validated against centrifuge experimental studies. The modal simulation is done in finite difference based software FLAC. Two pile failure mechanisms have been examined, bending and buckling. Material of pile and dimension of pile have significant effect the failure mechanism of pile. Failure modes defined by varying various parameters, predominant frequency, amplitude etc.

2.2.2 Bentley and EI Naggar (2000)

Single pile has been analyzed under the kinematic seismic loading. Three-dimensional finite element program ANSYS is used to model soil-pile system. Contact element is used to consider separation between soil and pile. Acceleration and Fourier amplitude slightly amplified in case of elastic soil. Effect of soil plasticity and separation have been analysed on response of pile under Loma Prieta (1989) earthquake motion.

2.2.3 Maheshwari et al. (2004)

In this study a pile group has been modelled by Three-dimensional finite element method. An advanced soil model with plasticity effect, HiSS(Hierarchical single surface) is used to simulate soil media. The soil-pile system analysed under harmonic and transient vibration. It is observed that effect of soil nonlinearity is more at low frequency of excitation and negligible at higher frequency. Dynamic stiffness decreases with nonlinearity of soil.

2.3 DESIGN APPROACHES AND CODAL PROVISION

Different approaches and codal provisions discussed in this chapter.

2.3.1 Seismic Pile Design

Different Approach

- Soil as a continuum with Linear Elastic properties and correctly representing Damping as well as Soil layer Resonance
- Discrete Approach Soil pile system as lumped masses and spring dashpots, Nonlinearity and Inelastic properties can be considered but radiation damping is not adequately represented
- Finite Element Method

2.3.2 Codal Provision

- IS 9716-1981 (Reaffirmed 2003)
  For design of pile foundation specially in earthquake zone
  IS 9716 suggests free and force vibration lateral load test to evaluate response of soil pile system under dynamic load and to obtain soul-pile stiffness, soil modulus, natural frequency, time period and damping characteristics of soil pile system. Typical acceleration records obtained using acceleration pick-ups for sensing the vibration. Imparted dynamic force is calculated for given eccentricity and frequency.

$$F_0 = mew^2 \tag{2.5}$$

Where ω is the forcing frequency from forced vibration.
m is eccentric mass
e is eccentricity

- IS 1893-2002 PART 1

  Design lateral load for the pile foundation is calculated by horizontal seismic coefficient.

While using code specific spectra, the horizontal seismic coefficient shall be calculated by

$$A_h = \left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)\left(\frac{I}{R}\right) \tag{2.6}$$

R Response reduction factor

I Importance factor

Z Zone factor

S_a/g : Spectral acceleration coefficient the sites for the time period calculated by the following (IS 1893-2005 PART IV) $T = C_t\sqrt{\frac{W_t H}{E_S A g}}$

Wt: Total weight of Structure

H: Total structure height from the base

A: Cross-section area at base

g: Gravity Acceleration

E_s Modulus of elasticity of material

C_t: Coefficient depending upon the slenderness ratio of the structure

Chapter 3

3 MODELING AND ANALYSIS

In this chapter detailed procedure of the modeling of soil pile system using finite difference based software package FLAC^{3D} (Fast Lagrangian Analysis of Continua) is given.

Single and pile group (2x2) are considered for dynamic analysis under earthquake motions. In this chapter different parts of the numerical model such as soil, pile, boundary condition and earthquake loading are explained in detail. The methodology adopted for nonlinear dynamic analysis is also discussed.

3.1 PHYSICAL MODELING

In the present study a single pile and a 2x2 pile group with two different spacing (i.e. s/d=3, s/d=5) are considered. Piles are taken as floating pile. Kinematic forces are considered due to seismic forces but inertial forces due to superstructure are neglected initially.

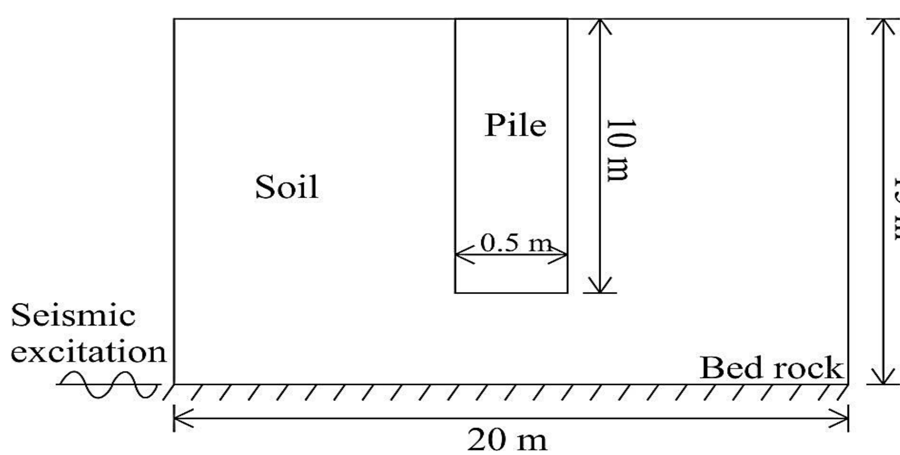


Fig.3.1: Geometry

Then superstructure is modelled as lumped mass to analyze the effect of structure. The geometry of pile is square in cross section. Pile is considered square in shape and cylinder ratio of pile length/side is 20. Near pile finer meshing is done as far away from pile coarser meshing is done. Pile soil model is shown in Fig.3.1.

3.2 FINITE DIFFERENCE SOFTWARE, FLAC^{3D}

FLAC^{3D} is based on explicit finite difference solving scheme to study the behaviour of three-dimensional continuous medium. To simulate mathematical model of a problem general strain and motion laws with appropriate constitutive model are provided in FLAC^{3D}. The output formulation is combination of partial differential equation, relating stress and strain variables, are to be solved for particular problem.

Three different approaches have been employed in FLAC^{3D} to solve a formulation:

1. Finite-difference approach
   Variation of the variables over finite space and time intervals are assumed to be linear.
2. Discrete-model approach
   The representation of continuous medium by discrete equivalents and forces are concentrated on grid point of three-dimensional mesh.
3. Dynamic solution approach
   In this the inertial term is taken into consideration in equation of motion to reach equilibrium state.

By these approaches the continuum motion laws are changed into discrete forms of Newton’s law at grids. The resulting ordinary differential equations are solved to adopt the finite difference approach.

3.3 PILE AND SOIL ELEMENTS

FLAC^{3D} contains an automatic 3D grid generator in which grids are created by manipulating and connecting pre-defined shapes such as bricks, cylinder, wedge and pyramid. In this study soil is modeled using radtunnel element in which radially mesh is generated around the perimeter of the square-shaped tunnel.

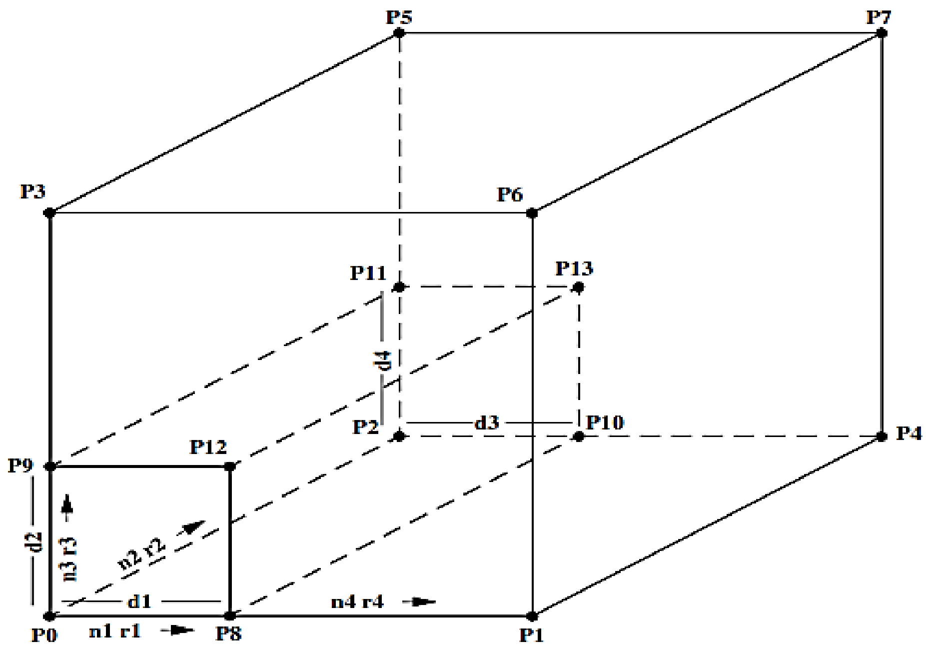


Fig.3.2: Radially graded mesh around parallelepiped-shaped tunnel radtunnel

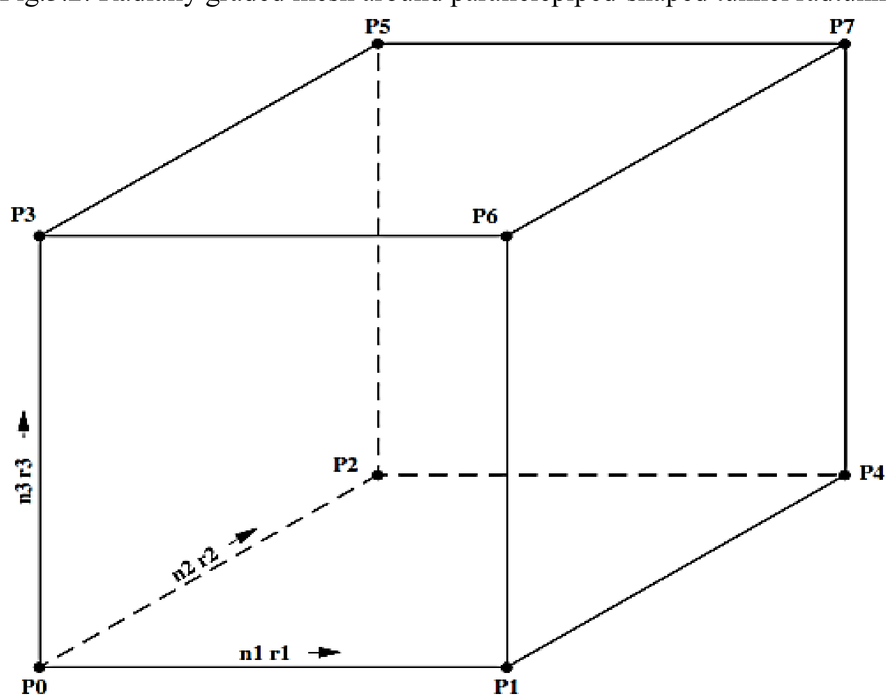


Fig.3.3: Brick Mesh-brick

Fig.3.2 shows the element numbering for a radtunnel mesh used to mesh the soil mass around pile. In this figure p0, p1....p13 shows reference corner points of the shapes, n1, n2, n3 and n4 number of zones in their respective direction and r1,r2,r3 and r4 specify ratios that is used to space zones with increasing or decreasing geometric ratio. Pile is modeled using brick element shown in Fig.3.3.

3.4 BOUNDARY CONDITION

Before carrying out dynamic analysis the model was brought in mechanical equilibrium under gravity loading. For static analysis the boundary condition was considered as fixed at base from x y z direction assuming bedrock.

The lateral sides of the mesh were taken far enough from the pile to avoid the effect of boundary. On other planes at x and y are fixed in normal direction of plane. After doing the static analyses the boundary condition was changed for dynamic analysis.

Modeling of geo-mechanics problems involve media which, are better represented as unbounded. Deep underground excavation are normally assumed to be surrounded by an infinite medium, while surface and near surface structures are assumed to lie on half space. Numerical methods relying on the discretization of a finite region of space require that appropriate conditions be enforced at the artificial numerical boundaries. In static analysis, fixed boundaries can be placed at some distance from the pile foundation. But in solution of dynamic problems such boundaries will create reflection of waves that propagating outward toward boundary. The use of large model can decrease the problem, since damping of material model will absorb most of reflected waves. But solution of this problem leads to large computational efforts. Another option is to use absorbing boundaries. In FLAC^{3D} viscous dampers has been used given by Lysmer and Kuhlemeyer (1969).

The quiet-boundary described by Lysmer and Kuhlemeyer (1969) apply dashpots and attach independently on the outer boundary in the shear and normal direction. Viscous normal and shear traction provided by dashpots are given by

$$t_n = -\rho C_p v_n \tag{3.1}$$

$$t_s = -\rho C_s v_s \tag{3.2}$$

Where v_n and v_s are the normal and shear components of the velocity at the boundary;

ρ is mass density

C_p and C_s are the p and s wave velocities and given by

$$C_p = \sqrt{\frac{K+4\left(\frac{G}{3}\right)}{\rho}} \tag{3.3}$$

$$C_s = \sqrt{\frac{G}{\rho}} \tag{3.4}$$

Where G and K are shear and bulk modulus.

For dynamic analysis, the fix boundaries in the static case were replaced by free field boundaries. The boundary conditions at the sides of model must be taken to be free field in absence of any structure. FLAC^{3D} has an option to apply the free-field motion via free field boundary command in such a manner that boundaries retain their absorbing properties i.e., outward waves reflecting from the structure are appropriately absorbed. The boundary condition formulations are shown in Figs. 3.4 & 3.5. The side boundaries of the main grid have been coupled to the free-field grid by attaching the viscous dashpots to put on a quiet boundary and the unbalanced forces from the free-field grid has been applied to the main grid boundary. The free-field boundary conditions are identical to those of an infinite model.

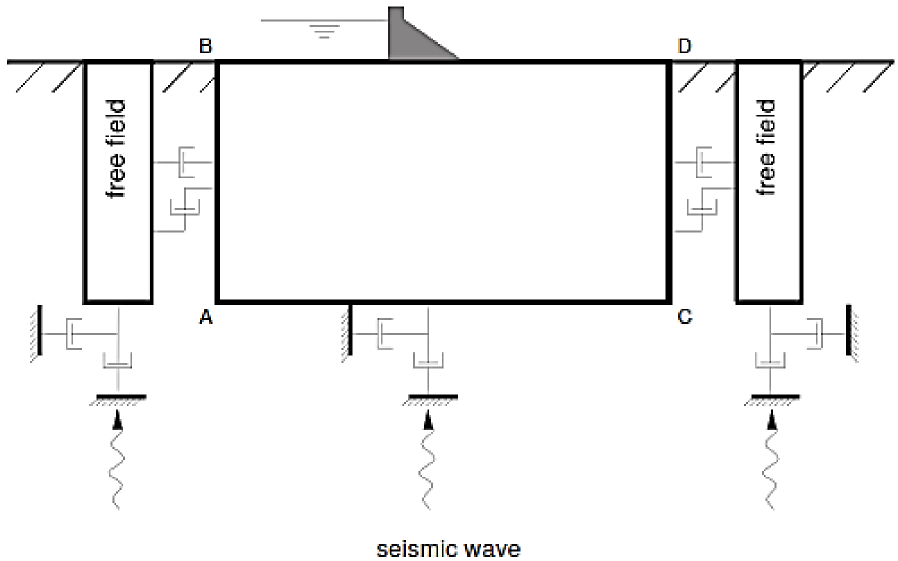


Fig.3.4: Free field mesh (Itasca manual 2006)

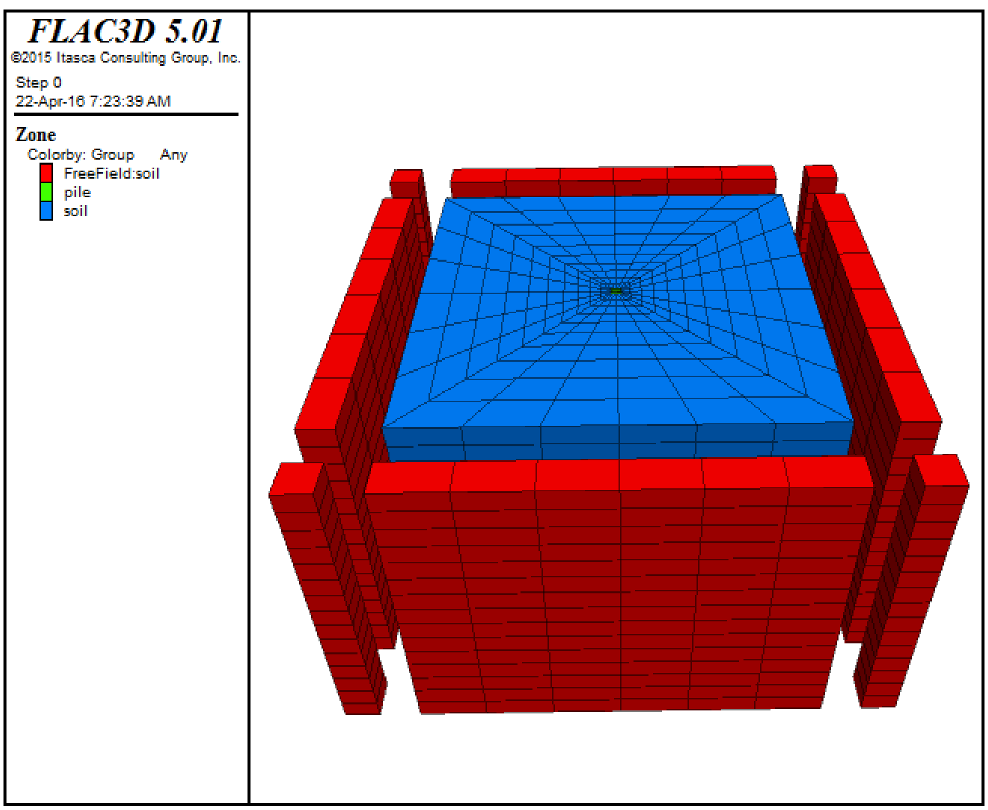


Fig.3.5: Free field boundary condition in soil-pile model at sides and corner

3.5 MESH FOR SOIL-PILE SYSTEM

The full model is taken into consideration. All piles are square in cross-section with dimension d. The base of soil mass taken as bed rock and piles are floating type. The pile group with different spacing (center to center) ratios are considered (s/d =3, 5). The meshing near pile material is fine and radially increases gradually to account for stress gradient.

The wave transmission through element size, the spatial element size as given by Kuhlemeyer and Lysmer (1973) should less than one-tenth of wavelength of applying wave.

$$\Delta l \leq \frac{\lambda}{10} \tag{3.5}$$

For single pile size of model 20m×20m×15m and pile size 0.5m×0.5m×10m are taken. Smallest size of element is 0.25 m and largest size of element is 1.25 m on outer side of soil block. In vertical direction the mesh size kept uniform of 1m length. Lateral boundary is 20d away from center of pile. Finite grid discretization is shown in Fig. 3.6.

For 2×2 pile group the element size varies same as a single pile 0.25 m to 1.25 m. Two spacing configuration (s/d =3, 5) is considered. For spacing s/d=3 mesh generation is shown in Fig. 3.7. A pile cap above from ground level is modelled on top of pile group.

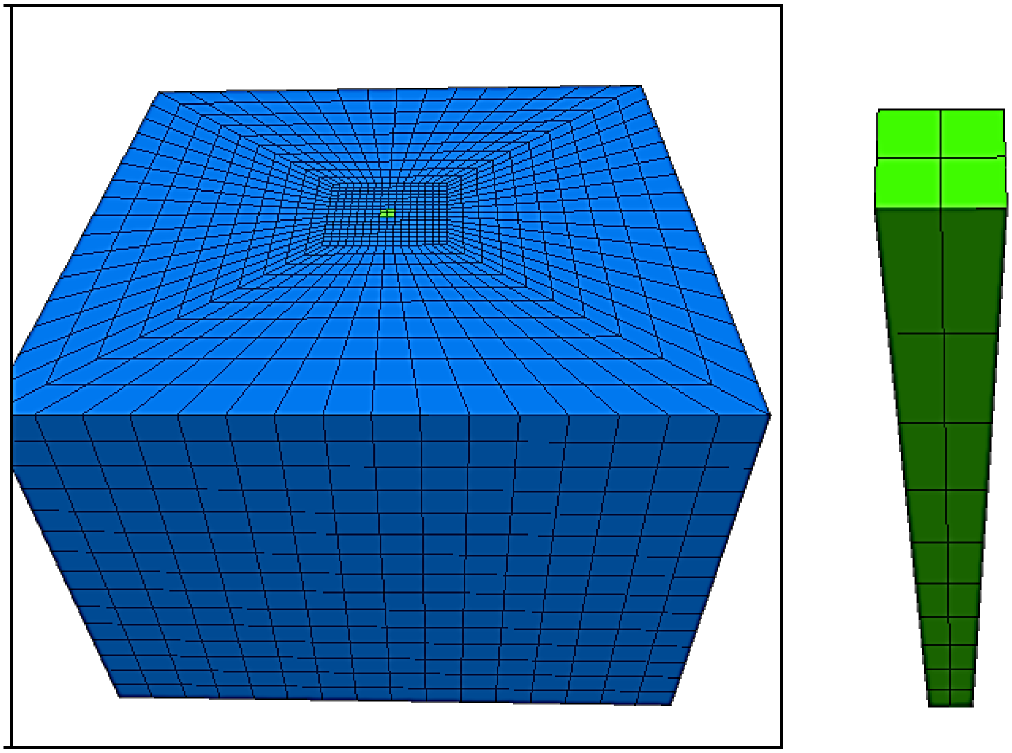


Fig.3.6: Mesh for single pile

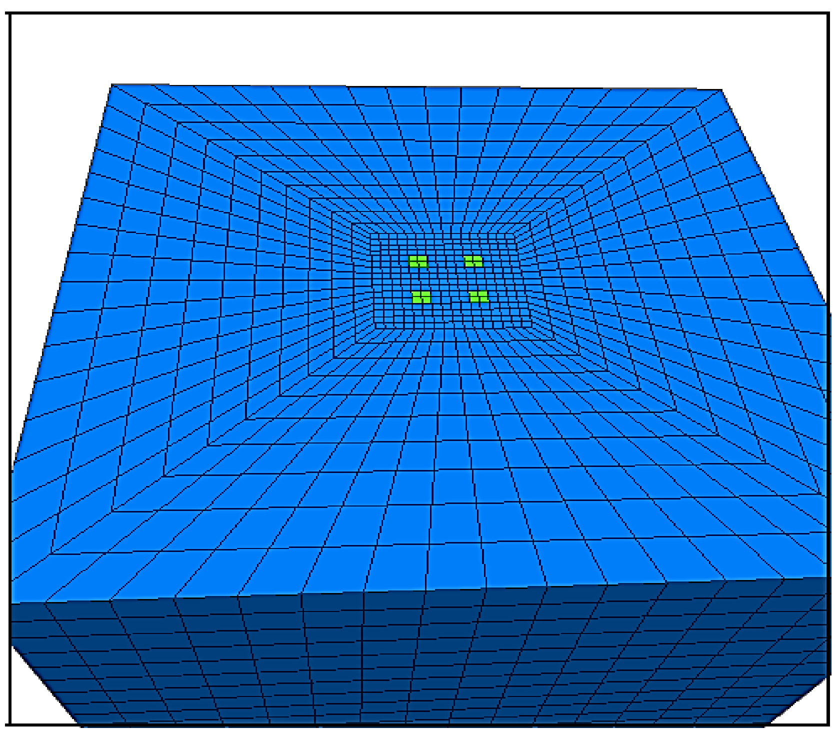


Fig.3.7: Pile Group mesh

3.6 CONSTITUTIVE MODEL

In $FLAC^{3D}$ there are various constitutive models are available to describing the behavior of geological materials. For pile elastic isotropic model is assigned. The required parameters for elastic model are bulk modulus (K) and shear modulus (G). For elastic analysis soil is also considered as elastic material.

For nonlinear analysis Mohr-Coulomb model is assigned to soil block. Mohr-Coulomb is the conventional model used to represent shear failure in soil. The parameters required for this model are bulk modulus (K), shear modulus (G), cohesion, friction angle and density. The failure pattern for this constitutive model relates to a Mohr-Coulomb criterion (shear yield function) with tension cutoff (tension yield function). The position of a stress at any point on this envelope is monitored by non-associated flow rule for shear failure and associated flow rule for tension failure.

Fig. 3.8 expressed Mohr-Coulomb criterion in terms of principal stresses σ1 , σ2, σ3 and corresponding principal strains ϵ1, ϵ2, ϵ3. Failure criteria from A to B is given by f^s in plane (σ1,σ3) is

$$f^s = \sigma_1 - \sigma_3 N_\emptyset + 2C\sqrt{N_\emptyset} \tag{3.6}$$

The tension failure criteria from B to C is given by

$$f^t = \sigma_1 - \sigma^t \tag{3.7}$$

Where $\emptyset$ is the friction angle, C is cohesion, σ^t is the tensile strength and

$$N_\emptyset = \frac{1+sin(\emptyset)}{1-sin(\emptyset)} \tag{3.8}$$

The maximum value of tensile strength is given by

$$\sigma^t_{max} = \frac{c}{tan\emptyset} \tag{3.9}$$

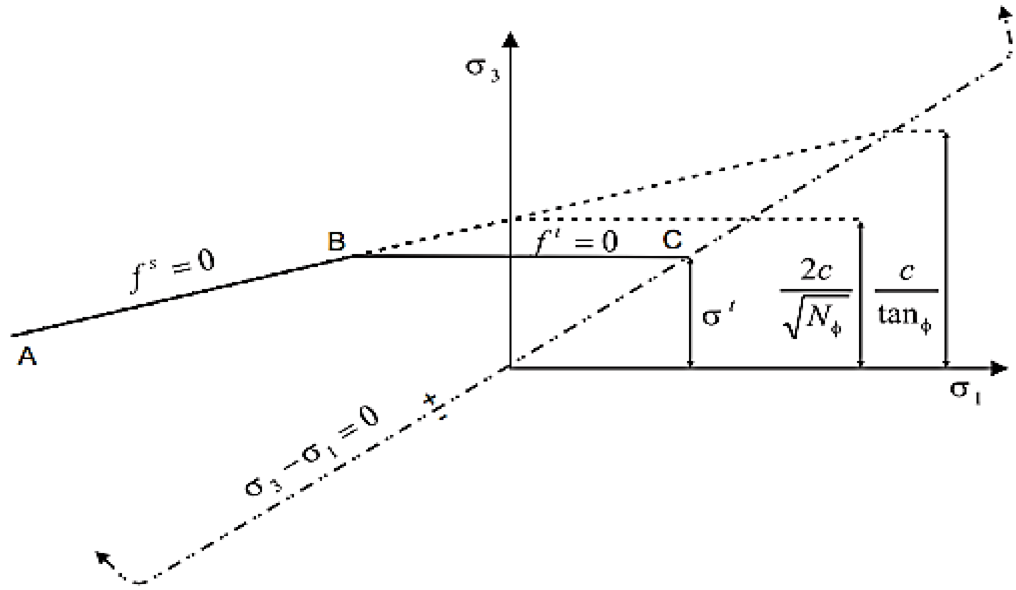


Fig.3.8: Mohr Coulomb failure criteria

3.7 DYNAMIC ANALYSIS

In this section approach for dynamic analysis is described. First we have to do static equilibrium analysis under gravity loading and then the subsequent changes in the boundary conditions and loading conditions are applied for dynamic analysis.

3.7.1 Static Equilibrium

First step is to generate soil pile grid then replace pile with soil material. Bring the soil-pile system to an equilibrium stress-state under gravity loading. The model is in perfect equilibrium when the net nodal-force (unbalanced force) at each grid point reached to zero. In numerical analysis the maximum unbalanced force will never reach to zero for a numerical analysis, but when the maximum unbalanced force is small compared to the tolerance value the model is considered to be in equilibrium.

In next stage model brought in equilibrium after installation of pile. Pile installed by changing properties of pile zone from soil material to concrete material. The dynamic simulation can be performed now by making certain necessary changes in the model.

3.7.2 Dynamic Simulation

Configured the model for dynamic analysis and assigned damping to the model for damp out the vibration. The plastic Mohr-Coulomb model dissipates energy at some extent. For this model 5% default hysteresis damping is assigned.

The fix boundaries assigned for static analysis made free. Then the quiet boundary condition applied in normal and shear direction to absorb the waves. Free field is simulated in all sides of model for infinite media simulation. The base freed in direction of applying motion (in x-direction) because we cannot apply acceleration or velocity motion in fixed grids. The time step of explicit solving equation is defined or FLAC^{3D} will take by default.

For dynamic input with high frequency the spatial mesh size required very fine and time step will also be very small. It will increase the time required to solve the problem. We can adjust the input motion by filtering high frequency component. A low-pass filter may be used for this purpose.

The base excitation we can apply in form of acceleration, velocity and stress time history. In present study the acceleration time history is applied at rigid base rock. The dynamic analysis is done by applying earthquake acceleration time history.

4 VERIFICATION OF MODEL

The soil-pile system is modeled in finite difference base software FLAC^{3D}, so verification problems have been done to check the reliability of the model. In this chapter first the simple cantilever problem solved. Single pile is modeled and static equilibrium analysis is done. For verification of model lateral load analysis is carried out and result compared with literature. For dynamic loading verification free field response is compared with analytical method and pile kinematic interaction factors compared with literature.

4.1 CANTILEVER BEAM

For verification of FLAC^{3D} a cantilever beam is taken in account and tip deflection is calculated. Result compared with analytical solution. The beam dimension is 2m×2m×10m and 320 zones created of size 0.5m×0.5m×0.5m. The elevation of beam and loading system are shown in Fig. 4.1. Linear material properties are used for cantilever beam. Young's modulus E=25 GPa and Poisson's ratio 0.25. A concentrated load of 1000 kN applied at the tip of the beam modeled using brick size elements. Results compared with FLAC^{3D} are shown in Table 4.1

Fig.4.1: Elevation of Cantilever Beam Model

Table 4-1: Comparison of tip deflection

Analytical Deflection	$FLAC^{3D}$ Deflection	Error (%)
10 mm	9.942 mm	0.58

The 0.58 % difference in $FLAC^{3D}$ result is found with analytical result.

4.2 STATIC LATERAL LOAD ON PILE

Verification for static lateral loading done by applying a horizontal load on pile head and deflection profile of pile along the depth of pile compared. For this case a load of 200 kN applied on pile head and deflection recorded. Dimension of circular Pile, dia=0.6 m length 10.1 m (0.1 m above from ground) and soil block of size 16m×16m ×15m is considered. The elevation of the soil-pile system with loading is shown in Fig.4.2.

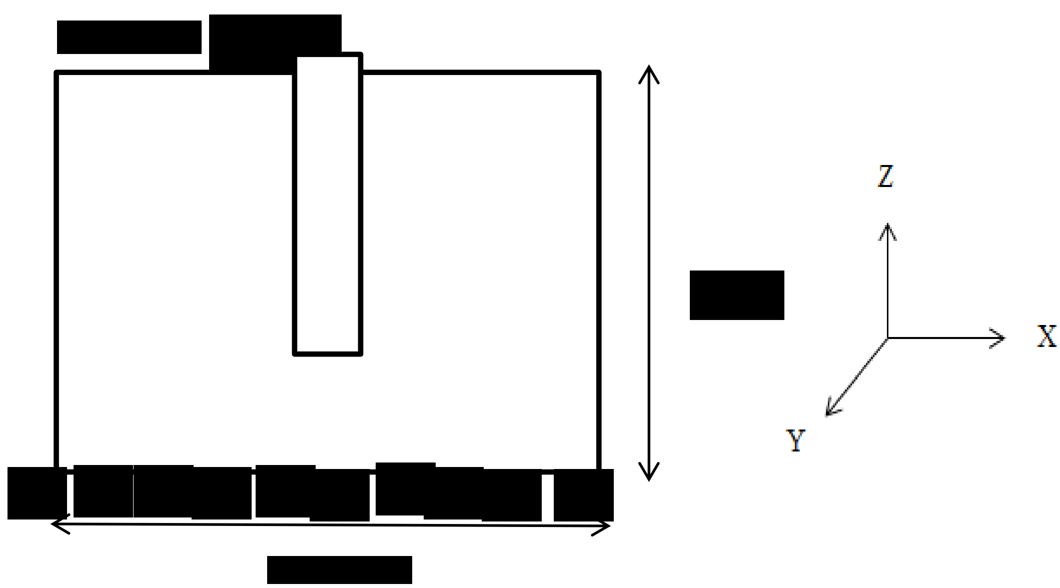


Fig.4.2: Soil-Pile system subjected to lateral static load

Table 4-2: Material properties for Soil-Pile system (Rao et al. (2013))

Material	Density (Kg/m^3)	Elastic Modulus (MPa)	Poisson's Ratio	Shear Modulus (MPa)	Bulk Modulus (MPa)	Friction Angle (degree)
Soil	1900	18	0.3	6.92	15	30 º
Pile	2400	27400	0.2	11410	15200	

Table 4-3 : Interface element properties (Rao et al. (2013))

Shear Stiffness (kN/m^3)	Normal Stiffness (kN/m^3)	Friction Angle (degree)
900	12000	30 º

The material and interface element properties are same given as Rao et al. (2013) in Table 4.2 and Table 4.3. 5% hysteresis damping is assigned to soil media.

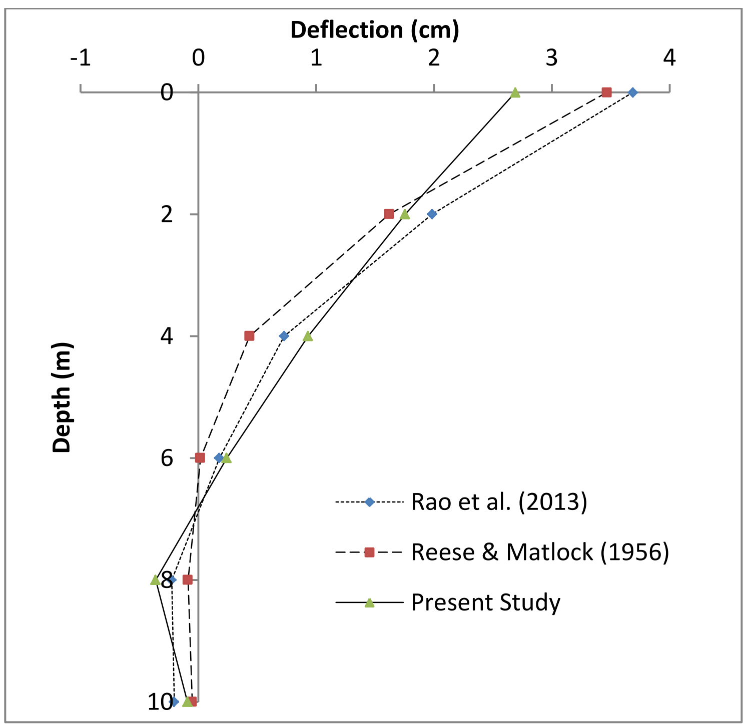


Fig.4.3: Lateral static load verification of single pile

The deflection profile along the pile obtained from literature and analytical method (Reese and Matlock 1956) is plotted with present study. The deflection profile along the pile has been plotted in Fig.4.3 is in good agreement, in both amplitude and behavior.

4.3 FREE FIELD RESPONSE

The verification of finite difference 3-D model with free field boundary condition for harmonic excitation was carried out by ground response analysis. Amplification ratio from finite difference model is compared with one-dimensional analytical method (Kramer 1996). A soil block is shown in Fig. 4.4 of size 50m×20m×20m (height 20m) and element size is unity in all direction. All grid points are restrained in y, z direction and free in x direction (applied motion direction) to simulate one-dimensional results. For this analysis free field boundary condition with dashpots (Lysmer and Kuhlemeyer 1969) is applied.

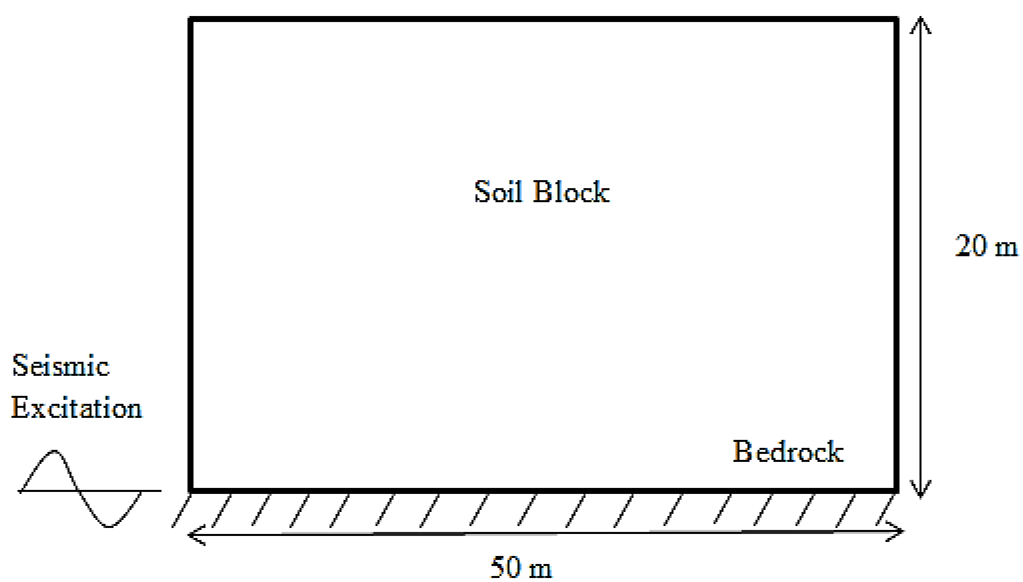


Fig. 4.4: Soil Block model for free field response

Soil is taken linear elastic with properties density ρ=2000 kg/m^3, elastic modulus E=163 MPa, Poisson's ratio 0.4 and hysteresis damping of 5%. Shear wave velocity of soil media is 170 m/sec. Base acceleration harmonic motion with amplitude 1 m/s^2 is applied for the different range of frequencies. The response of applied motion is recorded at the top of soil block. The free field amplification is the ratio of recorded ground acceleration with respect to applied acceleration.

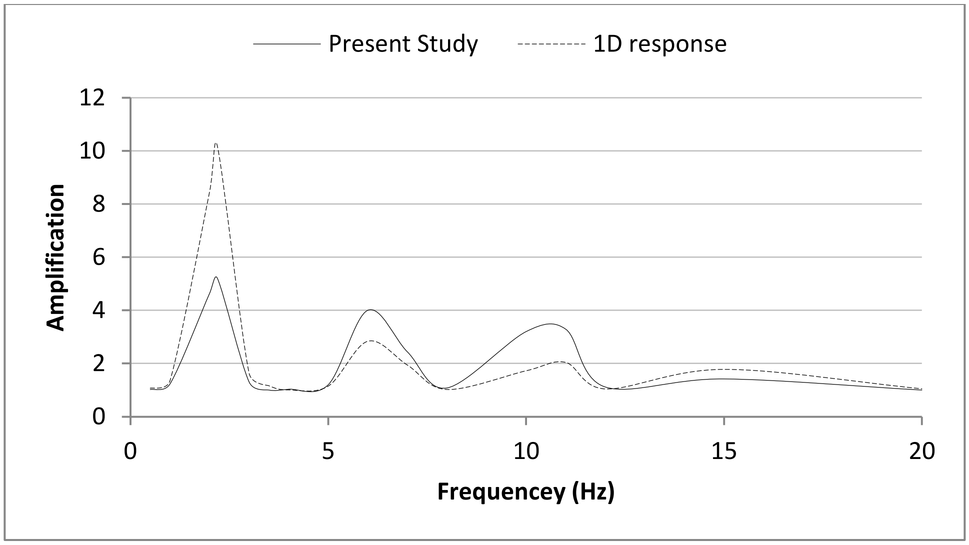


Fig.4.5: Verification for free field response

Free field amplification ratio from FLAC^{3d} three-dimensional model is compared with response obtained from one-dimensional analytical method. The amplification compared in Fig.4.5 is found in good agreement with analytical results. The peak amplification is lower in three-dimensional model at resonant frequency which is as expected.

4.4 GROUND RESPONSE ANALYSIS WITH DEEPSOIL

DEEPSOIL is a One-dimensional wave propagation analysis program for soil medium. It is based on nonlinear and equivalent linear analysis. This program is used to obtained Seismic site response of one-dimensional soil column (Hashash et al. 2015). On the above soil three-dimensional model harmonic motion of frequency 1Hz and unit amplitude is applied at base of model. In FLAC^{3d} to replicate one-dimensional condition (to compare with DEEPSOIL) other two dimensions are restrained at every grid point.

The soil block dimension for FLAC^{3D} is taken as Fig. 4.4 and for DEEPSOIL one dimensional vertical length is considered. Response of applied motion in terms of acceleration is plotted with DEEPSOIL output in Fig. 4.6 found similar after some time.

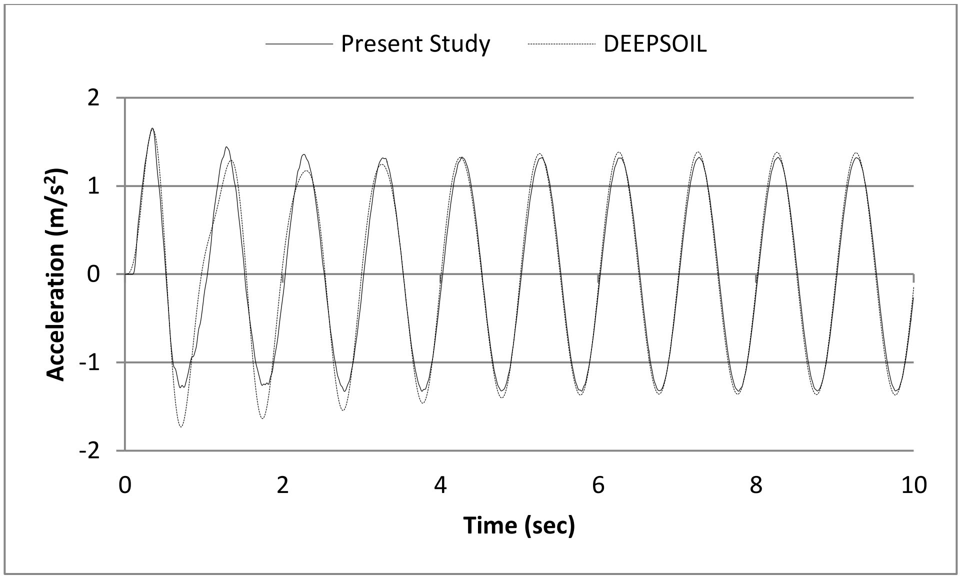


Fig.4.6: Verification for Harmonic loading on soil block

4.5 SEISMIC RESPONSE

Pile Group with spacing (s/d=3, s/d=5) is considered without superstructure. Vertically propagating shear waves at different frequency is applied at base of model.

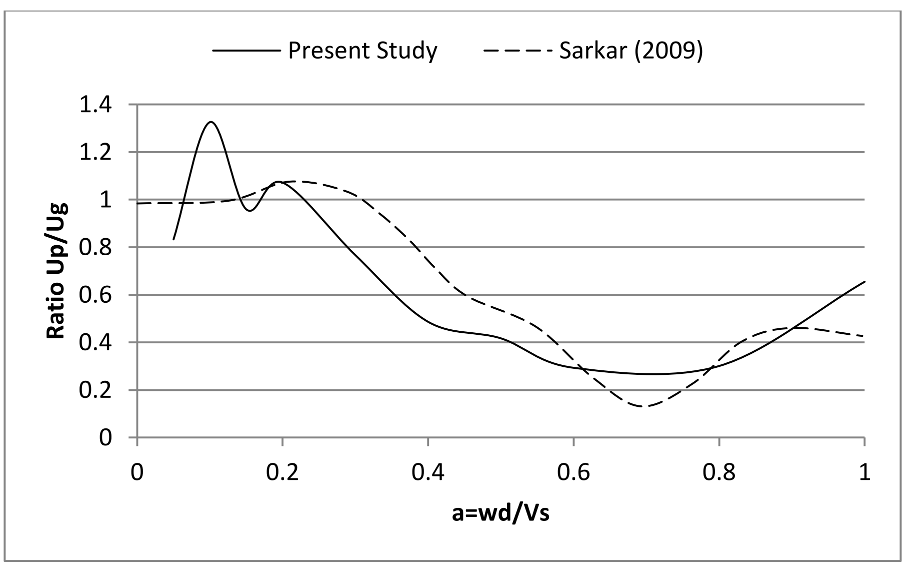


Fig.4.7: Kinematic interaction factor for 2x2 pile group with s/d=3

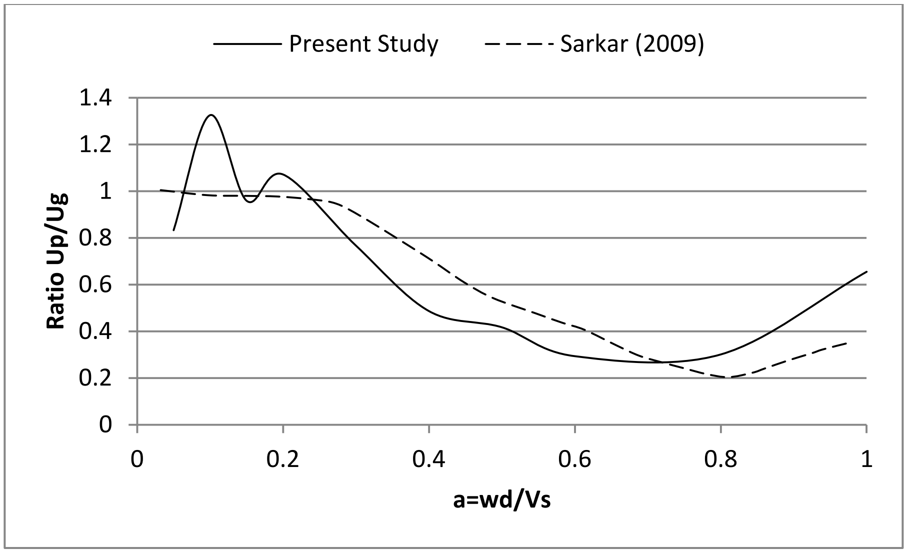


Fig.4.8: Kinematic interaction factor for 2x2 pile group with s/d=5

The base is considered rigid bedrock. The kinematic interaction factor for translation is U_p/U_g. Material properties are taken from Sarkar (2009) and results compared for pile group.

Where a is the dimensionless frequency ratio

U_g is the displacement at the soil block in absence of pile foundation.

U_p is the displacement at the top of pile foundation.

ω exciting frequency

d side of the square pile

V_s shear wave velocity of soil media

For pile group two configuration considered s/d=3 and s/d=5 to analyze for displacement amplification ratio. The kinematic interaction factor for pile is compared with Sarkar (2009) in Figs. 4.7 and 4.8. Initially higher amplification at low frequency after that similar trend find with Sarkar 2009). The similar pattern is found in Kaynia and Kausel (1982) also.

4.6 SUMMARY

In this chapter the elements are verified for static loading on cantilever beam. Static soil-pile system is verified with literature applying lateral load. The boundary conditions are verified with free field response comparing with analytical and DEEPSOIL program results. At resonant frequency there is some variation in peak amplitude. For dynamic loading soil-pile system is verified with kinematic interaction factors comparing with literature. After these verifications problems the model can be analyzed under different loading conditions.

5 RESPONSE OF SINGLE PILE

In this chapter square shaped single pile is analyzed under real earthquake excitation. Effect of increasing PGA (peak ground acceleration) of earthquake excitation on response of single pile studied. Earthquake motion of different predominant frequency applied on soil-pile system and effect of frequency is analyzed.

5.1 MODEL

The physical model is shown in Fig. 5.1 and material properties are taken from Table 4.2. The Mohr-Coulomb model is assigned to soil medium and elastic material properties to pile. The boundary condition is quiet boundaries with free field. The detailed description of modeling is given in Chapter 3.

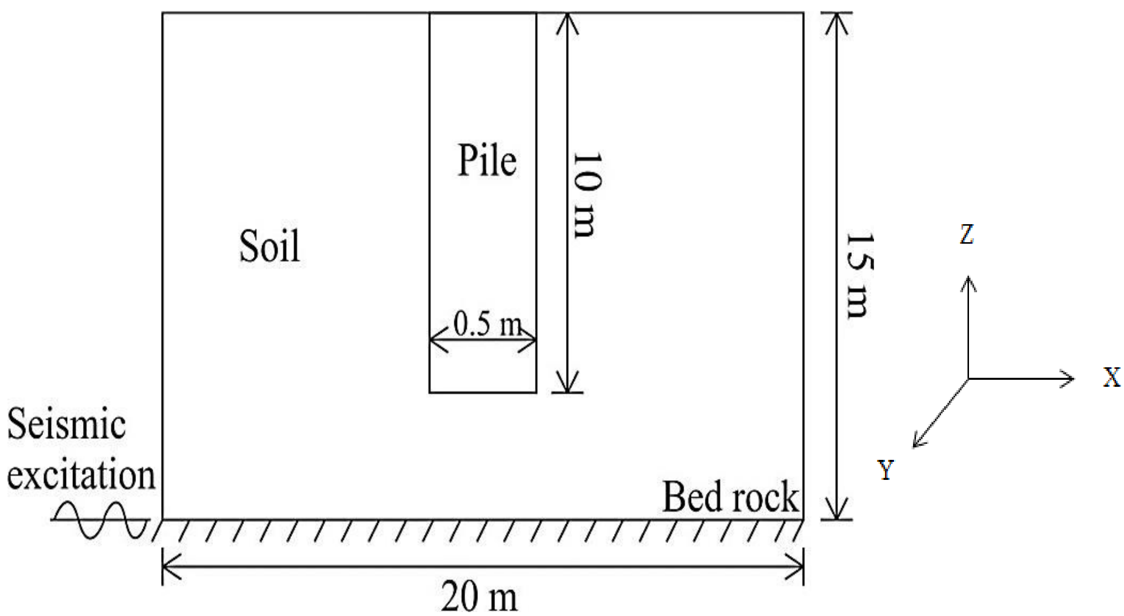


Fig. 5.1: Physical model for Single pile

Shear wave velocity calculated from the given formula (Kramer et al. 1996) is 70.3 m/sec.

$$V_s = \left(\frac{G}{\rho}\right)^{0.5} \quad (5.1)$$

Fundamental frequency of soil-pile system is 1.17 Hz.

5.2 INPUT MOTION

The input motion is acceleration time history of magnitude 7.0 Bhuj earthquake (January 2001), shown in Fig. 5.2. The time history represents horizontal component of acceleration history with maximum PGA 1.03 m/s^2 (strongmotioncenter.org). Predominant frequency of this earthquake motion is 1.19 Hz.

5.2.1 Base Line Correction

If raw acceleration or velocity motion is used, it may produce residual displacement after the motion has finished.

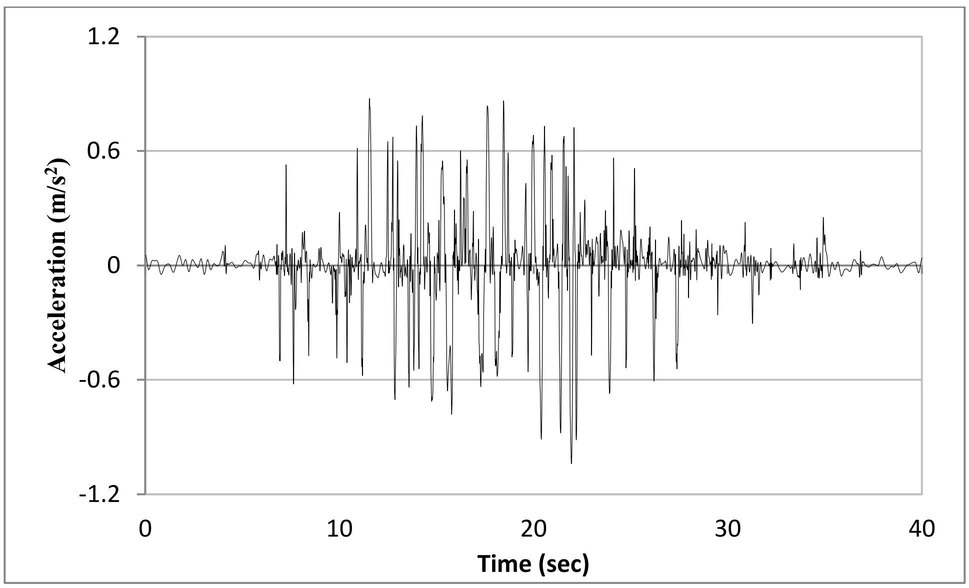


Fig. 5.2: Acceleration time history without base line correction

To avoid the residual displacement baseline correction should be done before applying on the soil-pile model. In base line corrected acceleration time history, final velocity and displacement will be zero. SeismoSignal program (Seismosoft 2016(b)) is used to make base line corrected time history (cubic base line correction applied). The displacement time history, which is obtained from double integral of acceleration time history without and with base line correction are compared in Fig. 5.3. By Fig 5.3 we can analyze that the residual displacement is zero for baseline corrected acceleration time history. It is found that the predominant frequency (1.19 Hz) and PGA remain same after base line correction.

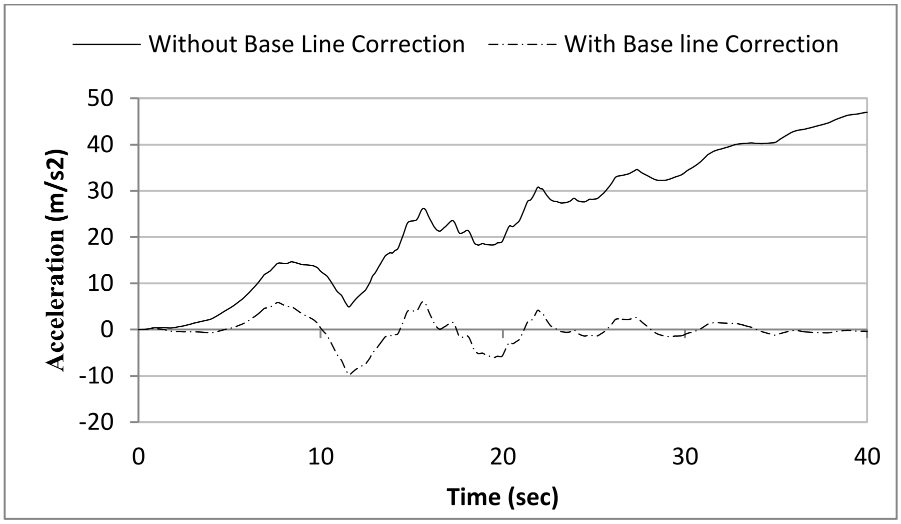


Fig. 5.3: Displacement time histories of Bhuj earthquake without and with base line correction

5.2.2 Compatible Ground Motion

The effect of different PGA ground motion for Bhuj earthquake which are made compatible to IS 1893:2002 (Part I) spectrum ground motions are analyzed. This has been done for different earthquake zones of India (i.e. with zone factor equal to 0.10g, 0.16g, 0.24g and 0.36g). SeismoMatch program (Seismosoft 2016(a)) is used to made compatible ground motion. This application is capable of adjusting earthquake acceleration to match a specific target response spectrum. The target spectrum is factorized with required PGA (i.e. for 0.1g target spectrum multiplied with 0.1).

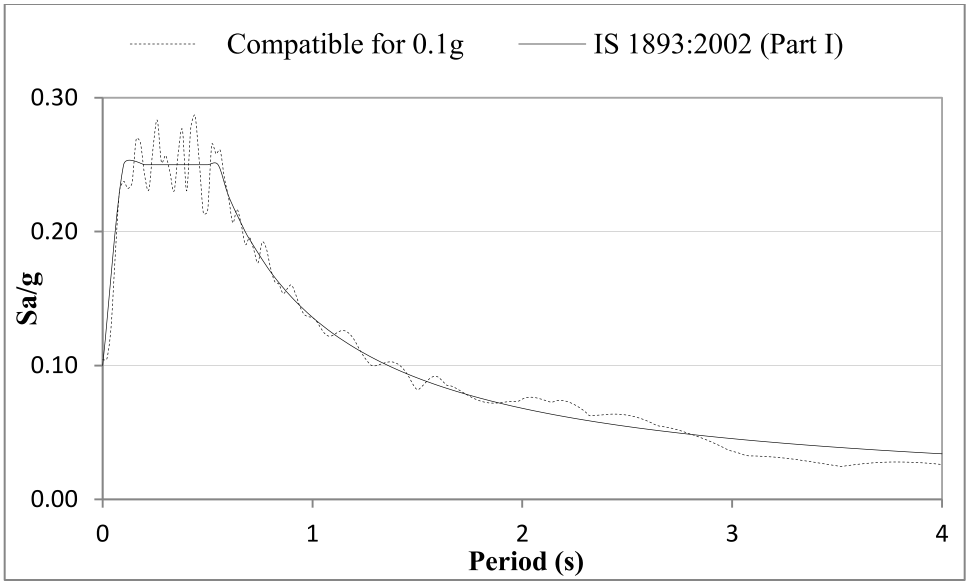


Fig.5.4: Compatible response spectra of Bhuj earthquake (2001) for PGA: 0.1g

The codal spectrum is taken for medium soil. The compatible response spectrum for for PGA 0.1g is plotted with codal spectra in Fig. 5.3, similar plots for other PGA also. The S_a/g value is slightly higher for Bhuj motion from codal spectrum.

5.3 EFFECT OF PGA

The above scaled acceleration time histories are applied on base of soil-pile model in lateral direction (x-direction). The time history analysis is time-consuming and resource-consuming analysis for three-dimensional continuum model. However for real earthquake it is the most precise analysis. Soil-pile system behavior is for PGA from 0.1g, 0.16g, 0.24g and 0.36g.

5.3.1 Time History Response of Pile

In this section, response of pile is obtained in terms of pile head displacement, acceleration and shear stress. Compatible time history of Bhuj earthquake with PGA 0.1g, 0.16g, 0.24g and 0.36g is applied on soil-pile system.

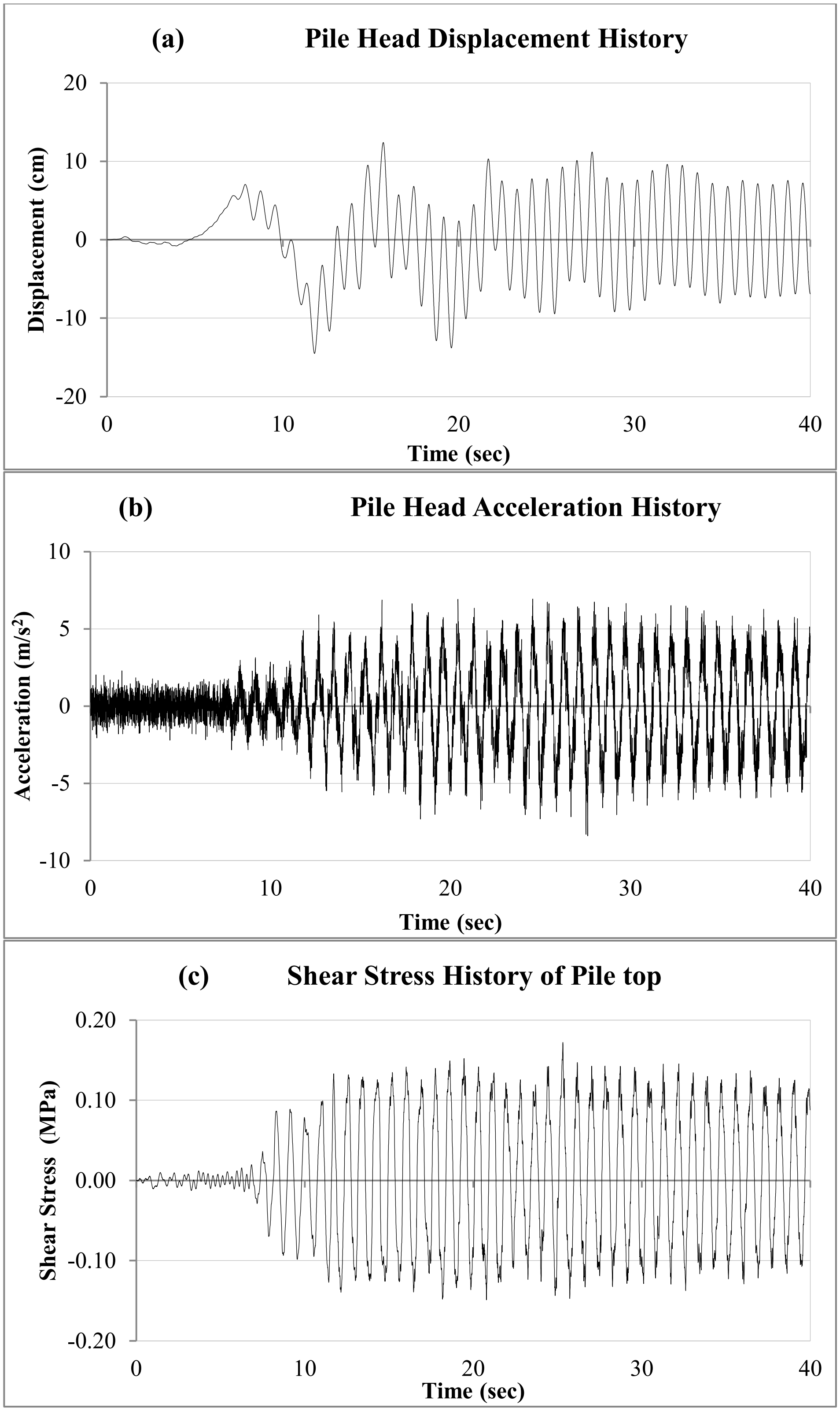


Fig.5.5: Plastic behavior of single pile: (a) Displacement (b) Acceleration (c) Shear stress time history at PGA 0.1g

To show the effect of permanent deformation on higher PGA, time history response for 0.1g and 0.36g are shown in Fig. 5.5(a) & Fig. 5.6(a). For PGA 0.16g and 0.24g, similar pattern of response time history is found.

The displacement shows the lateral movement of pile head due to loading. Acceleration time history indicates the amplification in applied motion. Shear stress along the depth of pile shows which part of pile carry larger soil pressure that may cause pile collapse (Taha et al. 2009).

Figs. 5.5 & 5.6 shows response of pile head in terms of displacement, acceleration and shear stress time history due to applied motion respectively 0.1g and 0.36g.

As we increased the PGA of applied motion, the displacement time history pattern changed. At 0.1 g, a small shift of displacement time history from origin and at 0.36g shift from base line is more. At higher PGA some permanent deformation takes place so displacement time history shifted from origin in Fig. 5.6(a). Similar type of displacement history trend is found in Haldar and Babu (2010).

From Fig. 5.5(b) & Fig. 5.6(b), amplification in acceleration time history is 8.4 times for PGA 0.1g and 3.1 times for PGA 0.36g. Proportional increase in acceleration is less for higher PGA due to the effect nonlinear soil model.

From Fig. 5.5(c) & Fig. 5.6(c), maximum Stress at pile head for applied motion of PGA 0.1g, is 0.172 MPa and for 0.36g maximum stress is 0.181 MPa. At pile head stress increment is less as increase in PGA due to the free headed pile.

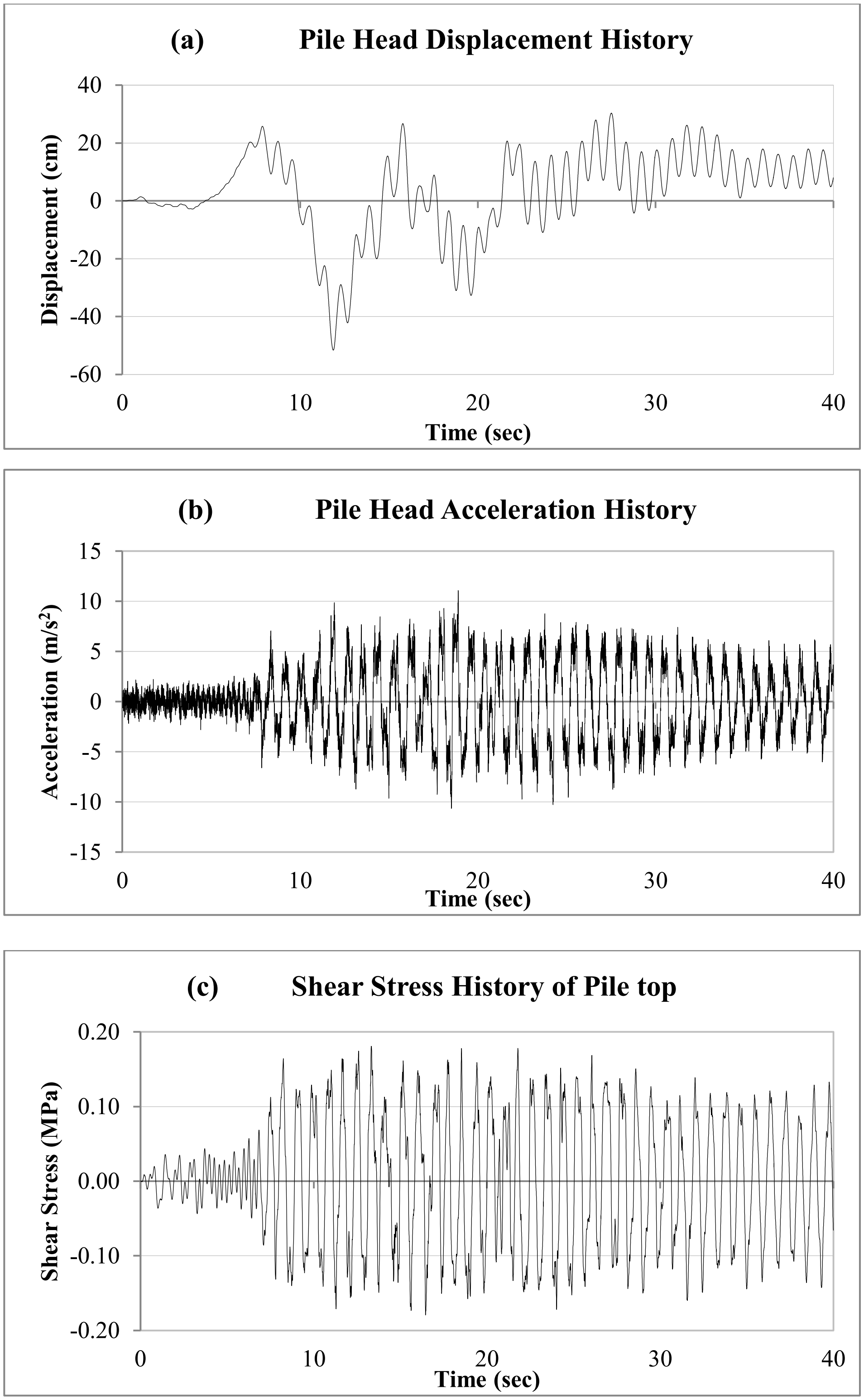


Fig.5.6: Plastic behavior of single pile (a) Displacement (b) Acceleration (c) Shear stress time history at PGA 0.36g

5.3.2 Response of Pile along the Depth

Displacement, acceleration and stress along the pile depth are important parameters for design of a pile. At any section, if some parameter is high that section is most vulnerable to damage. Fig. 5.7 shows these profiles along depth with variation of increasing PGA.

At 0.36g pile head displacement is 37.12 cm higher than displacement at 0.1g. Due to higher PGA, pile tip also displace more from its position. Variation of displacement along the depth is linearly increasing.

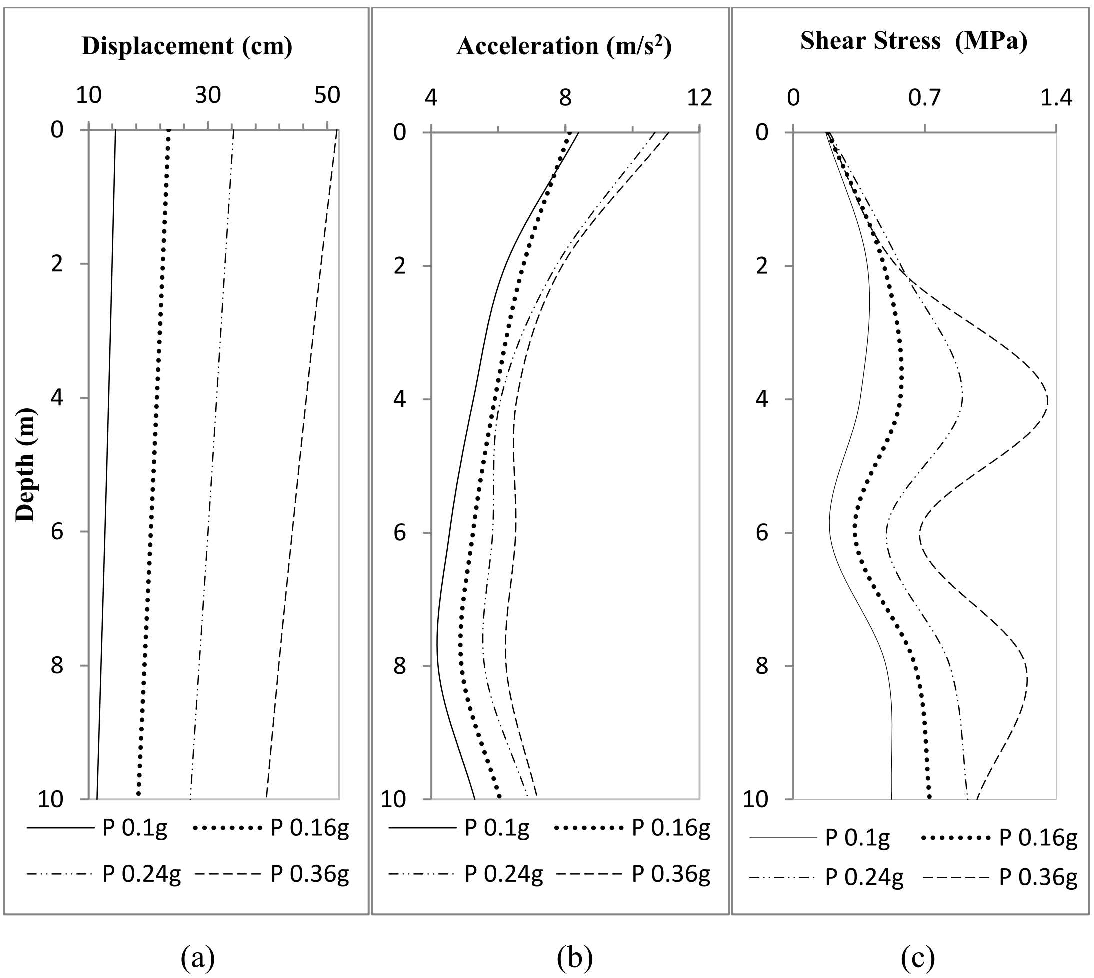


Fig.5.7: Effect of increasing PGA on the response of Single a Pile (a) Displacement (b) Acceleration (c) Shear Stress

*P 0.1g: P indicate plastic model (Mohr-Coulomb) and 0.1g is PGA of applied motion.

The pile acceleration is 2.68 m/s^2 higher at 0.36g than acceleration at 0.1g. Acceleration increment in not higher as PGA increment it is because of nonlinear plastic Mohr-Coulomb model. Acceleration reduces slightly at 8m depth from tip acceleration value for all PGA, due to change in material at soil-pile interface.Shear stress along the pile shows the applied lateral pressure due to earthquake loading. At depths 8 m and 4 m, shear stresses are high and indicate that these pile section are susceptible to collapse. For 0.36g highest shear stresses is 1.35 MPa at depth of 4m. At the pile head shear stresses variation is smaller compare to displacement and acceleration variation for increasing PGA.

5.4 EFFECT OF PREDOMINANT FREQUENCY

The response of soil-pile system under earthquake motion is affected by the excitation frequency. In this section, effect of predominant frequency for different earthquake is analyzed on response of pile foundation.

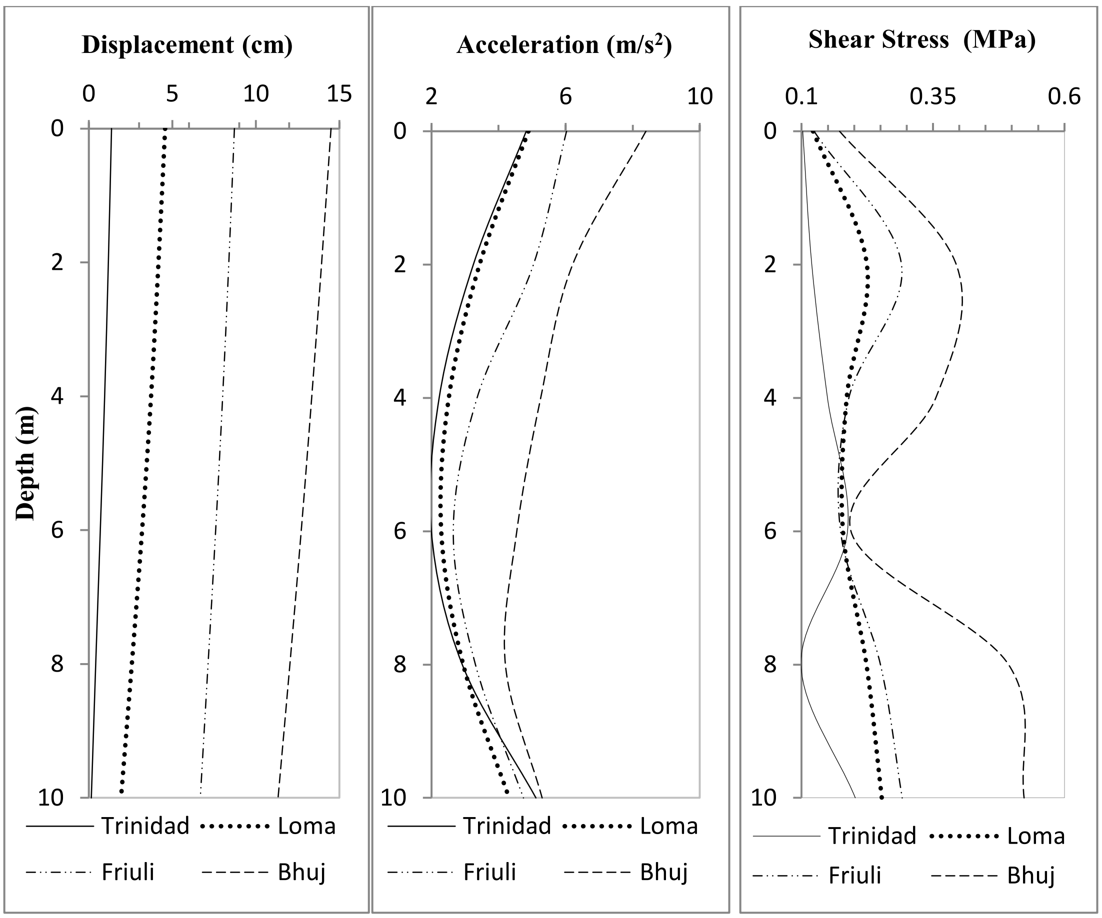
5.4.1 Response of Pile under Different Excitations

Earthquake motions and related other parameter detailed in Table 5.1. To analyze the effect of frequency, all motions are scaled to same PGA 0.1g and the same duration.

Table 5-1: Different input earthquake motion scaled to 0.1g and their related parameters

Earthquake Event	**Predominant Frequency (Hz)**	**Event Date**	**Recording station**	**Source**
Loma Prieta (USA)	0.51	October 18, 1989	090 CDMG STATION 47381	PEER Strong Motion
Bhuj (India)	1.19	January 26, 2001	IITR station Ahmadabad	Strong-Motion VDC
Friuli (Itly)	2.01	May 06, 1976	TOLMEZZO (Udine)	PEER Strong Motion
Trinidad (T & T)	2.76	August 24, 1983	090 CDMG STATION 1498	PEER Strong Motion
CDMG: California division of mines and geology, T & T: Trinidad and Tobago				

The response of pile is plotted in Fig. 5.8 along the depth of pile for given motions. From Fig. 5.8(a) we can analyze that displacement is smaller for Trinidad earthquake compare to other earthquake. Trinidad earthquake have higher predominant frequency which is 2.76 Hz, at high frequency small response is expected. Pile head displacement for Bhuj earthquake is found maximum which 14.5 cm. After Bhuj motion Friuli motion have significant displacement along the pile.



(a) (b) (c)

Fig.5.8: Effect of different earthquake with 0.1g PGA on pile (a) Displacement (b) Acceleration (c) Shear Stress

Maximum acceleration of pile head in Bhuj earthquake is 8.395 m/s^2 which is 3.57 m/s^2 higher than Trinidad earthquake. At pile tip variation in acceleration is small, above 8m depth acceleration rapidly increasing for Friuli and Bhuj earthquake. Fig. 5.8(c) shows maximum shear stress for all considered motions at 2 m depth.

5.5 CONCLUDING REMARKS

In this chapter dynamic loading and importance of base line correction are discussed. Method for compatible time history as IS code was shown. Response time histories of pile head for motion 0.1g and 0.36g are plotted. For higher PGA permanent deformation takes place. Response of parameters (displacement, acceleration and shear stress) are plotted along the pile and results compared.

In last section of chapter various earthquake motion with parameters are tabulated and effect of predominant frequency discussed. Bhuj earthquake frequency is nearby fundamental frequency of soil-pile system so the maximum amplification in displacement in this event. Trinidad earthquake have less displacement. Effect of predominant frequency on acceleration also is significant, more amplification for Bhuj and less for Trinidad earthquake. Shear stress also showing similar behavior, high values for Bhuj earthquake.

Pile is more susceptible to damage in motion that has predominant frequency near the fundamental frequency of model. This gives the range of interest for earthquake loading on soil-pile system.

6 RESPONSE OF 2×2 PILE GROUP

In this chapter 2×2 pile group (piles are square in shape) with different spacing ratio (s/d=3, s/d=5) is analyzed under earthquake motion. Effect of increasing PGA is examined on the response of pile group and compared with that response of single pile. In later section, response due to superstructure is also considered.

6.1 PHYSICAL MODEL

The physical model of a 2×2 pile group is shown in Fig. 6.1 and material properties are presented in Table 4.2. The Mohr-Coulomb model is assigned to soil medium and elastic material properties to pile. The boundary condition is quiet boundaries with free field. The detailed description of modeling is given in Chapter 3.

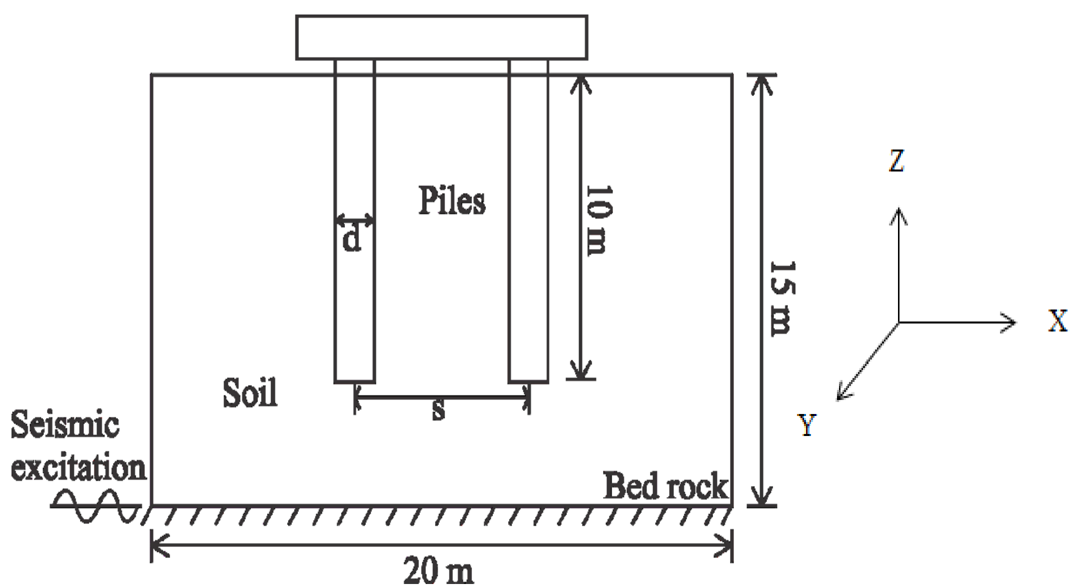


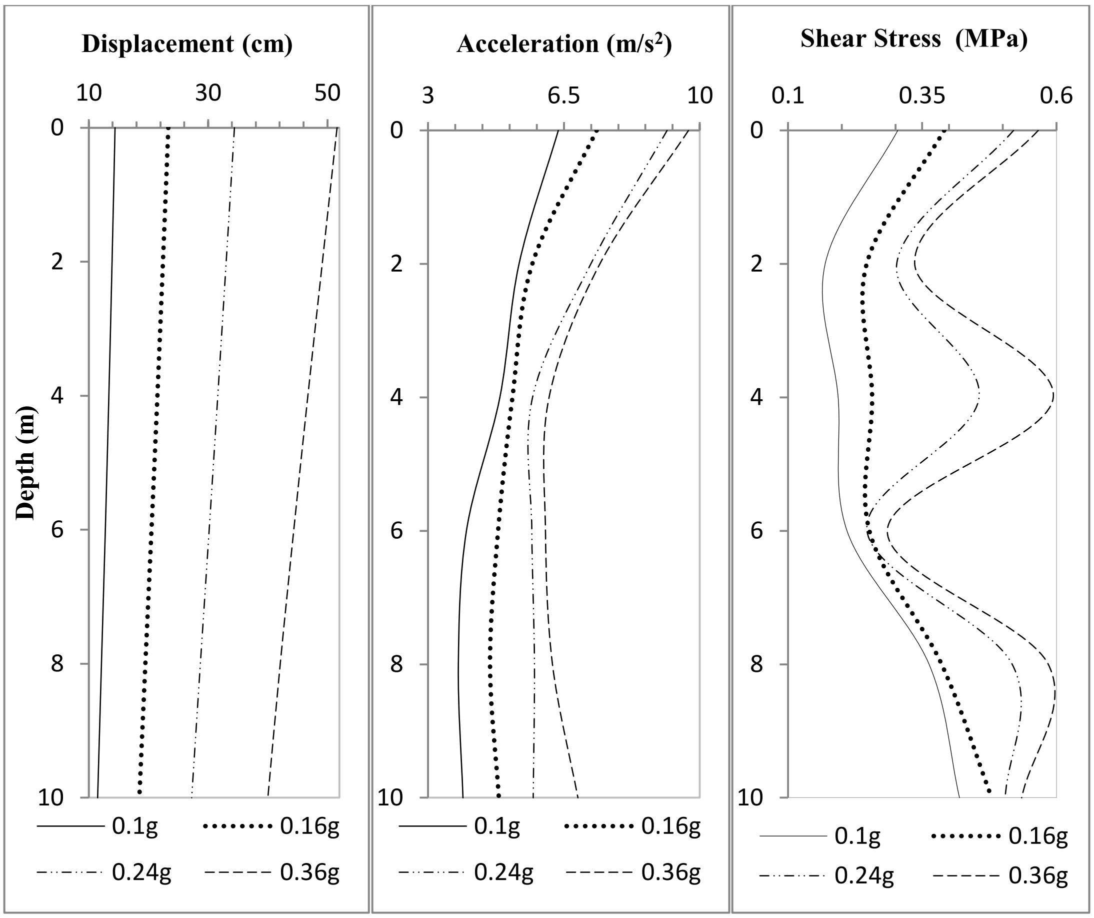
Fig.6.1: Physical model for pile group

6.2 EFFECT OF PGA AND GROUP EFFECT

The compatible earthquake motion of Bhuj is taken as section 5.2.2.

6.2.1 Effect of PGA

The pile behavior along the pile is analyzed for displacement, acceleration and shear stress using Fig. 6.2. Maximum pile head displacement is 51.61 cm for PGA 0.36g. With respect to the input motion the pile head acceleration for PGA 0.36g is 2.7 times amplified and for PGA 0.1, 6.3 times amplified. The maximum shear stress is 0.595 MPa for 0.36g PGA at 4 m depth, at this depth pile section will collapse.

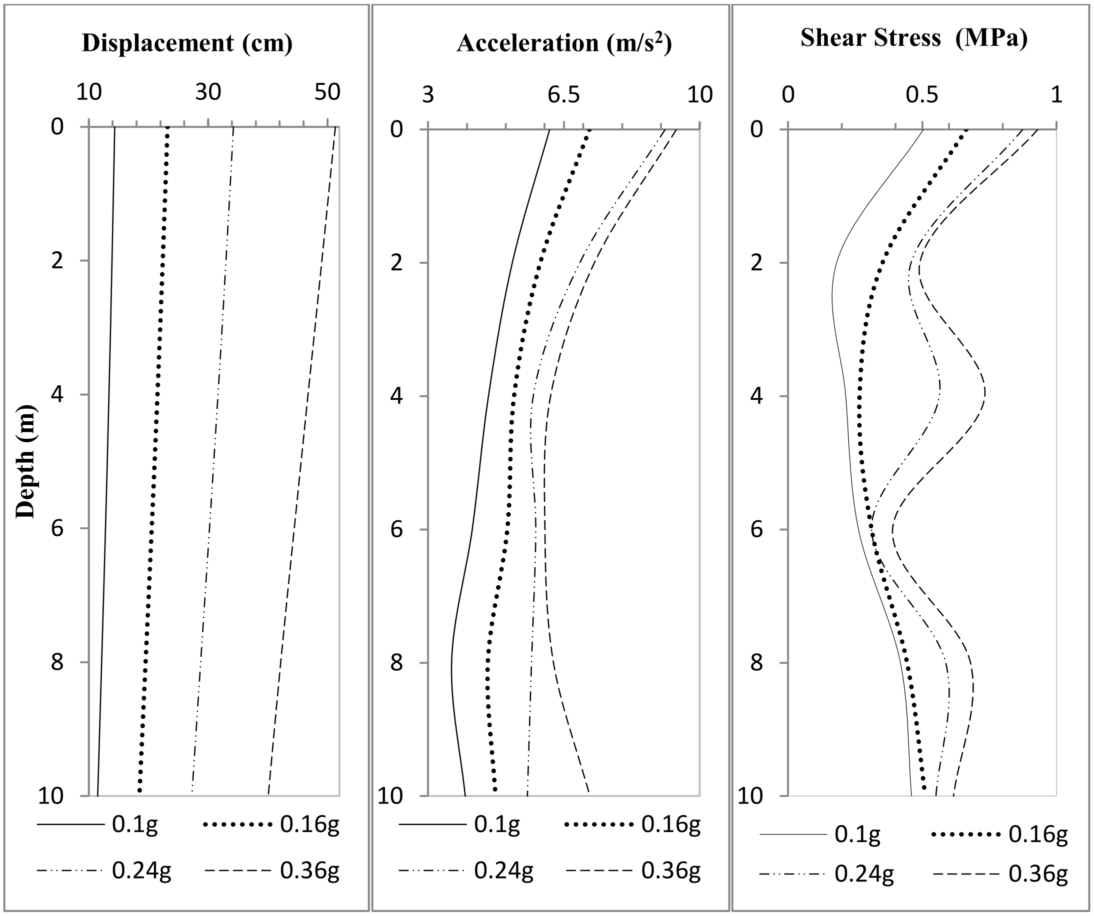


(a) (b) (c)

Fig.6.2: Effect of increasing PGA on response of 2×2 Pile group (s/d=3) (a) Displacement (b) Acceleration (c) Shear Stress

Response of pile foundation after increasing spacing (from s/d=3 to s/d=5) between piles is plotted along in Fig. 6.3. Pile head displacement for 0.36g PGA is 51.33 cm which is approximately equal to displacement for spacing s/d=3. With respect to the input motion the pile head acceleration for PGA 0.36g is 2.61 time amplified and for

PGA 0.1 amplification is 6.2 times. The maximum shear stress is 0.79 MPa for 0.36g which is higher from shear stress of pile with spacing s/d=3.



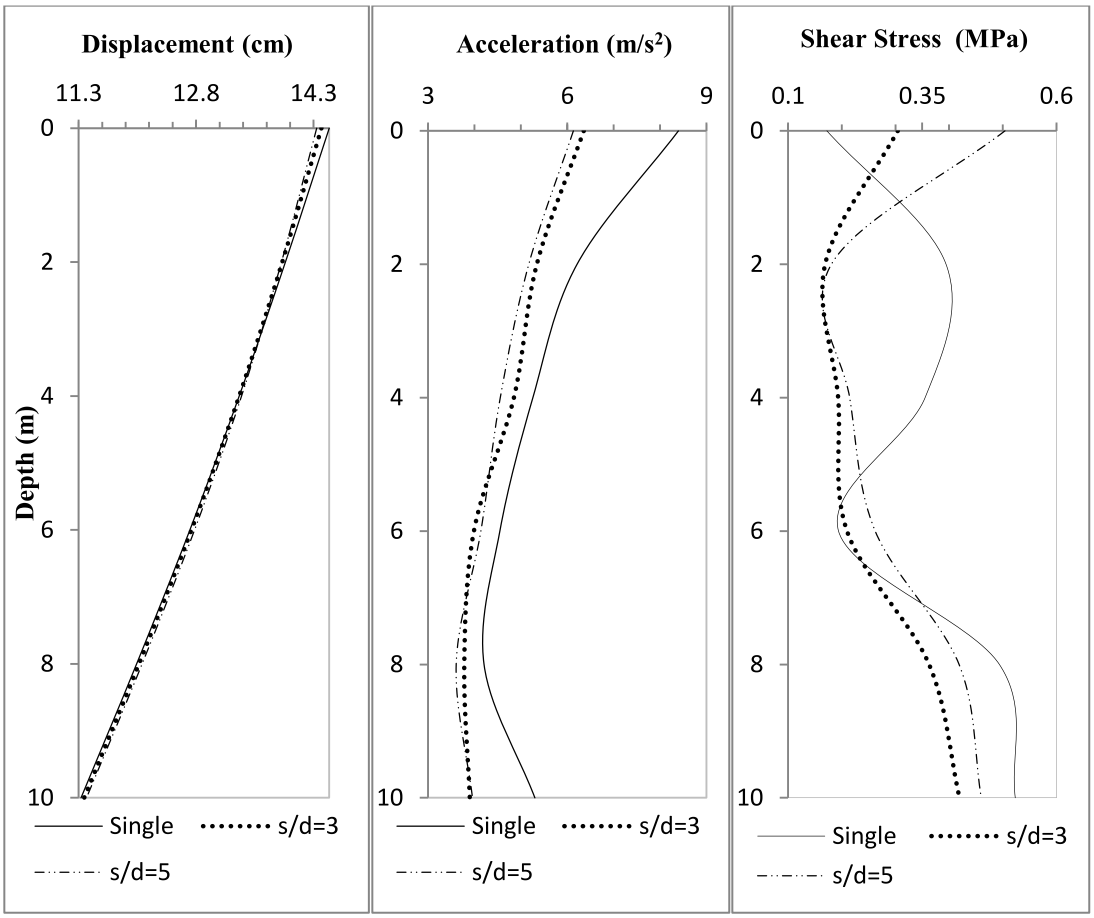
(a) (b) (c)

Fig.6.3: Effect of increasing PGA on response of 2×2 Pile group (s/d=5) (a) Displacement (b) Acceleration (c) Shear Stress

Displacement is increasing in proportion to with increased applied PGA. Amplification in acceleration is less as higher PGA compare to lower PGA input motion. For different spacing, displacement and acceleration response are in insignificant (Chu and Truman 2004) that is found in present study also. In the pile group shear stress is maximum near the pilecap.

6.2.2 Comparison of Pile Group Response with Single Pile

Displacement along the depth of pile for different pile configuration varies at the top of pile foundation. For single pile it is same to pile group below 4m depth of pile but at top it is 1.1 % higher than pile group of spacing ratio s/d 5. Acceleration is also higher for single pile as compared to the pile group which is expected due to more flexibility for single pile.



(a) (b) (c)

Fig.6.4: Effect of pile group on response of pile along the depth (a) Displacement (b) Acceleration (c) Shear Stress

Shear stress is varying along the depth for single pile and pile group with different spacing showing similar pattern. For single pile maximum lateral stress at 2 m depth but at the top of pile higher shear stress are found for pile group. It is due to stresses induced by the weight of pile cap. It is observed that lateral pile stress at pile head is higher in the presence of vertical load (load of pile cap) as compared to the pure lateral load case.

6.3 EFFECT OF SUPERSTRUCTURE

In this section, first load carrying capacity of pile group is calculated. To take the effect of superstructure on response of pile foundation, superstructure model as SDOF (single degree of freedom system) by a lumped mass on a column.

6.3.1 Pile Bearing Capacity

The pile capacity have been calculated as IS 2911 Part 1/sec2 (2010).Load carrying capacity of piles for granular soil is given by

$$Q_u = A_p \left(\frac{1}{2} D\gamma N_\gamma + P_D N_q\right) + \sum_{i=1}^{n} K_i P_{Di} \tan\delta_i A_{si} \tag{6.1}$$

A_p= cross-sectional area of pile tip, in m^2

D= diameter of pile

γ=effective unit weight of the soil at pile tip

N_q, N_γ (IS6403) Bering capacity factors depending upon internal friction

P_D=effective overburden pressure at pile tip

K_i=coefficient of earth pressure for ith layer

P_{Di} = effective overburden pressure for ith layer

δ_i=angle of wall friction between pile and soil

A_{si}=surface area of pile shaft for ith layer

The ultimate load carrying capacity for single pile is 575 kN. For pile group ultimate load carried capacity is 2300 kN assuming 100 % efficiency. Safe load carrying capacity is taken as Q_s =920 kN after applying factor of safety of 2.5.

6.3.2 Varying Vertical Load

The physical geometry and meshed model of soil-pile system with superstructure are shown in Fig. 6.5. The column height is 1 m and concentrated mass as pile capacity. Boundary condition is quiet boundary with free field.

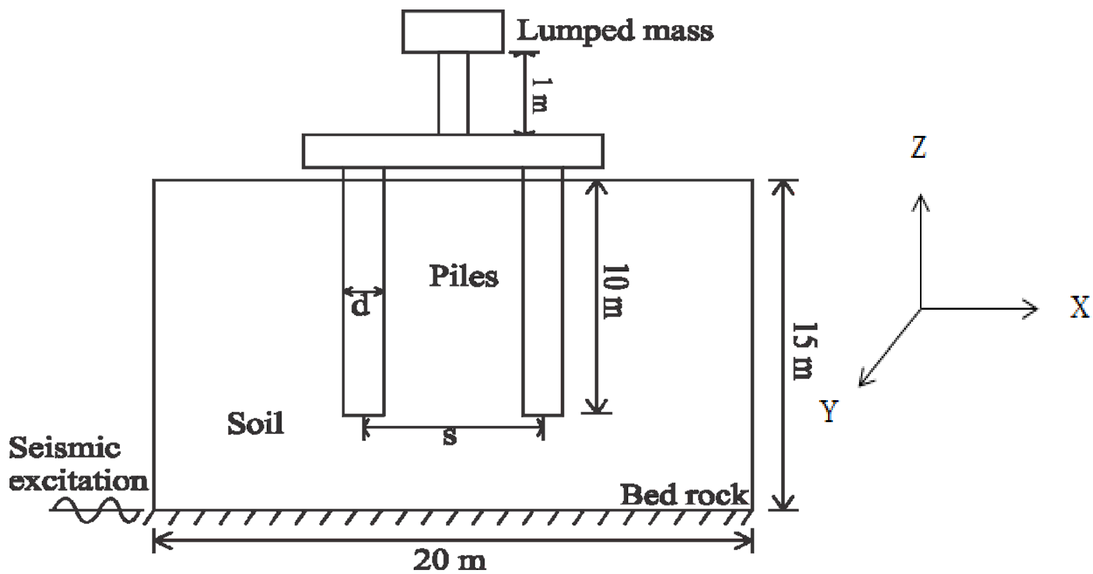


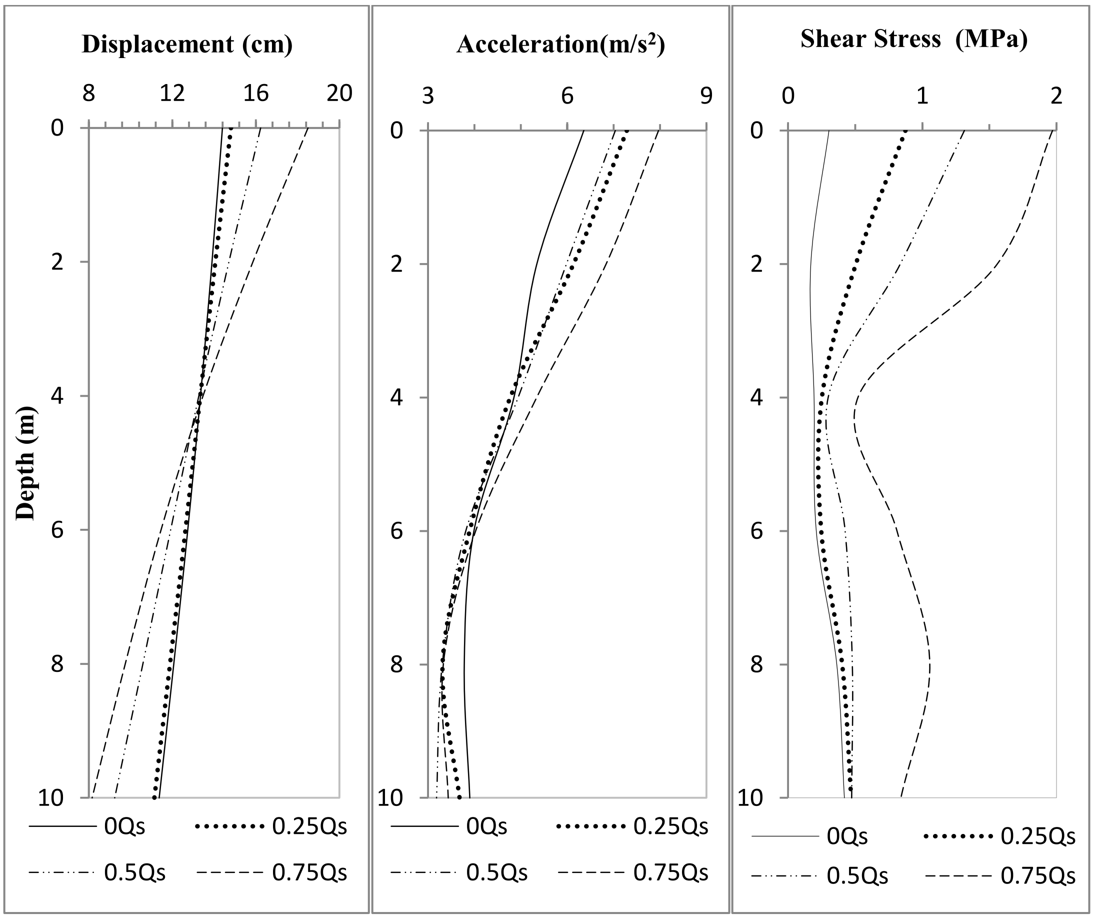
Fig.6.5: Physical model of soil-pile system with Superstructure

6.3.3 Effect of Inertial Forces

To analyze effect of superstructure lumped mass of 0 Q_s, 0.25 Q_s, 0.5 Q_s, 0.75 Q_s (Q_s = Safe load carrying capacity from IS2991 Part I/Sec 2) are applied. Fig 6.6 shows increase in displacement, acceleration and stress along the pile due to superstructure. For lumped mass of 0.75Q_s the pile head displacement is 18.50 cm that is 4.10 cm higher than the displacement without superstructure. This increment in displacement is due to effect of inertial forces. When superstructure mass is 0.25Q_s, increase in displacement is very small. Due to inertial forces pile starts rotating from 4m depth that can be seen in Fig. 6.6(a).

Effect of superstructure mass on response of acceleration along the pile is less. Fig. 6.6(b) shows that near the pile head acceleration has increased for high superstructure load. At $0.75Q_s$ amplification is 8 times from input motion. Pile head acceleration is amplified due to consideration of flexibility (Kanaujia et al. 2012).

Fig. 6.6(c) shows the maximum shear stress for vertical load $0.75Q_s$ is 1.97 MPa. shear stress increased at 8 m depth then decreased at 4 m, similar pattern for $0.5Q_s$ vertical loading also. At pile head higher shear stresses is due to inertia.



(a) (b) (c)

Fig.6.6: Effect of superstructure on pile foundation on (a) Displacement (b) Acceleration (c) Shear Stress

6.4 CONCLUDING REMARK

For single pile higher response is found in terms of shear stress but variation for displacement, acceleration is not too much. It is observed that lateral pile stress at pile head is higher in the presence of vertical load (load of pile cap) as compared to the pure lateral load case.

Design forces in piles are higher under combined loading as compared to the piles under pure lateral loading (Rajagopal and Karthigeyan (2008)), higher response are found in present study also. Rotation takes place due the higher superstructure mass.

Chapter 7

7 SUMMARY AND CONCLUSIONS

The investigation on Soil-Pile system using a 3D finite difference model in FLAC^{3D} has been reported in this chapter. Major conclusions based on the present analysis have been described. Scope for future work in the chosen topic is also suggested at the end of this chapter.

7.1 SUMMARY OF THE STUDY

In this study finite difference based software FLAC^{3D} is employed to model the soil-pile system. Three-dimensional modeling of problem was carried out using some of the most important inbuilt features of the software (*e.g.* free field boundary condition, nonlinear soil model). Effect of superstructure (lumped mass model) on the considered pile group is also examined. Step by step procedure for dynamic loading is described.

The soil-pile model developed in FLAC^{3D} was validated for lateral static loading from Rao et al. (2013) and Reese and Matlock (1956). In the case of dynamic loading boundary conditions (i.e. free field boundary) were validated by both analytical (i.e. free field response of soil block) and DEEPSOIL package. For dynamic loading 3-D model is verified by comparing kinematic interaction factors from existing literature.

Baseline correction was applied to all the considered earthquake motions, and then they were made response spectrum compatible as per IS 1893 Part I. Single pile was analyzed under dynamic loading to study the effect of increasing PGA (i.e. 0.1g,0.16g, 0.24g and 0.36g). Four earthquake motions with different predominant frequency (Bhuj, Loma Prieta, Friuli and Trinidad) have been considered to examine the effect of predominant frequency on single pile.

Pile group (2×2) with two different spacing (i.e. s/d =3, s/d=5) were analyzed to study the effect of PGA and the results were compared with single pile in the form of displacement, acceleration and lateral shear. A superstructure is modeled as lumped mass, to analyze the effect of inertial forces by varying the safe load carrying capacity of the foundation obtained as per IS2911 (Part 1/sec2) 2010 (i.e. 0%, 25%, 50%, 75% of Qs).

7.2 CONCLUSIONS

Major conclusions have been drawn from the present study, which are as follows:

- The response of Soil-Pile model has been validated for static loading by Rao et.al. (2013) and for dynamic loading with Sarkar (2009).
- The displacement and acceleration along the pile length was found to increase with increasing PGA. Lateral shear is also high along the pile but at pile cap variation is less for all PGA.
- Among the considered all four earthquakes, Bhuj earthquake results in higher displacement, acceleration and shear stress because of predominant frequency of Bhuj motion is near to fundamental frequency of soil-pile system.
- In the considered pile group (s/d=3 and s/d=5), the displacement and acceleration along the pile length is less in comparison to single pile.
- Extra self-weight of pile cap results in higher shear stress for pile group near pile head in compare to single pile.
- When there is a small increase in the weight of superstructure, then marginal amplification in the design parameters takes place. In contrast there was very significant amount of amplification observed when the weight of superstructure is high.

7.3 FUTURE WORK

The future work of scope can be summarized as following.

- Interface element between and soil and pile may be used to simulate more realistic behavior of soil-pile system
- For actual simulation of soil medium, shear modulus curve obtained from Cyclic tri-axial test may be assigned to plastic hardening soil model.

- To consider the effect of pore water pressure Finn and Byrne soil model can be used.
- In present work excitation applied only in one direction, study may be expanded for three dimension loading.

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