STUDY OF ECCENTRIC LATERAL LOADING ON ROCK-SOCKETED PILE GROUP

A DISSERTATION

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS

FOR THE AWARD OF THE DEGREE

OF

MASTER OF TECHNOLOGY

IN

GEOTECHNICAL ENGINEERING

Submitted by:

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CANDIDATE'S DECLARATION

I, MOHD FIZAZ, 2K20/GTE/13 student of M.Tech (GEOTECHNICAL ENGINEERING),hereby declare that the project Dissertation titled "STUDY OF ECCENTRIC LATERAL LOADING ON ROCK SOCKETED PILE GROUP" which is submitted by me to Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master Of Technology , is original and not copied from any source without proper citation .This work has not previously formed the basis for the award of any Degree, Diploma, Associateship, Fellowship or other similar title or recognition.

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CERTIFICATE

I hereby certify that the Project Dissertation titled "**Study of eccentric lateral loading on rock socketed pile group"** which is submitted by **Mohd. Fizaz, 2K20/GTE/13** in partial fulfillment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by the students under my supervision. To the best of my knowledge, this work has not been submitted in part or full for any degree or diploma to this university or elsewhere.

Place: Prof. A.Trivedi

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ABSTRACT

Piles must always be built to carry lateral loads in addition to compressive and tensile stresses. In the present study, a 2 × 2 steel pile group is framed in poorly graded soil subjected to eccentric lateral load. A gradual load of 0 to 10 kN is applied on the steel pile group socketed in the M50 grade of concrete mass. The depth of socketing is varied from 0 to 3D to determine the optimum depth of socketing. The effect of eccentricity on lateral deflection is observed along the pile length. A total of 12 numerical models are formulated to simulate the pile subjected to lateral eccentric loading. The increase in depth of socketing (120 to 360 mm) increases the lateral deformation at the centroid of pile cap. The increase in eccentricity from 0 to 240 mm decreases the lateral deformation at the centroid of the pile cap and vice versa. The pile head deflection reduces consistently as eccentricity varies from 0 to 240 mm for the set of selected depth of socketing. The optimum lateral deflection is observed at depth of socketing of 120 mm for $\frac{L_S}{R} > 4$.The results obtained for the present study are compared with codal provisions validating the study with minimal rock's effect in soil covers. The evaluation of lateral load bearing capacity of rock socketed pile group plays a vital role in the development of foundation design embeds in geomaterials.

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CHAPTER 1

INTRODUCTION

A pile foundation is a form of deep foundation that comprises of a thin column or long cylinder made of concrete or steel that holds the structure and distributes the load to the specified depth via end bearing or skin friction. Piles are used in foundations to transfer loads from a structure to underneath stratum capable of sustaining the weight. In general, piles transfer axial stresses mostly by skin friction along the shaft or primarily through the end bearing. When either of the foregoing weight transmission mechanisms is possible due to subsurface stratification at a specific location, piles are used. Pile foundations need careful piling system selection based on subsurface conditions, structural load characteristics, total settlement, differential settlement, and any other project-specific requirements. Pile placement necessitates meticulous attention to position, alignment, and depth, as well as specific knowledge and experience.

Pile group

A pile group is a collection of heaps with a pile cap that carry the load jointly. The pile cap would normally be in contact with the ground. The piles would be intended to divide the pile burden in an ideal situation. The pile cap would be built to connect the

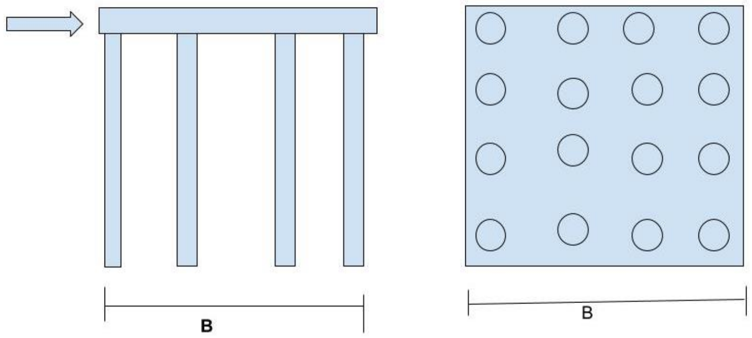

Fig. 1.1: Schematic diagram of a pile group subjected to lateral loading

Lateral loading on pile

In addition to compression and tension pressures, pilings must always be built to bear lateral loads. The soil conditions dictate the lateral capability of the piles. When lateral loads are present, the pile acts as a horizontally loaded beam, transferring lateral forces to the neighbouring soil via lateral resistance. Bending, rotation, and translation are all terms for when a pile flexes, turns, or shifts horizontally in reaction to external weight. Laterally laden piles are characterised as fixed head piles or free head piles based on whether the top part of the pile is fastened or not. Based on how they operate, frictional heaps and end bearing piles are indeed the two types of piles. The weight is transferred to friction piles via skin friction in between pile's embedded surface and the atmosphere soil. The loads are transmitted by the end bearing piles' bottom tips, which rest on a hard stratum.

Piles are used to support the building's substructure against vertical and lateral stresses. Tall chimneys, wharf, offshore constructions, tall retaining structures, towers, high-rise buildings, and jetties, and other structures are frequently subjected to high lateral stresses, which are then transferred to the foundations that sustain them as lateral shears. If the lateral loads are too great, inclined or batter piles can be utilised to help the vertical piles resist the lateral stresses (Murthy, 2012). For design purposes, analysing a laterally loaded pile to determine ground deflection, bending moment, shears, load-carrying capacity, and other factors is critical.

Eccentric lateral loading

The group effect in a pile group under eccentric lateral loads is significantly more difficult than in a pile group under lateral stresses due to the twist in the pile cap. Wind and wave movement, as well as ship collisions, cause large lateral stresses on tall structures, bridge spans, offshore structures, and wind turbines. The group effect in a pile group under eccentric lateral loads is significantly more difficult than in a pile group under horizontal stresses due to the twist in the pile cap. Insufficient foundation building to withstand such loads might have disastrous results. Tall buildings have been known to suffer major permanent deflections as a product of eccentric loads generated by strong winds. Individual piles' lateral, torsional, and axial resistances are all mobilised at the same time when a pile group is loaded eccentrically. The roles of these impedances to opposing the external load fluctuate dramatically under different situations. All transverse and torsionally stressed pile groups have been shown to have pile–soil–pile interactions. It's simply normal to assume analogous connections and the displacement coupling effect amongst isolated piles in tightly packed pile groups under eccentric lateral load.

Numerical solution of lateral loaded pile

The practical and theoretical solution to the problem of laterally stressed piles has been worked on by a number of researchers. For tackling laterally loaded pile difficulties, Reese et al (1974) and Matlock (1970) developed the notion of p-y curves. However, it has been shown that using p-y curves to forecast pile response is ineffective (Kim et al. 2004, Anderson et al. 2003).

The failure of p-y curves to accurately predict pile reaction is not surprising, according to Basu et al. (2008), because the p-y curves, which characterise the resistive qualities of soil as a function of pile deflection, were utilised in the p-y analysis to match the actual field pile-load test findings. As a result, p-y curves generated for one site do not apply to others. To use the p-y approach to accurately anticipate lateral pile response, p-y curves must be produced for each site through pile load experiments, which is an expensive option.

The difficulty with a horizontally loaded pile is similar to that of an elastic beam. External loads on a pile are expected either on or just above ground surface, unlike a beam over an elastic foundation, which can be loaded throughout its whole length. The principle of modulus of subgrade response is used in the majority of numerical solutions for laterally loaded piles. This is founded on the idea of a soil substance being represented by a group of tightly spaced separate elastic bands (Murty, 2012). The inaccuracy in Winkler's hypothesis has been shown to be insignificant. As a result, a succession of non-linear springs can be utilised to describe the soil's force-deformation characteristics in laterally loaded piles.

The key to solving the issue of laterally loaded piles is the modulus of subgrade reaction with respect to depth along the pile. The soil reaction p can be represented as at any point all along pile size x by

$$p = -E_s y \qquad (1.1)$$

Where the soil modulus is E_S, and y is the deflection at point x.

A nonlinear connection exists among soil resistance for every unit distance (p) and deflection(y). As a result, E_S is not constant, but fluctuates as the pile deflects. The pile width, flexural stiffness, applied load magnitude, and soil characteristics are all factors that determine the value of E_S. For any given load level, the variation in E_S with depth can be stated as:

$$E_S = \eta_h x \tag{1.2}$$

The soil modulus fluctuation coefficient is η_h.

Matlock and Reese (1960) developed a non-dimensional method for calculating deformation, slope, moment, shear, and soil reaction at any point x along a pile. The following are the equations:

Deflection $y = \frac{P_t T^3}{EI} A_y + \frac{M_t T^2}{EI} B_y$ (1.3)

Slope S = $S = \frac{P_t T^2}{EI} A_s + \frac{M_t T}{EI} B_s$ (1.4)

Moment $M = P_t T A_m + M_t B_m$ (1.5)

Shear $V = P_t A_V + \frac{M_t}{T} B_V$ (1.6)

Soil reaction $p = \frac{P_t}{T} A_p + \frac{M_t}{T^2} B_p$ (1.7)

T is the factor of relative stiffness, denoted by:

$$T = \sqrt[5]{\frac{EI}{\eta_h}} \tag{1.8}$$

Reese and Matlock theory is applicable for long pile i.e., $L > 5T$

The table may be used to generate non-dimensional parameter sets A and B, which is also the finite differences answer of the basic system of equations for a shaft on an elastic base. The coefficients are expressed in terms of the thickness coefficient Z:

$$Z = \frac{x}{T} \tag{1.9}$$

<u>Rock socketed pile</u>

A technique for inserting a pile in solid rock is known as "socket piles" (also referred as "rock sockets"). This might be required to fully use the piles' structural capability for both compressive and tensile stresses. It's a drilling technique for a rocky bottom when the depth appears shallow or the hard rock is sloping.

Drilling a socket somewhat larger than the pile through into the rocky layer is the technique. This leaves a void around the pile's exterior edge, which seems to be filled with grout, which is either portland cement or an ultra-high quality grout, depending on the prerequisites. By withstanding loads and uplift forces, this "socket" in the rock involves keeping the pile stable.

Pile capacity

The soil response induced and the strength properties of the shafts during bending determine the horizontal load behavior of a single pile. The horizontal load capacity of the pile would have to be determined using appropriate soil horizontal subgrade modulus values. Wind, earthquakes, water currents, earth pressure, the action of travelling cars or ships, plant and equipment, and other factors can all produce lateral stress on a pile. The horizontal load capacity of a pile is determined by the maximum stress of the pile length against bending caused by the introduction of a horizontal forces, as well as the lateral subgrade elasticity of the surrounding ground. When determining the structural capacities of the shaft, additional available loads, like the axial load on the pile, must be considered.

Fixed and free head conditions

A collection of three or even more piles linked by a hard pile cap is known as a fixed head condition. Grade beams must link caps for single piles in two directions, and grade beams must connect twin piles on a line lateral to the pair's common available axis. The pile shall be deemed free headed in all other circumstances.

Length of pile socketed in rock

Excessive cutting may cause a decline in resistance factor due to its smooth surfaces of holes drilled in rock to ensure precise socketing of pile onto surface layer, and severe chiselling may cause fractures in the rock mass. Both of these impacts may be damaging to piles' safe load bearing capacity. As a result, while deciding on the embedding of a pile in rock and determining the length of the socketing, extra care must be taken to maximise the actions of friction (on the rock stratum surface) and bearing on rocks. The kind of rock, the level below the pile cap where the rock is available, and the pile's pressure carrying capability are commonly used to estimate the socketing depth. When piles are laid on rock at short depths (with little depth of load) and/or on slanted solid rock, solid rock socketing is critical for all practical reasons. When solid rock is found at deeper levels, with a significantly bigger overburden to resist the lateral displacements, pile socketing as currently defined throughout most piling necessities may not be required, and heavy scraping for specified socket size will be a waste of time, as well as degrading the quality of hard rock. Cutting sound rock for a minimal of 1D is the current practise for socketing piles in rock (for large diameter piles).

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Wind & wave movement and ship impacts leads to substantial lateral pressure on foundations for a variety of structures, including as bridge bends, wind turbines, high rises and electric power transmission towers. Due to the eccentricity of the load imposed and/or the uneven geometry of large structures, eccentric lateral forces may be conveyed to the basement of these structures. A poor foundation design for such weights might spell tragedy. Wind activity has caused irrevocable damage to at least two onshore structures, including major torsion deformations. When a pile group is eccentrically loaded, the individual piles' lateral, axial, and torsional resistances are all deployed at the same time.

2.2 Lateral loaded pile

According to the study conducted by Phanikanth et al. (2015), earth pressure, earthquake or wave force, and wind forces must all be considered when designing pile groups and single piles. This necessitates the calculation of ultimate loads, which will be used to determine safe working loads, as well as the estimation of pile deflections, which will ensure that serviceability issues are addressed in the design. The failure mechanisms and behaviour of pile foundations are influenced by the pile's characteristic length (L), and many failure processes were discussed. The lateral force behavior of separate piles in cohesion less soils is investigated for a variety of subgrade moduli that indicate diverse soil types such as medium sand, loose sand, and thick sand. Both dry and rainy conditions are taken into consideration. The analysis is done with a hanging tip at the bottom of a free-headed pile. The influences of type of soil, pile depth, and pile radius upon pile reaction are investigated, with the findings reported. For various soil types, deflection and moment coefficients for a conventional pile are also determined. For the aforementioned study, the modulus of subgrade response method is implemented using a numerical solution technique and written in MATLAB.

According to the study conducted by Abdrabbo and Gaaver(2012), a tough soil–structure interaction problem is laterally loaded pile group reaction. Despite the development of very effective algorithms for predicting the lateral behaviour of individual piles, the horizontal reaction of pile groups has gotten less attention because to the significant cost and difficulty required. This study analyses horizontally loaded pile groups using a basic approach. P-multiplier factors are combined with the lateral modulus of subgrade action in the proposed technique. In a series of closely spaced piles, the influences of shadowing were investigated. Laterally laden piles buried in sand can be assessed within the operational load range provided the lateral load and displacements have a linear correlation, according to research. The suggested method calculates lateral load redistribution across piles in a pile group to predict the pile group's appropriate design lateral load. The recommended approach has the advantage of being simple and needing minimal computing work for the initial design stage.

Accordung to Sun(1994), a numerical approach and parametric study are used to calculate the earth and pile interface under lateral loading. Variational calculus is used to determine the controlling differential equations of the soil as well as pile systems. This model presents the elastic basis using a primary parameter (γ). To have a better grasp of the relationships among the primary model variables, such as Poisson's ratio v, deformation, and elasticity factor K_r, the quality and importance of those parameters are extensively explored. The importance of linear linkages among coefficients and simple analytical equations is underlined. Using the novel feature and related mathematical expressions, greater prediction capacity is obtained. This broad analytical perspective allows the proposed model to be modified and refined, as well as a path for laterally loaded pile design development.

In the study conducted by Duncan et al.(1994), the p-y technique of analysis is an useful approach to design footings that are prone to lateral stresses. Deflections and bending moments predicted by P-y analysis have regularly matched field measurements. The characteristic load approach (CLM) is a simpler alternative to p-y analysis that closely approximates p-y results. By leveraging correlations among dimensionless variables, the approach uses multidimensional analysis to understand the dynamic behavior of laterally loaded piles and drilled shafts. The new method is easy enough to be completed by manually, but it might be adapted for use on a computer. CLM's calculated lateral deformations and flexure moments matched the values obtained in field load testing.

According to Jayasree et al.(2018), laterally laden piles are often analysed utilising the IS 2911-2010 (Part 1/Section 2) approach, according to Indian Standards. However, practising engineers believe that the IS technique is overly cautious in its design. The goal of this study is to see how conservative the traditional IS design methodology is. This is accomplished by comparing the IS technique to a theoretical model on the basis of Vesic's equation. The Kerala PWD provided information on six separate bridges' bore logs. In situ fixed head piles were sunk in three soil conditions, so each end bearing and friction pile was tested and evaluated individually. STAAD was also used to model piles. The Matlock and Reese (1960) equation was used to validate the results of the Pro programme based on the IS technique. The findings were expressed as a percentage variation in between flexure moment and distortion values acquired using various techniques. When the findings from the experimental model on the basis of Vesic's (1961) equation were compared to those obtained using the IS method, the IS method was shown to be uneconomical and conservative.

According to Meyerhof et al.(1983), in sand the ultimate strength of stiff piles and pile groups has been established for a range of eccentricity and force tilt combinations from vertical to horizontal. The theoretical estimations are contrasted to the findings of single model pile and free-standing group load testing. Simple interaction formulas between failure load and moments, as well as the axially and normal sections of the ultimate load, may be used to demonstrate the effect of eccentricity and tilt on bearing capacity. Based on prior theoretical and model test results, the effect of a pile cap laying on the earth in piled foundations is investigated.

The Study conducted by Mehra and Trivedi (2021) considers pile group subjected to combined axial and torsional loads. A numerical model is prepared to observe the effect of nonlinear pile–soil interaction in a flow-controlled geomaterials. The variation is observed between parameters like normalized pile head displacement & normalized axial load, normalized pile head twist & normalized applied torque, twist parameter & normalized applied torque, displacement parameter & normalized axial load and dilation angle with displacement parameter & twist parameter. It has been observed that the resultant displacement increases with the torsional load for the large diameter pile and pile groups. Similarly, the twist increases with an increase in the axial load.

2.3 Eccentric lateral loaded pile

According to the study conducted by Gu et al.(2014), the response of a 1×2 pile group under non-concentric lateral loading was investigated using a large scale prototype test in silts. Two well-instrumented steel piles were set in a large dirt tank with three-pile radii on centres as the model pile group. The test findings demonstrated that lateral forces eccentricity had a minor influence on the 1×2 pile group's performance, but it did play a considerable role in internal pile force unevenness. In comparison to a torsionally stressed single pile, the coupled effect of deflection and torque led to a significant positive growth in the torsional impedance of each pile inside the group. As the imposed lateral load rose, the contribution of each pile torsional resistances to opposing external torque decreased. For the piling group, a three-dimensional finite-element calculation was also undertaken, and the simulated reaction is considered to be quite close to the actual test results. The same approach was utilised to investigate more cases with various loading scenarios. According to the assessments, every pile design within the group clearly impacted the behaviour of the 1×2 pile group when subjected to eccentric lateral stresses.

In the study conducted by Kong et al.(2019), since the grouped piles have various movement patterns caused by the rotation of the pile cap, the group effect inside a pile group with eccentric lateral loading is far more difficult than in a pile group during lateral pressure. The goal of this research is to create new p-multipliers for evaluating the total group effect in pile groups that have already been subjected to non-concentric lateral loading. The activities of two independent piles in a pile group were first investigated under eccentric lateral load, and the idea of the leading pile and the trailing pile was developed to characterise the two piles' relative placements. The interactions between two piles were then studied using numerical analysis, theoretical analysis and centrifuge model tests. The angles between the leading and following piles' motion directions, and the line through both piles, are represented by η and θ, are observed to vary between 0°–90° and -90°–90°, respectively. Both the leading and trailing piles' reduction factors are nonlinearly affected by η and θ. If interaction exists, the leading pile's reduction factor is greater than the trailing pile's. On the η-θ plane, the interaction of the two piles is minimal in certain areas. To define the boundary with and without contact for a given, the idea of critical angle θ_0 was established. Finally, empirical reduction factor equations and a p-multiplier calculation process were suggested and tested using existing test findings.

In another study conducted by Kong et al.(2015), he describes two 3×3 pile group studies in wet silt horizontally loaded at asymmetrical lengths of 6D and 11D. The model piles were 0.114 m in diameter and 5.95 m long. The tests' objectives are to examine: (1) the pile group's response to eccentric lateral forces and the effect of load eccentricity; and (2) the piles' stress distribution and their ability to sustain the imposed lateral loads and torques. According to the model trials, the lateral stiffness of the pile group reduced significantly as eccentricity increased. The pile group with just an eccentric distance of 6D produced 1.5 times the lateral pressure of the pile group with a non-concentric distance of 11D at the a horizontal displacement of 50 mm in the pile cap centre. Pile-head shear pressures, bending stresses, and horizontal pile displacements also varied widely in magnitude and orientation. The piles' shear resistance helped to resist the lateral stresses and torques that were applied. At varied load eccentric distances, different twist centres were discovered in the 33% pile group. In studies with eccentric lengths of 6 and 11D, the distance between the twist centre and the pile cap centre was 9 and 3.7D.

2.4 Rock-socketed pile

According to Prakash and Muthukkumaran (2021), massive rock socketed piles were employed to safely transmit a high amount of both vertical and horizontal load out through the building to a deeper depth without causing structural issues. The effectiveness of a rock socketed pile inside a soil-rock stratum profiles system under static lateral load was investigated using a model pile and a series of experiments. To understand the extent of pile motion and the load transmitted, the model piles were outfitted with motion and force transducers. The thicknesses of the pile's soil anchoring as well as the thickness of the rock plug had a significant influence on the horizontal capacity of rocks socketed piles. The lateral capacity of the pile improves significantly when the thickness of socketing into rock is three times the diameter of the pile with little embedment. The lateral capacity of non-socketed piles is around 18 times that of 3D socketed piles. According to the experimental evaluation, the depth of fixity increases as the piles are socketed deeper into in the bedrock (rock), and lateral movement diminishes significantly.

According to Srinivasamurthy (2009), socketing bored piles in rock is often a point of contention between consultants and contractors on the job site. The friction component formed all along length of the socket is typically neglected when rock socketed piles are thought to be end bearing piles. Due to the flat plane of drill holes in rock to provide perfect socketing of pile on surface of the material, heavy grinding may reduce friction component, and excessive chiselling may generate fissures in the rock mass. Both of these effects might cause piles to lose their safe load bearing capability. As a result, extra attention must be taken while determining just on socketing of a pile into rock and the depth of the socketing to maximise the effects of abrasion (on the rock layer surface) and wearing on rocks. To assess the socketing depth, the type of rock, the dimension underneath the pile cap where the stone is present, and the pile's pressure carrying capacity are typically employed. The length of the rock socketing should be sufficient for the pile to safely bear its structural strength. Various approaches are provided for determining the optimal length of socket in rock. Going for more socketing length than necessary is a waste of time, work, and money. Both the terminal bearing and the resistive element from friction force inside the socketed width should be considered when stone socketing in weathered/soft rock.

According to Singh et al.(2017), plaxis 3D has been used to predict the behaviour of a stone pile with isolated and combined loading in this study. The load test results reported in the literature were used to validate the numerical technique used in the study. Parametric evaluations of the rock socketed pile exposure to isolated and coupled pressure for various rock qualities, soil cover depths, and socketing lengths were done just after numerical studies. When the soil cover layer is thick, horizontal and vertical load capabilities under combined loading remain unaffected, according to the parametric investigation. When the soil depth is limited, the performance of a rock-socketed pile with various loading differs significantly from its performance under standalone loading. Under coupled loads, the rock qualities and socketing duration have a significant impact on pile behaviour. When measuring the load bearing capability of a pile in compression load, the full pile length is vital to consider. This is because the elastic compression of the pile is critical in determining the pile's ability to resist settling under the heavy intensities whereby the sockets are built. For varying socket lengths, the load carrying behaviour of stone piles during combined loading isn't much different from independent loading, although the soil cover depth has a major impact. Both socket height and cover depth affect the lateral force qualities of a rock-socketed pile under combined loads. Up to acceptable load limits, lateral applied load has little effect on the vertical load - carrying capability of piles with substantial soil cover. This is because the rock socket determines vertical load bearing capacity, but the stiffness of the top soil layer determines lateral load carrying capacity.

According to the study conducted by Jafari et al.(2019), the bearing capacity of steel pipes in igneous rocks is evaluated, which is significant in engineering since it affects the safety of anchored edifices as well as pile soundness during pile driving activities. Using a finite element model, the effect of longitudinal reinforcement depth on the circumferential base bearing capacity and crack pattern of typical open-ended pile foundation in rock strata masses is investigated, assuming the sedimentary rock is marked as a linear elastic and nicely plastic material merely following the Hoek-Brown criterion. The pipe pile's toe is rough, whereas the walls are polished. The circumferential toe resistance of pipe piles can be used to anticipate the rock mass resistance to totally coring drive, which is common for different diameter open-ended pipe piles. Round foundations and implanted strip bases socketed in intact rock are among the findings. The findings illustrate how the pipe pile's annular shape causes an uneven failure mechanism with respect towards the pipes centre, as well as a slant of rock mass reaction, which, if severe enough, can cause pile converging and damage throughout pile drive operations. The failure process depicts the plug's development and the pipe's upward travel. According to the research, the load carrying capacity of pile foundation approaches a theoretical limit in most realistic scenarios, which is less than or equal towards the terminal bearing capacity of an inserted foundation system with such a width equal to the plate thickness. A comparison with experimental data was done. In terms of rock mass impedance and the mechanism of rock plug production, the results are determined to be considerably in agreement with test data.

Seol and Jeong(2009), describes in his study that O-Cell testing is commonly used to determine the burden behavior of big drilled shafts filling it up in rock. The equivalent top load–settlement curve constructed by summing the produced shaft impedance and end stress at the same deformation ignores the pile-toe settling caused by the force transmitted along of the pile shaft. The use of the coupled load-transfer approach to assess the impact of linked compressive stresses, which is significantly connected to maximum load to soil modulus (D/E_s) and overall shaft resistance to total ultimate force (R_s/Q), is the emphasis of stone drilled shafts. A modified Mindlin's point load method was used to produce the recommended methodological approach, which takes into consideration the influence of linked shaft resistance. When evaluated to field case studies, the suggested technique correctly predicted ultimate load effectiveness and coupled results due to axis shear loading transfer. The O-Cell test is much less accurate in forecasting load–settlement behaviour of bored shafts subjected to bi-directional stress, according to these findings.

According to Chong et al.(2011), the p–y behaviour of laterally laden piles is related to pile head load-deflection response, is influenced by secondary properties such as joints. The three-dimensional unique element code 3DEC was used to do detailed mathematical analysis on piles socketed into mudstone rock in this work. The mudstone p–y curves were generated using the results of two full-scale pile load experiments to confirm the numerical model. Using the confirmed numerical model, the impact of different precisely described on the p–y and pile top load-deflection behaviour was examined next. The p–y behaviour of a rock mass and the consequent pile cap load-deflection response are significantly reduced as the number of joint sets increases. When mechanical joint sets were included, the previous p–y criteria were found to be insufficient to capture the variability in the stone's p–y responsiveness.

As per Kou et al. (2016), the findings of dynamic compressive stress testing on long rock-socked bored piles put in stratified soils are presented in this study. Three bored piles with a combined immersed height of 34 m were equipped to distinguish shaft and base opposition and examine the spread of tensile stresses along the pile shaft. Traditional methods were used to calculate shaft resistance. The shaft friction produced and over 78 percent of the compressive stresses at the conclusion of the testing. The basal resistance was not sufficiently mobilised during the test. The guidelines for assessing shaft resistance in lengthy rock-socketed footings were more conservative. Because of the varying geological conditions, empirical equations to determine shaft resistivity are restricted. Methods that directly evaluated factors from standard test procedure (SPT) data produced the most consistent estimates.

2.5 Research gap

The following research gap has been identified:

- The variable determined in lateral loaded piles are limited to lateral deflection and many other parameters like torsion, internal shear force etc., needs further exploration.
- The eccentric lateral loading on piles is still very less touched topic for the analysis.
- The literature on eccentric lateral loading on rock socketed piles is very limited.
- For different soil conditions the effect of rock socketing and the effect of water table on lateral analysis of piles needs further evaluation.

CHAPTER 3

PROJECT WORK

3.1 Objective

- To determine the response of rock-socketed pile group subjected to eccentric lateral loading.
- To determine the load-displacement response for varied eccentricity.
- To determine the effect of rock socketing against the lateral loading.
- To determine the optimum length of rock socketing.

3.2 IS code recommendation (IS 2911 (Part 1/Sec 1): 2010)

3.2.1 Analysis of laterally loaded piles

a- General

A vertical pile's maximum resistance to a lateral pressure, as well as its deviation at the extreme of loading values, are tough problems to solve and quasi-rigid structural member interacts with soil that contracts partly elastic deformation and partially plastically. An endlessly longer pile and a smaller stiff pile have fundamentally different failure mechanisms. Unrestricted and constrained pile heads both have different failure modes. Because of the problem's intricacy, only an approximate solution, which is appropriate in the vast majority of circumstances, is offered here. Situations that require a thorough examination will be handled accordingly.

The first stage is to decide whether the pile may become a compact, hard item or an endlessly long, flexible component. The rigidity factor R or T is determined for the specified pile and soil combination.

The conditions for operating as a short stiff pile or a long elastic pile are connected to the pile's incorporated length L that once rigidity factor has been computed. The distance from the ground surface to the virtual fixity point is then established, and the lateral deflection and bending moment are calculated using a typical elasto-plastic analysis.

b- <u>Stiffness factors</u>

The horizontal soil resistance is determined using the equation:-

For granular soils and generally cemented clays with variable soil modulus

$$\frac{p}{y} = \eta_h z \quad (3.1)$$

Where

p = Lateral soil response at a depth z per unit length of pile below ground level

y = Deflection of pile in lateral direction

η_h = Modulus of subgrade reaction for which the suggested values are given in Table 1

Table 3.1: Modulus of subgrade reaction for granular soils, η_h in kN/m^3 (IS 2911 (Part 1/Sec 1), 2010)

S. No.	Soil type	N(Blows/30 cm)	Range of modulus of subgrade reaction (kN/m^3 ×10^3)	
			Dry	Submerged
1.	Very loose sand	0-4	<0.4	<0.2
2.	Loose sand	4-10	0.4-2.5	0.2-1.4
3.	Medium sand	10-35	2.5-7.5	1.4-5.0
4.	Dense sand	>35	7.5-20.0	5.0-12.0

The lateral soil resistance is determined using the equation for preload clays with constant soil modulus:

$$\frac{p}{y} = K \quad (3.2)$$

Where $K = \frac{k_1}{1.5} \times \frac{0.3}{B}$ (3.3)

where k_1 is calculated using load-deflection observations on a square plate of dimension 30 cm, known as Terzaghi's modulus of subgrade response, and B is pile breadth. Table 3.2 shows the suggested values for k_1.

Table 3.2: Modulus of subgrade reaction for cohesive soil, k_1, in kN/m^3 (IS 2911 (Part 1/Sec 1), 2010)

S. No.	Soil consistency	Unconfined compressive strength, q_u (kN/m^2)	Range of modulus of subgrade response (kN/m^3 ×10^3)
1.	Soft	25-50	4.5-9.0
2.	Medium stiff	50-100	9.0-18.0
3.	Stiff	100-200	18.0-36.0
4.	Very stiff	200-400	36.0-72.0
5.	Hard	>400	>72.0

Note — For q_u less than 25, k_1 can be set to 0, implying that no lateral resistance exists.

Stiffness factors:

i- Piles in normal loaded clays and sands

Stiffness factor T in m, is expressed as $= \sqrt[5]{\frac{EI}{\eta_h}}$ (3.4)

where

E = Pile material Young's modulus, in MN/m^2

I = Moment of inertia of cross-section of pile, in m^4

η_h = Modulus of subgrade reaction, in MN/m^3 (see Table 1)

ii- Piles in preload clays

Stiffness factor R, in m, is expressed as $= \sqrt[4]{\frac{EI}{KB}}$ (3.5)

where

E = Pile material Young's modulus, in MN/m^2

I = Moment of inertia of cross-section of pile in m^4

$K = \frac{k_1}{1.5} \times \frac{0.3}{B}$ (See Table 2 for values of k_1, in MN/m^3)

B = Pile shaft width in m.

c- Criteria for long elastic piles and short rigid piles

As illustrated in Table 3, the embedded length L influences if the pile acts as a small stiff pile or a longer elastic pile.

Table 3.3: Pile behaviour criteria on the basis of its embedded length(IS 2911 (Part 1/Sec 1), 2010)

S. No.	Pile behaviour	Relationship between embedded length and stiffness factor	
		Linearly increasing	Constant
1.	Short (rigid) pile	$L \leq 2T$	$L \leq 2R$
2.	Long (elastic) pile	$L \geq 4T$	$L \geq 3.5\ R$

Note —The intermediate length L should be used to distinguish between elastic and rigid pile behaviour.

d- Moments and deflection in longer elastic piles

The comparable cantilever beam approach is a simple way to calculate deflections and moments caused by modest lateral loads. This necessitates determining the hypothetical fixity depth, z_f. The length to the point of hypothetical fixity can be calculated using the fig. 3.1. Either converting the moment to an appropriate horizontal load condition or calculating the true position of the horizontal applied load yields the efficient eccentricity of the location of force application. The stiffness variables R and T are as previously stated.

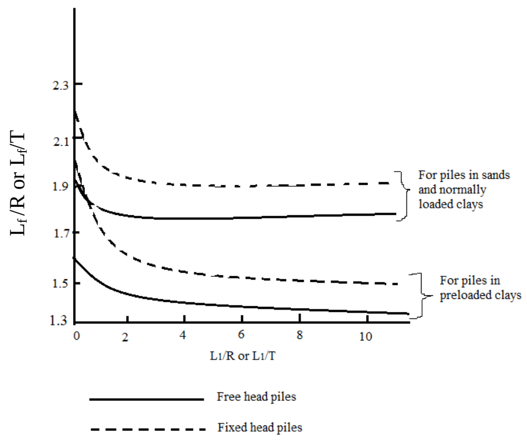

Fig. 3.1: Depth of fixity (IS 2911 (Part 1/sec 1), 2010)

The following formulae must be used to calculate the pile head deflection, y:

Deflection, $y = \frac{H(e+Z_f)^3}{3EI} \times 10^3$ for free head pile (3.6)

Deflection, $y = \frac{H(e+Z_f)^3}{12EI} \times 10^3$ for fixed head pile (3.7)

where

H = Lateral load on pile, in kN

y = Pile head deflection, in mm

E = Modulus of elasticity of pile material, in kN/m^2

I = Moment of inertia of cross-section of pile, in m^4

z_f = Depth to point of fixity, in m

e = Length of cantilever above ground/bed to point of applied load, in m

The following formulas can be used to calculate the pile's fixed end moment for the corresponding cantilever:

Fixed end moment, $M_F = H(e + Z_f)$ for free head pile (3.8)

Fixed end moment $M_F = \frac{H(e+Z_f)}{2}$ for fixed head pile (3.9)

3.3 Lateral load capacity of rock socketed pile

A socketed pile's stiffness, including its load-deflection qualities, soil depth, and the rock stratum in which it has been socketed, all affect its ability to sustain lateral loads.

The depth of fixity, lateral deviation, and peak moment all must be determined before determining the strength of a pile filling it up in rock. When three separate types of pile anchoring in rock are recognised, the design is altered.

a) Pile that passes through a deep layer of earth [Fig. 3.7(a)],

b) Intermediate soil mass and rock mass thickness [Fig. 3.7(b) and

c) Fully embedded pile in rock mass [Fig. 3.7(c)]

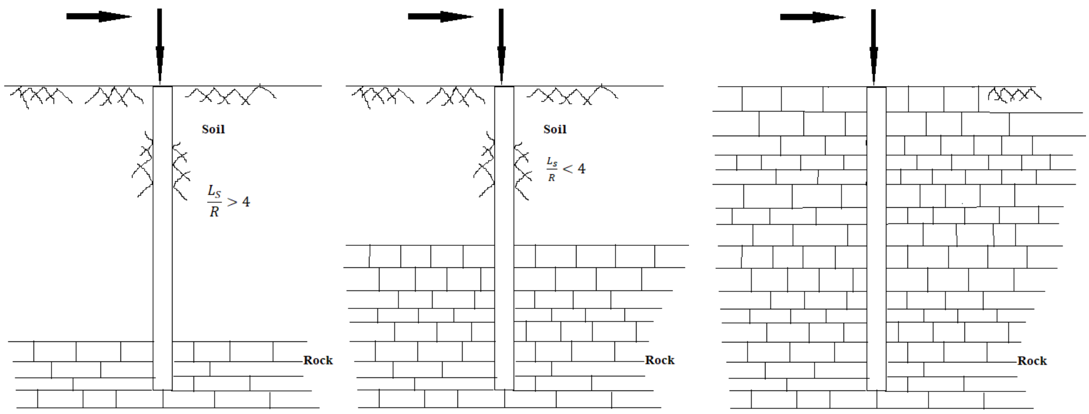

Fig. 3.2(a) Pile supporting rock with minimum rock penetration

Fig. 3.2(b) Pile fully fixed in rock below a thick rock layer

Fig. 3.2(c) Pile fully embedded in rock

Fig. 3.2: Different amount of embedment in rocks (IS 14593, 1998)

<u>3.7.1 Design considerations</u>

<u>Case (a)</u>

Whenever the rock by which the pile travels is covered by a sufficiently thick layer of the soil (Fig. 3.7(a), the impact of such rock is limited, and the ability evaluation approach can indeed be performed in the same way as IS 2911 (Part l/&c 2).

$$\frac{L_S}{R} > 4 \qquad (3.14)$$

Case (b)

In the event of intermediate pile Fig. 3.7(b), the end constraint situation of the socketed pile should be taken into account while determining lateral capacity:

$$\text{If } \frac{L_S}{R} < 4 \qquad (3.15)$$

Where

L_S = Pile length in soil, and

R = Factor of relative stiffness as per IS 2911 (Part l/Sec 2)

The contribution of the rock can also be accounted for by taking into account the difference in subgrade reaction modulus with depth.

Case (c)

As illustrated in Fig. 3.7(c), a pile's lateral capacity is defined by the capacity of its section to resist shear stresses when completely immersed in rock having a higher modulus than the pile material.

3.4 Experimental design and programme

The finite element programme abaqus was used to numerically simulate the 2×2 pile group at eccentric lateral stress. Finite element analysis (FEA) is a computer-aided method for predicting how a product will behave in the real world to pressures, disruptions, heat, fluid movement, and other physical factors. Finite element analysis is used to determine whether or not an object will fail, wear out, or function as planned. Analysis is a technique used for the development process to forecast what will happen when the product is used.

3.5 Specifications

Pile

Diameter = 120 mm = 0.12 m

Length = 6000 mm = 6 m

Material = Steel

Modulus of elasticity = 200 GPa = 200 × 10^9 N/m^2

Density = 7850 kg/m^3

Poisson's ratio = 0.28

Pile cap

Length = 800 mm = 0.8 m

Width = 800 mm = 0.8 m

Depth = 300 mm = 0.3 m

Material = Concrete grade M30

Modulus of elasticity = 27386.12 N/mm^2 = 27386.12 × 10^6 N/m^2

Density = 2.4 g/cm^3 = 2400 kg/m^3

Poisson's ratio = 0.2

Soil mass

Length = 5000 mm = 5 m

Width = 5000 mm = 5 m

Depth = 9000 mm = 9 m

Classification = Poorly graded sand (SP)

Cohesion = 0

Angle of internal friction = 34.6°

Specific Gravity = 2.62

Density = 1750 kg/m^3

Poisson's ratio = 0.25

Coefficient of uniformity C_U = 2.5

Coefficient of curvature C_C = 1.16

Modulus of elasticity = 18 N/mm^2 = 1.8×10^7 N/m^2

Rock properties

Concrete grade M50

Compressive strength = 50 N/mm^2 = 50×10^6 N/m^2

Modulus of elasticity = 35503.81 N/mm^2 = 35503.81×10^6 N/m^2

Density = 2.4 g/cm^3 = 2400 kg/m^3

Poisson's ratio = 0.2

Loading

An increasing loading is applied with the maximum capacity of 10000 N.

Load eccentricity

e = 0, 60mm, 120mm, 240mm

Depth of rock socketing

1D = 120mm, 2D = 240mm, 3D = 360mm

Software used- Abaqus

3.6 Methodology

Calculation of relative stiffness factor

E = 200 GPa = 200×10^3 MPa

I = $\frac{\pi D^4}{64} = 1.0178 \times 10^{-5}\ m^4$

η_h = horizontal modulus coefficient of subgrade response = 2.5 MN/m^3

$$T = \sqrt[5]{\frac{EI}{\eta_h}}$$

T = 0.9597 m

If depth of socketing = 1D = 120 mm then pile length embedded in soil L = 5880 mm

L (5880 mm) ≥ 4T (3838.8 mm) hence pile is long (elastic) pile.

$$\frac{L}{T} = 6.12 > 4$$

If depth of socketing = 2D = 240 mm then pile length embedded in soil L = 5760 mm

L (5760 mm) ≥ 4T (3838.8 mm) hence pile is long (elastic) pile.

$$\frac{L}{T} = 6.00 > 4$$

If depth of socketing = 3D = 360 mm then pile length embedded in soil L = 5640 mm

L (5640 mm) ≥ 4T (3838.8 mm) hence pile is long (elastic) pile.

$$\frac{L}{T} = 5.87 > 4$$

If depth of socketing = 6D = 720 mm then pile length embedded in soil L = 5280 mm

L (5280 mm) ≥ 4T (3838.8 mm) hence pile is long (elastic) pile.

$$\frac{L}{T} = 5.50 > 4$$

Hence in all the cases the pile is long (elastic) pile.

Different module involve in the model preparation are as follows:

<u>Part</u>

A 2×2 pile group is modelled in Abaqus software by using module part, in the following sequence:

<u>Pile</u>:- Circular cross-section, diameter = 0.12 m, length = 6 m

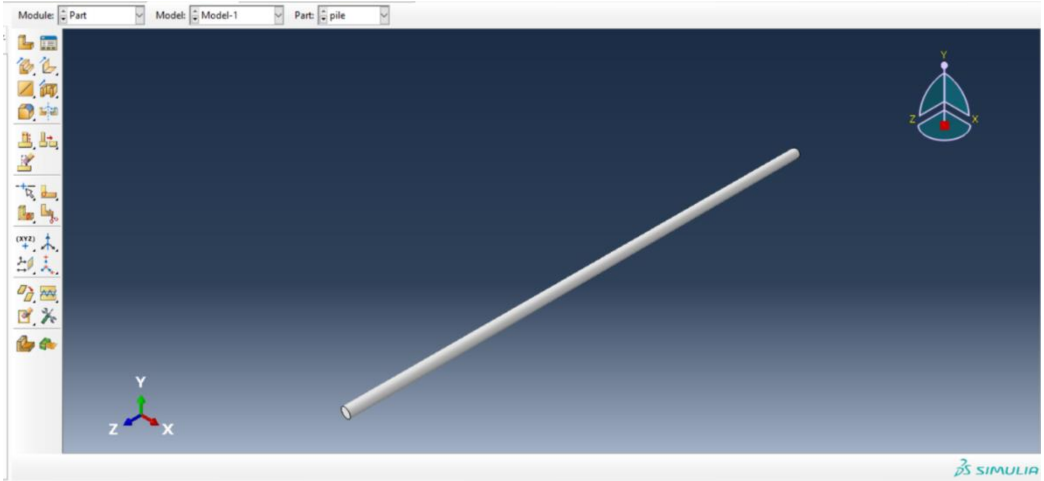

Fig. 3.3: Abaqus model of pile with diameter 0.12 m & length 6 m

Soil mass

Length = width= 5 m, depth = 9 m

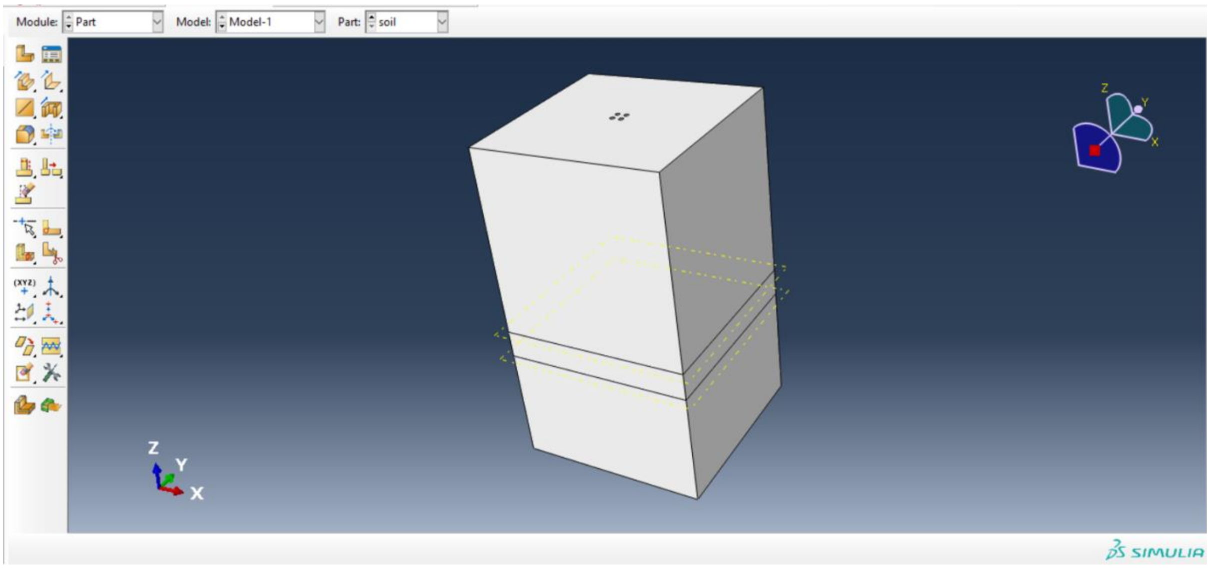

Fig. 3.4: Abaqus model of soil mass with length & width =5 m and depth = 9m

Pile cap

Length = width = 0.8 m, depth = 0.3 m

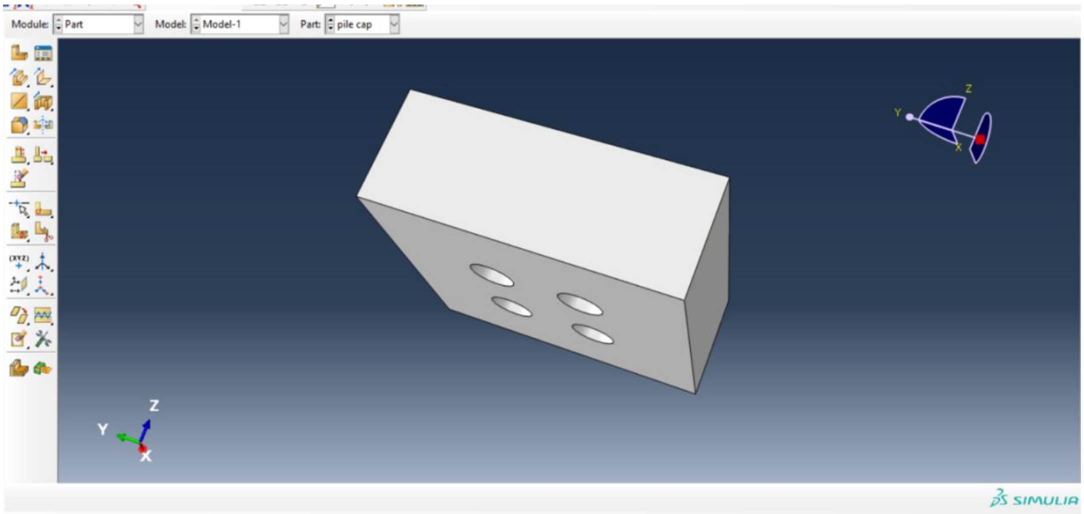

Fig. 3.5:Abaqus model of pile cap with length & width = 0.8m and depth= 0.3m

Property

The different elements are assigned by their respective properties by using module property as follows-

Pile:- Material = Steel

Modulus of elasticity = 200 GPa = 200×10^9 N/m^2

Density = 7850 kg/m^3

Poisson's ratio = 0.28

Soil mass:- Soil classification = Poorly graded sand (SP)

Cohesion = 0

Angle of internal friction = 34.6°

Specific gravity = 2.62

Density = 1750 kg/m^3

Poisson's ratio = 0.25

Coefficient of uniformity C_U = 2.5

Coefficient of curvature C_C = 1.16

Modulus of elasticity = 20 N/mm^2 = 2×10^7 N/m^2

Pile cap:- Material = Concrete grade M30

Modulus of elasticity = 27386.12 N/mm^2 = 27386.12 × 10^6 N/m^2

Density = 2.4 g/cm^3 = 2400 kg/m^3

Poisson's ratio = 0.2

Rock properties:- Concrete grade M50

Compressive strength = 50 N/mm^2 = 50 × 10^6 N/m^2

Modulus of elasticity = 35503.81 N/mm^2 = 35503.81 × 10^6 N/m^2

Density = 2.4 g/cm^3 = 2400 kg/m^3

Poisson's ratio = 0.2

Assembly

By using this module all the parts are combined together to form the combined assembly of the model.

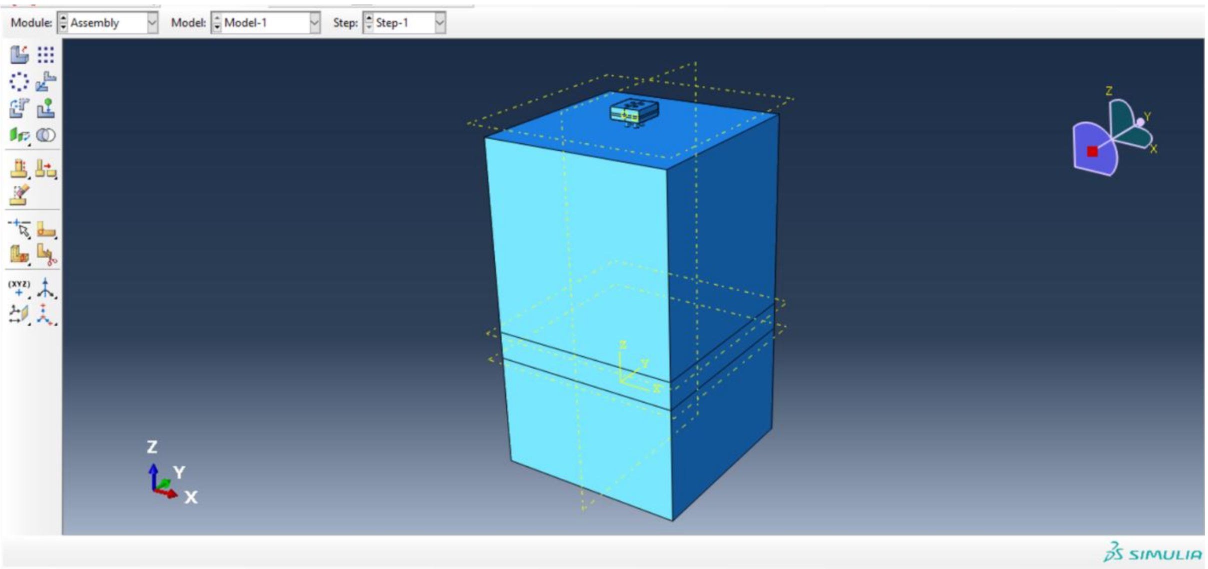

Fig. 3.6: Abaqus model of combined assembly of pile group in soil mass

Step

In this module by using the step manager, step is created with dynamic, explicit procedure and with time of 1 second. By using history output manager the variables which are to be determined are specified i.e., concentrated force in lateral direction and displacement in lateral direction, and the point where the variables are to be determined is mentioned.

Load

By using the boundary condition manager, the X-face and Y-face of the soil mass are given displacement zero and in the lateral direction a concentrated force of 10000 N is applied. At the point where variables need to be determined, this focused force is delivered to the model's pile cap. In this model the aim is to perform quasi-static analysis hence the amplitude of load applied is of smooth step type.

The amplitude and time provided in this model is as follows:

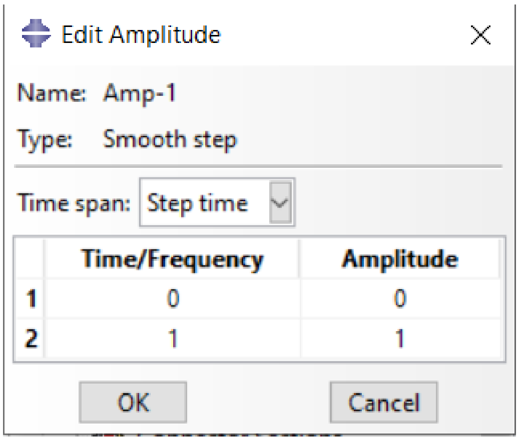

Fig. 3.7: Amplitude of loading on model

This implies at t=0 second, the concentrated force applied to the model is (0×10000)= 0 N and at t= 1second, the concentrated force applied to the model is (1×10000)=10000 N. In the step the time period for the concentrated force application is 1 second i.e., the concentrated force will be applied for 1 second and the magnitude variation of the concentrated force for the whole time period of 1 second is described in the figure below:

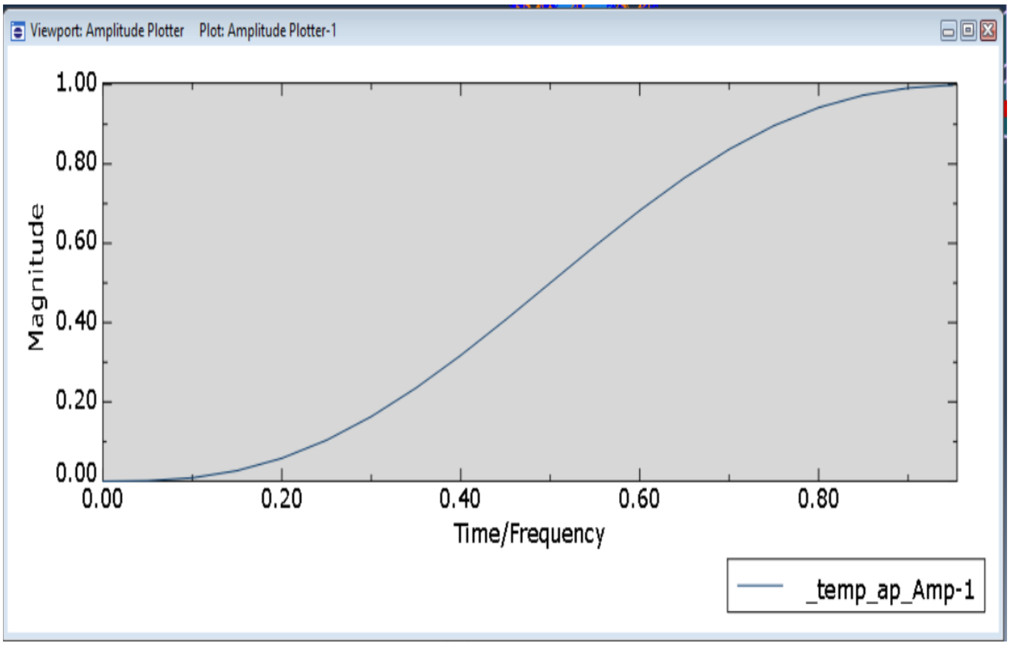

Fig. 3.8: Magnitude of smooth step loading on model with time

For the determination of the actual applied concentrated force at any time, magnitude of the above figure is to be multiplied by 10000 for any time value. From the above curve it can be observed that till 0.2 second the increment is very low and this helps in achieving quasi-static analysis. This type of load is used to determine the nature of load-displacement curve without actually failing the pile. This type of loading helps in determining the nature of displacement hence by keeping the same loading for all the different eccentricities and different depth of socketing we can determine the load-displacement curve.

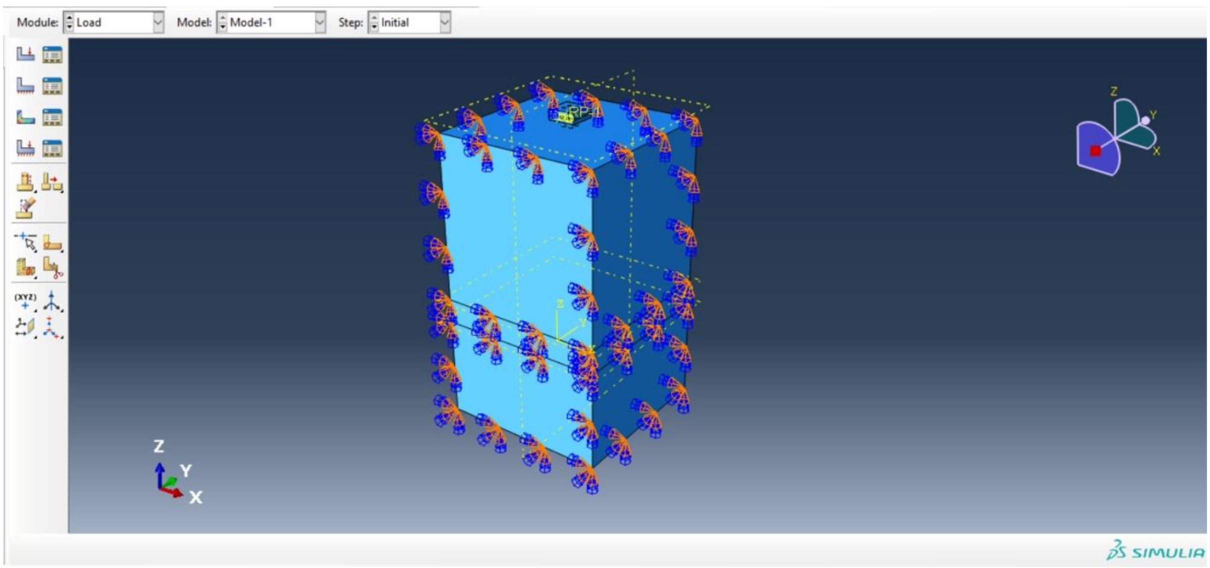

Fig. 3.9: Boundary condition on combined model of pile group in soil mass

Mesh

In this initially we seed edges of the model and then mesh the part.

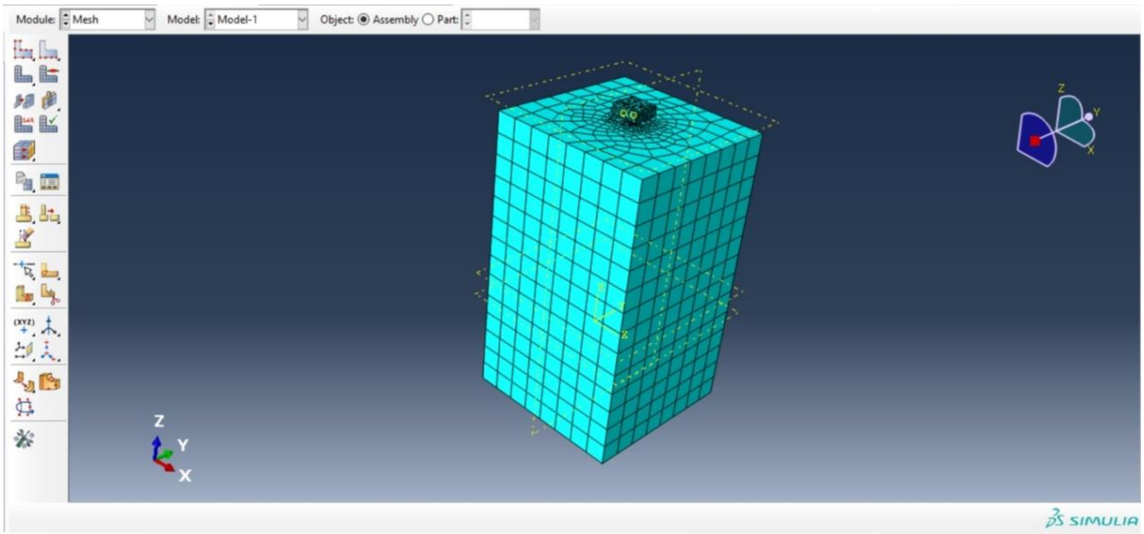

Fig. 3.10: Mesh model of combined assembly of pile group in soil mass

Job

In this by using job manager, a job is created and then the job is submitted for full analysis for the determination of the results.

Visualization

By using the results can be visualized and the displacement, stress can be observed and the graph can be plotted.

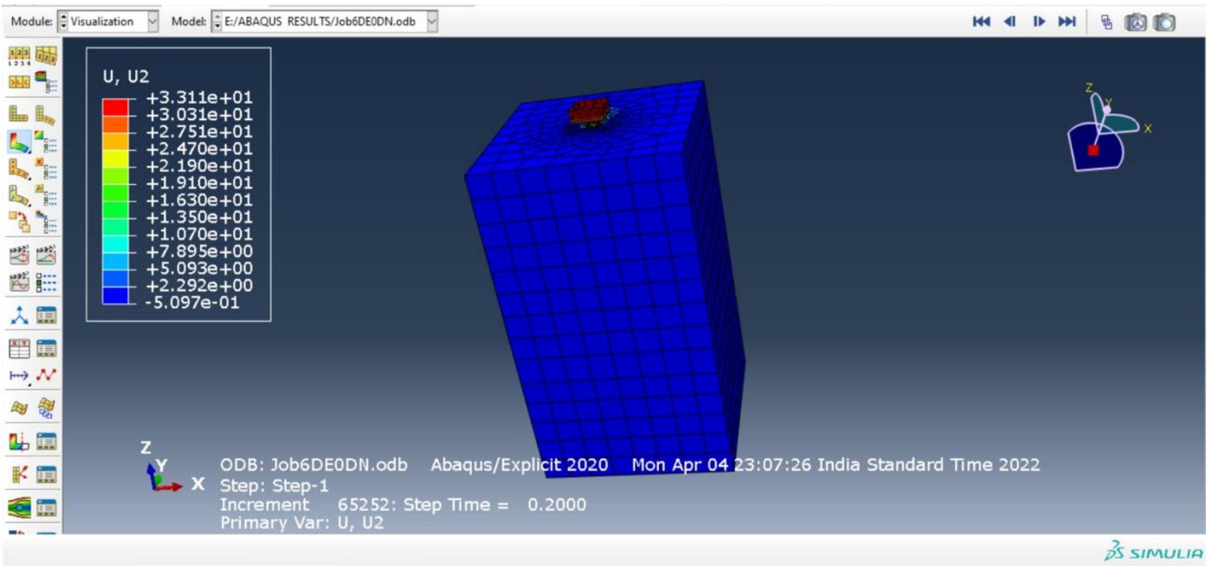

Fig. 3.11: Abaqus model showing lateral deflection after analysis

CHAPTER 4

OBSERVATION

The model is prepared in the abaqus software and an incrementing load of is applied over it. Two different effect has been studied here i.e., effect of depth of socketing and effect of eccentric lateral loading.

Three various depths of socketing were examined to investigate the influence of depth of socketing (120 to 360mm). Load eccentricity is changed from 0 to 240mm for a specific depth of socketing and lateral load - lateral deformation curve is created to evaluate the influence of eccentric lateral loading.

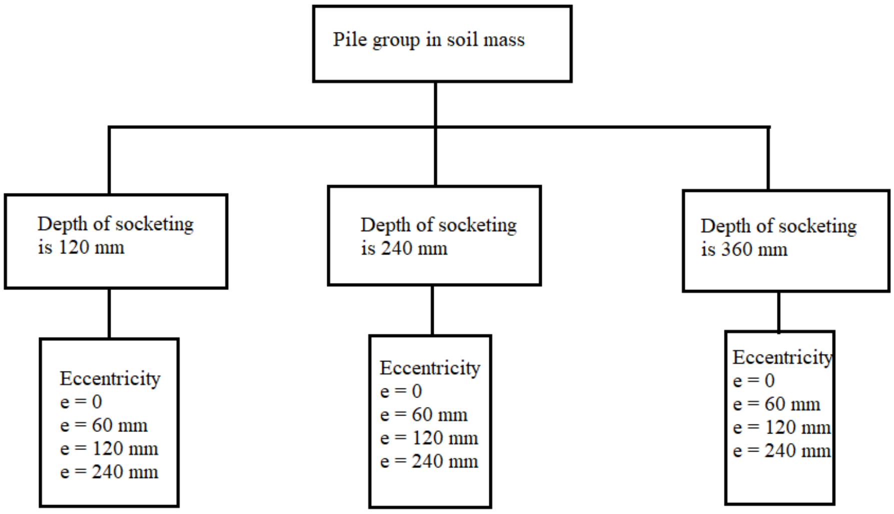

Fig. 4.1: Hierarchy flow of pile group in soil mass

4.1 Effect of depth of socketing

4.1.1 Socketing depth is one times of diameter (120mm)

In this scenario, the bottom section of the pile group is immersed in rock to a depth of 120mm, and then an incrementing load is applied in the lateral direction on the pile top. The lateral load vs lateral deformation curve is generated by first applying load in lateral direction at the centroid of the pile cap. To generate a lateral load-lateral deformation curve, the eccentricity is adjusted to 60mm and a comparable load in horizontal direction is applied. The load eccentricity is raised to 120 and 240mm, and the lateral load-lateral deformation curve is formed by applying the same load.

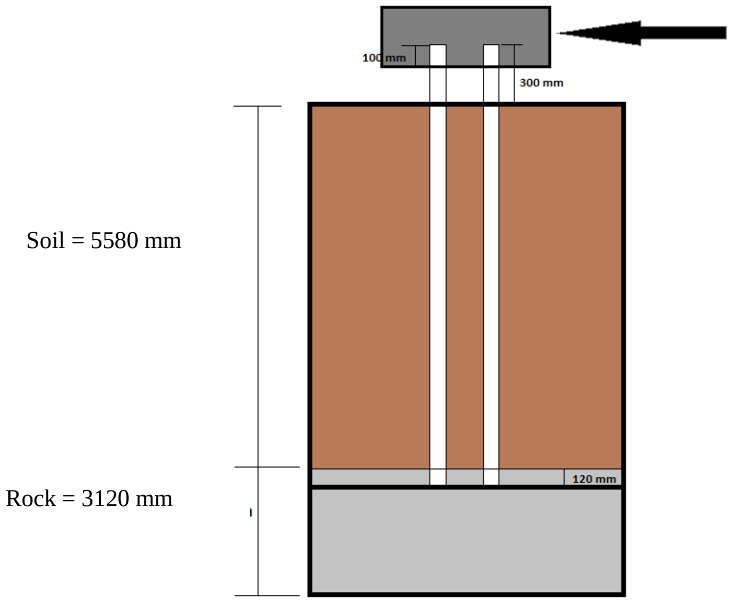

Fig. 4.2: Pictorial representation of pile group in soil mass

Effect of eccentricity

(a)- e = 0

The load is placed in the lateral direction at the centroid of the pile cap in this scenario, yielding a lateral load-lateral deformation curve.

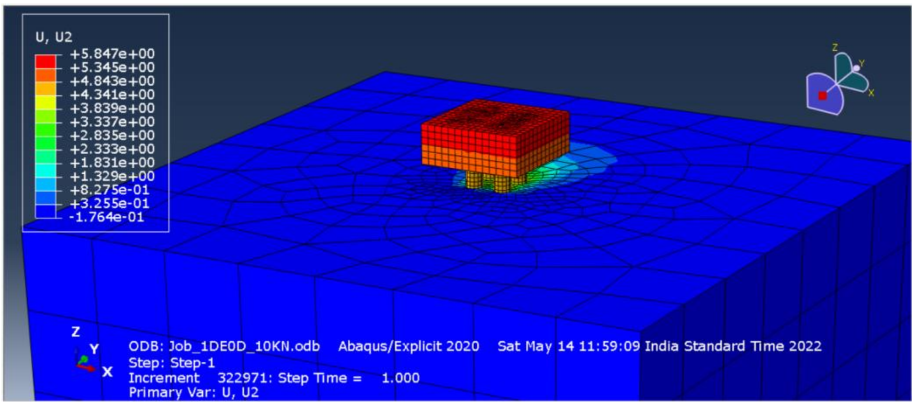

Fig. 4.3: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

The largest lateral deviation detected after analysis is 5.847 mm, which is at the pile cap.

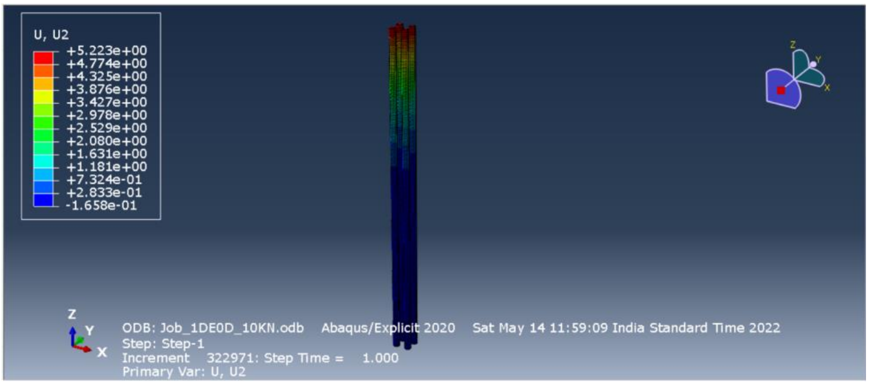

Fig. 4.4: Pile displacement in lateral direction along the length

In the lateral direction, the pile head deviation is 5.223 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral displacement curve all along length of the pile:

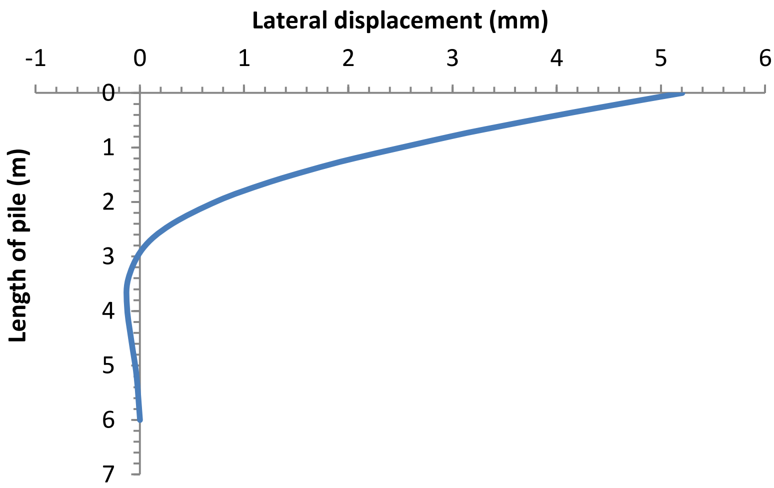

Fig. 4.5: Variation of lateral displacement of pile along its length

The lateral deflection is greatest at the pile head and dramatically reduces upto the depth of 3000 mm, after which the deflection is in the opposite direction and falls to zero at the pile end.

(b)- <u>e = 60mm</u>

The load is applied at an eccentricity of 60 mm from the pile cap's centroid, and the lateral load-lateral displacement curve is obtained.

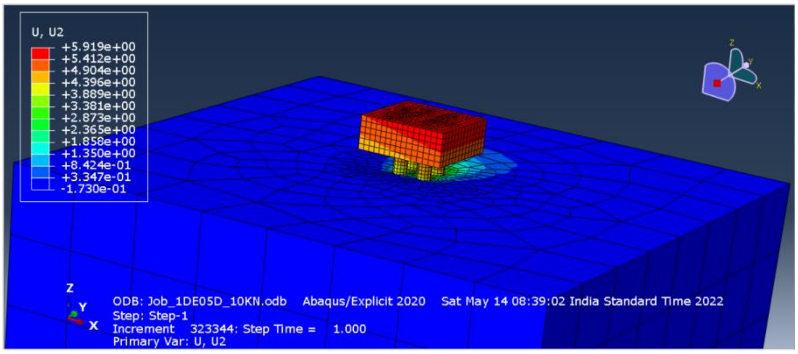

Fig. 4.6: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

The largest lateral displacement detected after analysis is 5.919 mm, which is at the pile cap.

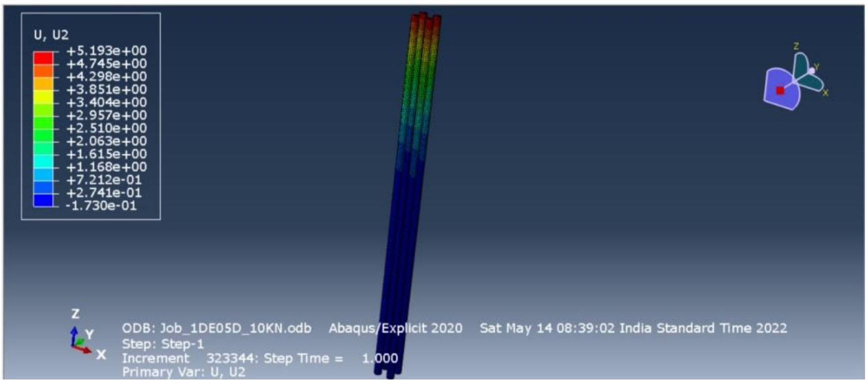

Fig. 4.7: Variation of lateral deflection of pile along its length

In the lateral direction, the pile head deflection is 5.193 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral deformation curve is along length of the pile:

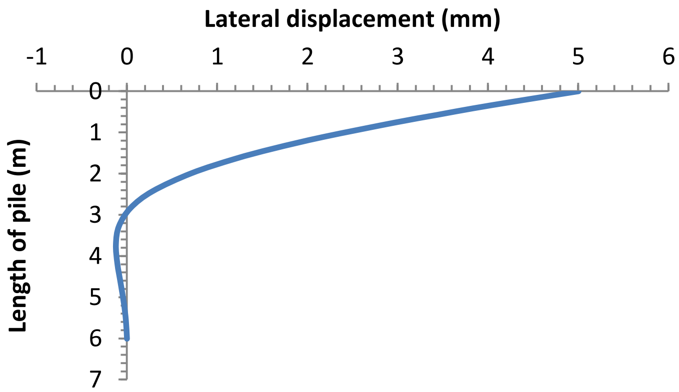

Fig. 4.8: Variation of lateral displacement of pile along the pile length

The above figure depicts that the lateral deflection is maximum at the pile head and decreases sharply till the depth of 3000 mm and beyond the 3000 mm depth the deflection observed in the opposite direction which decreases to zero at the end of pile.

(c)- e =120mm

In this scenario, the load is applied at a eccentricity of 120 mm from the pile cap's centroid, and a lateral load vs lateral deformation curve is obtained.

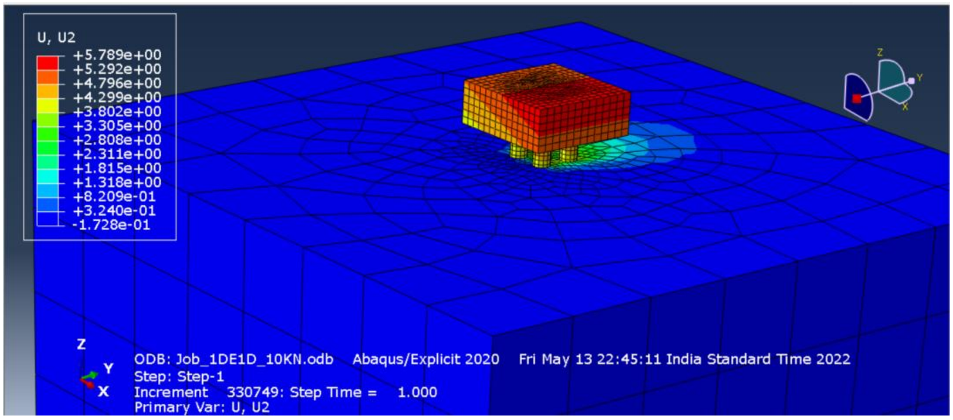

Fig. 4.9: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

The maximum lateral deflection observed is 5.789 mm, at the pile cap.

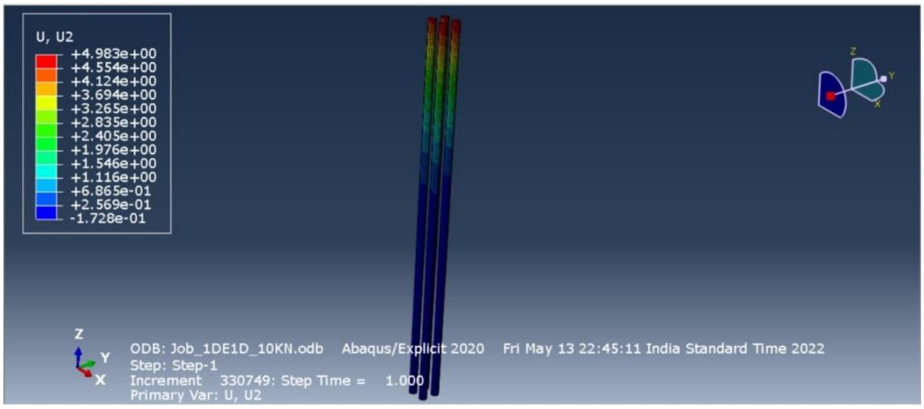

Fig. 4.10: Variation of lateral deflection of pile along its length

The pile head deflection is 4.983 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral displacement curve all along length of the pile:

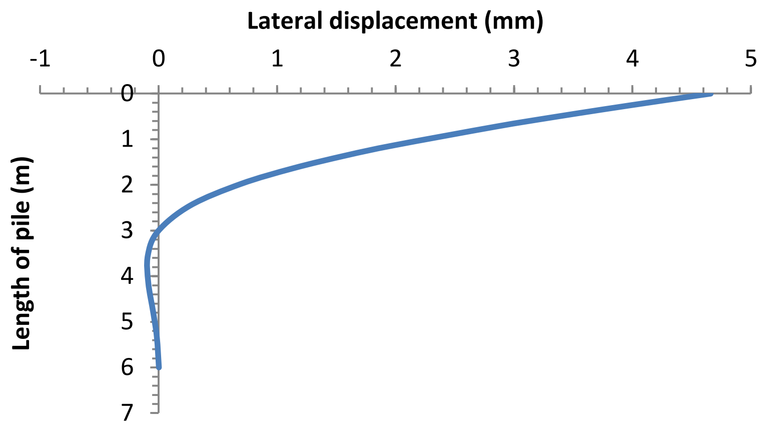

Fig. 4.11: Variation of lateral displacement of pile along the pile length

The above figure signifies that the lateral deflection is highest at the pile head and decreases sharply till the depth of 3000 mm and beyond the 3000 mm depth the deflection observed in the opposite direction which decreases to zero at the end of pile.

(d)- e = 240mm

In this scenario, the load is applied at a eccentricity of 240 mm from the pile cap's centroid, and a lateral load-lateral displacement curve is obtained.

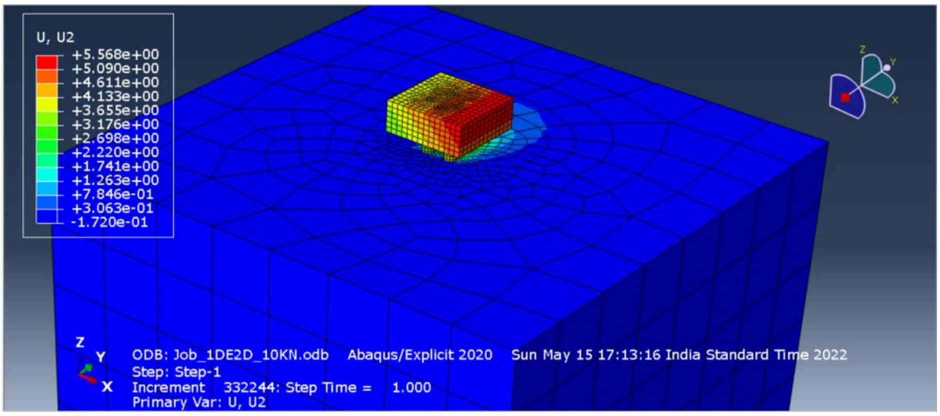

Fig. 4.12: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

The maximum lateral deflection observed is 5.568 mm, which is at the pile cap.

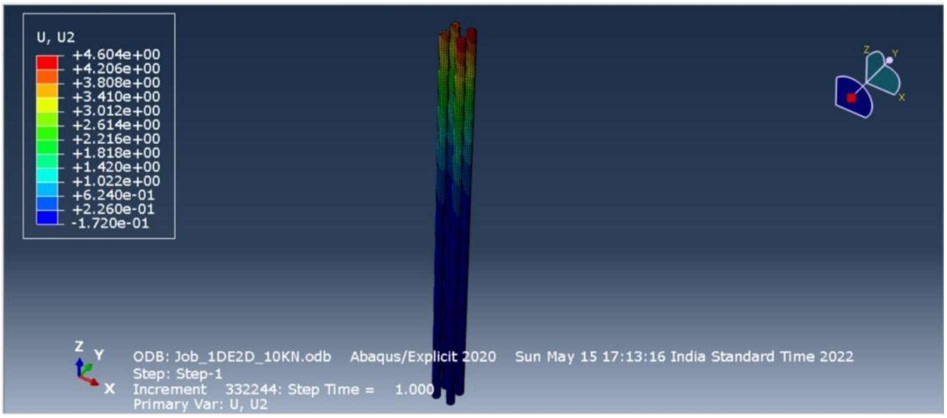

Fig. 4.13: Lateral deflection of pile

In the lateral direction, the pile head deflection is 4.604 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral deformation curve along the length of the pile:

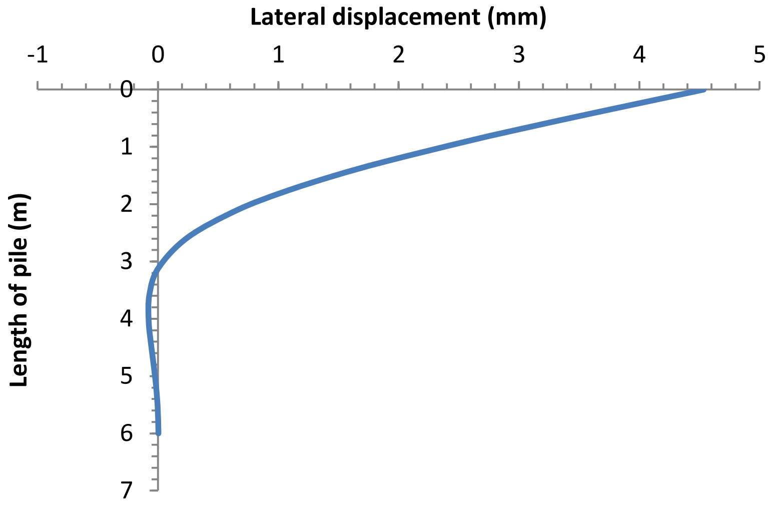

Fig. 4.14: Variation of lateral displacement of pile along the length of the pile

From the above figure it can be concluded that the lateral deflection is maximum at the pile head and decreases sharply till the depth of 3000 mm and beyond the 3000 mm depth the deflection observed in the opposite direction which decreases to zero at the end of pile.

4.1.2 <u>Socketing depth is two times of diameter (240mm)</u>

In this scenario, the bottom section of the pile group is immersed in rock to a depth of 240 mm, and then an incrementing lateral load is given to the pile top. The lateral load-lateral deformation curve is generated by first applying load in the lateral direction at the centroid of the pile cap. To generate a lateral load-lateral deformation curve, the eccentricity is adjusted to 60mm and a comparable load is applied. The load eccentricity is further increased to 120 to 240mm and by applying same load, lateral load- lateral deformation curve is obtained.

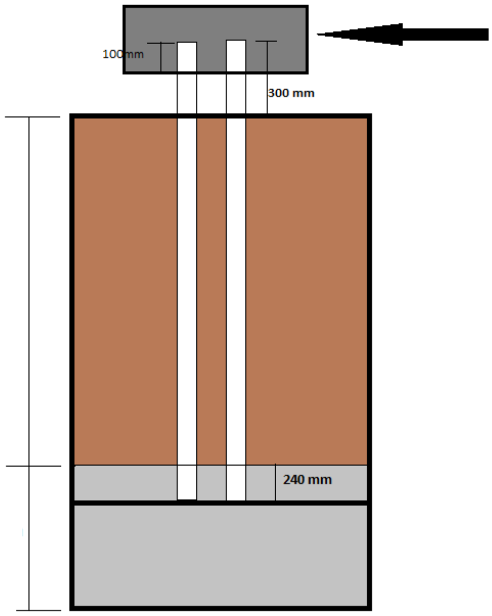

Fig. 4.15: Pictorial representation of pile group in soil mass

<u>Effect of eccentricity</u>

(a)- <u>e = 0</u>

The load is applied to the pile cap's centroid, and the lateral load-lateral deformation curve is obtained.

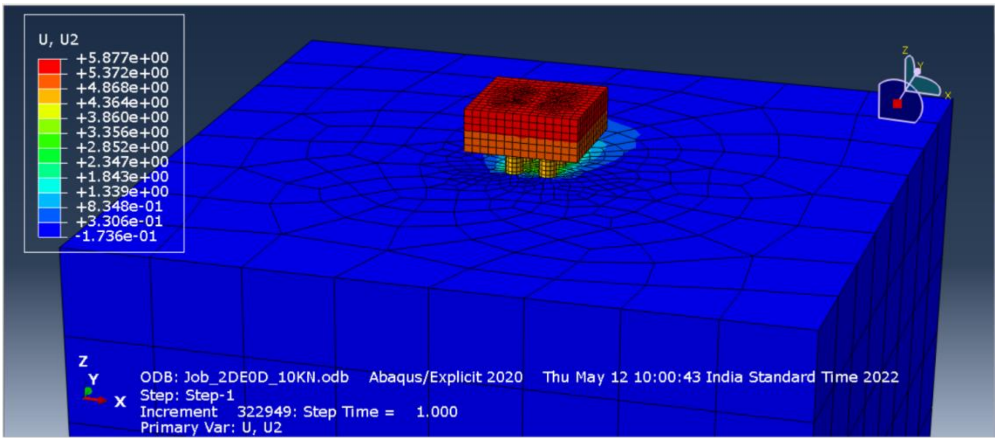

Fig. 4.16: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

After analysis the maximum lateral deflection observed is 5.877 mm, which is at the pile cap.

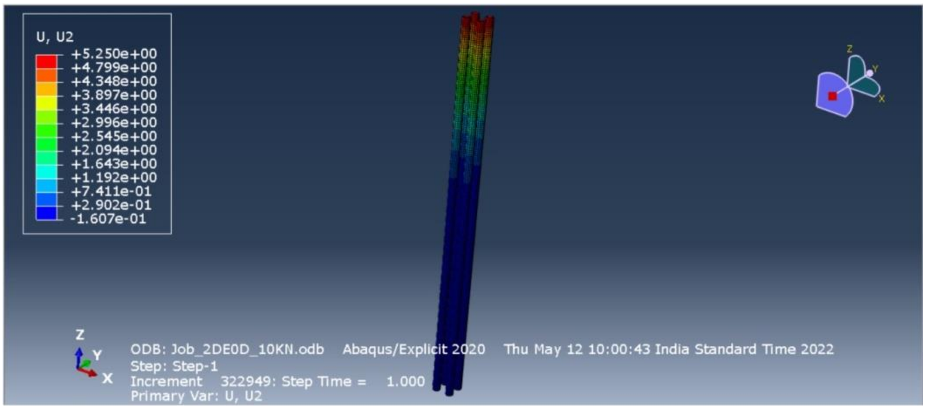

Fig. 4.17: Lateral deflection of pile

In the lateral direction, the pile head deviation is 5.250 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral deformation curve along the length of the pile:

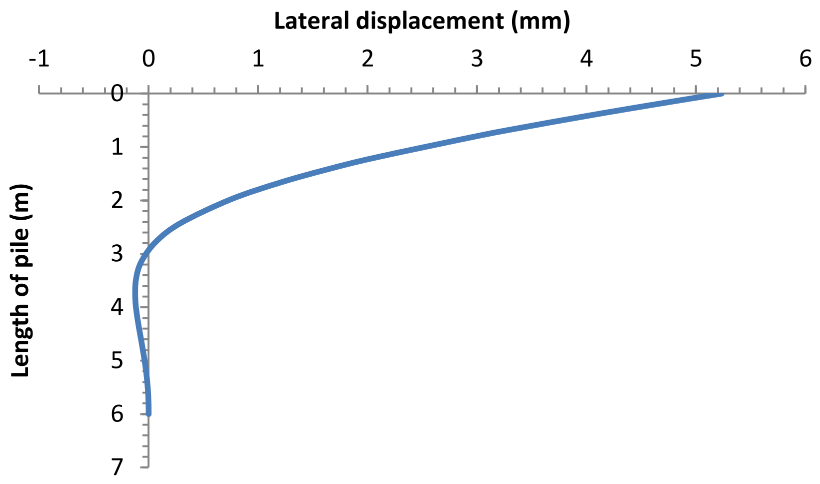

Fig. 4.18: Variation of lateral displacement of pile along the length

The lateral deflection is greatest at the pile head and dramatically reduces until the depth of 3000 mm, after which the deflection is in the opposite direction and falls to zero at the pile end.

(b)- <u>e = 60 mm</u>

The lateral load-lateral displacement curve is obtained when the load is applied at an eccentricity of 60 mm from the centroid of the pile cap.

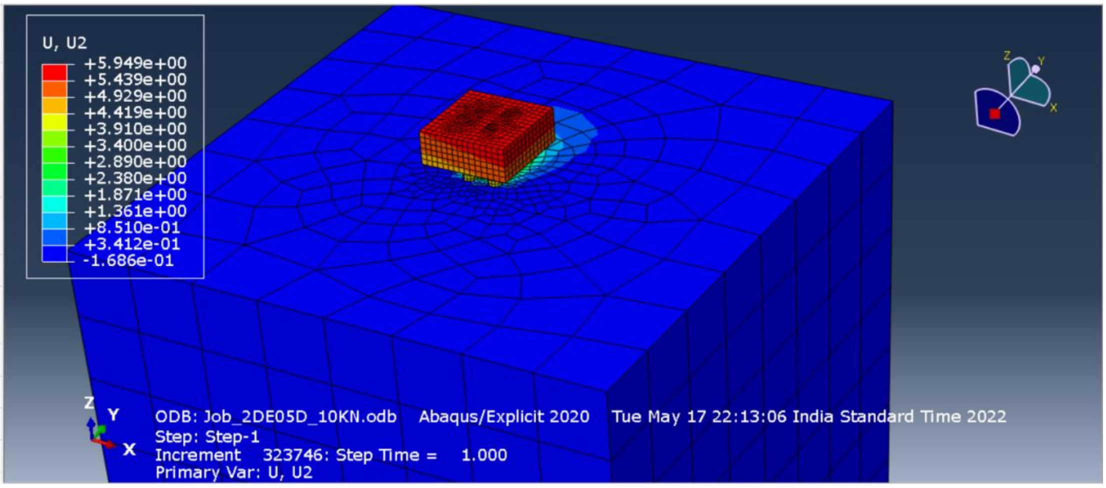

Fig. 4.19: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

After analysis the maximum lateral deflection observed is 5.949 mm, which is at the pile cap.

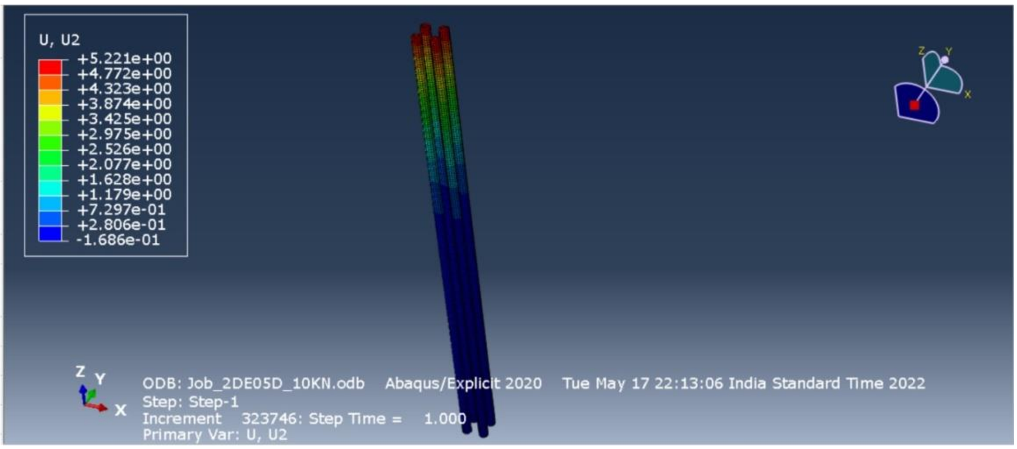

Fig. 4.20: Lateral deflection of pile

In the lateral direction, the pile head displacement is 5.221 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral displacement curve along the length of the pile:

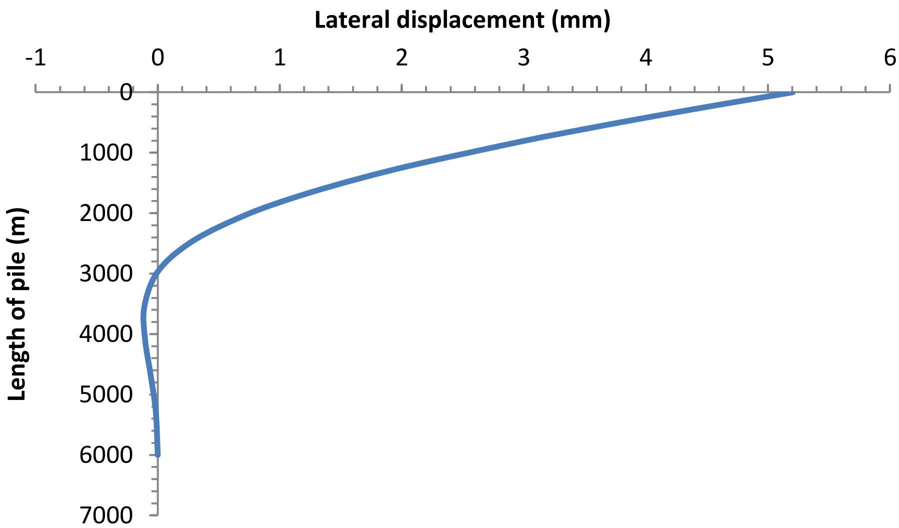

Fig. 4.21: Variation of lateral displacement of pile along the length

The lateral deflection is greatest at the pile head and dramatically reduces until the depth of 3000 mm, after which the deflection is in the opposite direction and falls to zero at the pile end.

(c)- e = 120mm

The load is applied at an eccentricity of 120 mm from the centroid of the pile cap, and the lateral load-lateral displacement curve is obtained.

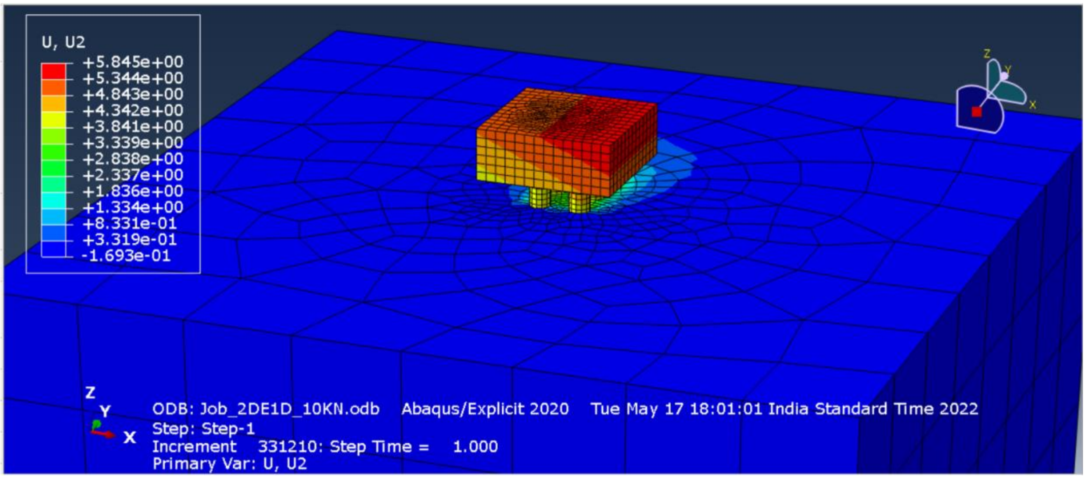

Fig. 4.22: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

After analysis the maximum lateral deflection observed is 5.845 mm, which is at the pile cap.

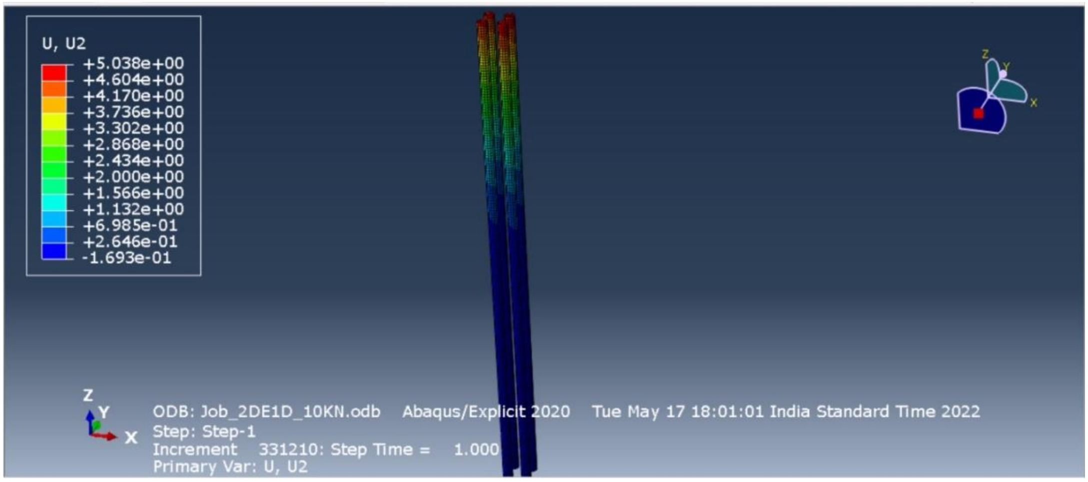

Fig. 4.23: Lateral deflection of pile

In the lateral direction, the pile head displacement is 5.038 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral displacement curve along the length of the pile:

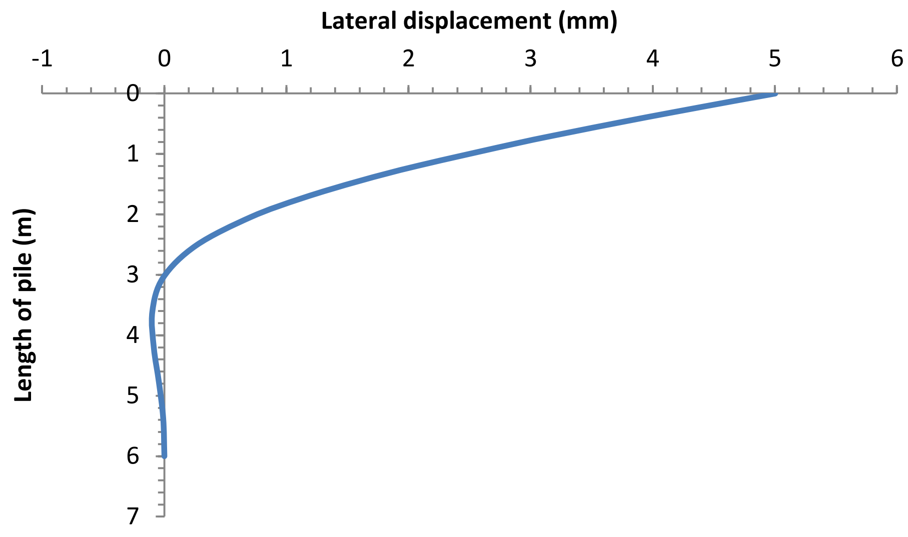

Fig. 4.24: Variation of lateral displacement of pile along the length

The lateral deflection is greatest at the pile head and dramatically reduces until the depth of 3000 mm, after which the deflection is in the opposite direction and falls to zero at the pile end.

(d)- <u>e = 240mm</u>

In this scenario, the load is applied at an eccentricity of 240 mm from the pile cap's centroid, and a lateral load vs lateral displacement curve is obtained.

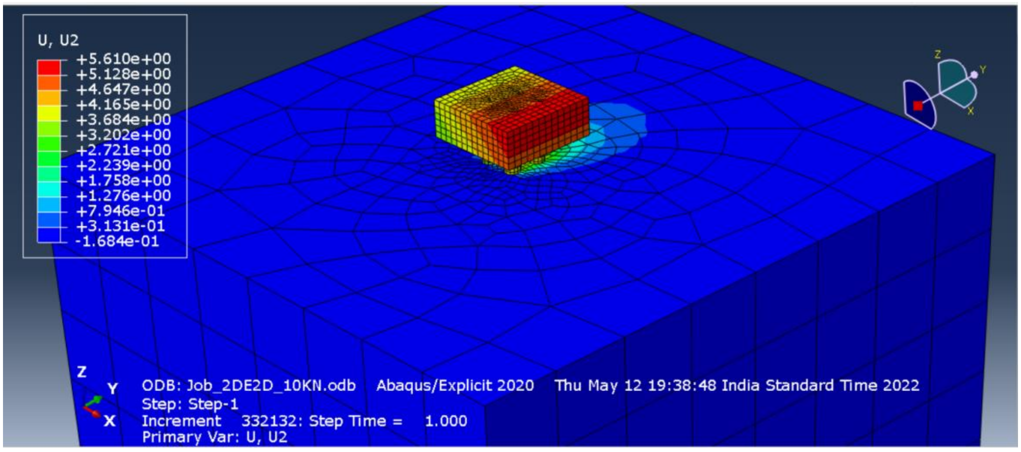

Fig. 4.25: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

After analysis the maximum lateral deflection observed is 5.610 mm, which is at the pile cap.

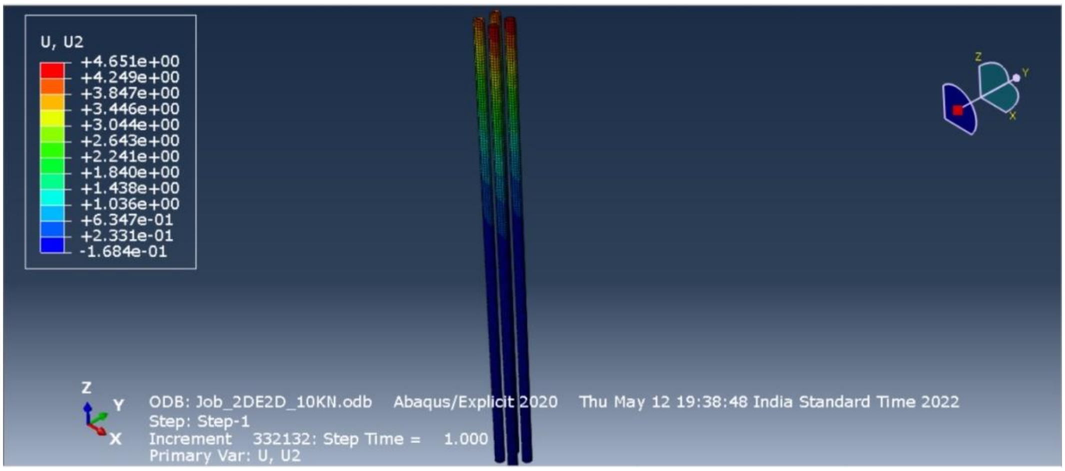

Fig. 4.26: Lateral deflection of pile

In the lateral direction, the pile head displacement is 4.651 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral displacement curve along the length of the pile:

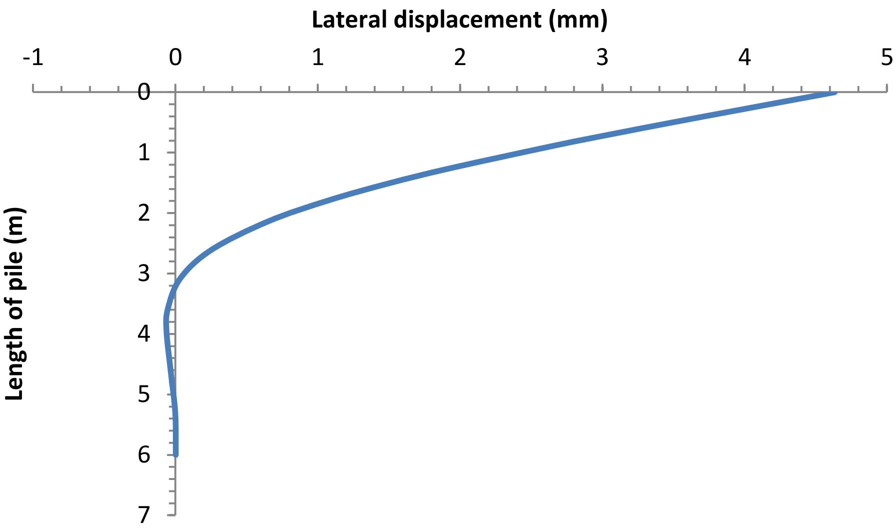

Fig. 4.27: Variation of lateral displacement of pile along the length

In the lateral direction, the pile head displacement is 4.651 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral displacement curve along the length of the pile:

4.1.3 <u>Socketing depth is three times of diameter (360mm)</u>

In this scenario, the bottom section of the pile group is immersed in rock to a depth of 360 mm, and then an increasing lateral load is given to the pile top. The lateral load-lateral deformation curve is generated by first applying load in the horizontal direction at the centroid of the pile cap. To generate a lateral load-lateral deformation curve, the eccentricity is adjusted to 60mm and a comparable horizontal load is applied. The load eccentricity is further increased to 120 & 240mm and by applying same load, lateral load- lateral deformation curve is obtained.

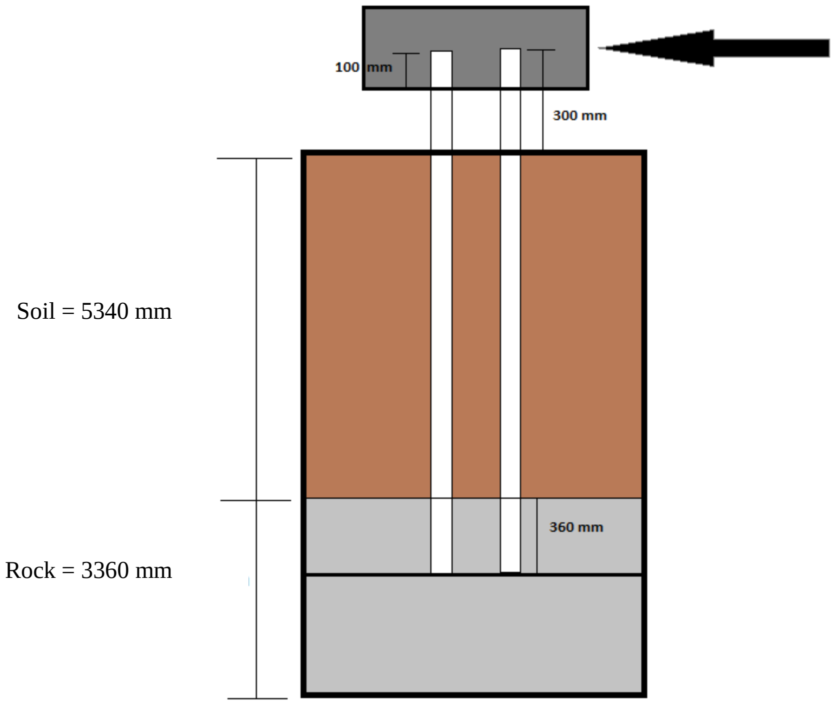

Fig. 4.28: Pictorial representation of pile group in soil mass

Effect of eccentricity

(a)- e = 0

The load is supplied to the pile's centroid, and the lateral load-lateral displacement curve is obtained.

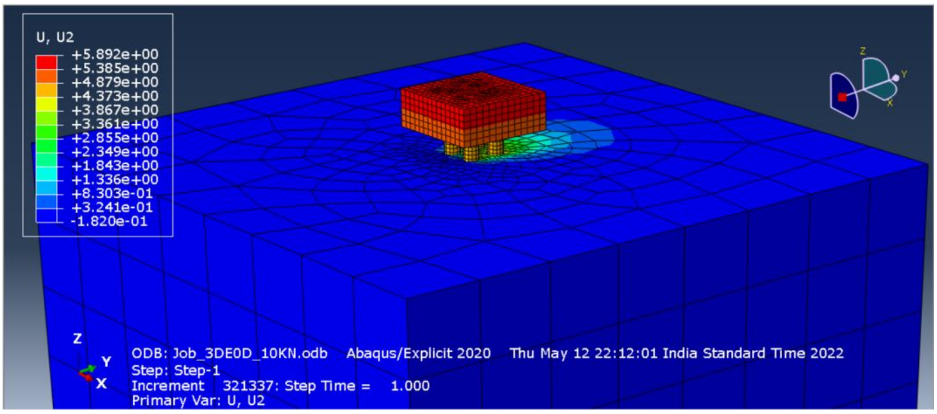

Fig. 4.29: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

After analysis the maximum lateral deflection observed is 5.892 mm, which is at the pile cap.

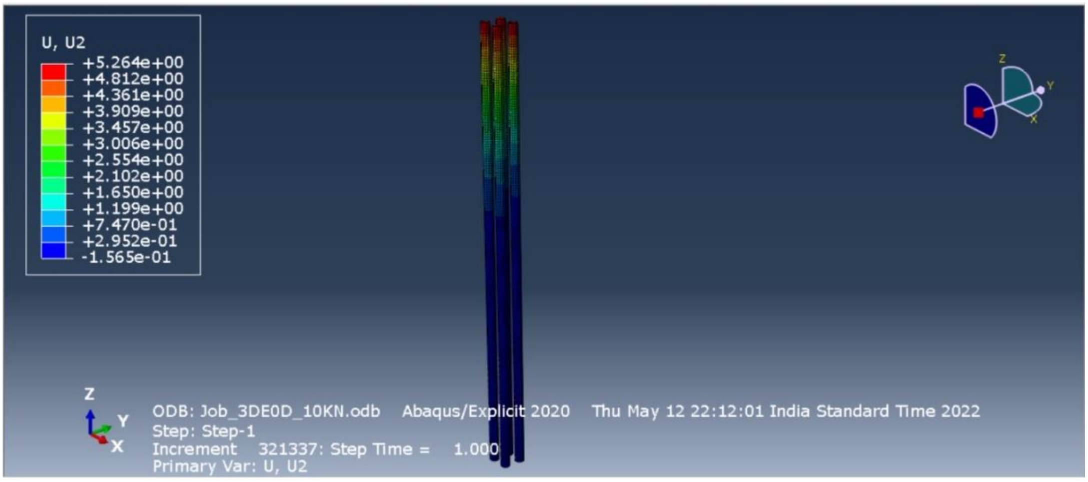

Fig. 4.30: Lateral deflection of pile

In the lateral direction, the pile head displacement is 5.264 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral displacement curve along the length of the pile:

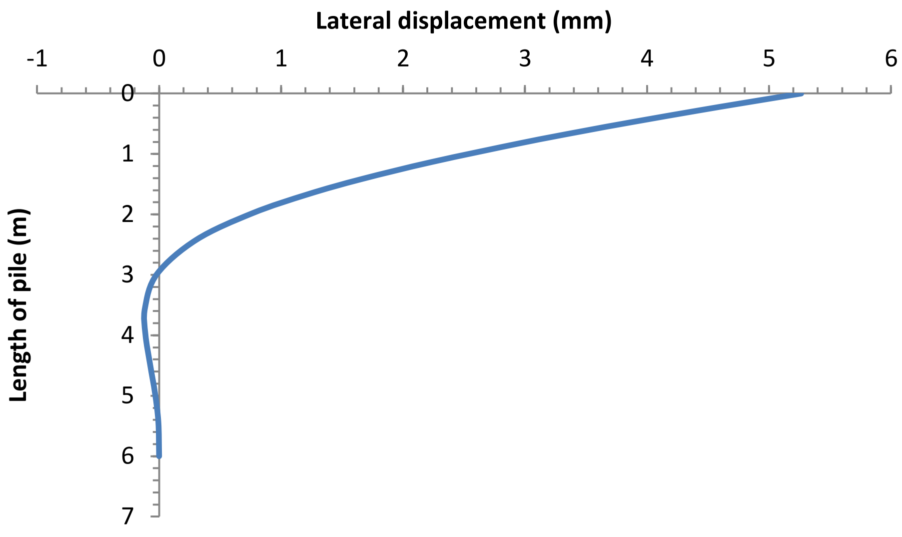

Fig. 4.31: Variation of lateral displacement of pile along the length

The lateral deflection is greatest at the pile head and dramatically reduces until the depth of 3000 mm, after which the deflection is in the opposite direction and falls to zero at the pile end.

(b)- <u>e = 60mm</u>

The load is applied at an eccentricity of 0.5D, or 60 mm from the pile cap's centroid, and the lateral load-lateral displacement curve is obtained.

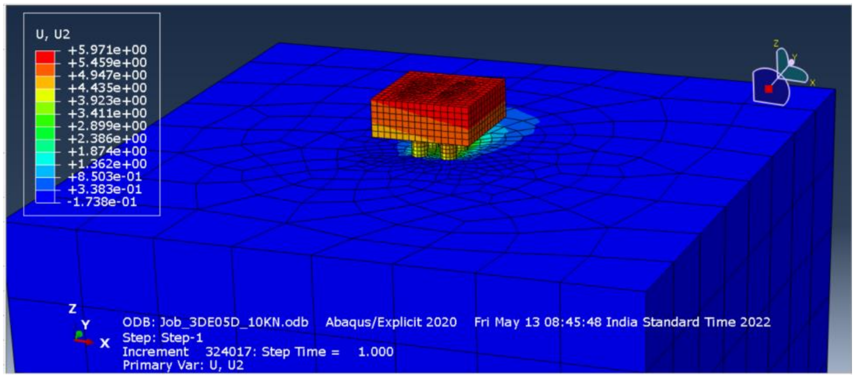

Fig. 4.32: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

After analysis the maximum lateral deflection observed is 5.971 mm, which is at the pile cap

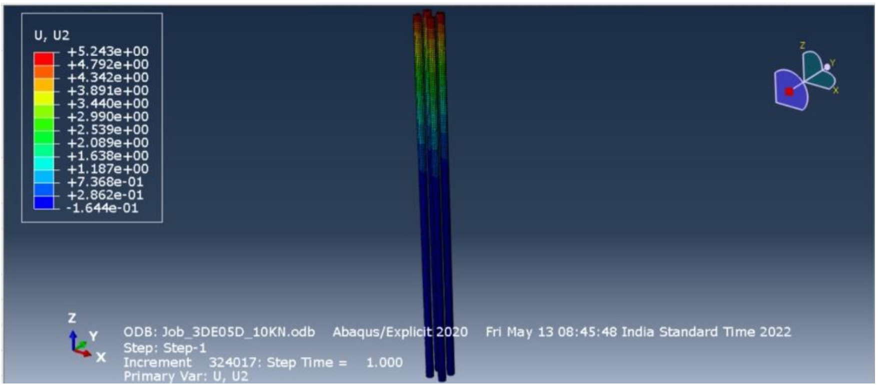

Fig. 4.33: Lateral deflection of pile

In the lateral direction, the pile head displacement is 5.243 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral displacement curve along the length of the pile:

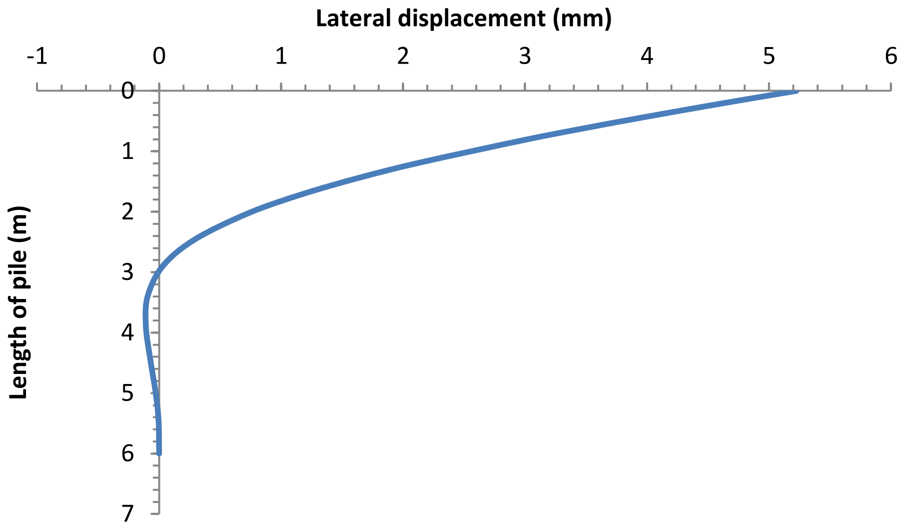

Fig. 4.34: Variation of lateral displacement of pile along the length

The lateral deflection is greatest at the pile head and dramatically reduces until the depth of 3000 mm, after which the deflection is in the opposite direction and falls to zero at the pile end.

(c)- <u>e = 120mm</u>

The load is applied at an eccentricity of 120 mm from the pile cap's centroid, and the lateral load-lateral displacement curve is obtained.

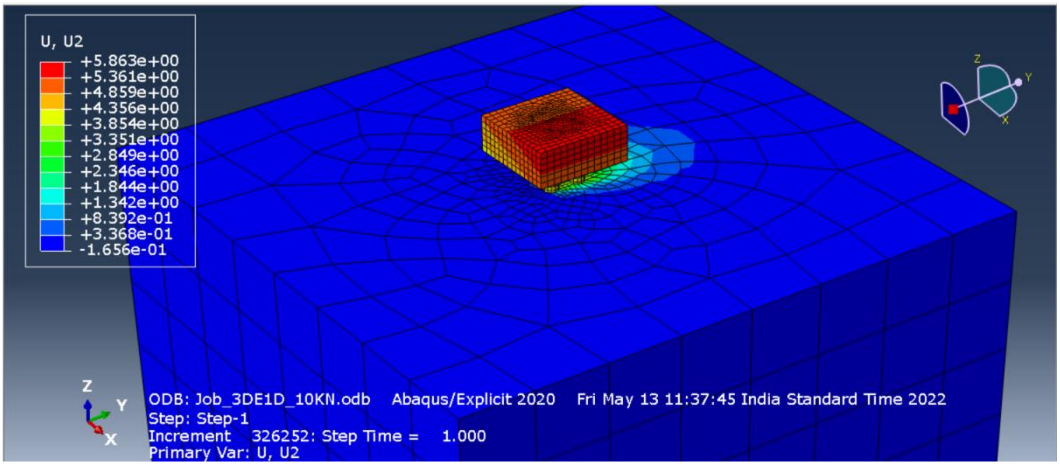

Fig. 4.35: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

After analysis the maximum lateral deflection observed is 5.863 mm, which is at the pile cap.

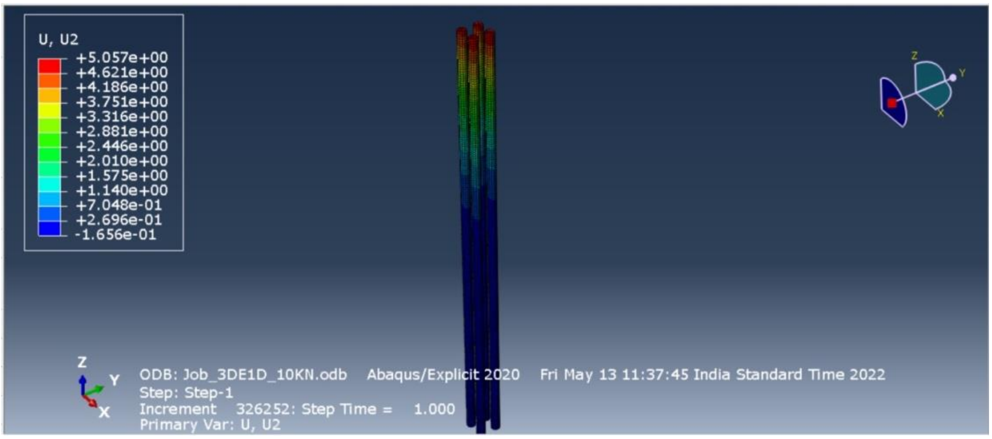

Fig. 4.36: Lateral deflection of pile

In the lateral direction, the pile head displacement is 5.057 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral displacement curve along the length of the pile:

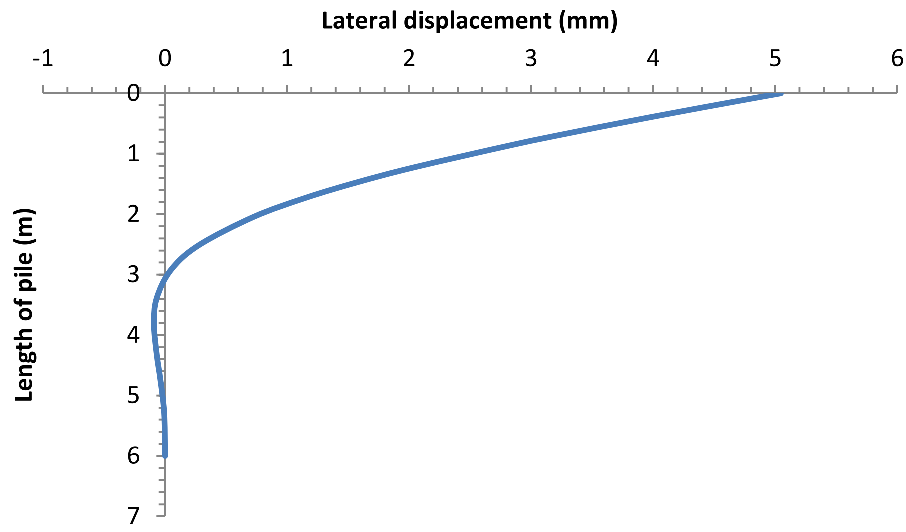

Fig. 4.37: Variation of lateral displacement of pile along the length

The lateral deflection is greatest at the pile head and dramatically reduces until the depth of 3000 mm, after which the deflection is in the opposite direction and falls to zero at the pile end.

(d)- <u>e = 240mm</u>

In this scenario, the load is applied at an eccentricity of 240 mm from the pile cap's centroid, and a lateral load-lateral displacement curve is obtained.

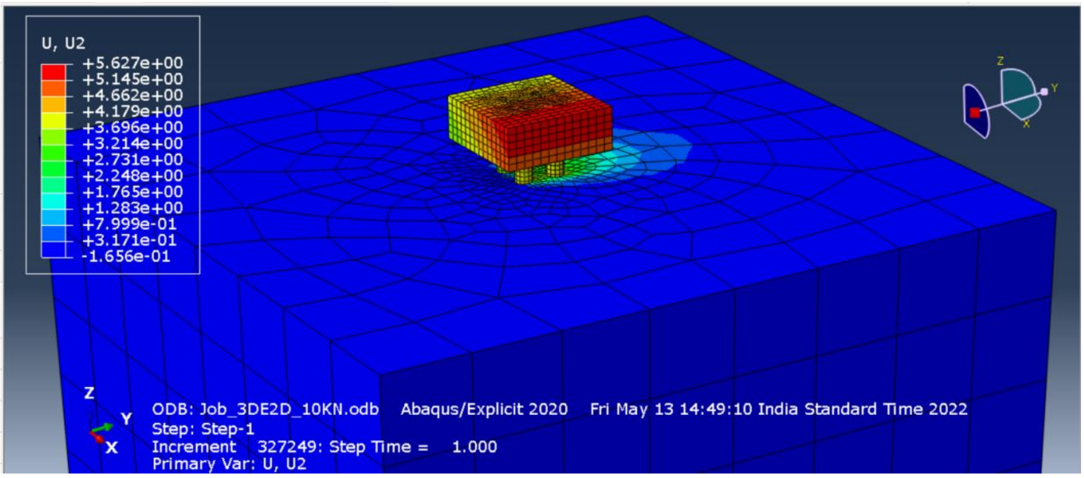

Fig. 4.38: Magnified view of lateral displacement of the combined assembly of pile group in soil mass

After analysis the maximum lateral deflection observed is 5.627 mm, which is at the pile cap.

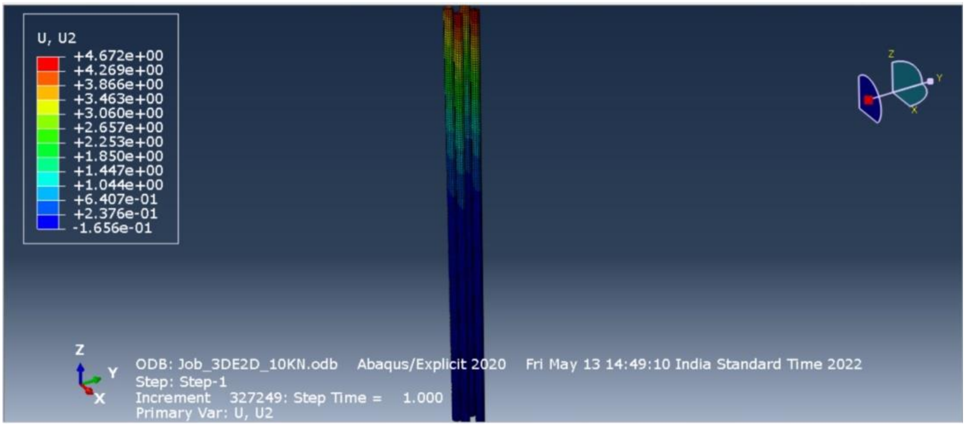

Fig. 4.39: Lateral Deflection of pile

In the lateral direction, the pile head displacement is 4.672 mm. The deflection of the pile head diminishes with depth, as seen in the figure. The figure below depicts the lateral displacement curve along the length of the pile:

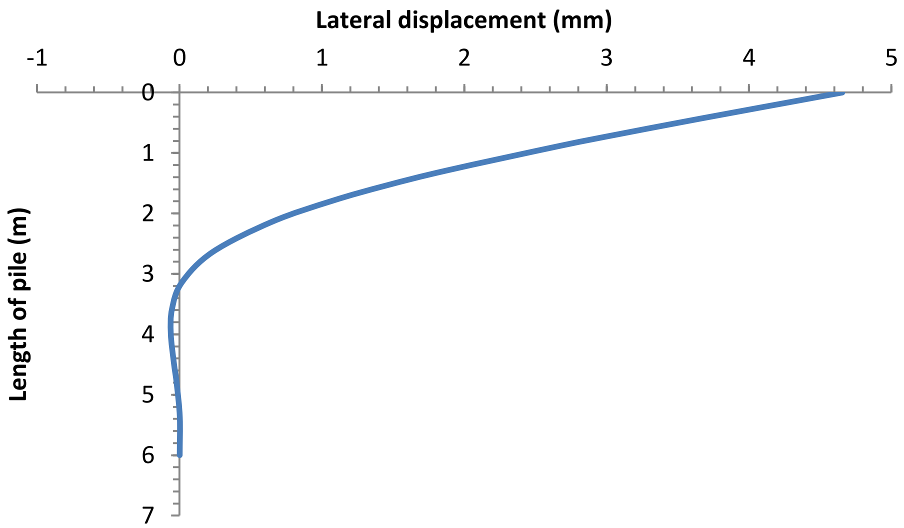

Fig. 4.40: Variation of lateral displacement of pile along the length

The lateral deflection is greatest at the pile head and dramatically reduces until the depth of 3000 mm, after which the deflection is in the opposite direction and falls to zero at the pile end.

CHAPTER 5

RESULT AND DISCUSSION

5.1 Effect of depth of socketing

(a)- Depth of socketing is 120mm (1D)

In this situation, the pile is lodged in the rock mass to a depth of 120mm and the load is applied laterally. In each situation, a rising load with a maximum limit of 10 kN is applied, and four different situations are examined in terms of load eccentricity.

The load is applied at an eccentricity of 0, which is the centroid of the pile cap, and subsequently raised to 60 to 240mm.

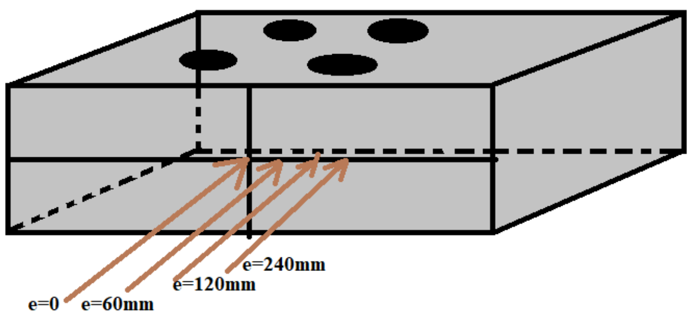

Fig. 5.1: Lateral loading direction at different eccentricities on pile cap

All the different cases are analysed and compiled in the figure below:

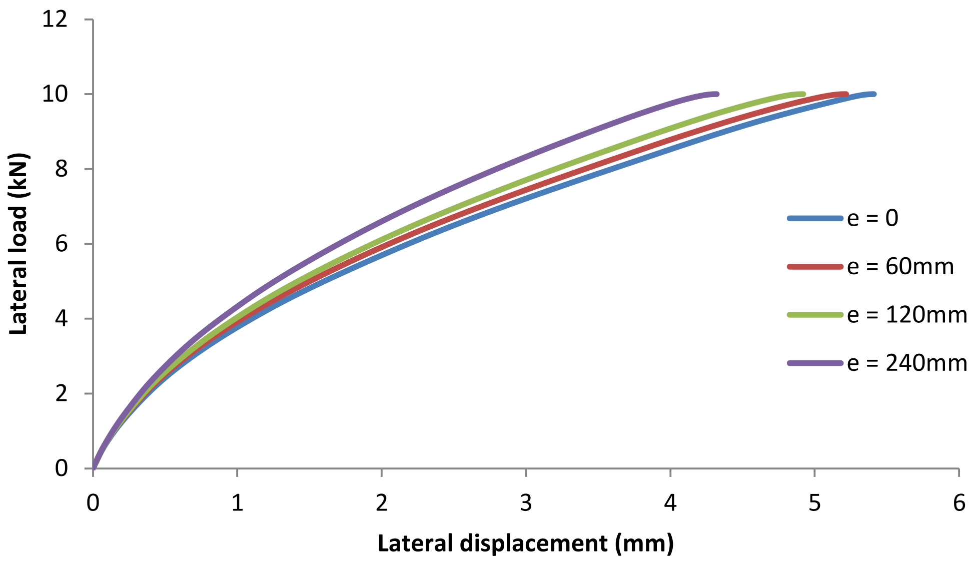

Fig. 5.2: Variation of lateral load with lateral displacement for different eccentricities at the centroid of the pile cap

From above figure it can be observed that with change in eccentricity from 0 to 240mm, the lateral displacement decreases in continued manner.

The combined figure of deflection of pile along its length with different load eccentricity is shown below:

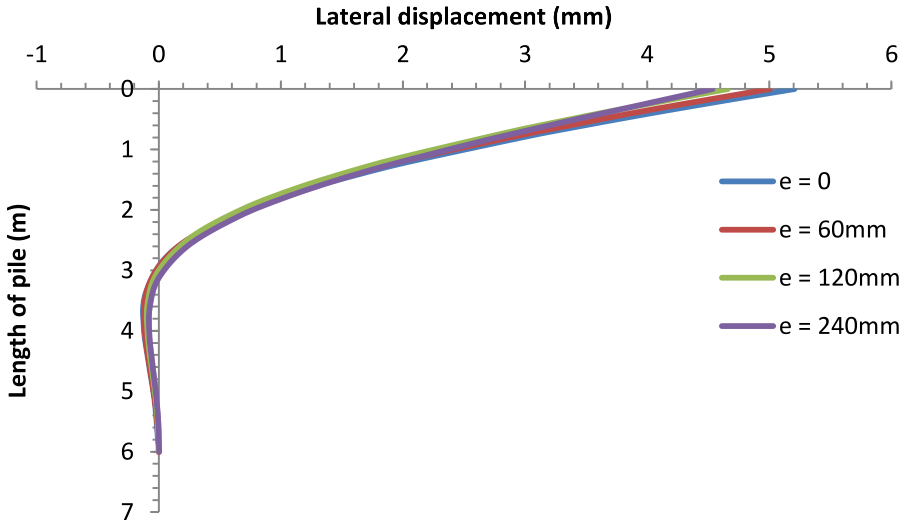

Fig. 5.3: Variation of lateral deflection of pile for different eccentricities along its length

From the above curve it can be evaluated that as eccentricity increases from 0 to 240mm, the pile head deflection decreases following the decreasing manner.

(b)- Depth of socketing is 240mm (2D)

In this situation the pile is embedded in rock mass upto the depth of 240mm and load is applied in the lateral direction. An increasing load is applied with maximum limit of 10 kN in every case and four different cases are analysed for different eccentricity of load.

The load is implemented at an eccentricity of zero, which is the centroid of the pile cap, and subsequently raised to 60 to 240mm. All the different cases are analysed and compiled in the figure below:

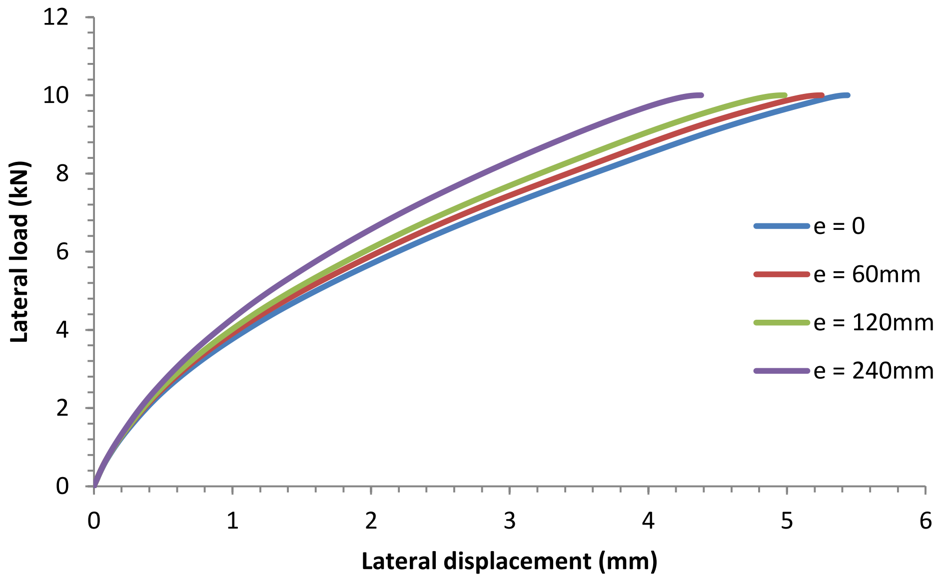

Fig. 5.4: Variation of lateral load with lateral displacement for different eccentricities at the centroid of the pile cap

From the above figure it can be observed that as eccentricity changes from 0 to 240mm, the lateral displacement decreases in continued manner. The combined figure of deflection of pile along its length with different load eccentricity is shown below:

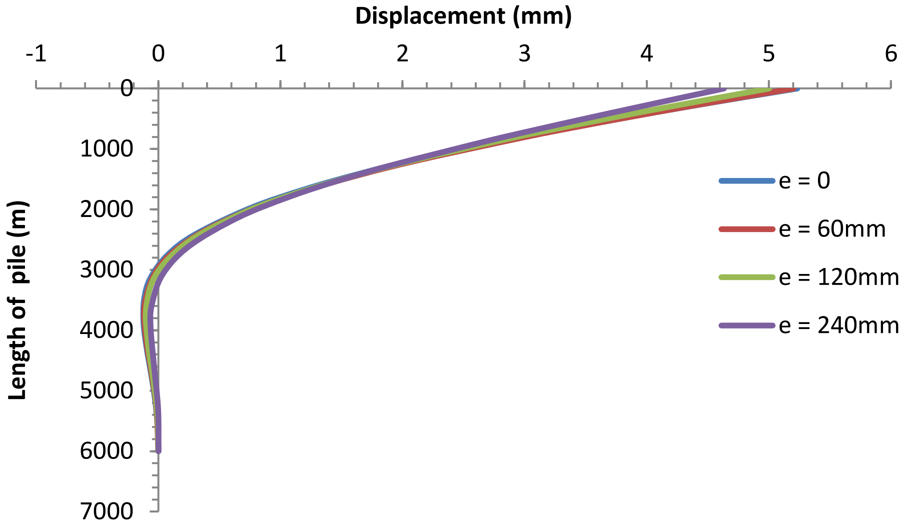

Fig. 5.5: Variation of lateral deflection of pile for different eccentricities along its length

From the above figure it can be observed that as eccentricity changes from 0 to 240mm, the pile head deflection decreases.

(c)- <u>Depth of socketing is 360mm (3D)</u>

In this situation the pile is embedded in rock mass upto the depth of 360mm and load is applied in the lateral direction. An increasing load is applied with maximum limit of 10 kN in every case and four different cases are analysed for different eccentricity of load.

The load is implemented at an eccentricity of 0, which is the centroid of the pile cap, and subsequently raised to 60 to 240mm. All the different cases are analysed and compiled in the figure below:

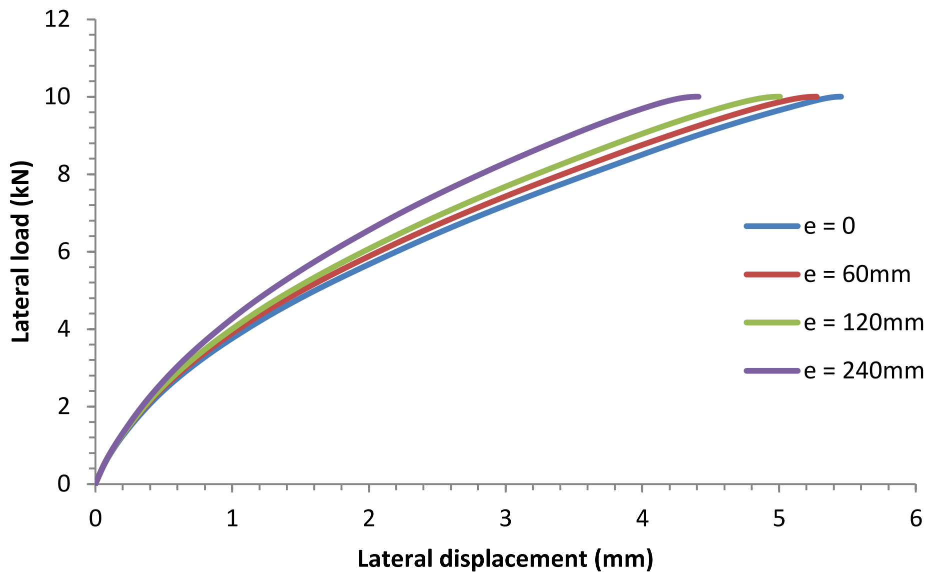

Fig. 5.6: Variation of lateral load with lateral deflection for different eccentricities at the centroid of the pile cap

According to the above figure, as eccentricity it can be seen that with increase in eccentricity from 0 to 240mm, the lateral displacement decreases in continued manner.

The combined figure of deflection of pile along its length with different load eccentricity is shown below:

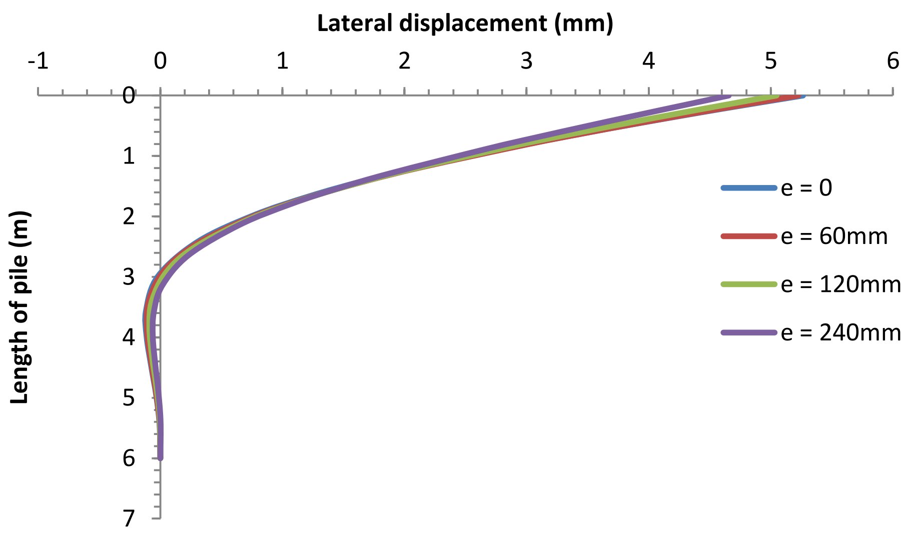

Fig. 5.7: Variation of lateral deflection of pile for different eccentricities along its length

From the above figure it can be observed that as eccentricity changes from 0 to 240mm, the pile head deflection decreases following the decreasing manner.

5.2 Constant eccentricity for different depth of socketing

In this case the load eccentricity is kept uniform and lateral load Vs lateral deformation curve is obtained for different depth of socketing. An increasing lateral load is applied with maximum limit of 10 kN and lateral deflection is observed. The curve is shown below. The lateral load vs lateral deflection are expressed as stacked area curve hence the lateral displacement are stacked up over the other for different depth of socketing i.e., 120 to 360mm. The different coloured areas represent lateral deflection for each depth of socketing.

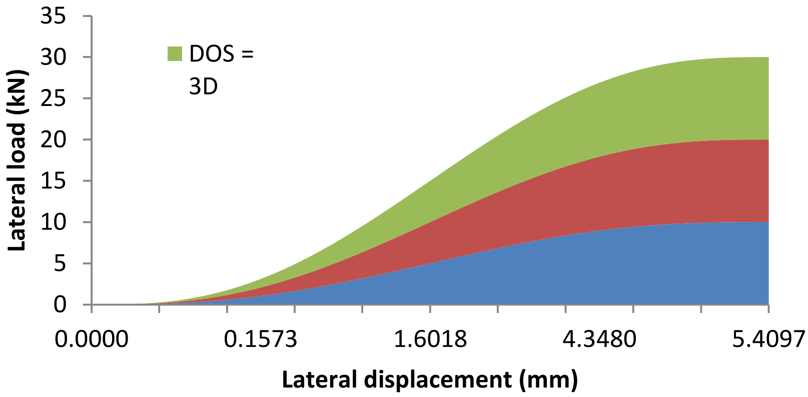

Fig. 5.8: Stacked area curve of lateral load Vs lateral deformation for different depth of socketing at eccentricity= 0

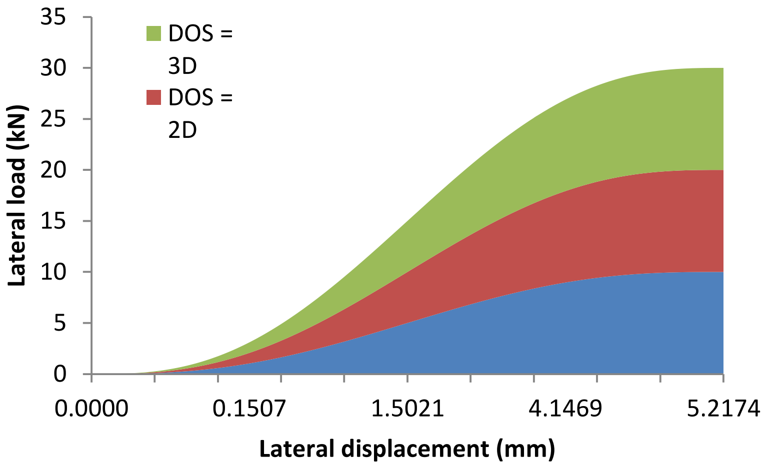

Fig. 5.9: Stacked area curve of lateral load Vs lateral deformation for different depth of socketing at eccentricity= 60mm

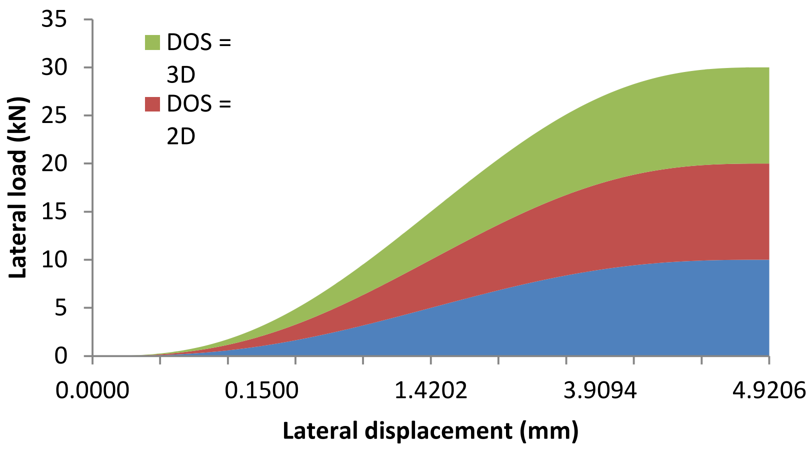

Fig. 5.10: Stacked area curve of lateral load Vs lateral deformation for different depth of socketing at eccentricity= 120mm

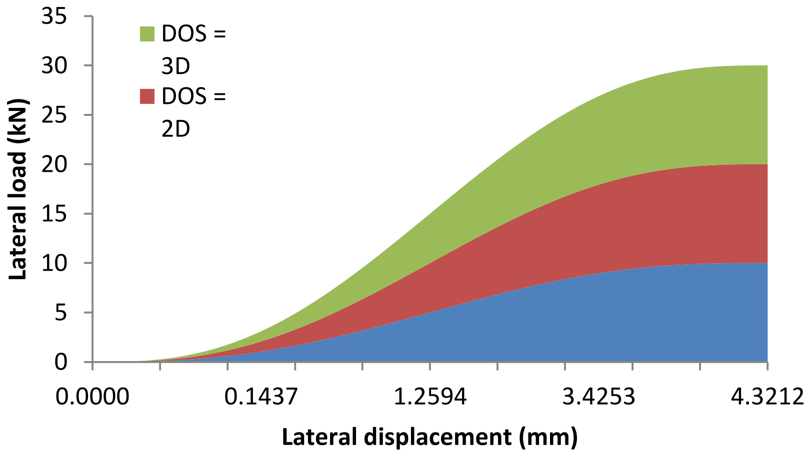

Fig. 5.11: Stacked area curve of lateral load Vs lateral deformation for different depth of socketing at eccentricity= 240mm

From the above curve it can be observed that at eccentricity = 0 i.e., at centroid of the pile cap the lateral displacement for each depth of socketing is highlighted by different colour. The area for each depth of socketing is nearly same. The different depth of socketing which is considered in the present study do not have much influence on the pile base rigidity. The stacked area curve represents same area for each depth of socketing for a maximum load of 10 kN. The pile head deflection is different for different depth of socketing but for the total load of 10 kN the entire curve obtained is approximately same. For eccentricities 60 to 240mm, the same follows and for the entire loading the area obtained for different depth of socketing is approximately same. The stiffness of a socketed pile, as well as its load-deformation qualities, soil thickness, and rock strata where the pile is socketed, determine its capacity to withstand lateral pressure., according to IS Code 14593:1998, whenever the rock through which the pile is driven is surrounded by a thick layer of earth., as illustrated in Fig. 5.1, the rock's effect is minor, and the technique of capacity assessment can be done similarly to that described in IS 2911 (Part l/&c 2) above.

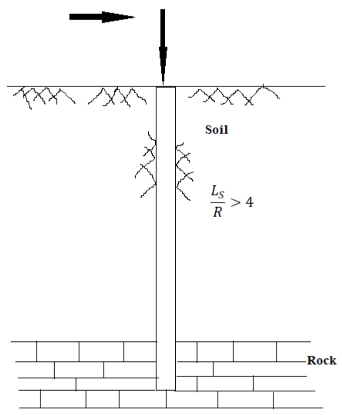

Fig. 5.12: Pile supporting soil with minimum rock penetration

E = 200 GPa = 200 × 10^3 MPa

I = $\frac{\pi D^4}{64} = 1.0178 \times 10^{-5}\ m^4$

η_h = Horizontal modulus coefficient of subgrade response = 2.5 MN/m^3

$$T = \sqrt[5]{\frac{EI}{\eta_h}}$$

$$T = 0.9597 \text{ m}$$

If depth of socketing = 1D = 120 mm then embedded length of pile in soil L = 5880 mm

L (5880 mm) ≥ 4T (3838.8 mm) hence pile is long (elastic) pile.

$$\frac{L}{T} = 6.12 > 4$$

If depth of socketing = 2D = 240 mm then embedded length of pile in soil L = 5760 mm

L (5760 mm) ≥ 4T (3838.8 mm) hence pile is long (elastic) pile.

$$\frac{L}{T} = 6.00 > 4$$

If depth of socketing = 3D = 360 mm then embedded length of pile in soil L = 5640 mm

L (5640 mm) ≥ 4T (3838.8 mm) hence pile is long (elastic) pile.

$$\frac{L}{T} = 5.87 > 4$$

From the above expression it can be concluded that at different socketing depth i.e., 1D, 2D, 3D, the $\frac{L}{T}$ is greater than 4, hence following the above condition of IS Code 14593:1998. Therefore there will no significant socketing effect on lateral capacity of pile.

5.3 Displacement of pile head & centroid of pile cap

5.3.1 Lateral displacement of centroid of pile cap

The lateral displacement of centroid of pile cap in different cases i.e., eccentricity 0 to 240mm at different depth of socketing (120 to 360mm) is analysed and tabulated below.

The lateral deflection decreases continuously as eccentricity increases from 0 to 240mm. The lateral deflection increases as depth of socketing increases from 120 to 360mm. Hence depth of socketing equals to 1D is the optimum depth of socketing to obtain minimum lateral deflection in the case when $\frac{L_S}{R} > 4$.

Table 5.1: Displacement of centroid of pile cap

Depth of socketing	Lateral displacement (mm) at e=0	Lateral displacement (mm) at e=60mm	Lateral displacement (mm) at e=120mm	Lateral displacement (mm) at e=240mm
1D	5.847	5.412	5.292	4.611
2D	5.877	5.439	5.344	4.647
3D	5.892	5.459	5.361	4.662

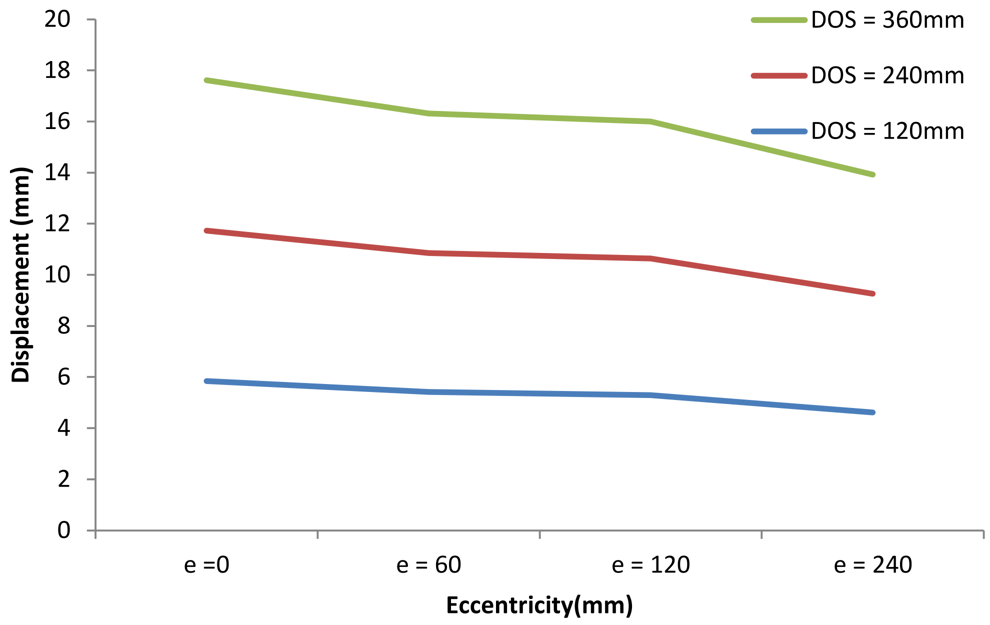

Fig. 5.13: Stacked line curve of displacement of centroid of pile cap at different depth of socketing and at different eccentricities

5.3.2 Lateral displacement of pile head

The lateral displacement of pile head in different cases i.e., eccentricity 0 to 240mm at different depth of socketing is analysed and tabulated below. For a particular depth of socketing the lateral deflection decreases continuously as eccentricity increases from 0 to 240mm.,The lateral deflection increases as depth of socketing increases from 120 to 360mm. Hence depth of socketing equals to 120mm is the optimum depth of socketing to obtain minimum lateral deflection in the case when $\frac{L_S}{R} > 4$.

Table 5.2: Displacement of pile head

Depth of socketing	Lateral displacement (mm) at e=0	Lateral displacement (mm) at e=60mm	Lateral displacement (mm) at e=120mm	Lateral displacement (mm) at e=240mm
1D	5.223	5.193	4.983	4.604
2D	5.250	5.221	5.038	4.651
3D	5.264	5.243	5.057	4.672

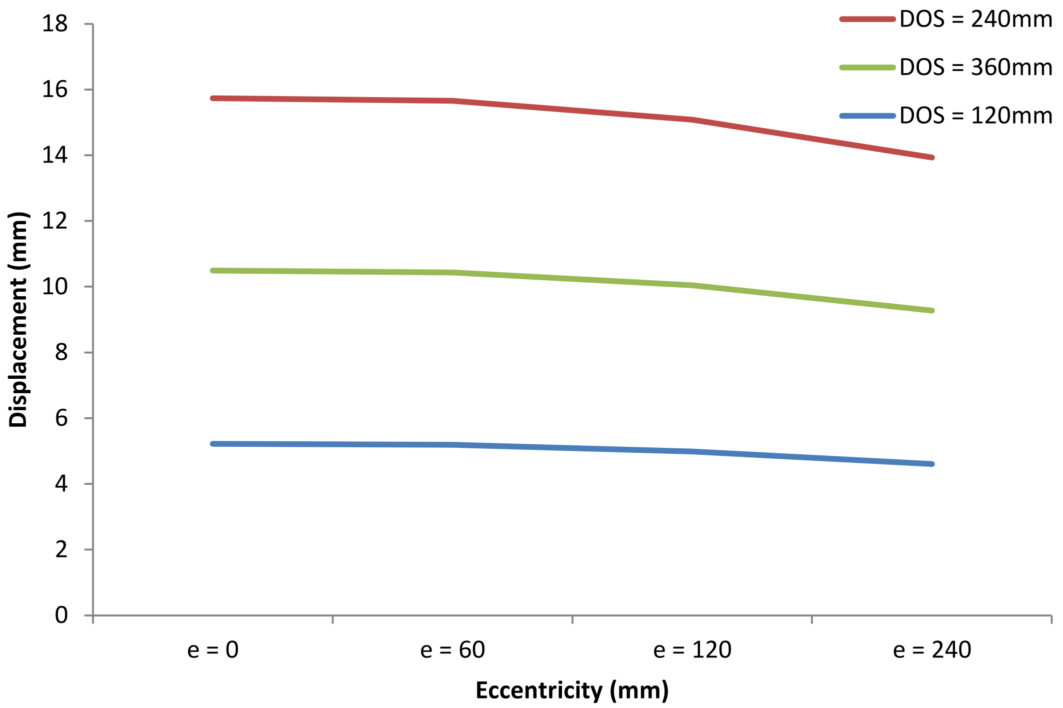

Fig. 5.14: Stacked line curve of displacement of pile head at different depth of socketing and at different eccentricities

5.4 Validation of model

According to IS 2911 (Part 1/Sec 1): 2010)

The following formulae must be used to calculate the pile head deflection, y:

Deflection, $y = \frac{H(e+Z_f)^3}{12EI} \times 10^3$ for fixed head pile

where

H = Load in lateral direction, in kN

y = Pile head deflection, in mm

E = Pile material Young's modulus, in kN/m^2

I = Moment of inertia of the cross-section of pile, in m^4

z_f = Depth to point of fixity, in m

e = Length of cantilever above ground/bed to point of load application, in m

e= 350 mm = 0.35 m

z_f = 2.111 m

E = 200 × 10^6 kN/m^2

I = 1.0178 × 10^{-5} m^4

H = 10 kN

$$y = \frac{10(0.35 + 2.111)^3 \times 10^3}{12 \times 200 \times 10^6 \times 1.0178 \times 10^{-5}}$$

y = 6.1018 mm

According to Reese and Matlock theory

Deflection $y = \frac{P_t T^3}{EI} A_y + \frac{M_t T^2}{EI} B_y$

P_t = 10 kN

T = 0.9597 m

A_y = 2.435, B_y = 1.023

M_t = −0.93× P_t × T

M_t is due to fixity of pile due to pile cap

$$y = \frac{10 \times 0.9597^3 \times 2.435}{200 \times 10^6 \times 1.0178 \times 10^{-5}} - \frac{0.93 \times 10 \times 0.9597^3 \times 1.023}{200 \times 10^6 \times 1.0178 \times 10^{-5}}$$

= 0.010573− 4.131165 × 10^{-3}

= 6.4418 × 10^{-3} m

= 6.4418 mm

Table 5.3: Comparison of abaqus model results with other available theoretical approach

Depth of socketing (mm)	Lateral displacement (mm) at e=0	Lateral displacement (mm) at e=240mm	Lateral displacement(mm) by IS code method (IS 2911: 2010)	Lateral displacement(mm) by Reese & Matlock solution, (1960)
120	5.223	4.604	6.1018	6.4418
240	5.250	4.651	6.1018	6.4418
360	5.264	4.672	6.1018	6.4418

Discussion of comparison of abaqus model results with IS code method

From the above results it can be evaluated that for socketing depth varied from 120 to 360mm, the maximum percentage variation between the results is 24.54% & minimum percentage variation in results is 13.73% for eccentricity 0 & 240mm respectively.

Discussion of comparison of abaqus model results with Reese & Matlock solution,(1960)

From the above table it can be evaluated that for socketing depth varied from 120 to 360mm, the maximum percentage variation between the results is 28.52% & minimum percentage variation in results is 18.28% for eccentricity 0 & 240mm respectively.

Therefore the results obtained from abaqus software are in permissible variation with respect to available theoretical study.

CHAPTER 6

CONCLUSION & FUTURE SCOPE

6.1 Conclusion

The results obtained in this study comes in well conjunction with the codal provisions for load eccentricity parameters ranging from 0 to 240 mm. The following conclusions may be taken from this research:

1. The compounded lateral displacement of the numerical model decreases with the increase in load eccentricity (0 to 240 mm) for different depth of socketing (120 to 360 mm).
2. The pile head deflection in lateral direction decreases with the variation in load eccentricity from 0 to 240 mm for different depth of socketing.
3. The optimum depth of socketing was observed as 120 mm. Further, the lateral pile head deflection increases as depth of socketing varies from 240 to 360 mm for the selected set of load eccentricity (0 to 240 mm).
4. The rate of increase of lateral displacement is highest for zero eccentricity and lowest for eccentricity of 240 mm for the depth of socketing ranging from 120 to 360 mm.

6.2 Future scope

The current study focuses on eccentric lateral loading on the rock socketed pile group. The depths of socketing studied in this project are varied from 120 to 360mm, and different eccentricities are 0 to 240mm for each level of socketing. The primary goal of this project is to determine and investigate the lateral load vs. lateral deformation curve. The following points should be addressed in future research:

- The current project may be expanded by increasing the depth of socketing so that $\frac{L_s}{R} < 4$ so that the in calculating the lateral capacity, the end confinement condition of the socketed pile should be taken into account,
- The current project may be expanded at different values of eccentricities other than considered in this work.
- The current project may be expanded to determine structural capacity, torsional capacity, uplift capacity etc.
- The current project may be expanded to study the effect of combined loading on pile group.
- The current project may be expanded by considering different pile group and different spacing of pile in pile group.

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05. Name of Department...Department of Civil Engineering
06. Admission Category i.e. Full Time/ Full Time (Sponsored)/ Part Time:....Full Time....
07. Applied as Regular/ Ex-student.....Regular........
08. Span Period Expired on ....May 2022
09. Extension of Span Period Granted or Not Granted (if applicable)..............................
10. Title of Thesis/ Major Project.....Study of eccentric lateral loading on rock socketed pile group........
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