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Non-linear 3-D FEA of Laterally Loaded Piles Incorporating No-tension Behaviour of Soil

D. M. Dewaikar*, S. P. Varghese, V. A. Sawant*** and H. S. Chore******

Introduction

Pile foundations are generally preferred when heavy structural loads have to be transferred through weak subsoil to firm strata. These foundations in some situations are subjected to significant amount of lateral loads besides vertical loads. Lateral forces may be due to impact of ships during berthing and wave action in case of offshore structures. Pile supported foundations of earth retaining and transmission tower structures will also subject to lateral loads. The problem of laterally loaded piles is of particular interest in the context of offshore structures. The analysis of laterally loaded piles is a complex soil-structure interaction problem. Flexural stresses developed due to combined action of axial load and bending moments must be evaluated in a realistic and rational manner for the safe and economical design of pile foundation.

The various analytical and numerical approaches available for solving this problem may be broadly grouped as subgrade reaction approach, elastic continuum approach and finite element approach. The subgrade reaction theory, also called as p-y method, idealizes a pile as elastic transversely loaded beam supported by a series of independent linearly elastic springs representing soil. Nonlinear behaviour of soil profile can also be represented by a series of p-y curves (Matlock and Reese 1961; Matlock 1970; Reese et al. 1975); but continuum property of the soil can not be properly accounted for. It is the most common approach used in the analysis of laterally loaded piles. The main advantages of these approaches are their simplicity and the relatively straightforward computations. But their main disadvantage is the difficulty of choosing an appropriate subgrade reaction modulus or p-y relationship for the given combination of pile size and soil type. These are usually estimated by empirical correlations which may lead in some cases to uncertainties and inaccurate solutions.

* Professor, Dept. of Civil Engineering, I.I.T. Bombay, Mumbai - 400 076, India. Email: dmde@civil.iitb.ac.in

** Formerly Research Scholar, Dept. of Civil Engineering, I.I.T. Bombay, Mumbai - 400 736, India.

*** Assistant Professor, Dept. of Civil Engineering, I.I.T. Roorkee, Roorkee – 247 667, India. Email: sawntfce@iitr.ernet.in

**** Assistant Professor, Dept. of Civil Engineering, D.M. College of Engg., New Mumbai - 400 708, India. E-mail: hschore@rediffmail.com.

In the elastic continuum approach, soil is represented as an elastic continuum; but treats it as linear-elastic material. The method is developed by Poulos (1971, 1972). Pile is represented as an infinitely thin linear elastic strip embedded in an elastic media. Banerjee and Davies (1978) reported elastic solutions for laterally loaded pile. Modified boundary element approach was also proposed by Poulos and Davies (1980) which employs the analytical point load solution in an elastic homogeneous half space and effects of soil non-homogeneity is also approximated by using some averaging process to obtain the soil modulus. These approaches however do not take into account the soil yielding and hence, suitable for prediction of load - deflection response of laterally loaded pile at small strain levels. Budhu and Davies (1988) reported elasto – plastic analysis of laterally loaded pile based on the boundary element method.

Development in high-speed large storage computers, leads to implementation of versatile finite element method for solving the three dimensional problems of continuum mechanics. Pressley and Poulos (1986) analysed group of piles using finite element technique with elastic - perfectly plastic soil model. Brown and Shie (1990) and Trochanis et al. (1991) studied the behaviour of single piles and group of piles with elastic plastic soil using 3-D finite element model. Muqtadir and Desai (1986) studied the behaviour of pile group with non - linear elastic soil model. Kimura et al. (1995) and Wakai et al. (1999) simulated number of model tests on fixed and free head pile groups by using 3-D elastic-plastic finite element method and found a good correlation between the experimental and analytical values. Zang and Small (2000) analysed capped pile groups subjected to lateral loads.

Above review of literature highlights the aspect of material non-linearity and has received significant attention. However above studies ignored the separation occurring between soil-pile interface and inability of soil to undergo tensile stresses. These aspects can not be modeled either by subgrade reaction approach or by elastic continuum approach.

In order to identify the significance of the material non - linearity and soil pile separation, a method has been developed using three - dimensional finite element technique for the analysis of piles subjected to lateral loads. Development of the analysis procedure and its validation using available theoretical solutions and model tests results are briefly discussed here.

Features of the Proposed Method

The proposed method is based on three - dimensional finite element technique in which the soil and pile media have been discretised into twenty - node isoparametric continuum elements. The pile elements have been assumed to remain in elastic state at all the time while soil elements have been assumed to undergo plastic yielding according to the yield criterion selected. It has been further assumed that soil is incapable of sustaining tensile stresses.

As the pile and soil follow different material behaviour, there is a possibility of relative displacement occurring at the interfaces of these media. This phenomenon is identified as the soil-pile separation occurring between soil and pile. It is simulated by introducing 8 node isoparametric interface surface elements at their interface. These elements are considered to transfer no tensile normal stresses across the interface. If any tensile normal stress develops at any instance during the loading, these elements are allowed to open up creating a gap at the soil-pile interface.

Soil Behaviour

Elasto-plastic behaviour

The soil is assumed to behave initially linearly within the elastic limit that is identified by a yield point beyond which soil is allowed to undergo plastic deformation. Three basic requirements for any elasto - plastic analysis are:

- a linear elastic constitutive relationship between stresses and strains,
- an yield criterion to identify the onset of plastic flow,
- a non-linear plastic constitutive relationship between stresses and strains to model the behaviour beyond yield.

The constitutive relationship between incremental stress {dσ} and incremental strain {dε} modeling the behaviour of soil within elastic limit is represented by generalized Hooke's law which is as given under:

$$\{d\sigma\} = [D]\{d\varepsilon\} \qquad (1)$$

where [D] is the elastic constitutive relationship matrix.

Yield criterion is a hypothesis defining the state of stress in a material at which yielding occurs and is generally expressed in the form of:

$$F(\sigma) = k \qquad (2)$$

where F(σ) is the yield function comprising of all six stress components and k is a parameter related to the material properties.

Potts and Zdravkovic (1999) suggested that von Mises criterion is suitable for analyzing the undrained behaviour of clay as it is expressed in terms of total stress. The parameters of the von Mises criterion are directly related to the soil properties, which can be measured in standard laboratory tests. The von Mises yield criterion is defined by following equation.

$$\sqrt{J_2} = k \qquad (3)$$

in which coefficient ***k*** is the cohesion of soil and J_2 is the second invariant of the deviatoric stress tensor and is given as:

$$J_2 = \frac{1}{6}\left[(\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2\right] + \left[\tau_{xy}{}^2 + \tau_{yz}{}^2 + \tau_{zx}{}^2\right] \qquad (4)$$

According to the von Mises criterion the yield function forms a right circular cylinder in the principal stress space (σ_1, σ_2, σ_3), with its axis along the space diagonal $\sigma_1 = \sigma_2 = \sigma_3$.

After initial yielding, the material behaviour will be partly elastic and partly plastic. During any subsequent increment of stress, the incremental strain, $d\varepsilon$ is assumed to be divisible into elastic component, $d\varepsilon_e$ and plastic component, $d\varepsilon_p$ and can be written as

$$d\varepsilon = d\varepsilon_e + d\varepsilon_p \tag{5}$$

The complete incremental relationship between stress and strain for elasto-plastic deformations is expressed as follows

$$\{d\sigma\} = [\mathrm{D}]\ \mathrm{d}\varepsilon_e = [\mathrm{D}]\ \mathrm{d}\varepsilon - \frac{[\mathrm{D}]\ \boldsymbol{a}\,\boldsymbol{a}^{\mathrm{T}}[\mathrm{D}]\ \mathrm{d}\varepsilon}{\boldsymbol{a}^{\mathrm{T}}[\mathrm{D}]\ \boldsymbol{a}} = [D]_{ep}\ \mathrm{d}\varepsilon \tag{6}$$

in which $[D]_{ep}$ is elasto - plastic constitutive matrix and the flow vector **a** can be written in the form

$$\boldsymbol{a}^{\mathrm{T}} = \frac{\partial \mathrm{F}}{\partial \sigma} = \left[\frac{\partial \mathrm{F}}{\partial \sigma_x} \quad \frac{\partial \mathrm{F}}{\partial \sigma_y} \quad \frac{\partial \mathrm{F}}{\partial \sigma_z} \quad \frac{\partial \mathrm{F}}{\partial \tau_{xy}} \quad \frac{\partial \mathrm{F}}{\partial \tau_{yz}} \quad \frac{\partial \mathrm{F}}{\partial \tau_{zx}} \right] \tag{7}$$

On simplification, the flow vector for the von Mises Yield condition is obtained as

$$\begin{gathered} \boldsymbol{a}^{\mathrm{T}} = \frac{1}{2\sqrt{\mathrm{J}_2}}\left[\sigma_x - p \quad \sigma_y - p \quad \sigma_z - p \quad 2\tau_{xy} \quad 2\tau_{yz} \quad 2\tau_{zx} \right] \\ where \quad p = \frac{\sigma_x + \sigma_y + \sigma_z}{3} \end{gathered} \tag{8}$$

No - tension criterion

Although a cohesive soil can sustain limited tensile stresses, it is considered to be incapable of sustaining tensile stresses in the proposed analysis for a conservative estimate. The no-tension criterion in soil is implemented through the initial stress method suggested by Zienkiewicz et al. (1968). During each incremental loading, the global Cartesian stresses $\{\sigma\}$ at every integration point are computed. The principal stresses σ_1, σ_2, and σ_3 at any integration point can be computed as the roots of the cubic equation

$$\sigma^3 - I_1\sigma^2 + I_2\sigma - I_3 = 0 \tag{9}$$

where I_1, I_2 and I_3 are the first, second and third invariant of stress $\{\sigma\}$.

For the principal stress σ_i, the direction cosines l_i, m_i and n_i are obtained by solving the following simultaneous equations

$$\begin{bmatrix} \sigma_x - \sigma_i & \tau_{xy} & \tau_{xz} \\ \tau_{xy} & \sigma_y - \sigma_i & \tau_{yz} \\ \tau_{xz} & \tau_{yz} & \sigma_z - \sigma_i \end{bmatrix} \begin{Bmatrix} l_i \\ m_i \\ n_i \end{Bmatrix} = 0 \tag{10}$$

and using the relation $l_i^2 + m_i^2 + n_i^2 = 1$ (11)

If any of the principal stresses σ_1, σ_2 or σ_3 at the integration point is tensile, that stress component is set to zero as the element cannot sustain any tension. The integration point is allowed to retain only compressive normal stresses. The cartesian stresses σ_x, σ_y, σ_z, τ_{xy}, τ_{yz} and τ_{zx} are then recomputed from the retained principal stresses and the corresponding direction cosines using the following relation

$$[\sigma] = [T]^T [\sigma_i][T] \tag{12}$$

in which [T] the co-ordinate transformation matrix $[\sigma]$ Cartesian stress matrix, and $[\sigma_i]$ is principal stress matrix defined by the following relations

$$[T] = \begin{bmatrix} l_1 & m_1 & n_1 \\ l_2 & m_2 & n_2 \\ l_3 & m_3 & n_3 \end{bmatrix}; [\sigma] = \begin{bmatrix} \sigma_x & \tau_{xy} & \tau_{xz} \\ \tau_{xy} & \sigma_y & \tau_{yz} \\ \tau_{xz} & \tau_{yz} & \sigma_z \end{bmatrix} and [\sigma_i] = \begin{bmatrix} \sigma_1 & 0 & 0 \\ 0 & \sigma_2 & 0 \\ 0 & 0 & \sigma_3 \end{bmatrix} \tag{13}$$

Soil-Pile Separation

In the proposed method, pile and soil elements are required to simulate behaviour of two different materials. The separation occurring between these elements is modelled by introducing an interface surface element at the contact between the pile and soil elements. Eight-node isoparametric interface elements as proposed by Buragohain and Shah (1978) are employed. These elements have zero thickness. The nodal degrees of freedom of this element are the relative displacements Δu, Δv, Δw at the nodes in the global directions (x, y, z). At any point on the surface, there are three stress components: two shear stresses, σ_{s1} and σ_{s2} acting along the two orthogonal tangential directions, and a normal stress σ_{sn} acting normal to the element surface. These stresses are directly related to relative displacements through shear and normal stiffness (k_{s1}, k_{s2}, k_{sn}) in respective direction.

$$\begin{Bmatrix} \sigma_{s1} \\ \sigma_{s2} \\ \sigma_{sn} \end{Bmatrix} = \begin{bmatrix} k_{s1} & 0 & 0 \\ 0 & k_{s2} & 0 \\ 0 & 0 & k_{sn} \end{bmatrix} \begin{Bmatrix} \Delta u_{s1} \\ \Delta u_{s2} \\ \Delta u_{sn} \end{Bmatrix} \tag{14}$$

Here, k_{s1} and k_{s2} are the stiffness of the element in the orthogonal tangential directions and k_{sn} is the stiffness in the normal direction. Further, Δu_{s1}, Δu_{s2} and Δu_{sn} are the corresponding relative displacements.

During an incremental load application, the normal stresses at the integration points in these elements are checked for tension for each stage of iteration. If normal stress σ_{sn} developed therein indicates positive normal relative displacement at the nodes, then adjoining solid elements are moving apart. At this stage, all the stresses at that point are set to zero as it has failed in tension and incapable of sustaining the computed stresses. The retained stresses are used to compute the internal forces along with the total stresses obtained for the solid elements after the no-tension and plastic yield checking. These internal forces are compared with applied load to estimate residual forces.

The system is allowed to deform further under these residual forces in an iterative process. As the solution converges during these iterations, the incremental normal relative displacements at the interface element nodes decrease and eventually, become negligible at convergence. Decrease in normal relative displacements at the nodes indicates that the normal stresses at the interface are also getting negligible at convergence. Ultimately, the interface elements will have negligible tensile normal stress within them. This guarantees that these elements will not transfer any tensile normal stresses across the interface and will remain open with the development of tensile stress.

Iterative Procedure

The global stiffness matrix [K] is computed only once based on elastic constitutive relation matrix [D]. In-situ stresses in the soil are accounted by applying self weight of pile and soil at the initial stage of the analysis, and displacements are initialized to zero for further analysis. In order to perform the elasto-plastic and no-tension analysis, external loading is applied in small increments (Δp).

1. In each load increment, the incremental stresses ($d\sigma$) are computed using the elasticity coefficient matrix [D] and the corresponding incremental strains $d\varepsilon$ using $\{d\sigma\} = [D]\{d\varepsilon\}$.
2. Incremental stresses $\{d\sigma\}$ are accumulated to the already existing stresses of previous iteration to get total stresses $\{\sigma\}$ which is then checked for tensile stresses so that all the normal stresses are of compressive nature.
3. These are further checked for plastic yielding as per the von Mises criterion selected. If yielding is not indicated, the point under consideration is still in elastic range and the total stress computed is treated as the converged total stress (σ) at that point.
4. If the point has yielded, the total stress is adjusted so as to bring the stress-state on to the yield surface by resorting to the procedure suggested by Owen and Hinton (1980) as mentioned below:

   For a point that has yielded, only part of incremental stress which satisfies the yield criterion is computed and added to the total stress. For the r^{th} iteration, the incremental stress $\{d\sigma\}$ satisfying the yield function is given by

   $$\{d\sigma\} = [D]_{ep}\{d\varepsilon\} \tag{15}$$

   The total stress state (σ^r) at the end of r^{th} iteration is then given as

   $$\{\sigma\}^r = \{\sigma\}^{r-1} + \{d\sigma\} \tag{16}$$

   In this way, it is ensured that the state of stress always remains on or inside the yield surface.

5. Any tensile normal stress developed in the interface elements indicates positive normal relative displacement at the nodes that implies that the adjoining solid elements are moving apart. At this stage, all the stresses at that point are set to zero as it has failed in tension and can't sustain the computed stresses.
6. Accumulated total stresses are converted to equivalent nodal forces

$$\{f\} = \iint_V [B]^T \{\sigma\}^r \, dV \tag{17}$$

7. Equivalent nodal forces are compared with the applied total load and the difference is estimated as residual forces. These residual forces are applied back on to the system and solved for corresponding incremental displacements.
8. From these incremental displacements, incremental stresses are computed and added to the total stress that is further subjected to the check mentioned above. The iterative procedure is repeated until a convergence in incremental displacement is observed.

Validation of Proposed Method

In order to validate the formulation of elasto - plastic and no - tension model, a surface strip footing on purely cohesive and weightless soil has been analysed. From the classical bearing capacity theories, it is known that the ultimate bearing pressure q_u for a strip footing on the surface of a cohesive soil is given as

$$q_u = c\,N_c \tag{18}$$

where N_c is a bearing capacity factor having value ranging from 5.14 as suggested by Prandtl (1920) to 5.7 that by Terzaghi (1943). In the test problem, a strip footing has been modeled with boundary conditions to simulate the plane strain conditions. A plot of the load-displacement curve in the form of vertical displacement of the centre of the rigid footing with the normalized applied load (with respect to soil cohesion) has been presented in Figure 1 where from N_c is estimated to be 5.1 which, further, is in good agreement with the theoretical value of N_c as suggested by Prandtl (1920).

From this it can be concluded that the elasto - plastic behavior of purely cohesive soil has been modeled properly and in a most realistic manner, as per *von Mises* yield criteria.

Comparison with Analytical Solutions

For verification of proposed method, a single pile has been analysed and the results obtained from the there from have been compared with those obtained using the methods suggested by Poulos and Davies (1980), and Randolph (1981) for a square pile of width 1m and length 20m embedded in soil with uniform strength. Both these methods consider the soil as elastic medium. In addition, Poulos and Davies consider the local yielding of soil along the pile length through a yield displacement factor. Two different soil modulus values of 5 MPa and 25 MPa were considered. The lateral displacement at the top of the pile under the applied lateral load has been compared in Figure 2. A very good agreement is observed in the initial tangent parts of the load- displacement curves obtained resorting to all the three methods. The non-linear part of the results presented by Poulos and Davies (1980) depends on the ultimate load that was determined by the method suggested by Broms (1964). In the present case, since the ultimate load was lower than that obtained from the Poulos and Davies (1980), the load displacement curve with proposed method is observed below. Similar trend was observed in the load-displacement curve with reference to the soil modulus of 25 MPa.

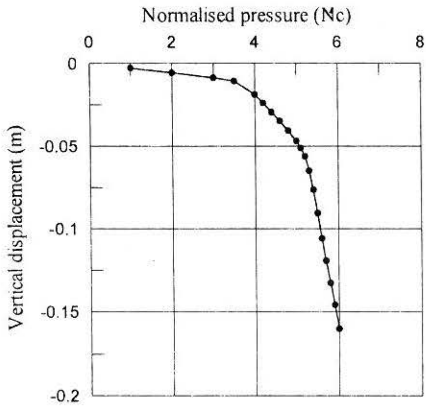


Fig. 1 Load - Displacement Curve - Surface Strip Footing on Cohesive Soil

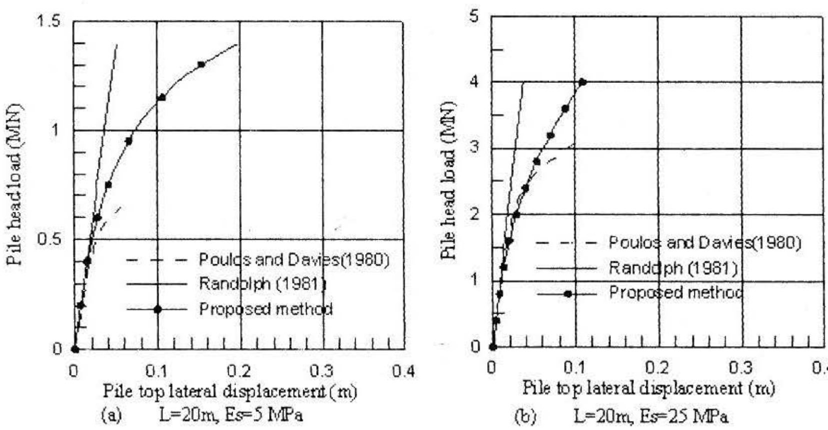


Fig. 2 Comparison with Poulos and Davies (1980) and Randolph (1981) for Soil with Uniform Strength

Similarly, comparison of results was also made with the results obtained from Randolph (1981) and Budhu and Davies (1988) for soil with strength increasing with the depth. A square pile of width 1m and length 20m was considered in the analysis. Two different soil strengths were assumed and the rate of increase of shear strength for these soils were c_m =1 kPa/m and 2 kPa/m. The results have been compared in Figure 3 and it shows a very good agreement in the initial linear range. Budhu and Davies considered yielding of soil. The results are in good agreement with the results of Budhu and Davies.

Randolph (1981) approach is based on linear elastic behaviour of soil; the results indicated by Randolph's approach do not fall in line with that of the proposed method at higher load level.

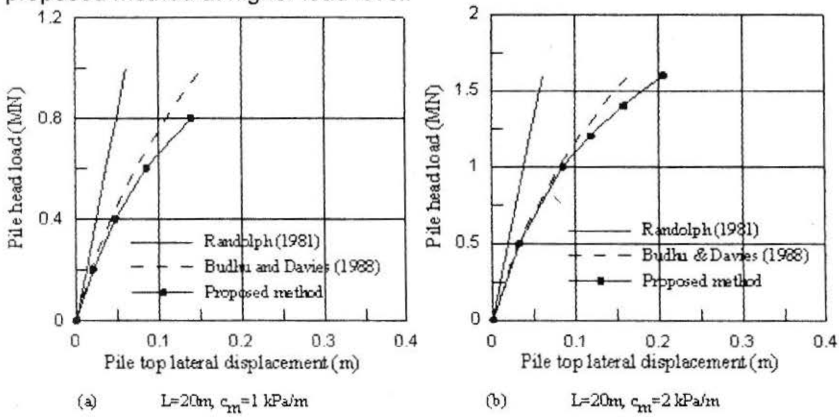


Fig. 3 Comparison with Randolph (1981) and Budhu and Davies (1988) for Soil with Strength Increasing with Depth

Comparison with Model Test Results

Gabr et al. (1994) have reported the results of field model tests on single piles loaded laterally at the top. These cases have been analysed using the proposed method and the results have been compared with those measured at site. The data and the results of the two cases have been presented below.

Case 1: Cannons Park, England

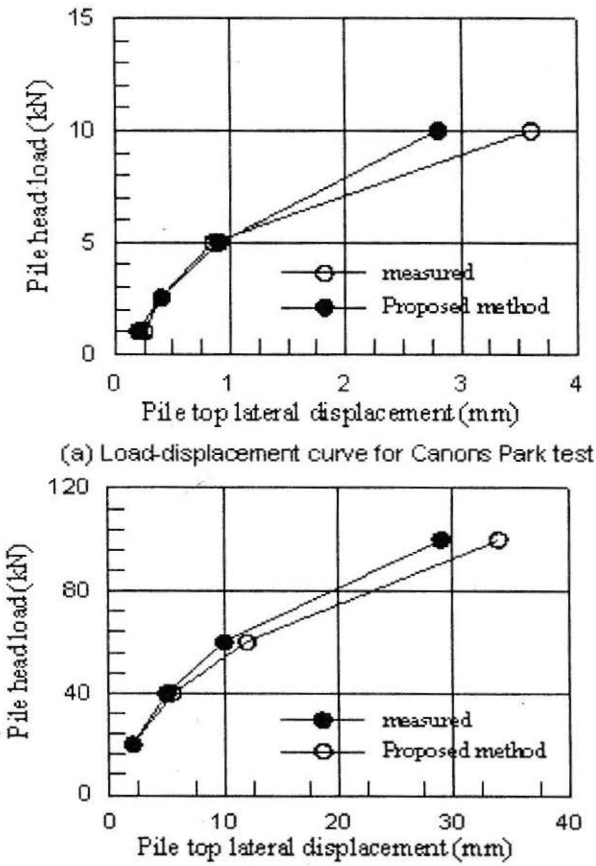
In this test, a cast-in-situ pile of diameter 0.17 m and length of 4.5 m was subjected to a lateral load at the top. The soil at the site was normally consolidated clay, with its strength increasing with depth. Results of the finite element analysis employed in the present study along with the field observation have been presented in Figure 4 in the form of load-displacement curve, for comparison. Results of the analysis show a very good agreement with the observations made on the field.

Case 2: Brent Cross, England

In this test, a tubular steel pile embedded in clay with a penetration length of 16.5 m was subjected to the lateral load at the pile head. The clay at the site was having average shear strength of 50 kPa. Stiffness of the pile was estimated to be 51.4 MN m^2. The deflection at the top of the pile obtained from the finite element analysis employed in the present investigation has been compared with those measured at the site in Figure 4. In this case also, the results obtained from the program computations were found to be in good agreement with field observations.

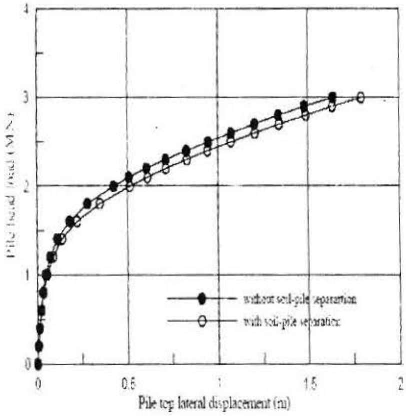
Parametric Study

A single pile was analysed in two different cases to examine the effect of soil-pile separation. In the first case, surface interface elements were introduced between soil and pile to allow separation between them. In the second case, no interface elements were considered. The lateral load was applied at the pile head in small increments until it deflected through a distance of about twice the pile size. A series of load-displacement curves obtained from the analysis are shown in Figures 5(a) to 5(e). These results correspond to a pile penetration length of 20 m and modulus of 210 GPa. It is clear from these plots that the capacity of a single pile in the case where soil-pile separation was allowed, is less than when soil-pile separation was not allowed. In other words, the analysis that does not account for the separation between soil and pile under lateral load, tends to over-estimate the capacity of a single pile. Because, the load-deformation curves do not show a well-defined collapse load, the ultimate capacity was assumed to be the load carried by the pile corresponding to a pile top deflection of 20% of the pile size. In all the cases studied, it was found that separation effect reduced the capacity of the pile. The over-estimation of capacity, in cases where separation was not considered, was determined and found to be in the range of 10 - 12% for a pile length of 10 m, while it was in the range of 4 - 6% for rest of the pile lengths examined.

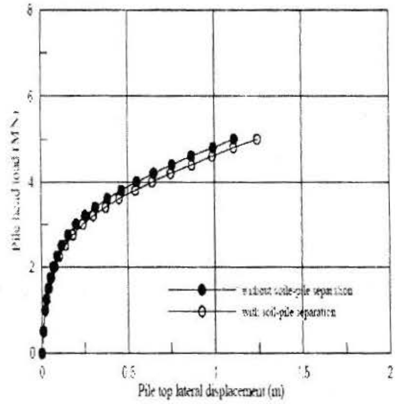


(b) Load-displacement curve for Brent Cross test

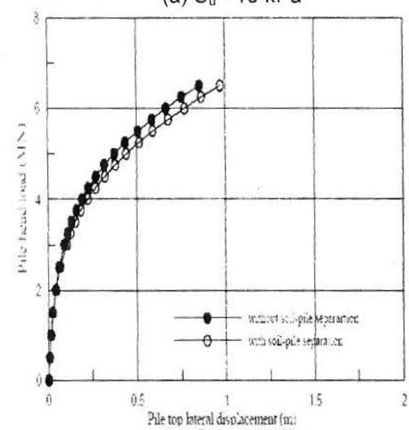
Fig. 4 Comparison of Results with Field Tests, Gabr et al. (1994)



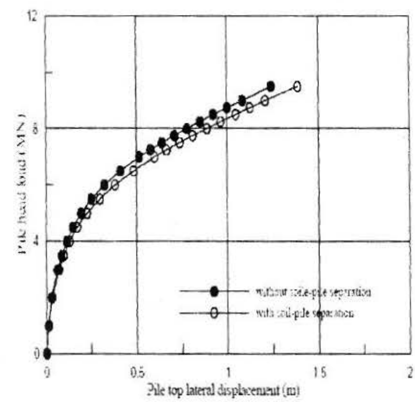
(a) S_u= 10 kPa



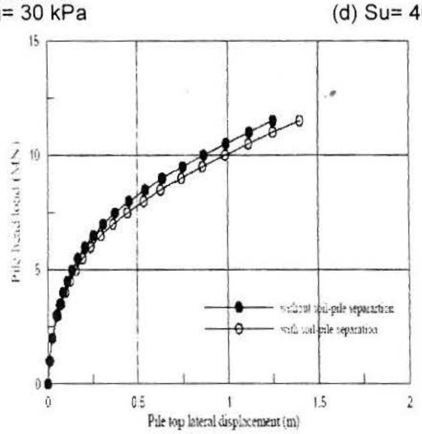
(b) S_u= 20 kPa



(c) Su= 30 kPa



(d) Su= 40 kPa



(e) S_u= 50 kPa

Fig. 5 Load - Displacement Curves for a Free Head Pile

The variation of over-estimation for different pile lengths and soil undrained shear strength values has been presented in Figure 6. In general, piles having shorter penetration lengths of 10 m and 20 m are more affected by the separation than the piles having larger lengths of 30 m and 40 m. Short piles undergo rigid body rotation under lateral load and soil-pile separation develops both at rear of the pile near ground line and in front of the pile near its tip. On the other hand, longer piles have their significant lateral deflection only near the upper portion of the penetration depth, thereby the separation occurs only near the top portion of the pile. Since the lateral capacity reduction is directly related to the pile – soil separation, short piles will get affected more than the longer piles.

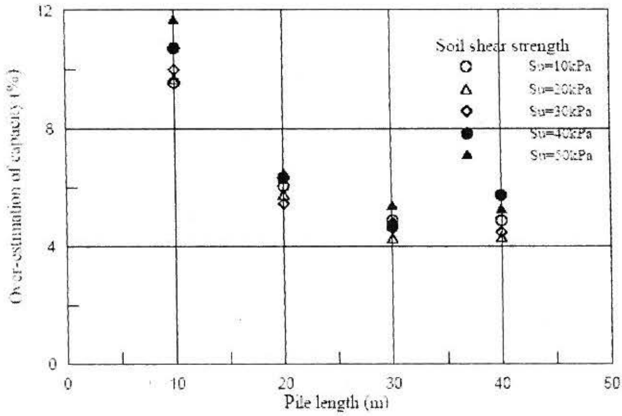


Fig. 6 Overestimation of Pile Capacity due to Effect of Soil - Pile Separation

The bending moments in the pile at various applied pile head loads were computed from bending stresses in the pile. Distribution of the bending moment along the pile at various pile head loads for a length of L = 20 m and soil modulus of E_s = 20 MPa are presented in Figure 7. This Figure also shows the bending moment distributions for a similar case where soil-pile separation was not allowed for the benefit of comparison. It is observed that the separation effect causes an increase in the bending moment in the piles. This increase was found to be in the range of 7 - 10% for all the cases studied with the separation effect being prominent at lower load levels. The maximum bending moment occurred at a depth of 0.40 L below the pile head in the case of a free head pile. It is clear from these results and the load displacement plots that the pile – soil separation causes a reduction in pile capacity and an increase in the bending moments in the pile.

The variation of ultimate capacity of a single pile with the soil shear strength is presented in Figure 8. These are obtained for various pile lengths. It is observed that capacity increases almost linearly with the shear strength for a given pile penetration length. Further, there is a general increase in capacity with increase in pile penetration length. The variation of capacity with pile penetration length L is presented in Figure 9 for a pile modulus value of 210 GPa. It can be seen that there is a general increase in the capacity with increase in pile penetration length for all the soil shear strength values used and for penetration lengths up to 30 m. The computations indicated no further increase in capacity when the pile length increased from 30 m to 40 m. It is evident from these results that capacity of a single pile generally, increases with increase in pile length with other parameters remaining the same. However, there appears to be a limiting value of pile penetration, beyond which capacity remained constant.

A parametric study was also carried out to identify the effect of ratio of soil modulus values to shear strength on single pile capacities. In addition to the values of 500 S_u, E_s values of 250 S_u, 750 S_u and 1000 S_u were also considered in this analysis. Variations of capacity with ratio of E_s/S_u, for a pile penetration length of 20 m are presented in Figure 10. It is observed that the single pile capacity increases almost linearly with shear strength for any E_s/S_u values. This is following the reason that the yield envelope gets bigger for higher shear strength values. Further, at any given soil shear strength; the capacity is more for higher soil modulus values.

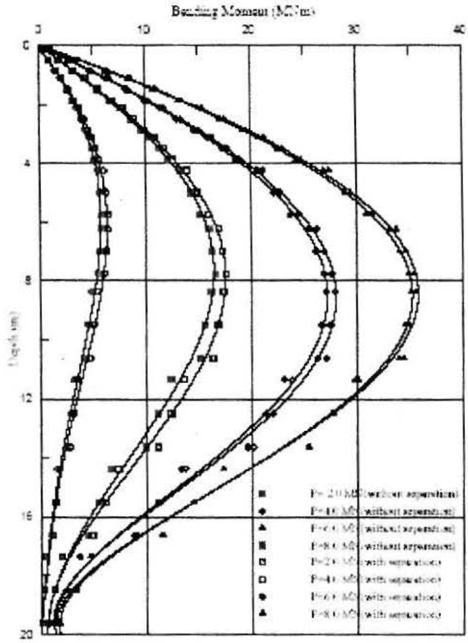


Fig. 7 Bending Moment Distribution in a Free Head Pile with L = 20 m and E_s = 20 MPa

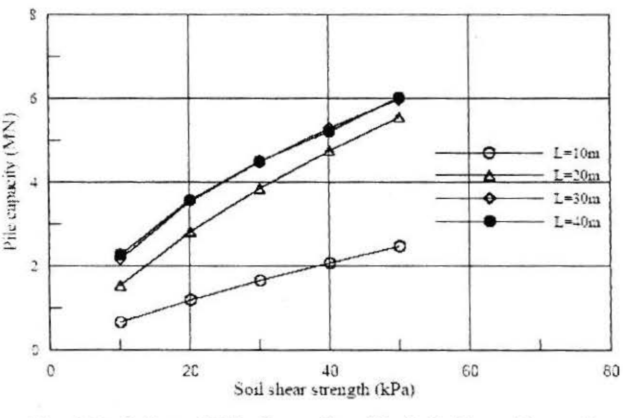


Fig. 8 Variation of Pile Capacity with Soil Shear Strength

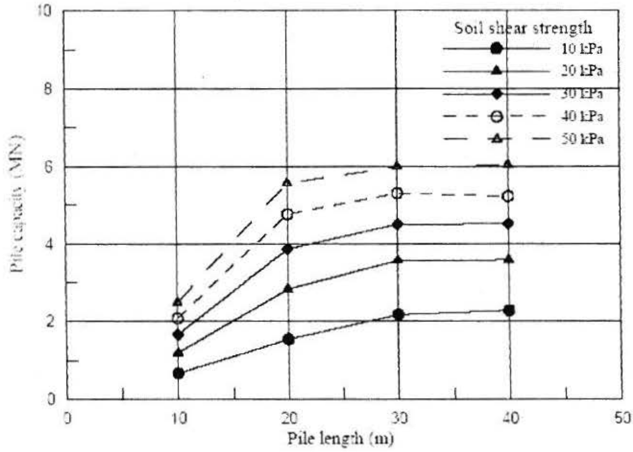


Fig. 9 Variation of Pile Capacity with Pile Length

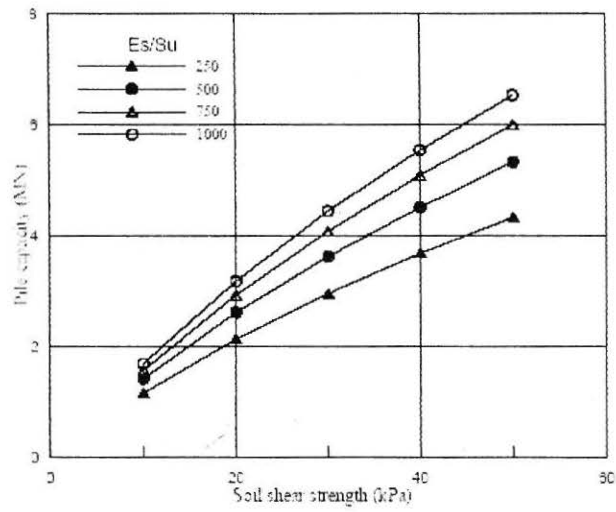


Fig. 10 Variation of Pile Capacity with E_s/S_u Ratio

Conclusions

A new method is proposed to analyse laterally loaded piles which model three important aspects of material behaviour. The soil has been assumed to undergo plastic yielding based on von Mises yield criterion. In addition, the soil has been considered to be incapable of sustaining any tensile stresses. Effect of soil-pile separation is incorporated with modeling interface element. An incremental iterative procedure is suggested to account these aspects. The proposed method has yielded the satisfactory results that are in good agreement with the available analytical solutions and the field results reported in literature. It has been observed that soil-pile separation causes reduction in pile lateral capacity and increase in bending moments developed in the pile. It is evident from these results that capacity of a single pile generally, increases with increase in pile length. However, there appears to be a limiting value of pile penetration, beyond which capacity remained constant. It is observed that the single pile capacity increases almost linearly with shear strength for any E_s/S_u values.

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