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Full Length Research Paper

Reinforcement in concrete piles embedded in sand

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In this study, the necessity of reinforcement in concrete pile (bored or driven) is assessed. The soil was assumed to be unsaturated and homogeneous sandy soil. Throughout the study, a finite element computer program was used and the pile was modeled as a beam-on-elastic foundation. The soil is represented by discrete spring. The stiffness of each spring is considered to be linearly variable with depth. The moment loading, lateral loading, pile length, pile diameter, in addition to the angle of internal friction and soil density were taken as parameter to study their effect on the extent of reinforcement along the pile shaft. It is concluded that for piles embedded in sand, a length of reinforcement not less than 40% of pile length for bored piles and 20% for driven piles is needed.

Key words: Sand, reinforcement requirements, bored piles, driven piles, pile reinforcement.

INTRODUCTION

The cost of steel reinforcement is increasing and the demand is also high. Hence, it is necessary to study the possibility of reducing this material to the minimum during pile construction. In the past, piles were fully reinforced. Nowadays, the designers prefer to minimize the length of reinforcing bars so that they may reduce the cost of piles. This minimization requires well separation for the cases where the piles need fully or partially reinforcement and the cases where the reinforcement can be completely eliminated. After making a survey on the codes of practice and studying their recommendations in such field, it was found that all the codes of practice give specifications and limitations for the percentage of bars that should be provided in the pile cross-sectional area. But the depth of extension of this reinforcement along the pile is not specified and thus left to the designer discretion. The main objective of this study can be divided into two main categories, that is, (a) make a survey on the codes and their requirements on pile reinforcement, and (b) investigating whether the pile needs to be provided with reinforcement or not, and to what length the pile reinforcement is needed.

Past studies of pile reinforcement

The reinforcements are required in concrete piles to resist bending and tensile stresses, but may be used to carry a portion of the compression load. The extension of the reinforcement required at any section of the pile depends upon the loads and stresses applied to that section.

Reinforcement is required if the pile is subjected to bending moments. The bending moment and shearing force in a pile subject to lateral loading may be assessed using the method of Matlock and Reese (1960) as given in Figure (1). This method models the pile as an elastic beam embedded in a homogeneous or non-homogeneous soil. The structural capacity along flexible pile is likely to govern the ultimate capacity of a laterally-loaded pile.

The pile reinforcement undergoes the needs and the requirements. Therefore, there is no specific limit where the pile should be reinforced. The needs are determined by one of the pile analysis theories, where field observations and some theoretical consideration specify the requirements.

REINFORCEMENT REQUIREMENTS

Precast concrete piles

The reinforcement should be provided in all precast concrete piles to take up the stresses caused in handling, pitching and driving and this greatly exceeds what is needed once the pile is in the ground (Saurin, 1949; Whitaker, 1976; Mohan, 1990).

Indian Standard (IS 2911 - part I, 1964-this code of practice is more than 40 years old, and its relevance in today’s practice may

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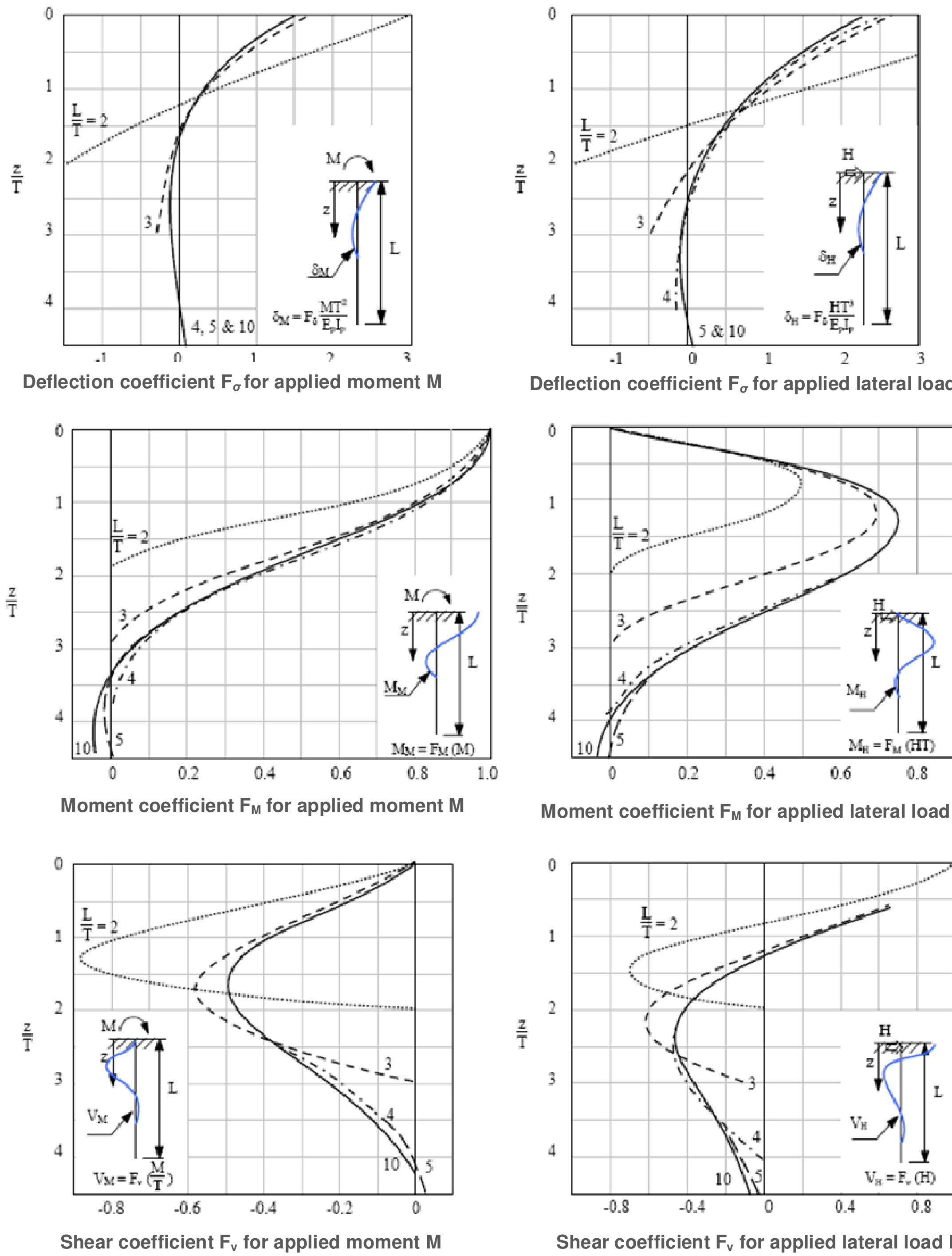

Figure 1. Influence coefficients for piles with applied lateral load and moment (flexible cap or hinged end conditions) (Matlock and Reese, 1960).

not be significant) recommended that the area of the main longitudinal reinforcement shall be not less than the following percentages of the cross-sectional area of the pile:

(a) 1.25% for piles with length less than 30 times the least width
(b) 1.5% for piles with length between 30 to 40 times the least width
(c) 2.00% for piles with length greater than 40 times the least width

Notes: (1) Stiffness factor, $T = \sqrt[5]{\frac{E_P I_P}{n_h}}$

where E_p, I_p = bending stiffness of pile and n_h = constant of horizontal subgrade reaction. (2) Obtain coefficients Fσ, FM and Fv at appropriate depths desired and compute deflection, moment and shear respectively using the given formula.

Where the lateral reinforcement shall be in the form of hoops or links and shall be not less than 5 mm in diameter. The volume of lateral reinforcement shall be not less than the following percentage of the gross volume of the pile:

(a) 0.20% in the body of the pile,
(b) 0.60% at each end of pile for a length of about 3 times the least width.

The transition between the closer spacing and the maximum shall be gradual over a length of 3 times the least width.

South African Bureau of Standards (SABS 088, 1972) recommended that the cross-sectional area of longitudinal reinforcement should be at least 0.8% of the cross sectional area of the pile, and lateral ties should be at least 6 mm in diameter, closely spacing at both ends of the pile.

American Concrete Institute (ACI 543R, 1974) recommended that the longitudinal steel cross-sectional area should not be less than 1.5% or more than 8% of the cross-sectional area of the pile. At least six longitudinal bars should be used for rounds or octagonal piles and at least four bars for square piles. The lateral steel should not be less than 0.25 in. (6 mm) in diameter and spaced not more than 6 in. (150 mm) on centers except that the spacing should be closer at each end of the pile.

What is DIN? German Institute for Standardization (DIN 4026 1975) recommended that the longitudinal reinforcement of the piles, at length not exceeding 10 m, shall be not less than 0.8% of the cross-section of the pile. For solid rectangular piles, at least 4 longitudinal bars of 14 mm diameter must be arranged in the corners; for round piles, at least 5 longitudinal bars of 14 mm diameter have to be placed and evenly spaced, without end hooks. The transverse reinforcement should be at least 5 mm in diameter. The axial spacing (pitch) of a helix should be not exceed 120 mm and reduced to about 50 mm over a length of 1 m at top and bottom of the pile.

Japanese Industrial Standard (JIS A5310, 1987) recommended that the longitudinal reinforcement shall consist of 6 mm or more bars, with a steel ratio not less than 0.8% and it is desirable that they are arranged uniformly along the circumferences of the concentric circles in the respective cross sections of reinforced concrete pile. The minimum spacing shall not be smaller than 0.75 times the maximum dimension of the coarse aggregate.

The spiral bars shall be arranged outside the longitudinal reinforcement. The additional bars shall have a diameter not smaller than 3 mm, and a pitch not larger than 110 mm.

Many literatures recommended the same specifications for the reinforcement of precast concrete piles (Chellis, 1961; Rennie, 1986; Jha and Sinha, 1995).

Cast *in-situ* concrete piles

The extent of reinforcement in cast-*in-situ* concrete piles is governed by the loads involved and the design analysis. Some codes differentiate between the recommendations of reinforcement in both driven and bored cast-*in-situ* concrete piles (BS 8004, 1986, and DIN 4014-part I, 1975), where others consider them as one unit under the main article, cast-*in-situ* concrete piles, (IS 2911-part I, 1964, and ACI 543R, 1974).

IS 2911-part I (1964) recommended that any reinforcement in cast-*in-situ* concrete piles should be made up into cages sufficiently well wired to withstand handling without damage. The bars should be so spaced as not to impede the placing of the concrete and the lateral ties or spiral should not be closer than 150 mm center to center. Reinforcement in the pile may reflect the manner of the transmission of the load by the pile to the soil, and need not normally exceed 0.8% of the cross-sectional area of the pile.

ACI 543R (1974) recommended that the reinforcement is used in cast-*in-situ* concrete piles for any unsupported section of the pile, uplift loads, or lateral loads when the analysis indicates. Unsupported sections (which extend through, air, water, or even through very fluid soil) should be designed to resist buckling under the imposed loads. Sufficient longitudinal and lateral steel should be used for the loads and stresses to be resisted.

For lateral loads, the pile should be designed and reinforced to take loads and stresses involved. In general, the amount of reinforcement required will be governed by the loads involved and the design analysis. Except for uplift loads, it is recommended that not less than four longitudinal bars be used. The extent of reinforcement below ground surface depends on the flexural and load distribution analysis.

DIN 4014-part I (1975) recommended that bored piles normally contain both longitudinal and transverse reinforcement extending over the entire length of the pile. The reinforcement shall be made in the form of a reinforcing cage and installed in the casing pipe in such a way that it cannot be displaced during the concreting or lifted with the casing when the latter is being extracted. A reinforcement extending over the full length of the pile may be dispensed with if the piles are vertical and are not less than 300 mm in diameter and not more than 7.5 m in length. Provided there is no likelihood of the piles being subjected to bending by either earth pressure, the lateral pressure of plastic soft soils, eccentric loading or any other cause.

The longitudinal reinforcement shall comprise not less than five reinforcing bars of 14 mm diameter, spaced at intervals of not more than 200 mm. The total of the cross-section area of the longitudinal reinforcement must be not less than 0.8% of pile cross-section. If any permanent casing is used, it shall not be reckoned as part of the reinforcement because of the risk that it may rust through.

The transverse reinforcement shall be arranged in helical form with a bitch between 150 to 200 mm. It must have a diameter of not less than 5 mm, when the pile diameter is not more than 350 mm, or 6 mm with thicker piles.

BS 8004 (1986) and CP 2004 (1972) recommended that the reinforcement should normally be carried down for the full length for bored piles and into the enlarged base, if piles are required to resist tensile force. Where the tensile forces are small, the reinforcement need only be of the length necessary to transmit fully the tensile forces. Reinforcement should be provided for tensile forces, which are not expected to exist when the structure is completed.

For driven cast-*in-situ* concrete piles, it was recommended that the reinforcement may be provided over the whole of their length, over part of their length, or merely provided with short splice bars at the top for bending into the pile cap. The extent of the reinforcement will depend on whether the pile is used to resist tensile or bending forces, on the type of foundation, and on the possibility on horizontal or vertical movements due to the installation of other piles nearby or to moisture changes in the soil.

Derrington (1966) stated that if piles of 3 ft (0.9 m) diameter and over do not generally require reinforcement unless passing through a considerable depth of very soft ground. Only nominal reinforcement is required at the pile head for connection to pile cap or column. In 2 ft (0.6 m) and 2.5 ft (0.75 m) diameter piles it may be considered desirable to reinforce the upper part of the pile shaft if this passes through weak ground. Large diameter piles may be reinforced to resist bending moment resulting from horizontal forces, these forces being balanced by the passive resistance of ground against the pile.

Fleming et al. (1985) recommended that for bored piles loaded in compression alone, it is only necessary to reinforce the shaft to a depth of 2 m greater than the depth of temporary casing, to prevent any tendency for concrete lifting when pulling the casing. Piles subjected to tension or lateral forces and eccentric loading (possibly being out of position or out of plumb) require suitable reinforcement to cope with these forces. Nominal reinforcement for piles in compression only would comprise about four 12 mm diameter bars for a 400 mm diameter pile to five 16 mm diameter bars for a 550 mm diameter pile. A special cage of 5 mm steel, or hoops of flat steel, are employed as lateral ties. Bars should not be so densely packed that concrete aggregate cannot pass freely between them and hoop reinforcement is not recommended at closer than 100 mm centers. Provided the cage can be oriented, maximum steel need only be placed over that part of the pile subjected to maximum stress, and a reduced density can be used in the plane of the natural axial.

For driven cast-*in-situ* concrete piles, Fleming et al. (1985) recommended that widely spaced reinforcement bars being necessary to allow the low workability mix to penetrate to the interior of the pile. If the pile is to resist compressive forces only, the reinforcement may be restricted to the upper section.

Bowles (1988) stated that, for bored piles, the reinforcing bars may be required only in the upper region for moments that are carried by the shaft, because these moments dissipate with depth are hence the shaft load is primarily axial at about L/2. At this depth, temperature changes are not great; therefore, longitudinal and spiral reinforcements are not required.

Tomlinson and Woodward (2008) stated that reinforcement is not needed in bored piles unless uplift loads are to be carried (uplift may occur due to the swelling and shrinkage of clays). Reinforcement may also be needed in the upper part of the shaft to withstand bending moments caused by any eccentricity in the application of the load, or by bending moments transmitted from the ground beams.

Design aspects

Laterally loaded piles are analyzed by means of two main categories, one using Winkler modulus of sub-grade reaction concept as the soil model, and the other using and elastic continuum as soil model. Each one has its advantages and disadvantages.

Matlock and Reese (1960) formulated and solved the differential equation for the deflection of the pile using a beam-on-elastic foundation approach. The soil strength is characterized using coefficient of sub-grade reaction. They obtained a series of non-dimensional curves so that a user could enter the appropriate curve with the given lateral load and estimate the ground-line deflection and maximum bending moment in the pile shaft.

Broms (1965) presented methods for the calculation of lateral deflections at working load based on the concept of a coefficient of subgrade reaction. It has been assumed that the coefficient of sub-grade reaction increases linearly with depth in case of cohesionless soils, and that it is constant with depth for cohesive soils.

Poulos (1971) analyzed the behavior of piles that were subjected to lateral load and moment using the continuum theory. It was found that the major factors influencing the pile behavior are the length to diameter ration, L/D, and the pile flexibility ratio, K_R, which is defined as:

$$K_R = \frac{(EI)_P}{E_s \times L^4} \tag{1}$$

where

K_R is the pile flexibility ratio,
E is the modulus of elasticity of the pile,
I is the moment of inertia of the pile,
E_s is the modulus of elasticity of soil, and
L is the length of pile embedded in soil.

Randolph (1981) studied the response of flexible pile to lateral loading using finite-element method and treated the soil as an elastic continuum with a linearly varying soil modulus. It was found that the maximum bending moment induced in a free-headed pile subjected to lateral force, H, can be estimated as:

$$M_{\max} = \frac{0.1}{\rho_c} \times H \times l_c \tag{2}$$

where

M_{max} is the maximum bending moment induced,
H is the lateral force,
ρ_c is the factor giving relative homogeneity of soil, and
l_c is the critical length of the pile.

Gleser (1984) suggested a generalized solution applicable to laterally loaded vertical piles of any configuration of stiffness throughout their length, embedded in foundation comprising any arrangement of layers of any type of soil. The soil behavior at any point along the length of the pile can vary from elastic through semi-elastic to plastic as a known function of the applied stress at that point. He took full recognizance of the behavior of soils having nonlinear p-y response curves in predicting the behavior of pile in such soils when subjected to lateral loads.

Horvath (1984) presented the theoretical development of the application of the simplified continuum approach to the laterally loaded pile problem, using the analysis procedure, suggested by Reissener (1958). He showed that solving such problem could be simplified if certain stress components (σ_y, σ_z, and τ_{yz}) were assumed to be equal to zero. In addition, all displacements were assumed to be equal to zero at some horizontal distance from the pile. He also demonstrated that there were difficulties in adapting this approach to handle nonlinear behavior, a Young's modulus that varies linearly with depth, and other practical considerations.

Amir (1985) analyzed the behavior of shear piles in rock by the spring model method, assuming an exponential relationship between sidewall shear and displacement. The resulting nonlinear differential equation, in terms of dimensionless force, may be solved by iterative finite-differences. The load settlement curves and axial force distribution obtained from this solution show good agreement with field measurements.

Budhu and Davies (1987) presented results of a numerical analysis of single laterally loaded piles embedded in cohesionless soils. The soil is modeled as an elastic material. They used the results of instrumented lateral load test carried by Cox et al. (1974), to compare between their results and the results obtained from the analysis of the test pile, carried by Reese et al. (1974). By modeling the laterally loaded pile as a beam element and the soil pressure as independent nonlinear springs (p-y method).

The test pile, 610 mm diameter steel pile with flexural rigidity 172 MN.m^2, was embedded 21 m in a deposit of medium dense to dense fine sand. Lateral load was applied at a height of 305 mm above ground level. The ground water level was kept above ground level during the tests. The properties of sand as reported by Budhu and Davies (1987) are: ϕ□= 39^o,γ = 10.5 kN/m^3 .The agreement between the results are quite good.

Bowels (1988) generalized a computer program to analyze laterally loaded piles using Winkler foundation approach and assumed the modulus of sub-grade reaction increases linearly with depth.

Verruijt and Kooijman (1989) presented a numerical model for a laterally loaded pile in a horizontally layered elastic continuum, and obtained a quasi-three-dimensional analysis. They combined the finite-element and finite-difference methods with a relatively simple and compacted method of analysis. A comparison between their solution and the solutions obtained by Poulos (1971) and the sub grade theory showed a good agreement for intermediate and large values of flexibility ratio. In general, the values of sub grade theory are somewhat larger than those obtained by Poulos; the agreement is good over the entire range of flexibility factors.

Theoretical approaches for determination of K_h

Many theoretical approaches were used to determine the values and variations of sub-grade reactions. Some of these studies are given in this section.

Palmer and Thompson (1948) suggested the following expression for the variation of K_h with depth:

$$K_h = \left(\frac{Z}{L}\right)^n K_L \tag{3}$$

where
K_h is the horizontal modulus of sub-grade reaction,
Z is any depth along the pile,
L is the pile embedded length,
K_L is the value of K_h at the pile base (Z = L) and
n is an empirical index equal to or greater than zero.
The most common assumptions are that (n = 0) for clay where the modulus is constant with depth and (n = 1) for granular soils where the modulus increases linearly with depth. For the case (n = 1), it is convenient to express the variation of K_h as:

$$K_h = \frac{Z}{B} n_h \tag{4}$$

where
B is the diameter or width of the pile, and
n_h is an empirical value ranging from (271.5 - 542.9) kN/m^3 for soft normally consolidated clay.

Glick (1948) proposed the following equation to find K_h :

$$K_h = \frac{22.4 E_s (1-\nu_s)}{(1+\nu_s)(3-4\nu_s)\left[2\ln\left(2L/B\right)-0.443\right]} \tag{5}$$

where:
E_s is the soil modulus of elasticity, and
ν_s is the soil Poisson's ratio.

Alizadeh and Davisson (1970) analyzed the results of the field tests on laterally loaded piles by means of the theoretical expression presented by Matlock and Reese (1960). This expression is based on the triangular distribution of horizontal subgrade modulus, K_h, with depth, in which:

$$K_h = n_h Z \tag{6}$$

For design purposes, n_h should be selected compatible with the anticipated deflections. Sogge (1981) proposed the following simple relationship to obtain a range of n_h values for shallow piles:

$$K_h = (2 \text{ to } 30)\frac{Z}{B} \quad \text{(in kcf unit (kcf = 159 kN/m}^3\text{))} \tag{7}$$

Bowles (1996) gave the most general form for either horizontal or vertical modulus of sub-grade reaction, which is:

$$K_s = A_s + B_s Z^n \tag{8}$$

Where
A_s is a constant for either horizontal or vertical members,
B_s is a coefficient for depth, and
n is an exponent to give K_s the best.
At the ground surface, A_s is zero for horizontal K_s, but at any small depth A_s will be greater than zero. For footing and mats, $A_s > 0$ and $B_s \approx 0$. This means that K_s is considered constant because the depth of influenced zone is small compared to piles.

THE COMPUTER PROGRAM

If the pile is not designed for buckling, then the main causes of tensile stresses in a pile section are the lateral loads and/or bending moments, that is, the reinforcement should be provided for all sections subjected to tensile stress. For this reason, a computer program (PLRN) is modified from that given in Bowles (1988) to check the depth through which the reinforcement will only be required to cover the tension zone of the pile.

(PLRN) program is coded in Fortran-77 language and based on Winkler foundation model where the pile is treated as beam element and the uniaxial soil resistance is represented by independent springs.

Problem description and modeling

The basic parameters that are used in this study are as follows: (expand the abbreviation)

For pile:

Moment, M = 0.1 × B × Q_a (kN.m)
Horizontal load, H = 0.1 × Q_a (kN)
Q_a = 100 kN
L= 25.0 m
B= 1.0 m (for bored piles)
B= 0.5 m (for driven piles)

For soil:

Unit weight of soil, γ = 15 kN/m^3
Angle of internal friction, ϕ = 30°
Cohesion, c_u = 0 kN/m^2

Bored piles are usually constructed with larger diameters compared to driven piles. Therefore, for a constant depth, the depth ratio in bored piles will be smaller than that in driven piles. This paper does not deal with the effect of construction of the pile on its behaviour, but when the pile is loaded laterally, its behaviour will depend on whether it has large diameter (bored) or small (driven).

ANALYSIS AND DISCUSSION

The effect of different parameters on the stress distribution and, hence, on the extension of reinforcement below the ground surface are thus explained.

Effect of pile type

Figure 2 presents a relationship between the minimum

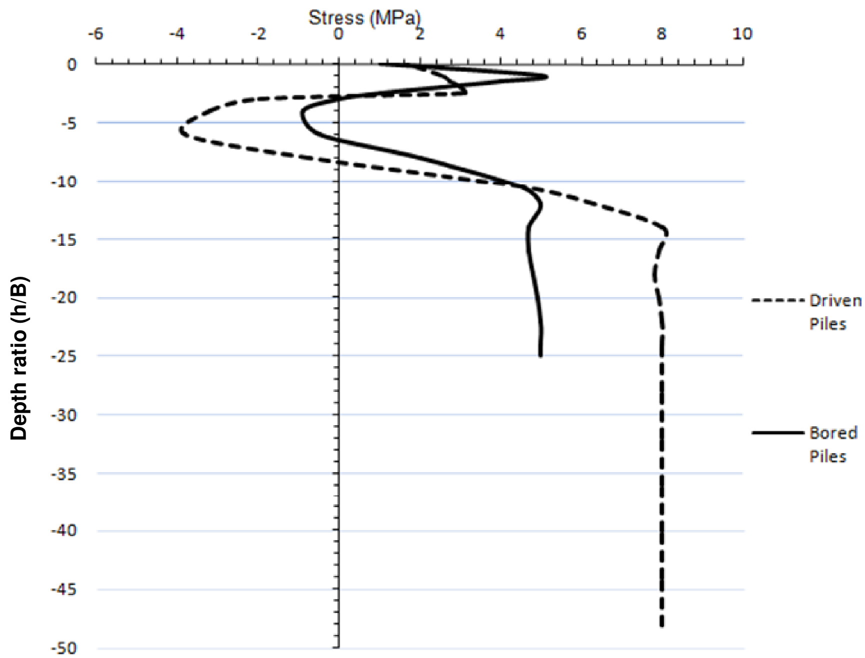

Figure 2. Stress distribution for driven and bored pile embedded in sand.

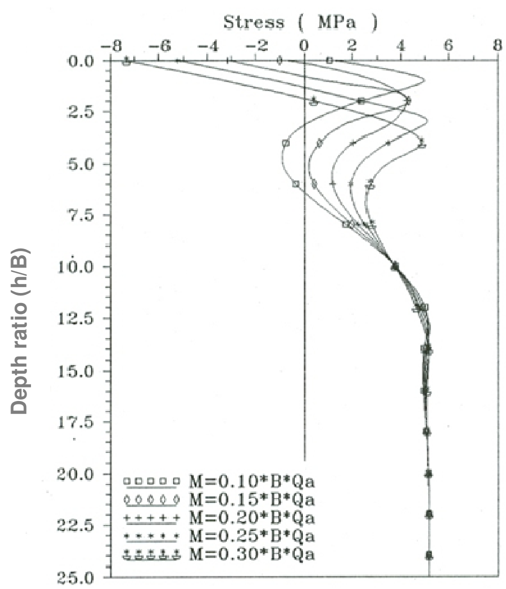

Figure 3. Effect of moment loading on the stress distribution along the shaft of bored pile in sand.

bending stress (tension or compression) within the pile sections under the general working load and the depth ratio for both bored and driven piles embedded in sand. The pile will no longer be subjected to tensile stress and, therefore, it will act as a compression member. The variation in the stress distribution along the pile shaft is due to the change in the distribution of bending moment along the shaft and the decrease in the allowable load with depth, h. The tensile stresses in bored piles are smaller than in driven piles because the diameters and hence the moments of inertia of the pile section are greater which lead to decrease in the stresses.

Bored piles

Effect of moment loading

Figure 3 shows the effect of moment loading on the

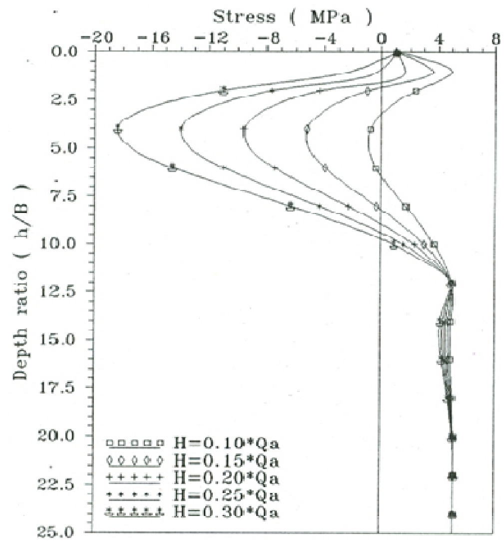

Figure 4. Effect of lateral loading on the stress distribution along the shaft of bored pile in sand.

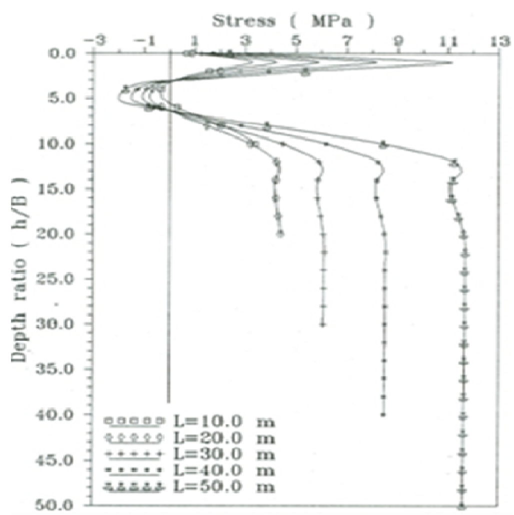

Figure 5. Effect of pile length on the stress distribution along the shaft of bored pile in sand.

stress distribution along the pile shaft, as the applied moment increases the tensile stress will increase at the pile top and decreases or vanishes as it goes down. It was found that the zero tensile stress occurs at a depth of about 6.5 diameters for 10% of the applied moment loading. As such, the steel reinforcement needs to be extended along this depth only.

Effect of lateral loading

Figure 4 shows the effect of lateral loads on the stress distribution along the pile shaft, as the applied lateral load increases the tensile stress increases to its maximum value at a depth ratio of 4 with 30% of the applied load, then decreases with depth to reach a constant value in the compression side. The constant value in the stress distribution curves is the same as for all values of the applied lateral loads. The maximum tensile stress, for all curves, located at about 4 diameters. The depth where the tensile stress equals to zero will increase as the lateral load increase. The effect of applied moment vanishes at a depth ratio of about 10 while this depth ratio is about 15 for lateral load.

Effect of pile length

Figure 5 represents the effect of pile length on the stress distribution along the pile shaft, as the pile length

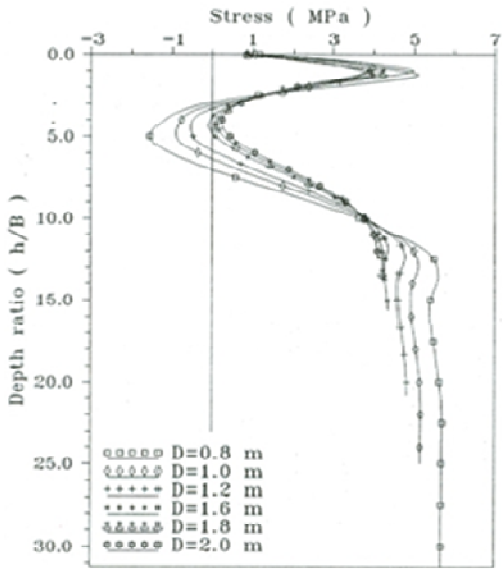

Figure 6. Effect of diameter on the stress distribution along the shaft of bored pile in sand.

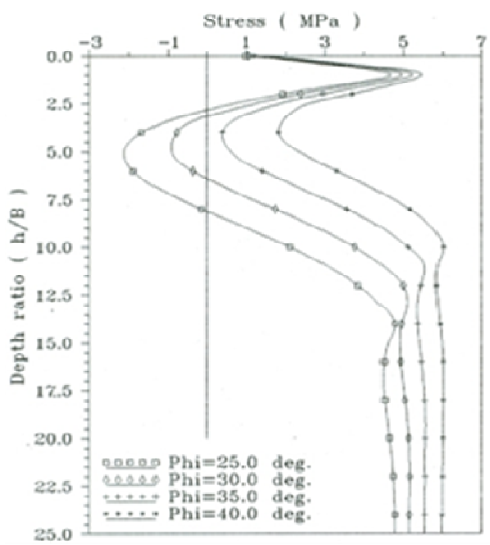

Figure 7. Effect of friction angle on the stress distribution along the shaft of bored pile in sand.

increases, the stress will increase too in both compression and tension sides, but will not affect the depth of zero tensile stress nor the location of its maximum value. The maximum tensile stress appears at approximately 5 diameters and it will be equal to zero at about 6 diameters.

Effect of pile diameter

Figure 6 shows the effect of pile diameter on the stress distribution along the pile shaft, the increase in pile diameter, increases the pile stiffness, and accordingly will decrease the value of the tensile stresses along the shaft. The maximum tensile stress will occur at a depth ranging between 4 to 5 diameters for a pile diameter of 2.0 to 0.8 m, respectively. The depth of zero tensile stress decreases as the pile diameter increases. But generally it does not exceed 7 diameters. When the pile diameter increases, the moment of inertia of its section will increase too, which causes reduction in stresses.

Effect of angle of internal friction

Figure 7 shows that the effect of angle of internal friction on the stress distribution for bored pile embedded in

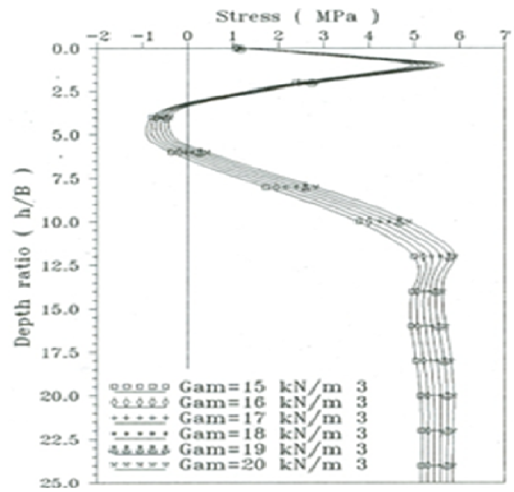

Figure 8. Effect of unit weight of soil on the stress distribution along the shaft of bored pile in sand.

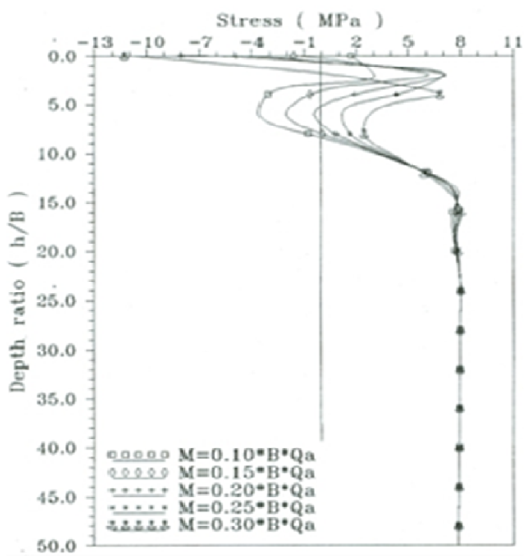

Figure 9. Effect of moment loading on the stress distribution along the shaft of driven pile in sand.

sand. The angle of internal friction has a significant effect on the stress distribution. As it increases in value, increasing soil stiffness, the bending moment will decrease and the tensile stress will not appear. For minimum value of the angle, 25°, the zero tensile stress appears at about 8 diameters with maximum tensile stress located at about 5 diameters.

Effect of soil unit weight

Figure 8 shows the effect of soil density on the stress distribution along the pile shaft embedded in sand. The behavior is somewhat similar to the effect of angle of internal friction, increasing in compression stresses, decreasing to reach a maximum tensile stress then increasing again to reach a constant compression value. The soil density has little effect on the stress distribution, at least in the upper portion. The maximum tensile stress located at approximately 5 diameters where it reaches zero at about 5.5 to 6.5 diameters.

Driven piles

Effect of moment loading

Figure 9 shows the effect of moment loading on the stress distribution along the shaft of driven pile embedded in sand. The behavior is similar to those of

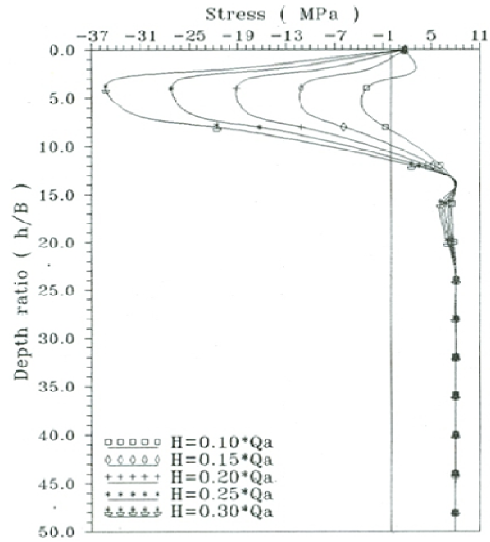

Figure 10. Effect of lateral loading on the stress distribution along the shaft of driven pile in sand.

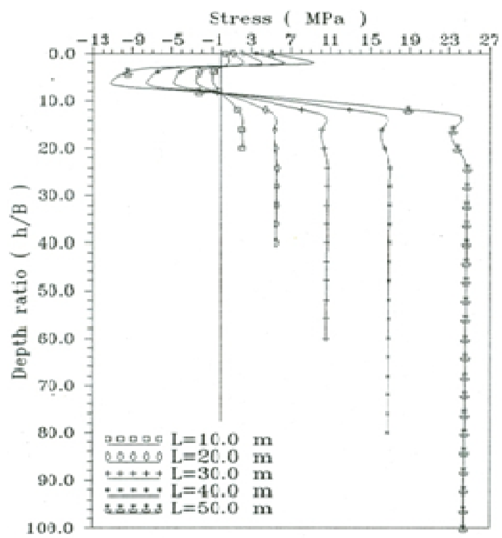

Figure 11. Effect of pile length on the stress distribution along the shaft of driven pile in sand.

bored pile in sand, but with greater values of stress in both compression and tension and less depth for maximum and zero tensile stresses. Generally, the depth of zero tensile stress is located at about 8 diameters for the moment of 10% of the applied load.

Effect of lateral loading

Figure 10 shows the effect of lateral loading on the stress distribution along the pile shaft. The behavior is similar to that of bored piles but with greater values of stress and less depth. In general, the maximum tensile stress appears at approximately 4 to 6 diameters for a range of lateral load from 30 to 10% of the applied load, respectively. The depth of zero stress increases with increasing lateral load. Values ranging between 8 and 11 diameters for lateral loading caused by 10 to 30% of the applied load respectively.

Effect of pile length

Figure 11 shows the effect of pile length on the stress distribution along the shaft of driven pile in sand. It is clearly seen that the stress increases as the pile length

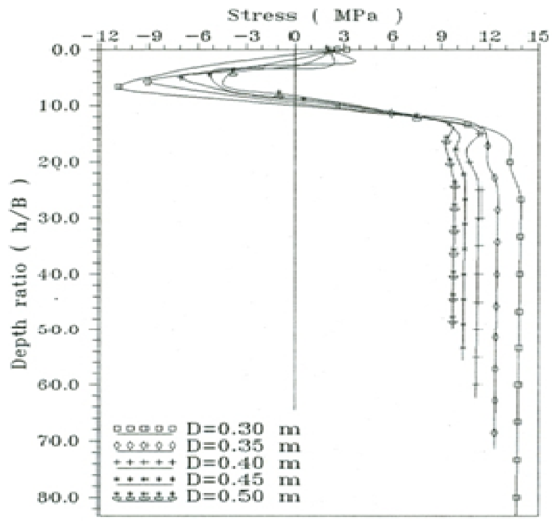

Figure 12. Effect of diameter on the stress distribution along the shaft of driven pile in sand.

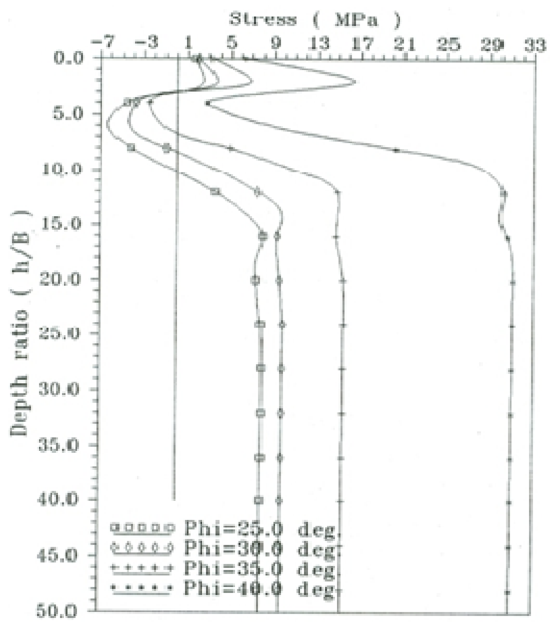

Figure 13. Effect of friction angle on the stress distribution along the shaft of driven pile in sand.

increases, but the pile length has no effect on the location of the maximum tensile stress or on the depth of its zero value, similar to bored piles the depth of maximum tensile stress is at approximately 6 and 8 diameters for the depth of zero tensile stress.

Effect of pile diameter

Figure 12 represents the effect of pile diameter on the stress distribution of driven pile embedded in sand. The stresses decrease as the diameter increases for both tension and compression, due to increasing pile stiffness. Subsequently the depth of zero tensile stress is at about 6 diameters where the depth of its zero value varies between 8 to 10 diameters for a diameter of 0.5 to 0.3 m, respectively.

Effect of angle of internal friction

Figure 13 shows the effect of the angle of internal friction on the stress distribution for driven pile in sand. The value of the tensile stresses will decrease as the soil

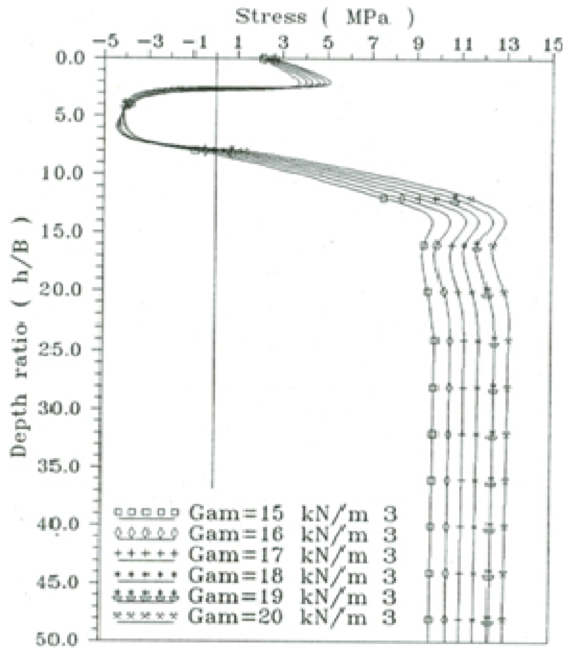

Figure 14. Effect of unit weight of soil on the stress distribution along the shaft of driven pile in sand.

stiffness increases, based on the increase in the angle of internal friction, similar to bored piles. The location of maximum tensile stress will increase as the soil stiffness decreases and it ranges between 4 to 6 diameters. The same thing is also true for the depth of zero tensile stress, which is located at about 7 to 10 diameters for $\phi =$ 35 and 25° respectively.

Effect of soil unit weight

Figure 14 represents the effect of soil density on the stress distribution along the shaft of driven pile embedded in sand. Similar to bored piles, the soil density has little effect on the stress distribution. The maximum tensile stress is located at about 5 diameters while the zero tensile stress is at approximately 8 diameters for different soil density ranging from 15 to 20 kN/m^3.

Conclusions

A beam-on-elastic foundation model was used to analyze a loaded pile in order to investigate its need and necessity for reinforcement. This model is performed using the finite element method as a numerical tool for the analysis. The pile is discretized into a number of elements while the soil is represented by a number of springs. The stiffness of these springs is considered to be variable with depth.

Based on the results obtained, the following conclusions can be drawn:

1. For cast-*in-situ* bored or driven piles, the codes did not recommend a specific depth for the reinforcing bars that should be provided to resist the tensile stresses. This issue is left to the designer.
2. Bored piles embedded in sand must be provided with reinforcing bars extending to a depth of not less than 0.4 times the pile length. While for driven piles this length may be reduced to 0.2 times the pile length, approximately.
3. For bored piles in sand, the pile will not be subjected to tensile stresses below an approximate depth ratio of 10; accordingly reinforcement is not needed below this depth.
4. For bored piles, the depth of zero stress in sand is greater for small values of friction angle and this depth will be about 8 diameters.
5. Driven piles in sand need a depth of reinforcement to be extended to approximately 8 diameters to resist the tensile stresses, while it does not need any reinforcement at a depth of about 11 diameters because the zero moment will start at that depth.

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