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A Critical Review of Some Important Aspects of the Indian Practice of Geotechnical Design of Bored Piles

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Abstract. This paper presents a critical review of some important aspects of the geotechnical design of bored piles with a view to evaluate the relevant provisions of Indian codes of practice in the context of international best practices. The following aspects are reviewed – critical depth concept of limiting skin and end bearing resistance, coefficient of earth pressure, minimum embedment into bearing stratum, and reduction in end bearing resistance due to the presence of weak layers below the pile tip. It is shown that arguments against the validity of critical depth concepts seems to have gained more acceptance and the international practice is moving away from the concept of limiting the skin and end bearing resistance on account of critical depth. Values of coefficient of earth pressure for bored piles recommended by Indian codes appear to be appreciably higher than those used internationally. The minimum embedment of two times the pile diameter into the bearing stratum for piles installed through weak strata and terminated in a competent bearing stratum recommended by IS 2911 looks reasonable. The presence of a weak stratum below the pile tip could have an effect on the end bearing resistance of piles. The limitations of the Indian codes with respect to this issue are highlighted and recommendations from various sources are discussed. The implications of the above aspects for design are discussed and suggestions for updating the Indian codes of practice are presented.

Keywords: bored piles, design, Indian practice.

1 Introduction

The most commonly used type of deep foundation in India is bored and cast in-situ concrete piles. The geotechnical design of bored piles in India is carried out in accordance with various codes like IS 2911 (Part 1/sec2) [1], IS 14593 [2], IRC 78 [3] and Indian Railways Bridge Substructure and Foundation Code [4]. The objective of this paper is to critically review some important aspects of the geotechnical design of bored piles as stipulated by the Indian codes and to compare with international practices. The following aspects are examined in this paper:

- Critical depth beyond which unit skin friction and unit end bearing resistance remains more or less constant with depth

- The coefficient of earth pressure for calculation of skin friction resistance
- Minimum depth of embedment for piles passing through a weak stratum and terminated in a competent bearing stratum
- Effect of weak stratum below pile tip on end bearing resistance

2 Critical Depth

2.1 Background

The unit skin friction resistance and the unit end bearing resistance at any depth depend on the vertical effective stress. Since vertical effective stress increases linearly with depth, one would expect that unit skin friction and unit end bearing resistance should also increase linearly with depth. However, experimental research by Kerisel [5] and Vesic [6,7] suggested that the unit skin friction and unit end bearing resistance increased linearly up to a critical depth and beyond this critical depth skin friction and end bearing resistance remained more or less constant. This idea was accepted by most engineers and several leading text books and codes and design guidelines endorsed the concept of critical depth. Several researchers like Kulhawy [8], Randolph [9] and Fellenius [10] have challenged the concept of critical depth, saying this is a myth or fallacy arising from problems in interpretation of experimental data. A review of the relevant literature suggests that design practice is moving away from the concept of critical depth and limiting skin and end bearing resistance.

2.2 Indian Codes of Practice and Critical Depth

IS 2911 (Part 1/sec 2) Annex B clause B-1 for piles in granular soils [1] stipulates that *in working out pile capacity by static formula, the maximum effective overburden at the pile tip should correspond to the critical depth, which may be taken as 15 times the diameter of the pile shaft for $\phi \leq 30°$ and increasing to 20 times for $\phi \geq 40°$*. The IRS Bridge Substructure and foundations code [4] recommends that design is to be carried out in accordance with IS 2911 and hence the above recommendations will be applicable for pile foundations for railway bridges.

The design of foundations for road bridges is carried out in accordance with IRC 78 [3]. In clause 1.1 of Annex 5 of IRC:78, which gives the procedure for calculation of end bearing resistance, it is stated that *effective overburden pressure at pile tip limited to 20 times diameter of pile for piles having length equal to more than 20 times diameter*. However in clause 2 of Annex 5, which describes the method for calculating skin friction, no mention is made about critical depth.

2.3 Arguments for Critical Depth

The concept of a limiting value of unit end bearing resistance and unit skin friction for piles in sand was first proposed by Kerisel [5]. Experimental studies carried out by Vesic [6, 7] played a major role in getting wide acceptance for the concept of critical depth and limiting values of end bearing and skin friction. Vesic [6,7] carried out a series of laboratory and field tests on instrumented piles with diameter ranging from

50 mm to 450 mm and with a range of length-diameter ratios installed in dry, moist and saturated using different installation techniques – buried, jacked-in and driven. End bearing resistance and skin friction were measured separately. He found that for shallow depth of penetration there was a liner increase in both unit skin friction and unit end bearing resistance. However at greater depths, both unit skin friction and unit end bearing resistance became more or less constant with depth. The maximum value of resistance appeared to depend on relative density of sand and the method of installation of piles. The depth up to which a linear increase observed in unit resistance was in the range of 10 to 20 times the pile diameter and appeared to depend on the relative density of sand and the method of installation of pile.

Vesic [6,7] suggested that the explanation for the observed phenomena is that the vertical stress in the vicinity of the pile is different from the initial overburden pressure. He postulated that both unit skin friction and unit end bearing resistance are linear functions of the vertical stress in the vicinity of the pile at failure, which is not necessarily equal to nor proportional to the overburden pressure. Vesic [6,7] put forward the notion that if the unit skin friction and unit end bearing resistance attained a constant value at greater depth, it is because the vertical stress at failure also becomes constant at greater depth. The departure of the vertical stress around and below the pile from the initial overburden stress was attributed by Vesic [6,7] to arching. When the pile is loaded, the sand below the tip and around the pile is compressed downward. If the depth of embedment of the pile is sufficiently large, and if the base displacement is sufficiently small, there is a distance above the pile tip above which the effect of the downward movement of the soil is not significant. Above this level, the vertical stress at failure in the vicinity of the pile will be equal to the initial overburden stress and the unit skin friction and unit end bearing resistance will increase linearly with depth up to this level.

Tavenas [11] conducted load tests on instrumented steel H piles and precast concrete piles driven into a thick homogenous layer of medium dense sand. The piles were driven and tested incrementally to depths of 6, 9, 12, 15, 18 and 21 m. The results showed that for both H-piles and the precast concrete piles, the capacity was not a liner function of the embedded depth in sand. In particular, the precast concrete pile, a critical depth was observed to be 23 times the width of the pile below which both unit skin friction and end bearing resistance remained perfectly constant.

Hanna and Tan [12] presented results of tests on small scale model piles which showed that the unit skin friction and end bearing resistance increased linearly with depth up to length-diameter ratios of 30 – 40 and thereafter the resistance remained virtually constant until length-diameter ratio of 112 corresponding to the maximum length of pile tested. Meyerhof [13] in his Terzaghi lecture stated that large scale experiments and field observations showed that point resistance and average skin friction of a pile in homogenous sand will increase with depth up to a certain critical depth only and below this critical depth both end bearing resistance and average skin friction remains practically constant owing to the effects of soil compressibility, arching, crushing and other factors.

Textbooks like Pile Foundation Analysis and Design by Poulos and Davis [14], Pile Foundations in Engineering Practice by Prakash and Sharma [15], Geotechnical

Engineering Handbook edited by Smoltczyk [16] and design manuals by Naval Facilities Engineering Command [17] and US Corps of Engineers [18] recommended the concept of critical depth and limiting unit end bearing and skin resistance for the design of pile foundations. Canadian Foundation Engineering Manual [19] states that both unit skin friction and unit end bearing resistance may continue to increase with depth, but at a decreasing rate and recommended that for practical design purposes it is advisable to adopt limiting values for long piles in cohesionless soils.

Vesic [6], Poulos and Davis [14] and Prakash & Sharma [15] suggest that where a sand layer is overlain by a clay layer, the critical depth is to be measured from the top of the sand layer. This is not explicitly mentioned in any of the Indian codes of practice. Some coastal regions in India like Kochi have thick deposits of marine clays underlain by sands and it is a common practice to support structures on end bearing piles embedded into the sand layer. In such cases, if the critical depth is measured from the ground surface, instead of from the top of the sand layer, the calculated end bearing and skin friction in the sand layer would be too low.

2.4 Arguments Against Critical Depth

Kulhawy [8] argued that the concept of critical depth below which the end bearing and skin resistance are constant is not correct. He attributed the trend of unit skin friction increasing linearly up to a certain depth and then remaining more or less constant with depth observed in studies to the effects of over consolidation. If the soil is normally consolidated the unit skin friction will increase linearly with depth. However, if the sand is over-consolidated, the behavior changes appreciably. Kulhawy [8] suggested that some overconsolidation is the rule, rather than exception in most soil deposits. Using a hypothetical example of a sand stratum whose upper part is overconsolidated and lower part is normally consolidated, Kulhawy [8] demonstrated how the variation of the coefficient of earth pressure at rest (K_0) will result in unit skin friction will remain more or less constant for certain range of depth. Based on this demonstration, Kulhawy [8] argued that the apparent limiting value of unit skin friction is only a coincidence due to the product of β ($\beta = K \tan\delta$, where K is the coefficient of earth pressure and δ is the pile-soil interface friction angle) and z (depth) nullifying each other over a limited range of depth.

Kulhawy [8] also showed based on Vesic's theory of the bearing capacity of deep foundations [20] that with increasing depth, the unit end bearing resistance increases at a decreasing rate. This was attributed primarily to the effect of decreasing rigidity with depth and partly to the reduction in the angle of shearing resistance with increasing confining stress. However, Kulhawy [8] suggested that there is no critical depth beyond which the unit end bearing resistance is constant.

Kraft [21] presented a detailed critical review of the concept of critical depth and limiting skin and end bearing resistance of piles. He commented that an explanation to dismiss the existence of limiting values as proposed by Kulhawy [8] is easy to develop, but such an argument does not eliminate the experimental evidence that gave rise to the creation of limiting values in the first place. Kraft [21] suggested that while arching can explain a trend of resistance increasing with depth at a decreasing rate, it does not necessarily follow that a limiting unit resistance is reached. He listed factors which could result in a trend of unit resistance increasing with depth at a decreasing rate – angle of shearing resistance of soil and pile-soil interface friction angle may decrease with depth due to an increase in effective overburden stress; soils may exhibit more contractive behavior with depth as a result of increasing confining stresses; because of longer duration of pile driving there may be larger reduction in lateral stresses. Kraft [21] discussed several *artificial reasons*, which could influence the interpretation of pile load test data leading to an apparent limiting value of unit skin friction and end bearing resistance. These data interpretation problems include residual stresses, toe effects, failure criterion adopted for pile capacity, variability in soil stratigraphy and density of pile instrumentation, boundary effects in model tests and scale selected for plotting of test data. Kraft [21] concluded that no data from full-scale field load tests could be found which provided conclusive evidence for the existence limiting values. Although model tests show evidence of limiting shaft and tip resistance, it is likely that the data are being influenced by the boundary effects of the test chambers and limiting values do not exist in general. However, the rate of increase of resistance, especially end bearing resistance, decreases with an increase in overburden stress in homogeneous sands.

Randolph [9] observed that the basis for the concept of critical depth is empirical and quantitative analytical justification for limiting values of skin friction and end bearing resistance were lacking. He suggested that two factors may contribute to end bearing resistance increasing at a decreasing rate with depth – reduction in angle of shearing resistance with increasing stress level and a reduction in rigidity index, since shear stiffness of soil increases with stress level at a slower rate than frictional strength. He also suggested that the so-called limiting value of skin friction below a certain depth, is not a true limit on skin friction, but due to the degradation of shaft friction at shallower depths as the pile was advanced.

Fellenius and Altaee [10] stated that *the critical depth is a fallacy which originates in the failure to interpret the results of full and model-scale pile tests properly*. In the case of load tests on instrumented piles, the instruments would probably register only the loads applied to the pile during the test and disregard any loads present in the pile before the test. However, residual loads are induced in both driven and bored piles during and after installation through several phenomena – wave action during driving, soil quakes along the pile and reconsolidation of the soil after the installation disturbance. Residual loads are present in all piles even before measurements are taken and analysis performed and their effect is commonly overlooked. In the interpretation of the results of load tests on full-scale instrumented piles, if the residual loads are not accounted for in the analysis, the measured load distribution may show a trend of resistance linearly increasing up to a depth and remaining constant below that depth. In the case of small scale model tests, the residual loads may not be significant. However, because of the very low stress levels, the sand will exhibit a dilatant response resulting in significantly higher values of lateral pressure at shallow depths, and the results may show a distribution of skin friction which first increases and then remains constant, which could be mistakenly interpreted as the existence of a critical depth.

Poulos et al. [22] expressed the view that limiting values of unit shaft friction and end bearing resistance probably do not exist although the rate of increase with depth is not linear. However it remains convenient from a design view point into impose an upper limit for shaft and base resistance and it is unlikely that this common practice would be discarded easily. Fleming et al. [23] stated that *modern approaches to pile design have generally moved away from limiting values of end bearing pressure, but accept that in a uniform sand deposit there will be a gradually decreasing gradient of design end bearing pressure with depth*. Tomlinson and Woodward [24] state that for long piles the assumption of a constant unit end bearing resistance below a depth of 10 to 20 diameters has been shown to be over-conservative. Many widely used methods for pile design like FHWA [25], API [26] and ICP [27] methods do not recommend use of a critical depth. However, it is accepted that unit skin friction and end bearing resistance does not necessarily increase linearly with depth and limiting values of skin and end bearing resistance may be specified.

2.5 Discussion

From the review of literature it is evident that the concept of critical depth is no longer considered to be credible and the practice of limiting the value of effective vertical overburden stress for calculation of unit skin friction and end bearing resistance to its value at a depth of 10-20 times the pile diameter is unduly conservative. However, unit skin friction and end bearing resistance may not continue to increase linearly with depth for large depths and it is prudent to restrict the maximum design values to appropriate limits based on relevant experience.

3 Coefficient of Earth Pressure

3.1 Theoretical Background

A widely used method for the calculation of skin friction resistance of piles in sand is using an expression of the form equation (1).

$$Q_s = \sum_{i=1}^{n} K_i \ \sigma_{vi} \ tan\delta_i \ A_{si} \tag{1}$$

where, Q_s is the total skin friction resistance, K_i is the coefficient of earth pressure for the ith layer, σ_v is the vertical stress at the center of the ith layer, δ_i is the pile-soil interface angle for i$_{th}$ layer and A_{si} is the surface area of the pile in ith layer. The coefficient of earth pressure depends on number of factors including the relative density and stress history of the soil, geometrical and material characteristics of the pile, method of installation [24].

Prior to the installation of the pile, K will be equal to K_0, the coefficient of earth pressure at rest, which is largely a function of the relative density and stress history of the soil. The original in-situ state of stress will be altered during the installation of a bored pile depending on a number of factors – use of temporary casing or permanent liners, use of mineral or polymer slurries, method of concreting and quality of construction practices. When installation of bored piles is carried out using casings or liners which are advanced ahead of boring, the initial at-rest earth pressure conditions may not be altered significantly. However, in case a borehole stabilized by drilling fluids, the K value will decrease. But, when wet concrete is placed, because of the fluid pressure exerted by the fresh concrete, the K value may become equal to or even exceed K_0. When the pile is loaded, shear stresses are induced in soils adjacent to the pile shaft. If relative density of the soil and the stress conditions are such that the soil tends to dilate, which is being restrained by the pile and surrounding soil mass, the lateral stresses on the pile will increase. Hence, the lateral stresses acting on the pile shaft at failure depend on a large number of factors and hence it is very difficult to determine K accurately.

3.2 K values from Literature

The K values for bored piles recommended by various references are summarized in table 1.

Table 1. Coefficient of earth pressure for bored and cast in-situ concrete piles

Reference	K
IS 2911 (Part 1/sec 2) [1]	1.0 – 1.5
IRC:78 [3]	1.5 – 1.8
BS 8004 [28]	0.7
AASHTO [29] / FHWA [25]	K_0
Canadian Foundation Engineering Manual [19]	K_0
Bangladesh National Building Code [30]	
Dry construction – good workmanship	K_0
Slurry construction – good workmanship	K_0
Slurry construction – poor workmanship	(2/3) K_0
Casing under water	(5/6) K_0
Tomlinson and Woodward [24]	0.70 K_0 to K_0
Fleming et al. [23]	0.7
Prakash and Sharma [15]	0.5
Viggiani [31]	
Loose sand	0.5
Dense sand	0.4
Kulhawy [8]	(2/3) K_0 to K_0

From Table 1 it may be seen that the values recommended by IS 2911 and IRC: 78 are appreciably higher than the values recommended by other references.

4 Minimum Embedment into Bearing Stratum and Effect of Weak Layer below Pile Tip

4.1 Theoretical Background

When a pile passes through relatively weak strata and terminated in a competent bearing stratum, it is generally considered that some embedment into the bearing stratum is required to mobilize the full end bearing resistance which can be offered by the bearing stratum. Similarly if there is a weaker stratum below and close to the pile toe, the end bearing resistance will be reduced. Both effects are related to the shape of the failure surface for the pile toe. The shape of the failure surface assumed in different theories of ultimate bearing capacity of piles is shown in figure 1 [31].

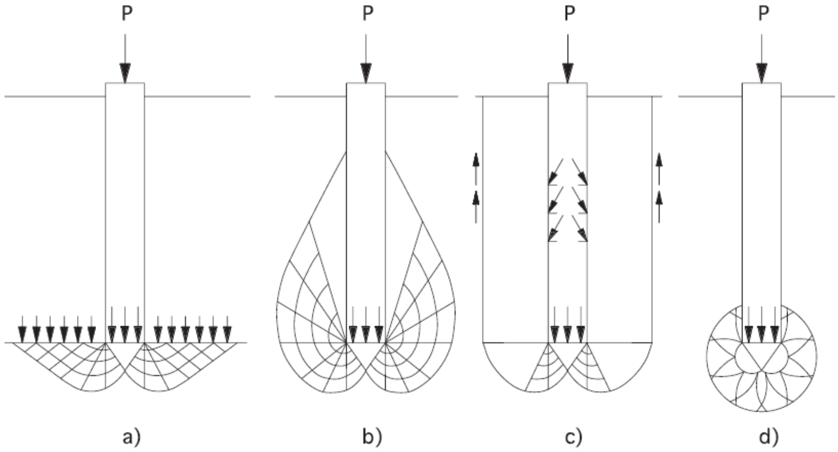

Fig. 1. Failure surfaces under deep foundations, after [31]

Here a) is considered by Prandtl and Caquot, b) by De Beer and Meyerhof, c) by Berezantsev and d) by Bishop, Skempton and Vesic. It may be seen that for most theories, the influence zone is in the vicinity of the pile toe, except in the case of theories of De Beer and Meyerhof, wherein the failure surface extends to a large distance above the pile tip. In this context, it is also interesting to note the comment by Fellenius [32] that the concept of bearing capacity does not appear to a pile toe and instead the load-settlement relationship is a function of the compressibility of the soil below pile toe and effective overburden stress.

4.2 Recommendations on Minimum Embedment into Bearing Stratum and Effect of Weak Layer below Pile Toe

The recommendations on minimum embedment of pile into a bearing stratum and effect of weak layer below pile toe extracted from various sources are summarized in table 2.

Table 2. Recommendations on minimum embedment into a bearing stratum and effect of weak layer below pile toe on the end bearing resistance of piles

Reference	Recommendation
IS 2911 (Part 1/sec 2) [1]	For piles passing through cohesive strata and terminating in a granular stratum, a penetration of at least twice the diameter of the pile shaft should be given into the granular stratum
BS 8004 [28]	The design of the pile foundation should consider the influence of different soil/rock layers below the pile toe.
AASHTO [29]	When a shaft is tipped in a strong soil layer overlying a weak layer, the base resistance shall be reduced if the shaft base is within a distance of 1.5 times the shaft diameter of the top of the weak layer
Eurocode 7 [33]	The strength of a zone of ground above and below the pile base should be taken into account when calculating the pile base resistance. This zone may extend several diameters above and below the pile base. Any weak ground in this zone has a relatively large influence on the base resistance. Punching failure should be considered if weak ground is present at a depth of less than four times the base diameter below the base of the pile.
AS 2159 [37]	Where a pile is funded on a stratum that overlies a softer or weaker stratum, allowance shall be made for the possible reduction of end bearing resistance due to the presence of weaker or softer stratum.
Canadian Foundation Engineering Manual [19]	Where a weak soil layer overlies a dense sand layer, the full toe capacity is not mobilized until the pile penetrates six diameters into the dense sand. Where a weak layer underlies a dense sand layer, toe capacity would be affected if the pile toe is less than three times the pile toe diameter above the weak layer.
API [26]	In the case of piles embedded in a layer with adjacent weak layers, reduction in end bearing resistance need not be considered if the pile achieves penetration of two to three times into the layer and the tip is approximately three times the diameter above the bottom of the layer to preclude punch through. Where these distances are not achieved, some modification in the end bearing resistance may be necessary.
German Geotechnical Society [34]	The zone governing base resistance is from one diameter above and four diameters below the pile base for pile diameters up to 600 mm and from one diameter above and three diameters below the pile base for pile diameters greater than 600 mm.
Salgado [35]	Pile has to penetrate a bearing stratum by at least two diameters if that layer's bearing resistance is to be fully developed.

Coduto [36]	Soil between the depth of about one diameter above the toe and about three diameters below the toe has the most influence on the toe bearing capacity.
Meyerhof [13]	In the case of a bearing stratum overlain by a weaker stratum full toe resistance is mobilized if the penetration into the bearing stratum is at least 10 times the pile diameter. Similarly in the case of a bearing stratum underlain by a weak stratum, toe resistance will be reduced if the pile tip is within a distance of 10 times the pile diameter from the top of the weak stratum.

4.3 Discussion

From table 2 it may be seen that most references suggest that the influence zone for end bearing resistance extends to a distance of one to three pile diameters above the pile toe and three to four diameters below the pile toe. An exception is the recommendation of Meyerhof [13], where the influence zone is considered to extend to a distance of 10 times the diameter above and below the pile toe. This could possibly due to the shape of the failure surface considered by Meyerhof. The recommendation of IS 2911 for a minimum embedment of two times the pile diameter into a bearing stratum is generally satisfactory. However, Indian codes do not have a provision to account for possible reduction in end bearing capacity due to the presence of weak layer below pile toe. It is advisable to include a recommendation for appropriate reduction in end bearing capacity if there is a weak layer within three diameters below the pile toe.

5 Conclusions

A review of literature concerning some important aspects of the design of bored piles was carried out and the following conclusions may be drawn:

1. The concept of critical depth is no longer considered to be credible and the practice of limiting the value of effective vertical overburden for calculation of unit skin friction and end bearing resistance to its value at a depth of 10-20 times the pile diameter is unduly conservative. However, unit skin friction and end bearing resistance may not continue to increase linearly with depth for large depths and it is prudent to restrict the maximum design values to appropriate limits based on relevant experience.
2. The coefficient of earth pressure recommended by Indian codes seems to be appreciably higher than the values used internationally. A suitable revision of the Indian codes appears to be desirable.
3. The recommendation of IS 2911 with respect to the minimum embedment of pile into a bearing stratum seems reasonable.
4. Indian codes do not have a provision to account for the influence of a weak layer below the pile toe on the end bearing resistance. It is advisable to include a recommendation to suitably reduce the end bearing resistance when

there is a weak layer within a distance of three times diameter below the pile toe.

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