Geomechanics of Soft Ground Improvement by Perforated Piles: Review and Case Study

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Abstract: - Civil Infrastructure built on soft and compressible soil is likely to collapse due to undrained shear failure or unacceptable settlement of supporting foundations. Incorporation of adequate ground improvement technique with the aim of upgrading the strength and stiffness of the weak soil is essential in such cases. Amongst various established methods adopted worldwide for improving soft ground, using perforated piles is a relatively emerging technique. Such piles not only transmit the structural load into the subsoil beneath in a manner similar to the conventional piles, but also assist in radial consolidation of soft soil due to perforated side walls. This paper presents a brief overview on the investigations carried out on this new technique. Also, a typical case study has been presented. As observed, the axial pile capacity progressively increased while settlement reduction took place, with accelerated radial consolidation.

Key-Words: - Axial capacity, Ground settlement, Radial consolidation, Skin friction, Soft ground, Stress concentration ratio

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1 Introduction

Many parts of the world including alluvial plains and coastal regions consists of soft compressible soils with average shear strength seldom exceeding 20-30 kPa. Infrastructures build on such soft ground are likely to fail unless the foundation soil is significantly improved. In particular, foundations on soft clay deposits can cause excessive settlement initiating undrained failure of infrastructure if proper ground improvement is not carried out [1], [2].

Reducing long-term settlement of infrastructure and providing cost-effective foundations with sufficient load-bearing capacities are national priorities for infrastructure development in most countries. Much modern infrastructure is constructed over poor quality ground and is subject to greater static and dynamic loading than previously experienced. Appropriate ground improvement techniques can be adopted to prevent unacceptable excessive and differential settlement and increase the bearing capacity of the foundations at much lower cost. Over several decades, different ground improvement techniques have been developed, which include stone columns, preloading with prefabricated vertical drains, piling, geogrids and chemical stabilization [3], [4], [5].

Among various methods of soft soil improvement, installation of prefabricated vertical drains (PVDs) or stone columns (SCs) with preloading is one of the well-established and effective techniques practiced worldwide [6]. The method involves acceleration in the soft soil consolidation by shortening of the drainage path via radial consolidation. Piles without reinforcement, known as concrete injected columns [7], and installation of strong piles are other methods of soft soil improvement [8]. In case of vertical drains with preloading, the consolidation takes place for several years, if not vacuum assisted which is costly, and simultaneously PVDs do not possess additional stiffness to withstand the majority of imposed loading from superstructure [9]. In case of SCs, the time of consolidation is much reduced due to higher hydraulic conductivity of stone aggregates, and simultaneously the column-soil relative stiffness provides additional bearing capacity to the reinforced soft clay [10]. Through the pile foundation, the structural load is transmitted to the stiffer soil layers underlying the soft soil deposits. In absence of any radial consolidation, the soft soil is not improved [11]. In case of chemical stabilization, the admixtures undergo chemical reaction with the soft clay particles and thus the strength and stiffness of soil is improved [5], [12]. The relative merits and demerits of different methods are summarized in Table 1.

The piles transmit the structural load to the deeper soil stratum. Basically, the load transmission takes place by means of frictional resistance between the pile surface and soil. Such piles are termed as 'frictional pile' or 'floating pile'. In the cases where the bases of the piles are embedded into stiffer soil layer or rock, significant base resistance is offered to the pile compared to the friction. Such piles are termed as 'end bearing pile'. This is illustrated in Fig.1 below.

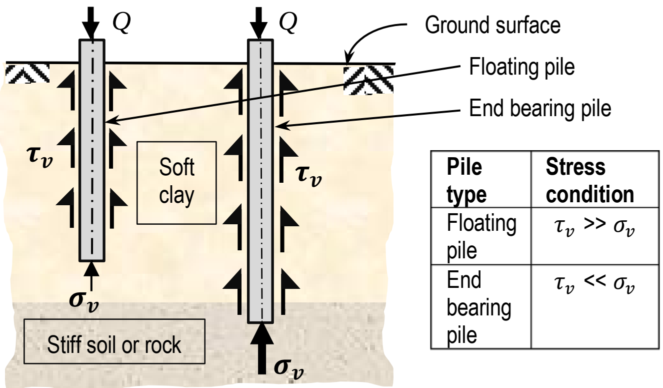


Fig 1. Load transfer mechanism of pile

Table 1. Comparison of various ground improvement techniques

Technique	Advantage	Disadvantage
PVD with preloading	Accelerated radial consolidation; convenient and cost effective; suitable for deep soft clay deposit.	Speed of consolidation is lesser than sand drains and SC; no relative stiffness; biodegradation of PDV material.
Sand drains	Faster radial consolidation than PVD; suitable for deep soft clay deposit; moderate relative stiffness.	More costly than PVD; clogging affects drainage; constructional difficulty may take place.
Stone columns	Faster radial consolidation than sand drains; high relative stiffness.	More costly than sand drains; higher clogging rate than sand drains; requires specialized installation techniques.
Piles	Significantly high load bearing capacity; very high relative stiffness; suitable for deep soft clay deposit.	No radial consolidation; high installation cost; not environmentally friendly.
Chemical stabilization	Effective rise in strength, durability and workability of soil; Convenient	Permeability of virgin soil deposit may be affected; unsuitable for deep clay deposit.

It is expected that the pile foundation can also assist in radial consolidation in case the side wall is permeable. Such pile may be termed as perforated piles. Limited research carried out on perforated piles indicate that they are expected to assist in accelerated soft soil consolidation as well as carry the foundation load as conventional piles. This paper presents a brief overview on the existing knowledge on the behavior of perforated pile and a typical case study to illustrate its load transfer and soft ground improvement characteristics.

It is observed that conventional piles, having possessed significant relative pile-soil stiffness, transmit the imposed load to the subsoil beneath through skin friction and end bearing, but they do not assist in soft soil improvement by consolidation. The PVDs and stone columns, although assist in consolidation, have limited relative stiffness, hence not effective in load-transfer mechanism. Perforated piles serve both the purpose. Although past contributions investigated the consolidation characteristics of perforated piles, studies conducted on capacity and settlement analysis of perforated piles are rather limited. The current paper aims to carry out a preliminary study to bridge up the knowledge gap.

Available information on perforated pile performance is quite limited. The primary aim of this paper is to carry out a literature review and to provide preliminary analytical formulations pertaining to load bearing capacity, consolidation characteristics and settlement analysis of soft ground improvement by perforated piles.

2 Literature Review

The perforated piles, unlike conventional piles, assist in radial consolidation, apart from transmitting the superstructure loads to the subsoil below. The concept of perforated piles was initially proposed by Mei et al. [13] and later, its radial consolidation and load transfer characteristics were studied through theoretical and laboratory studies by various researchers [14], [15], [16].

Ni *et al.* [14] carried out laboratory model tests on concrete permeable piles with drainage hole on its circumference. A series of uniaxial and flexure tests were conducted with permeable concrete pile and the results were compared with equivalent conventional piles. The crack pattern, deflection profile and the induced strains were investigated. As observed, although the permeable piles assist in accelerated radial consolidation, the axial compressive strength of such piles have been decreased to some extent because of reduced cross sectional area due to the drainage holes. The flexural strength, on the other hand, was found to increase due to redistribution of the bending stresses.

Ni *et al.* [15] conducted a numerical study based on 3-dimensional finite and infinite element approaches. The soil displacement and excess pore water pressure dissipation were studied through consolidation analysis of perforated driven piles. The numerical results were compared with existing analytical solutions and available field data. It was observed that an optimum radial consolidation was achieved for the drainage area below 50% of the overall circumferential area of the piles.

Ni *et al.* [16] performed laboratory investigations on the radial consolidation by perforated driven piles in soft clay. The perforated piles were found to be effectively dissipate the excess pore water pressure. When such piles are used in groups, the consolidation characteristics were found to improve further. Based on the test results, zone of influence of the permeable piles were proposed.

Wang *et al.* [17] developed a semi-analytical consolidation solution of perforated pile using mixed boundary condition. Instead of providing drainage hole, PVDs were installed in the pile circumference. It was observed that increase the number of PVD strips would provide a better consolidation performance, instead of increase in its width.

Chen *et al.* [18] provided analytical solution to characterize the pile-soil interface boundary for consolidation analysis driven permeable pipe pile in soft clay. Also, a set of laboratory model tests were performed with permeable pipe pile, single and group, embedded in remoulded saturated soft clay bed prepared in a rectangular confining chamber. The excess pore water pressure was measured by a series of piezometers installed in the soil bed. It was observed that the optimum improvement was achieved at the centre of pile group; the influence zone was found as function of depth.

The consolidation characteristics of perforated driven piles in soft saturated clay is illustrated in Fig.2.

3 Geomechanical Characteristics of Perforated Pile

The load transfer mechanism of perforated pile is similar to that of a conventional driven pile in soft clay. The consolidation characteristics, on the other hand, is like a vertical drain. Due to pile-soil relative stiffness, arching does take place on ground surface around the pile in case of embankment loading, the mechanism being similar to a typical stone column. These aspects have been described herein.

3.1 Load Transfer: Ultimate Axial Capacity

The ultimate axial capacity of a typical pile in clay is obtained by the following correlation [8]:

$$Q_u = Q_s + Q_b - W_p \tag{1}$$

where, Q_u is the ultimate axial capacity, Q_s is the shaft capacity, Q_b is the base restraint and W_p is the self-weight of pile, which are given respectively by:

$$Q_s = \int_0^L \pi D_o \, \alpha(1-\chi)\, c_u^{zt}\, dz \tag{2}$$

$$Q_b = \frac{\pi\left(D_o^2 - D_i^2\right)}{4}\left(c_u^{Lt} N_c + \sigma_v\right) \tag{3}$$

$$W_p = \frac{\pi\left(D_o^2 - D_i^2\right)}{4} L\gamma_p \tag{4}$$

where, D_o and D_i are the outer and inner diameter of the pile, α is the adhesion factor, χ is the perforation density defined as the ratio of the total areas of the drainage holes to the circumference, c_u^{zt} is the undrained cohesion of soil at depth z below ground surface, N_c is the Terzagi's bearing capacity factor, σ_v is the vertical stress imparted on the pile base, and γ_p is the unit weight of pile.

The undrained failure of foundation is important in case of soft saturated clay. Although drainage of excess pore water pressure from soil occurs via the pile perforations, the undrained pile capacity has been derived by Equations (3) and (4). The drained capacity of piles is usually taken for over-consolidated clay on long-term basis [19].

The undrained cohesion is one the most significant soil strength parameter of soft clay which is required to evaluate the undrained pile capacity. In most cases, the undrained soil strength has been idealized to increase linearly with depth [20]. Simultaneously, the undrained cohesion also increases with time due to consolidation [21]. Hence, the undrained cohesion (c_u^{zt}) in the above Equations has been expressed as a function of both z and t.

The correlation for the non-dimensional perforation density χ is given by:

$$\chi = \frac{\frac{n_h \pi D_h^2}{4}}{\frac{\pi\left(D_o^2 - D_i^2\right)}{4} L} \tag{5}$$

where, n_h is the total number of holes in the pile circumference, D_h is the diameter of an individual hole.

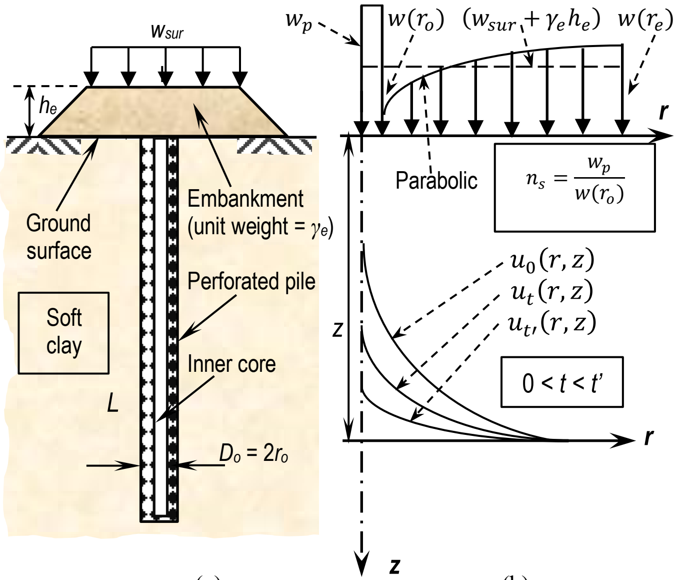
Various correlations are available for the adhesion factor α. After the American Petroleum Institute [22], the following correlation holds good:

$$\left.\begin{aligned} \alpha &= 1.0 \; for \; c_u^{zt} \le 25\, kPa \\ &= 0.5 \; for \; c_u^{zt} \ge 75\, kPa \\ &= 1.0 - (c_u^{zt} - 25)/90 \; for \; 25\, kPa < c_u^{zt} < 75\, kPa \end{aligned}\right| \tag{6}$$

The above correlation has later been modified as follows [23]:

$$\left.\begin{aligned} \alpha &= \tfrac{1}{2}\sqrt{\sigma_z'/c_u^{zt}} \; for \; \sigma_z'/c_u^{zt} \ge 1 \\ &= \tfrac{1}{2}\sqrt{\sqrt{\sigma_z'/c_u^{zt}}} \; for \; \sigma_z'/c_u^{zt} < 1 \end{aligned}\right| \tag{7}$$

where, σ_z' is the effective overburden pressure at the interface at a depth of z.



(a) (b)

Fig 2. (a) Typical perforated pile, and (b) load-transfer and consolidation mechanisms.

3.2 Soil Arching

The arching effect is evident in case of embankment loading. Due to significant pile-soil relative stiffness, the soil arches over the pile, initiating a parabolic vertical stress distribution on the ground surface. This phenomenon is termed as the soil arching [24].

It is well established that the soil strength progressively increases with the consolidation. Therefore, the arching is essentially time dependent. The vertical stress on the ground surface is given as [21]:

$$w(r) = [1 - F_s(N - 2r/D_o)^2]\, w(D_o/2) \tag{8}$$

where, $q(r)$ is the vertical stress on the ground surface at a radial distance of r from the pile surface, F_s is a stress function which depends upon the pile-soil stress concentration ratio n_s and N is a geometrical parameter, given as,

$$N = \frac{r_e}{D_o/2} \tag{9}$$

The stress concentration ratio is given by:

$$n_s = \frac{w_p}{w(D_o/2)} \tag{10}$$

where, r_e is the radius of influence and w_p is the vertical stress on pile. Since the soft clay undrained gradually increases with time as the consolidation progresses, the parameter ns also increase with time, till a steady state value is attained.

In case of PVDs, the value of n_s is unity, while for SCs, its value varies between 2 to 5 [25]. In case of pile, however, the value of n_s is expected to be more than that for SCs due to significantly higher pile-soil relative stiffness, although no definite information is available on this aspect. In addition, unlike PVDs and SCs, the pile-soil interface is subjected to shear stress initiating a differential settlement between the soil and the pile, which is likely to alter the load transfer mechanism in a different way [26].

3.3 Soft Clay Consolidation

Due to perforations, the piles also act as vertical drains to assist radial consolidation. Due to larger diameter and higher hydraulic conductivity, the rate of consolidation in case of these perforated piles is expected to be much greater than PVDs and SCs.

The flow of pore water during the consolidation is predominantly radial (horizontal towards the pile), although there shall also be a minor vertical component as well. Neglecting this vertical component, the differential equation for radial consolidation is given by [27]:

$$c_{vr}(\nabla_r^2 - \nabla_t)u_{rt} = 0 \qquad (11)$$

where, $\nabla_r^2 \equiv \frac{\partial^2}{\partial r^2} + \frac{1}{r}\frac{\partial}{\partial r}$, $\nabla_t \equiv \frac{\partial}{\partial t}$, u_{rt} is the excess pore water pressure at the coordinate (r, t) in the space-time frame and c_{vr} is the coefficient of radial consolidation, given by:

$$c_{vr} = \frac{k_h}{m_v \gamma_w} \qquad (12)$$

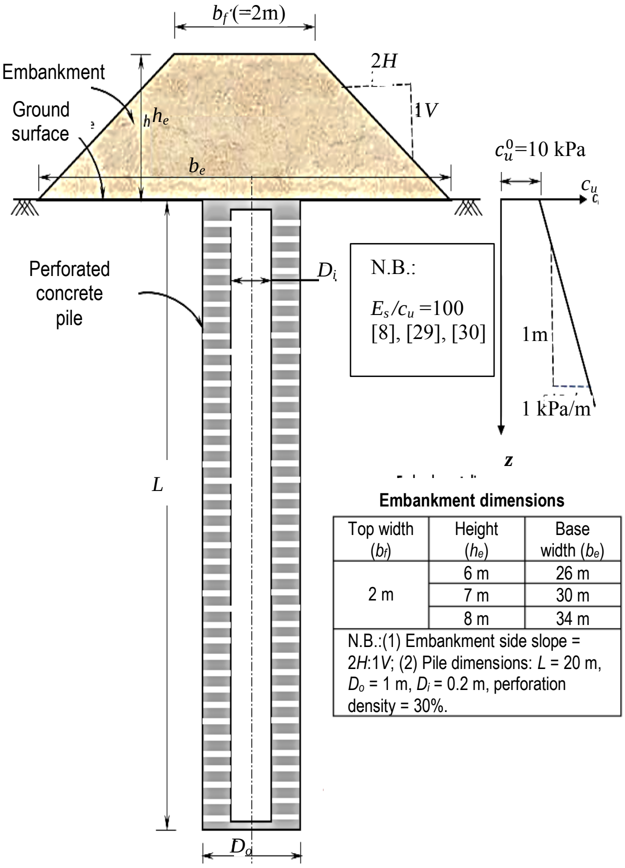
where, k_h is the horizontal permeability of soft clay, m_v is its volumetric compressibility and γ_w is the unit weight of water.

Due to excess pore water pressure dissipation, the soft soil will strengthen and stiffen significantly, resulting in enhanced ultimate pile capacity at the end of consolidation. The concept of installation of perforated piles involves partial replacement of weak soil with the piles which act as in-situ reinforcement to the soft cohesive soil. Ground improvement by perforated piles are adopted to support embankments, bridge abutments, tanks as well as large industrial, commercial and marine structures [28].

4 Case Study

In this section, the field performance of a prototype perforated pile supporting a typical rectangular embankment has been studied. The post-consolidation pile capacity due to enhanced soil strength, settlement reduction and radial consolidation characteristics have been analyzed through simplified formulations. In absence of appropriate theoretical model, these parameters were evaluated using existing results valid for stone columns. Although they might not be specifically applicable in case of perforated piles, those results were used as a preliminary study. However, a more accurate analysis demands rigorous theoretical modelling.

The practical application of perforated pile at the site has been illustrated by a hypothetical case study, where a semi-infinite soft clay deposit has been improved by a concrete perforated pile and the preloading is imparted by an embankment, as shown in Fig.3, [29], [30]. The analysis and the performance of the perforated piles in terms of soft ground improvement has been studied, as described herein.



Embankment dimensions

Top width (b_f)	Height (h_e)	Base width (b_e)
2 m	6 m	26 m
	7 m	30 m
	8 m	34 m

N.B.:(1) Embankment side slope = 2H:1V; (2) Pile dimensions: L = 20 m, D_o = 1 m, D_i = 0.2 m, perforation density = 30%.

Fig 3. A typical case study

4.1 Ultimate Axial Capacity of Pile

The ultimate axial pile capacity has been evaluated using Equations (1-4) above. The values of α have been computed using Equations (6-7) above. Choosing bulk unit weight of soft clay as 18 kN/m^3, the depth-wise variation of α has been shown in Fig.4.

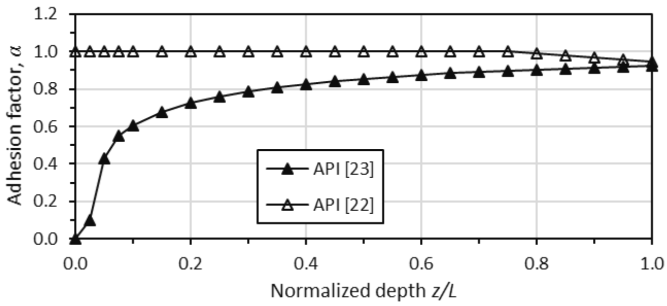


Fig.4. Depth-wise variation of adhesion factor

Considering N_c = 9.0 [8] and γ_p = 25 kN/m^3, the pre-consolidation values of ultimate axial capacity of the pile is evaluated, as given in Table 2.

Basack *et al.* [31] carried out parametric studies to provide design curves for radial consolidation and resulting improvement of bearing capacity for stone column reinforced soft clay. The soft ground improvement was quantified by a non-dimensional improvement factor β defined as the ratio of post-consolidation to pre-consolidation undrained cohesion of soft clay.

For different embankment heights, the value of N has been estimated from the following correlation:

$$N = \frac{2(b_f + 4h_e)}{D_o\sqrt{\pi}} \tag{13}$$

The average vertical stress on the ground surface is given as,

$$w_{av} = h_e\gamma_e \tag{14}$$

where, γ_e is the bulk unit weight of the embankment material.

The stress concentration ratio has been estimated from the following correlation:

$$n_s = E_p/E_s \tag{15}$$

where, E_p and E_s are the Young's modulus of the pile and the soil respectively.

A set of design curves portraying the variation of β with w_{av}/c_u^0 for different values of N and n_s is available [31]. Assuming γ_e=20 kN/m^3 and E_p=30 GPa, different values of β have been extrapolated for different embankment heights. The post-consolidation ultimate axial pile capacity has been estimated from the following correlations:

$$Q_s^f = \beta Q_s \tag{16}$$

$$Q_b^f = \frac{\pi(D_o^2 - D_i^2)}{4}(\beta c_u^{Lt} N_c + \sigma_v) \tag{17}$$

$$Q_u^f = Q_s^f + Q_b^f - W_p \tag{18}$$

where, Q_s^f, Q_b^f and Q_u^f are the post-consolidation values of shaft capacity, base restraint and net ultimate capacity of the pile, respectively.

For different values of embankment heights, the imposed vertical stress on the ground surface is likely to vary. Accordingly, the post-consolidation pile capacities shall also alter. Using the above correlations, the different values of post consolidations have been calculated [32]. This is depicted by a bar chart shown in Fig.5.

4.2 Settlement Analysis

Apart from the requirement of adequate factor for pile design, the settlement under imposed load must not exceed the acceptable limits. Hence, the settlement analysis has also be included in the present case study. The method of computation is described below.

Assuming the volumetric compressibility of the clay as m_v = 3 x 10^{-6} m^2/N, the settlement of the clay layer at a degree of consolidation of U = 90% without the pile has been computed from the following correlation [31]:

$$\rho_{90}^s = \int_0^L U m_v \Delta p \, dz \tag{19}$$

Table 2. Pre-consolidation ultimate axial pile capacity

Method \ Capacity	Shaft capacity, Q_s (kN)	Base capacity, Q_b (kN)	Self weight, W (kN)	Net capacity, Q_u (kN)
API [23]	730	475	377	828
API [22]	835	475	377	930

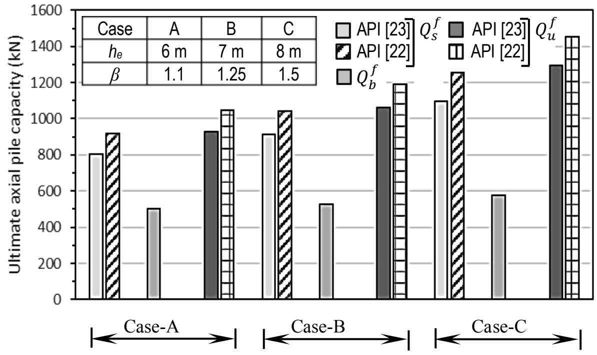


Fig 5. Post-consolidation axial pile capacities

where, Δp is the change in the effective overburden pressure at the centre of the clay element.

To compute the settlement of ground surface with pile, the methodology proposed by Basack *et al.* [31] has been followed, where a settlement factor ξ was introduced, which was defined as the ratio of average ground settlements with and without reinforcement at 90% consolidation. The variation of ξ with N for different values of n_s were proposed by a set of curves. The average ground settlement with piles has been computed using these curves by extrapolation. The values of ground settlements are shown in Fig.6.

As observed, the average ground settlement increases with the ascending embankment height following a fairly linear pattern. Due to the presence of the pile, the average ground settlement has been significantly reduced compared to those obtained without the pile. For the embankment heights of 6 m, 7 m and 8 m, the values of settlement factor ξ has been estimated as 0.165, 0.175 and 0.189 respectively. Accordingly, the percentage of settlement reduction compared to unreinforced soft ground has been calculated [$= (1-\xi)x100\%$]. This implied that the values of relevant settlement reduction have been 83.5%, 82.5% and 81.1%, respectively.

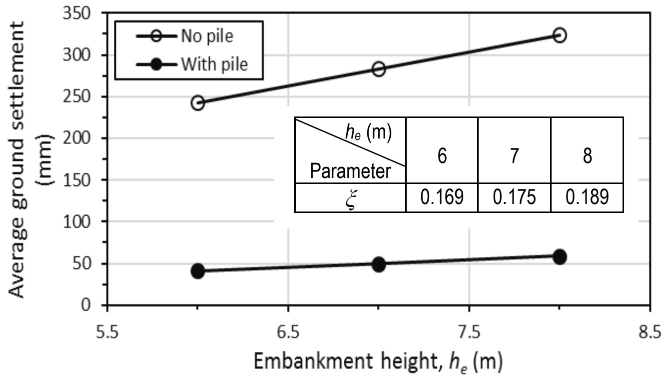


Fig 6. Variation of ground settlement with embankment height

4.3 Consolidation Characteristics

To study the consolidation characteristics, appropriate theoretical analysis and experimentations (laboratory or field) are necessary. In absence of such data, an approximate analysis has been carried out in the present case study. The existing consolidation test data with the perforated single pile from the laboratory model tests by Chen *et al.* [18] has been utilized herein. To obtain the consolidation time results, the appropriate laboratory data has been multiplied by a non-dimensional conversion factor ς given by:

$$\varsigma = \left(\frac{c_{vr}^{f}}{c_{vr}^{l}}\right)\left(\frac{\chi_f}{\chi_l}\right) \tag{20}$$

where, the suffixes *f* and *l* refer to the values of the relevant parameters corresponding to field and laboratory, respectively.

Considering the horizontal permeability of the soft clay in the field as 1 x 10^{-9} m/s, the value of ς has been evaluated as 208.33. Using the laboratory test results [18], the time pattern of variation of the degree of consolidation is plotted, as depicted in Fig.7. The details of analysis have been available elsewhere [32].

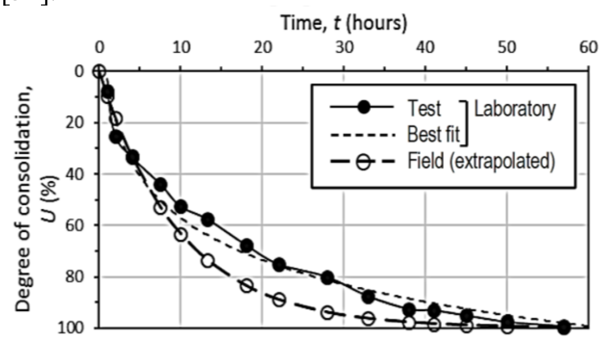


Fig 7. Variation of degree of consolidation with time

The field data has been estimated based on extrapolating the laboratory test results using Eq. (20) above. It has been observed that the improvement factor β has been 1.1, 1.25 and 1.5 in the cases of embankment heights of 6m, 7m and 8m, respectively (see Fig.5). Thus, due to increased soil strength initiated by radial consolidation, the ultimate pile capacity has been increased by 10%, 25% and 50%, respectively. Moreover, the perforated piles have been quite effective as a vertical drain, assisting the radial consolidation, as observed in Fig.7. The field consolidation using extrapolated laboratory test data progressed faster compared to the laboratory results. Level of accuracy of such observation requires a more rigorous analytical or numerical modelling.

5 Critical Analysis and Research Directives

Soft ground improvement using perforated piles is a relatively new but quite effective technique. However, due to limited study, theoretical and experimental knowledge available is rather limited. This necessitates the importance of conducting a thorough and detailed study including theoretical analysis (analytical and numerical), laboratory model tests incorporating the critical parameters and instrumented field trials. In parallel, the cost effectiveness of the new technique should as well be analyzed to critically understand the benefits achieved over the other established soft ground improvement techniques. This should be followed by developing design recommendations associated with appropriate charts and curves to assist the practicing engineers.

Accordingly, the research directions should follow the path satisfying each of the above steps sequentially. The proposed flow-chart of execution of the research is presented in Fig. 8 below.

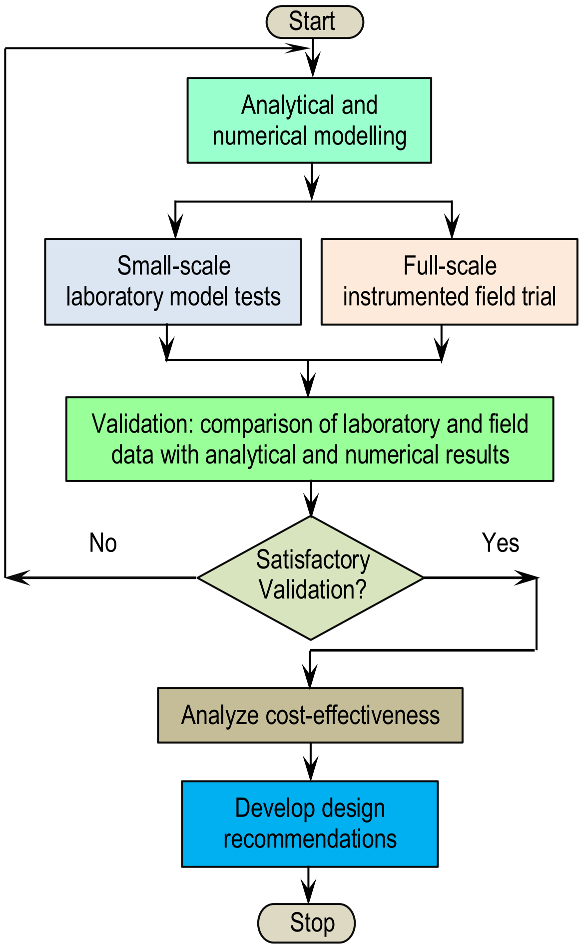


Fig 8. Research directives: proposed flow-chart

6 Conclusions

Perforated piles used for soft ground improvement is relatively new but promising technique. This paper presents a brief overview the existing studies carried out on this emerging study area. The literature review indicates that the numerical, analytical and laboratory investigations carried out has been rather limited, thus the field is in infantry stage.

The study reveals that the perforated piles not only transmit the structural loads quite effectively to the subsoil beneath through shaft friction and end bearing, but they are also quite effective in reducing the settlement and accelerating radial consolidation. Simplified correlations have been proposed to estimate ultimate capacity of these perforated piles.

A typical case study on the effectiveness of perforated piles embedded in soft clay deposit under embankment loading has been carried out. While the pile capacity has been found to increase by 10-50% due to consolidation, significant settlement reduction of 81.3-83.5% was observed. The consolidation characteristics were also studied by reasonable extrapolation of the available laboratory test data, which indicated acceleration.

It is proposed that extensive theoretical and experimental investigations should be conducted together with cost effectiveness, followed subsequently by appropriate design recommendations for practicing field engineers.

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Notations:

b_f	=	Top width of embankment
b_e	=	Bottom width of embankment
c_u^{zt}	=	Undrained cohesion of soil at point (z, t)
c_u^{Lt}	=	Undrained cohesion of soil at point (L, t)
c_{vr}	=	Co-efficient of radial consolidation
c_{vr}^f	=	Co-efficient of radial Consolidation in field
c_{vr}^f	=	Co-efficient of radial consolidation in laboratory
D_o	=	Outer diameter of pile
D_i	=	Inner diameter of pile
D_h	=	Diameter of an individual hole
E_p	=	Young’s modulus of pile
E_s	=	Young’s modulus of soil
F_s	=	Stress function
$H_{e\backslash}$	=	Height of embankment
H_s	=	Height of soil layer
k_h	=	Horizontal permeability of soft clay
L	=	Length of pile
m_v	=	Volumetric compressibility
n_s	=	Stress concentration ratio
n_h	=	Total number of holes in the pile circumference
N_c	=	Bearing capacity factor
Q	=	Load carrying capacity of pile
Q_b, Q_s, Q_u	=	Base restraint; shaft friction, axial capacity of pile
Q_b^f, Q_s^f, Q_u^f	=	Post-consolidation base restraint, shaft friction and net axial capacity of pile
Q_b^f	=	Post-consolidation net ultimate base restraint
Q_u^f	=	Post-consolidation net ultimate capacity of pile
r	=	Radial distance
r_p	=	Radius of pile
r_e	=	Radius of influence
U	=	Rate of consolidation
u_{rt}	=	excess port water pressure at point (r, t)
$w(r)$	=	Vertical stress on ground surface
w_p	=	Vertical stress on pile
w_{av}	=	Average vertical stress on ground surface
w_{sur}	=	Surcharge load
w_p	=	Self-weight of pile
z	=	Depth
α	=	Adhesion factor
β	=	Non-dimensional improvement factor
γ_e	=	Unit weight of embankment
γ_p	=	Unit weight of pile
γ_s	=	Unit weight of soil
σ_v	=	End bearing capacity
σ_z'	=	Effective overburden pressure
τ_v	=	Skin friction
ς	=	Non dimensional conversion factor
χ	=	Perforation density
χ_f	=	Perforation density in field
χ_l	=	Perforation density in laboratory
Δp	=	Change in overburden pressure

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