Critical State Modelling of Soil-Structure Interface for Advanced Design

Modélisation à l'état critique d'interface sol-structure pour la conception avancée

Sarma D., Sarma M.D
LM-IGS, M-ISSMGE, Independent Consultant (Southern Africa), Gaborone, Botswana

ABSTRACT: Information on the influence of impregnation of cementitious slurry at the soil-structure interface of bored cast in-situ foundation is inadequate in available literatures. Moreover, influence of such impregnation on negating the detrimental effects of smear zone, formed by construction tools, surrounding the borehole is also unknown. In classical foundation engineering, influences of smear and impregnation are neither considered as dependent functions in determining contributory or negative shaft resistances, nor in shaft and base resistance interaction. This ignorance contributes empiricism in bearing capacity evaluation recognising it as one of the possible causes of variation of field performance with respect to prediction. Solution to these problems has been explored through field and simulated laboratory studies of smear and impregnation, developing new device and technique. Further, an approach to interface modelling of soil-structure is presented considering impregnation.

RÉSUMÉ : L'information disponible dans la littérature sur l'influence de l'imprégnation de coulis de ciment à l'interface sol-structure de fondations coulées en place est inadéquate. En outre, l'influence de l'imprégnation sur la négation des effets néfastes de la zone de souillure, dus aux outils de construction, entourant le trou de forage est également inconnue. Dans les travaux de fondation classiques, les influences des souillures et l'imprégnation ne sont jamais considérées comme des paramètres liés dans la détermination du frottement négatif ou positif, ni dans la résistance de pointe. Cette ignorance contribue à l'empirisme en cours dans la détermination de la capacité portante et peut être l'une des causes possibles de variation des performances sur le terrain par rapport à la prédiction. Une solution à ces problèmes a été explorée in situ et en laboratoire en développant un nouveau appareillage et une nouvelle technique. En outre, une approche de modélisation des interfaces de sols structure est présentée en prenant en compte l'imprégnation.

KEYWORDS: Impregnation, smear zone, soil-structure interface, effective diameter, shaft resistance.

1 INTRODUCTION

Barring the situations where a permanent casing is left in the borehole, in all other bored cast in-situ deep foundations, fresh concrete comes directly in contact with the ground. During the process of concreting, cementitious slurry from the body of the unset concrete of the cast in-situ deep foundation starts impregnating and upon setting, strengthens the surrounding soil within the impregnation depth. Physical evidence of surrounding cement-impregnated soil becoming a part of the foundation shaft, stated by many authors, was reconfirmed (Sarma, 1992). In such a case it is apparent that the adhered soil shall behave as an integral part of the foundation, increasing its effective diameter. Other researchers (Berezantzev, 1965; Sowers, 1979) also found evidences of such a phenomenon, investigation on which is, however, limited. Such impregnation may increase negative shaft resistance causing serious treat to both friction and end bearing foundations, positive contribution of which, may enhance shaft resistance for all types of cast in-situ foundations. Consideration of such phenomenon in design can contribute immensely in economic aspects. Therefore, extent and effect of such impregnation into the surrounding soil have been studied in this paper for wide variety of soils. These are aimed at ascertaining the soil-structure interface strength mobilisation for bored cast in-situ foundations considering smear zone, which is formed by construction equipments. Necessary equipment for field simulation was designed and developed (Sarma, 2000) for impregnation study and eventually for simulation modelling. Investigations through microscopic and staining techniques were carried out for insights into the effects of smear and impregnation with associated effect of shrinkage. Ignorance of this complex soil-structure interface interaction contributed empiricism in bearing capacity evaluation witnessing uncertainty in field performance.

2 FORMATION OF SMEAR ZONE

The process of installation of cast in-situ deep foundation disturbs the sub-soil formation surrounding the borehole with respect to its virgin state. The mechanical process involved in the boring changes physical characteristics weakening the structure of soil to reduce its shear strength within the zone of influence of the boring tools. During the progress of boring, soil in the upper portion of borehole squeeze inward due to the loss of lateral support and their shearing during withdrawal and reinsertion of boring-tool further disturbs the structure of clays or in case of sand reducing its density despite stabilisation under drilling mud. This process repeatedly continues during each insertion and withdrawal of the boring tool. Further, the surging effect that occurs during repeated withdrawal of boring equipment further loosens the soil. The process of boring also changes the particle orientation at the interface. Thus a smear or distortion zone is formed circumscribing the borehole. The extent of smear zone is thus function of the sensitivity or relative density of the surrounding soil, magnitude of vibration and disturbances caused by the boring equipments, properties of the soil responsible for propagation of velocity wave for the vibration and disturbances caused during boring etc.

3 DEPTH OF SMEAR ZONE AND INTENSITY OF SMEAR

Meyerhoff G.G (1976) reported that depth of smear surrounding bored pile might exist up to 1 inch (25 mm). Besides that, there is not much information in the literatures about the depth of smear zone associated with cast in-situ deep foundations. This is perhaps due to the fact that it is rather a problem primarily associated with workmanship for installation of such foundation along with a host of secondary factors and thus any effort for rational analysis of the problem bristles with difficulties. The depth of smear or distortion zone depends on the type of boring equipments, method of boring and more importantly on the nature and status of the soil. Effort has been made to fulfil parts of the objectives of the research (Sarma, 2000) to evaluate the depth of smear zone, intensity of smear, and effect of impregnation under laboratory test conditions.

4 IMPREGNATION AND ITS EFFECT ON INTERFACE STRENGTH

Fresh concrete forces the softened/loosened soil partially outward from the borehole owing to its larger unit weight and pressure head developed during placement and concreting operations. This creates further disturbances to the soils. Specific gravity of the fresh concrete being higher than that of the in-situ soil and due to positive pressure developed particularly in tremie action cementitious slurry, which is one of the constituents of the fresh concrete, always exists at higher than the ground water pressure unless an artesian flow of higher order reverse the situation. Therefore, in general, a tendency for impregnation of cementitious slurry from the body of the freshly cast deep foundation towards the less stressed zone surrounding the borehole always exists until a state of equilibrium of slurry pressure in the pores of surrounding soil is attained. Such impregnation of cementitious slurry alters the physico-chemical characteristics of soil within the impregnated zone and upon setting strengthens smear/distortion zone within the impregnation depth.

5 MOBILISATION OF SHAFT RESISTANCE

The philosophy of soil-structure interface strength based on the effect of distortion or smear zone and impregnation of cementitious slurry together, can be visualised in the following way:

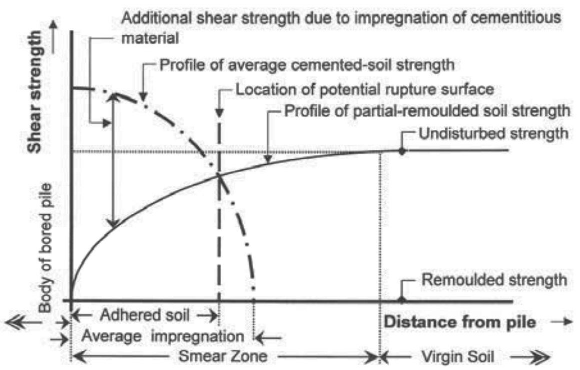
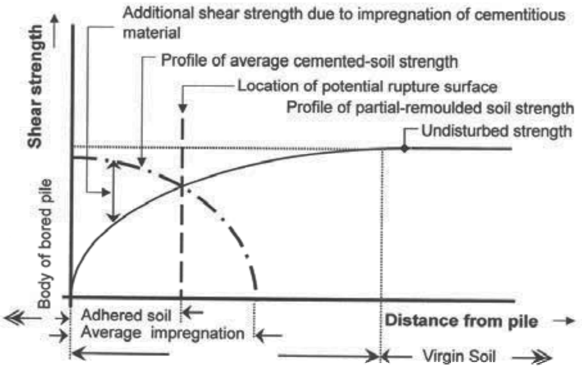


Figure 1 (a). Idealised profiles at soil-structure interface, cemented strength more than undisturbed strength (After Sarma, 1992)

It was asserted that the impregnated cement slurry improves the shear strength of part of the distortion zone surrounding cast in-situ deep foundation (Sarma, 1992). Also during curing, concrete absorbs moisture from the surrounding soil and thus soil gets consolidated to give better strength. This gain in strength diminishes with increasing distance from the foundation. On the other hand, intensity of remoulding / loosening effect that causes shear strength to decrease in the surrounding soil reduces away from the foundation. Hence the rupture surface for mobilisation of shaft resistance does not exactly lie at the interface of the concrete of the foundation and soil. It is possible that the weakest surface may exists away from the body of foundation as a result of the two opposing effects mentioned above. Therefore soil up to the weakest surface may adhere to the foundation surface behaving as its integral part. In such a case slip or local yield that occur when the shear stress reaches the adhesive (or yield) strength may not occur at soil-structure interface rather between adhered and surrounding soil. Two different possibilities have been presented. Fig. 1(a) portrays potential rupture surface in case cement impregnated strength of soil is higher than its undisturbed strength and Fig. 1(b) portrays potential rupture surface in case cement impregnated strength of soil is lower than its undisturbed strength

Figure 1(b). Idealised profiles at soil-structure interface, cemented



strength less than undisturbed strength (After Sarma, 1992)

In both the figures soil-structure interface shear strength is presented in the ordinates. Therefore, any point towards left of the ordinates will represent the body of foundation. The profile of the shear strength of soil due to the effect of smear is shown with minimum remoulding strength at the soil-structure interface where extent of smear is maximum and strength increases away from the interface. It will be undisturbed shear strength at the end of smear zone due to the diminishing affect of smear. Any point at the profile of shear strength of soil within smear zone will represent partial remoulded strength depending upon the severity of smear.

On the other hand cemented soil strength, i.e., soil shear strength due to the effect of impregnation of cementitious material, will be maximum at the soil-structure interface. The cemented soil strength has a diminishing trend away from the body of the foundation up to the remoulded shear strength of soil at the end of impregnation. This may be idealised within the extent of average impregnation depth.

The combined effect of cemented soil strength, which has diminishing tendency, and remoulded shear strength which has increasing tendency from the body, may act as the mobilised shear strength of soil surrounding the shaft. It is clear from the figure that potential rupture surface exists at the point where mobilised shear strength attains a minimum value. The soil up to the potential rupture surface will act as a part of the shaft that affects increased diameter. With the fact that the potential rupture surface exists away from the shaft, the average shear strength of the soil within the impregnated zone may be either lower or higher than that of the undisturbed state due to the combined effect of smear and cemented strength.

While formulating the philosophy of this radical concept of soil-structure interface strength it is considered that the depth of impregnation is less than the depth of smear. The depth of impregnation being more than the depth of smear is possible only in case of granular soil. This is ruled-out as bentonite slurry, which is generally used to stabilise the borehole, impregnates prior to cement slurry.

6 FIELD EVIDENCES OF IMPREGNATION

Piles pulled out of soil frequently appear with a skin of soil sometime several mm thick adhering tightly to the surface of the pile thus becoming a part of the pile itself (Bowles, 1988). Field evidence of such a phenomenon was noticed and reconfirmed by this author too during excavation for construction of deep pile caps. Due to soil adhering to the pile surface the effective diameter, at which shaft resistance mobilises, increases.

Field investigation reveals that soil becoming a part of pile, with a thick skin of adhered soil, is prominent in case of cast in-situ deep foundation installed by conventional / hydraulic assisted augers or bailers where no drilling mud is used. Such installations where Direct Mud Circulation (DMC) technique is used, a thick soil becoming a part of the foundation is not generally observed during excavation for caps. This has been confirmed during bored pile foundation for the hostel building of Indian Institute of Technology, Guwahati, and in several other cases. This is due to the fact that the impregnation of bentonite that takes places during the process of boring, fills the voids adjacent to the bored surface. Moreover, the thixotropic property of the bentonite particles left no room for cement slurry to impregnate further, thus negating the chances for a thick skin of soil becoming a part of the structure. This confirms the possibility of mobilisation of shaft resistance closer to the surface of the structure or at the surface itself depending upon the nature of soil. In such a case the adhesion that occurs between two different materials may govern the process of mobilisation of shaft resistance rather than cohesion between adhered to surrounding soil.

7 DEVELOPMENT OF DEVICE FOR IMPREGNATION STUDY

Notwithstanding the effect of impregnation and soil becoming part of bored cast in-situ deep foundation, available literatures do not give much information on the magnitude of impregnation and its dependent factors. Furthermore, no technique for measuring such impregnation could be known from present literatures. Therefore, a new method was developed in which the pressure exhibited by cementitious slurry during placement of fresh concrete in borehole was simulated in laboratory for allowing impregnation through the soil sample collected from borehole. Such laboratory simulation involved development of concept, fabrication of device, and performing trial tests. Final version of the impregnation test equipment, incorporating minor modification upon trial tests, was used for impregnation study.

The equipment comprised of a closed rectangular concreting compartment of size 150x200x200 mm fabricated from thick steel plates with detachable top lid fixed by high tensile nuts and bolts. At the top lid a non-return-air valve and a pressure gauge were fixed to pump compressed air in and to monitor air pressure inside the compartment respectively. In one of the side plates of the compartment a hole of 75 mm diameter was made and a threaded socket was welded along the circumference of the hole so that a sampler could be threaded into the socket. Cylindrical hollow samplers of 75 mm diameter, 150 mm long, and 2 mm thick having threads at both the ends were used for sampling and for fixing with the socket for impregnation test. At the other end of the sampler, a threaded cap was fixed with a hole in it, plugged by jute wick, to allow water to come out.

All the joints have been made airtight. The non-detachable joints are sealed by resinous epoxy, threaded joints that require frequent removals are sealed by jute fibre soaked in zinc solution and non-threaded joints are sealed by rubber gasket kept in highly a compressed state. A remote control pneumatic pump was used to compress air inside the first compartment.

8 IMPREGNATION TEST PROCEDURE

Samples for impregnation tests were collected from shallow depth, generally, two numbers at the same depth from each auger-borehole by horizontal sampling. Shallow sampling depth was preferred in order to collect samples experiencing maximum disturbance from repeated insertions and withdrawals of boring tool. Collected samples were kept for twenty-four hours inside the sampler to regain its natural state to the possible extent. A pumice stone wrapped by filter paper was placed by trimming soil in the driven end of the sampler. After covering the end by cap, the sampler was inserted into the socket with smeared face of the sample towards the rectangular compartment. Nominal mix of concrete of ratio 1:2:4 (1 part cement: 2 parts sand: 4 parts 20 mm nominal size aggregate) with 10% extra cement, having slump of 120 mm was poured in the rectangular compartment and properly placed. Without any delay the top lead was fixed and air was pumped by the pneumatic pump till air pressure inside the rectangular compartment reaches the required limit.

Bowles (1982) indicated that the critical maximum pressure of concrete would occur at a depth within 10 to 20 times the diameter of pile. So for 400 mm diameter pile, the extent of height of fresh concrete was worked out within 4 and 8m and corresponding extents of impregnation pressures were adopted as 1 and 2 Kg/cm^2 for fresh concrete of specific gravity 2.5.

The pressure had to be maintained at required limit as sometime it was found necessary to compensate little pressure drops as a result of impregnation. Although initial setting time of cement is 30 minutes, pressure in the compartment was maintained for 90 minutes to simulate field condition to the possible extent. The pressure in the compartment was released and the sampler was then removed from the compartment. Sampler cap, pumice stone and pieces of filter paper were also removed for extrusion of the sample from sampler. For extrusion, pressure was applied in the face at the outer side where pumice stone and G.I. cap was fixed such that no disturbance occurs to the impregnated and smeared face of the sample. The samples were stored in dry place without any disturbance to the impregnated and smeared face.

9 PREPARATION OF BLOCK SAMPLES FOR OBSERVATIONS

For visual observation it is necessary to prepare block samples with at least one plane face. Of the whole sample, since the face through which impregnation occurs was important, it was necessary to cut along the direction of impregnation. Prior to cutting, the samples were saturated with toluene-epoxy solution under vacuum desiccators. With the process of de-airing, the sample absorbed toluene-epoxy solution and upon curing gets strengthened. After 15 days of strengthening the samples were found to be suitable for cutting. The cutting was done by a thin metal saw without causing any undue damage to the required plane.

10 MICROSCOPIC OBSERVATION FOR SMEAR ZONE AND IMPREGNATION

Microscopic investigation was carried out under a high resolution polarised microscope. The prepared sample for investigation was placed under microscope and observed through lens ‘X100’ and subsequently with ‘X10’. In case of study of samples under high magnifying lens, the particle re-orientation or particle crushing due to smear effect was found very difficult to identify from other randomly oriented soil particles that naturally exist in the soil. Furthermore, it was found to be very difficult to distinguish between the particles of impregnated cement from that of other similar whitish materials scattered in the soil matrix. Under low magnifying lens (X10), however, some irregular cylindrical veins of deep brown tinted materials and whitish materials were seen at random. Later those deep brown tinted material was identified as the epoxy resin used for stabilisation of the study samples and whitish material was identified as impregnated cement particles respectively. The identification was confirmed upon comparing the samples of hardened epoxy and hardened cement under the same microscope. While in case of epoxy resin, cracks formed due to desiccation of surface during preparation of samples was identified as the chief reason for impregnation, two possibilities were identified for impregnation of cementitious materials in the form of cylindrical veins. These two possibilities might occur separately or simultaneously.

In the first possibility, the flow of cementitious slurry might take place through the shear channels formed during the process of boring. In such an event there had the possibility that the smear zone would comprises series of lumps at random surrounded by shear channels without altering soil properties within the lumps. In the second possibility, flow of cement slurry had not taken place through the smeared face of the borehole wall uniformly and depending upon variations in permeability cement slurry impregnated in the shape of cylindrical veins. Less permeability might be either due to undisturbed state of soil or due to a lesser effect of smear. As undisturbed state of soil at the face of the borehole was not in conformity with practical experience, then, the lesser effect of smear would seem to govern. However, it could not be ascertained that impregnation of cement particles took place through the smeared face to the extent of formation of cylindrical veins, since it was found very difficult to distinguish between the particles of impregnated cement from that of other similar whitish materials scattered in the soil matrix.

The findings on the smear zone and impregnation could not therefore be properly interpreted from overall perspective due to limitation of the study under microscope. However, the only distinct photomicrograph showing soil-structure interface is presented in Fig. 2 (a). From the other photomicrographs, mapping of a distinguishable demarcation line between natural soil and its impregnated counterpart was not found convenient and therefore alternative technique had to be explored.

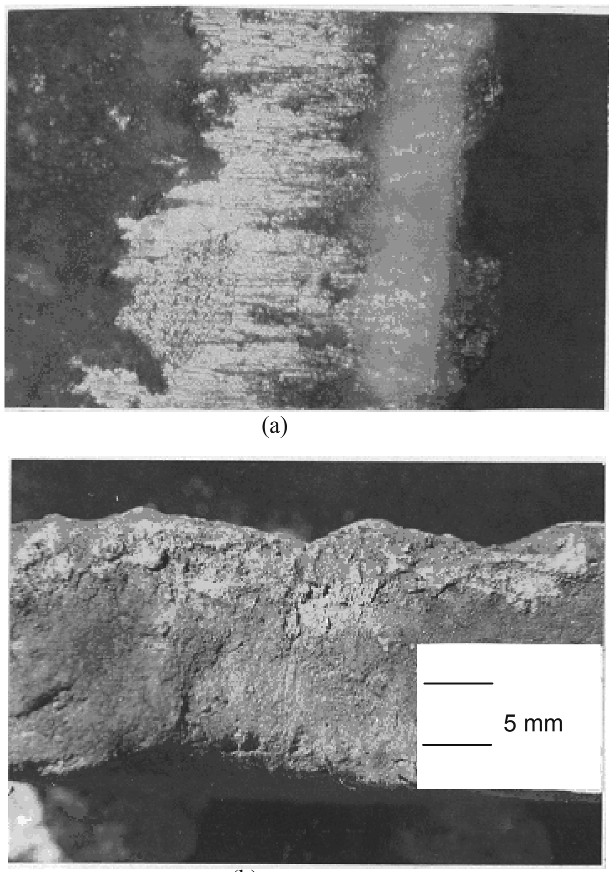
11 STAINING TECHNIQUE

Hutchison (1974) formulated a procedure to identify carbonated ingredients in sedimentary rocks. The chemical treatment carried out on rock sample turns the carbonated ingredient into pink colour. Initially, this 'staining technique' was adopted to identify the extent of impregnation in natural soil as cementitious material is predominantly enriched with carbonate particles. The staining technique was started by washing the exposed face of the sample with 1.5% HCl solution for 10 to 15 seconds and then immersing in the reagent for 10 to 15 seconds and drying under sun rays. The basic chemicals used for staining tests were 1.5% HCl solution, alizerin red-s and potassium ferocyanide. For treatment, two solutions were prepared from the above basic chemicals, one alizerin red-s solution (ARS) and other potassium ferocyanide solution (PFS). Both the solutions were prepared by dissolving solvent in 100cc of 1.5% HCl acid. For ARS, quantity of alizerin red-s was 0.2 gm while for PFS, quantity of potassium ferocyanide was 2.0 gm. The reagent was prepared by mixing ARS and PFS in ratio 3:2 for 30 to 45 seconds. This process, however, gave slight tint of colour from which distinguishable demarcation of natural soil with impregnated counterpart was not possible.

Eventually after many trials an appropriate staining technique, suitable for the impregnated soil samples, was developed modifying over the staining technique suggested by Hutchison (1974). Prominent demarcation was noticed after the treatments, resulting in two distinguishable colours of the sample viz., pink and green. In order to identify material of impregnation, hardened epoxy and cement pellet were treated by the same technique. Hardened epoxy did not give traceable change in colour while cement pellet turned pink. Thus cement-impregnated portions of the samples were ascertained. The reason behind natural soil turned green was explored and found that alkaline material turned pink while acidic material turned green upon reaction with the reagent. pH test conducted in Government Agricultural Laboratory confirmed pH values of soil ranges from 5.00 to 6.70, which confirmed acidic nature.

12 MEASUREMENT OF IMPREGNATION

To facilitate the measurement of impregnation close-up photographs of the samples were taken with a magnification of about two times rendering easy measurement of impregnation. A typical photograph with a pasted scale showing dimension of 5 mm is presented in Fig 2 (b). The scale of each photograph was derived by dividing measured length of the pasted scale in mm by 5. Impregnation mapping of all samples were then drawn from the photographs. A typical mapping is presented in Fig. 3 (a).



(a)

(b)

Figure 2. (a) Photomicrograph of soil-structure interface, (b) Coloured profile of cement impregnated soil at the soil-structure interface (After Sarma, 2000)

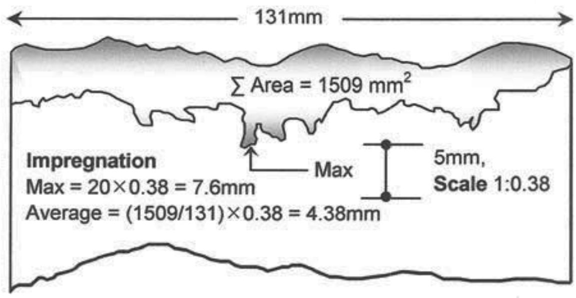
The mean depth of impregnation (I_{av}) was determined from the relationship as follows:

$$I_{av} = \frac{A_{iz}}{L_{iz}} \times S \quad (1)$$

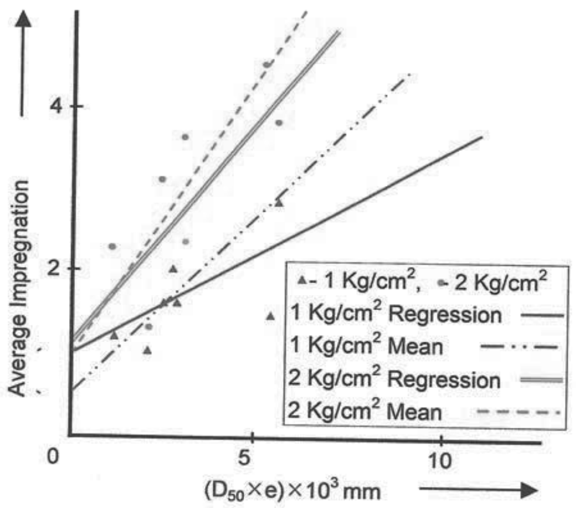
where, A_{iz} is the area enveloped by impregnation zone, L_{iz} length of impregnation zone, and 'S' the scale. Similarly, the maximum depths of impregnation were marked and measured multiplying by scale factor.

13 FACTORS AFFECTING IMPREGNATION AND FINDINGS

From the test results of samples collected from nine different locations, the average impregnations of different types of samples were found varying from 0.57 to 4.56 mm and maximum impregnations were varying from 1.42 to 7.6 mm. The peak values of average and maximum impregnation depths corresponding to 1 kg/cm^2 slurry pressure were 2.84 mm and 6.3 mm respectively. Similarly, peak values of average and maximum impregnation depths corresponding to 2 Kg/cm^2 slurry pressure were 4.56 mm and 7.6 mm respectively.



(a)



(b)

Figure 3. (a) Typical impregnation mapping, (b) Response of impregnation versus $D_{50}\times e$ (After Sarma, 2000)

As a general trend it was observed that both average and maximum impregnation depths increase with increase in slurry pressure. Individual plots between impregnation and other parameters like, over consolidation ratio, liquid limit, plasticity index, activity, coefficient of permeability, void ratio, revealed no definite trends. Therefore, plots were tried with composite parameters. For plots with maximum impregnation versus $D_{50}\times e$ (where D_{50} is the size of mesh through which 50% of soil passes and 'e' is the in-situ void ratio), no mean line could be drawn as points were scattered. This may be due to the fact that maximum impregnation depth depends on the depth of local fissures and as the depth of fissures vary considerably from sample to sample, the depth of maximum impregnation too would vary without it being a single function of void ratio and pore size. However, points of average impregnation versus $D_{50}\times e$ for slurry pressure of 1 Kg/cm^2 and 2 Kg/cm^2 respectively were seen in a trend more or less around a straight line drawn through the mean points for each pressure (Fig. 3 b). Regression analysis was done for average impregnation versus $D_{50}\times e$ and following equations of straight lines were developed for slurry pressure of 1 Kg/cm^2 and 2 kg/cm^2 respectively:

$$I_{av} = 0.23 \times 10^3 V + 1.05 \quad (2)$$

$$I_{av} = 0.54 \times 10^3 V + 1.12 \quad (3)$$

where, 'V' is $D_{50}\times e$ and 'Iav' is average impregnation in mm. The above equations show that average impregnation is a direct function of '$D_{50}\times e$', which is a measure of both pore size and overall void ratio.

14 EFFECT OF SHRINKAGE

Shrinkage of concrete, which is a phenomenon associated with curing, is an important factor as it introduces an element of uncertainty in the location of the potential rupture surface that results in uncertainty in the prediction of shaft resistance (Sarma, 1992). It is therefore of considerable importance in mobilisation of shaft resistance.

With the setting action of concrete, the process of shrinkage also continues. The magnitude of shrinkage, however, depends on various factors among which effects of size of aggregate, elastic properties of aggregate, concrete used, contamination of concrete by clay particles are important.

Among the influencing factors as stated above, the most important influence is exerted by aggregates. The size and grade of aggregate do not influence the magnitude of shrinkage directly but large aggregate permits use of linear mix and hence results in lower shrinkage (Nevile, 1981).

The elastic properties of the aggregate determine the degree of restraint offered. Presence of clay particles in concrete lowers its restraining effect increasing shrinkage. Even if, aggregates used in concrete are free from clay particles, during the process of tremie concreting it may carry clay particles from borehole wall and prone to higher shrinkage.

Based on the shrinkage strain of 3×10-4, recommended by Indian Standard (I.S. 456-1978), reduction of diameter is expected varying from 0.09 to 0.36 mm for diameter of pile 300 to 1200 mm. This reduction of diameter shall give rise to virtual gap around the shaft leading to virtual loss of contact with borehole wall.

There may be mixed opinion whether such gap has any practical significance or not. Generally it is expected that the soil of the borehole trends to fill-up such gap by collapsing under active pressure mitigating the effects of shrinkage. However, from the evidence of cemented-soil becoming part of such structures, it is possible that progressive collapse with gradual shrinkage occurs outside the zone of impregnation as concrete brings-in cement-impregnated surrounding soil during its shrinkage. Eventually, shrinkage may have effect outside impregnation zone and strain softening the potential failure surface further. Therefore, separate effect of shrinkage has not been considered while presenting the alternative concept of shaft resistance mobilisation based on impregnation. Nevertheless, effect of shrinkage may be prominent in case of bored cast in-situ deep foundation installed by DMC technique where, impregnation depth and soil becoming part of the pile may not be significant.

15 CONTINUING DEVELOPMENTS

The model of average impregnation is presented for the range of maximum concrete pressure for pile of 400 mm diameter. As the information on critical depth of maximum concrete pressure is available, the model can be extended for higher diameter shaft of deep foundation. With these findings determination of effective diameter of bored cast in-situ deep foundation is possible for variety types of soil more importantly information on the location of potential rupture surface for mobilisation of shaft resistance. Among other two indeterminate factors, namely, shrinkage and smear zone, development of a virtual collapse model due to shrinkage and its effect on the shaft resistance had already been completed. Formation of smear zone is a problem primarily associated with types of equipments used and installation workmanship. Developments over the conventional construction equipments and method of installation had already been done, new equipments fabricated, and prototype piles constructed for performance evaluation. The objectives of these new equipments were to keep the depth of smear minimum and within the impregnation depth. With these new patented equipments (viz., valve auger, scraper unit, and vibratory tremie) full scale piles constructed for performance tests which indicated promising results in comparison to the piles installed by conventional technique.

16 CONCLUSIONS

The extent of impregnation and smear zone surrounding borehole is a function of the soil properties and installation technique. Impregnation of cementitious slurry alters the physico-chemical characteristics of soil within the impregnated zone and upon setting strengthens smear zone within the impregnated depth. Cement-impregnated soil becoming a part of the cast in-situ deep foundation is almost certain. The resulting larger effective diameter is causing potential threat from negative shaft resistance. However, its positive contribution enhances shaft resistance vis-à-vis design economy. Due to the opposing nature of smear and cemented-soil characteristics, soil-structure interface strength depends on the properties of partially remoulded soil at the potential rupture surface, location of which is dictated by the depth of impregnation. Using newly developed simulation device soil samples from various auger-boreholes were impregnated at 1 and 2 Kg/cm^2 slurry pressure. Microscopic observations on impregnated samples were not found promising for determination of the extents of smear and impregnation. Modified staining technique enabled mapping of impregnation profile for determination of maximum and average impregnation depths. Although both impregnation depths increase with increase in slurry pressure, their plots with common soil parameters revealed no definite trends. In the plots of impregnation depth with composite parameter $D_{50}\times e$, which is a measure of both pore size and overall void ratio, no mean line could be drawn for maximum impregnation, however, regression analysis for average impregnation rendered model of straight line. The model indicates location of rupture surface for auger bored soil-structure interface that can be used for determination of shaft resistance. Further due to linear shrinkage the progressive collapse (strain-softening) may occur at the rupture surface or at the interface when impregnation is not prominent. This may be detrimental particularly for higher diameter shafts. The maximum unit shaft resistance is possible when the depth of smear is less and remains within impregnation depth. In order to minimise the extent of smear zone, new equipments were fabricated, patented, and used for construction of prototypes, performance of which were found to be promising..

17 ACKNOWLEDGEMENT

In order to supplement a research (Sarma, 2000), Mr. A. Deb undertook detailed investigation on impregnation (Deb, 1995) implementing the scheme and using the device developed by the Author[1]. His support is gratefully acknowledged. The experimental investigations referred to in this paper were part of Author[1]'s PhD work (Sarma, 2000), which were conducted under the supervision of Prof. P. K. Bora, PhD (Birmingham), Head of Civil Engineering (Retired), Assam Engineering College, India. His encouragement is gratefully acknowledged.

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