Geotechnical Aspects of the Indian Offshore Environment*

by

*Shashi K. Gulhati***

INTRODUCTION

Soils on main land India cover a wide spectrum in variety : from clayshales in the lower Himalayas, alluvial silts in the Indo-Gangetic plain, windblown deposits of sand in the Rajasthan desert, soft marine clays in the coastal belts, organic clays in the north-east, montmorillonitic Black Cotton Soils in the Deccan to laterites further south. These soils have provided rich fare for the Indian geotechnical engineer. This variety is further enhanced when one proceeds seaward from India's coast and focusses attention on soil deposits on the continental shelf.

Continental shelf is the land below the sea adjoining the coast where water depth is less than 200 m. In India, the continental shelf is very wide on the west coast, more than 200 km at Bombay-High and is quite narrow on the east coast, only 30 km in Madras and 40 km here at Visakhapatnam. The total area of this continental shelf is 3,80,000 sq. km which is about 8 per cent of India's land area.

From this additional 8 per cent of land under the sea, ONGC is currenlty obtaining 21 millon tonnes of oil and 7,300 million cubic metres of gas each year. On this land, more than 100 platforms have been erected for oil extraction and 80 new platforms are likely to be installed in the next four years. A network of submarine pipelines running into 1,800 kms now criss-crosses this land connecting platforms to each other and to the shore. About 900 kms of pipelines are to be laid in the next three years. Clearly, civil engineering activity is only going to increase with time on the continental shelf and then extend into deeper waters on the continental slope.

It is time, therefore, that due cognizance is given to the soils and the soil engineering problems that exist off-shore India. It is to this pursuit that this Lecture is dedicated.

TERRESTRIAL, TERRIGENIC AND PELAGIC SOILS

Soils below the sea are either terrigenic, that is, originating on land and then transported to sea, or they are pelagic, that is originating in the sea itself. Soils on the continental shelf are primarily terrigenic although they are often modified by the sea environment.

A close relation thus exists between terrestrial soils on land near the coast and those in the nearby off-shore region. Twenty years ago, Goldberg and Griffin (1970) presented data to show how the mineralogical content of surface soils off-shore India in any region correlates with mineralogical content of soils which are carried by the rivers that flow into the sea in that region. Thus, soils on the continental shelf in the Bay of Bengal, near Calcutta, are rich in Illite, Kaolinite and Chlorite, since the soils being brought there originate in the Himalayas and are being transported by the rivers Ganges and Brahmaputra. On the other hand, soils on the continental shelf, surrounding the peninsular coast line are rich in Montmorillonite, since rivers of the Deccan flow into the sea in this region.

Soils on the Indian continental shelf are either the familiar terrestrial sands, silts and clays as well as these soils modified by the sea environment, for example, through the development of cementatious bonds arising from the deposition of Calcium Carbonate or these soils are pelagic, originating either biogenetically from the skeletal remains of sea organisms or chemically through precipitation of chemicals and minerals in the sea water. These pelagic soils can then be further modified by the sea environment through mechanisms such as cementation.

On land, one has now learnt to expect some definite patterns of soil deposition. One expects to find coarse alluvial soils to be deposited by a river when it is flowing at high velocity and finer grained material when it flows gently. In the off-shore environment, one would expect coarser material near the coast line and finer soil as one moves seaward. This is indeed so, unless the sediment deposition process is interfered with by currents arising from thermal gradients, waves and tides. One then gets confused, as for example, at off-shore Madras where Rao

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**Professor of Civil Engineering, Indian Institute of Technology, Delhi, New Delhi - 110016, India.

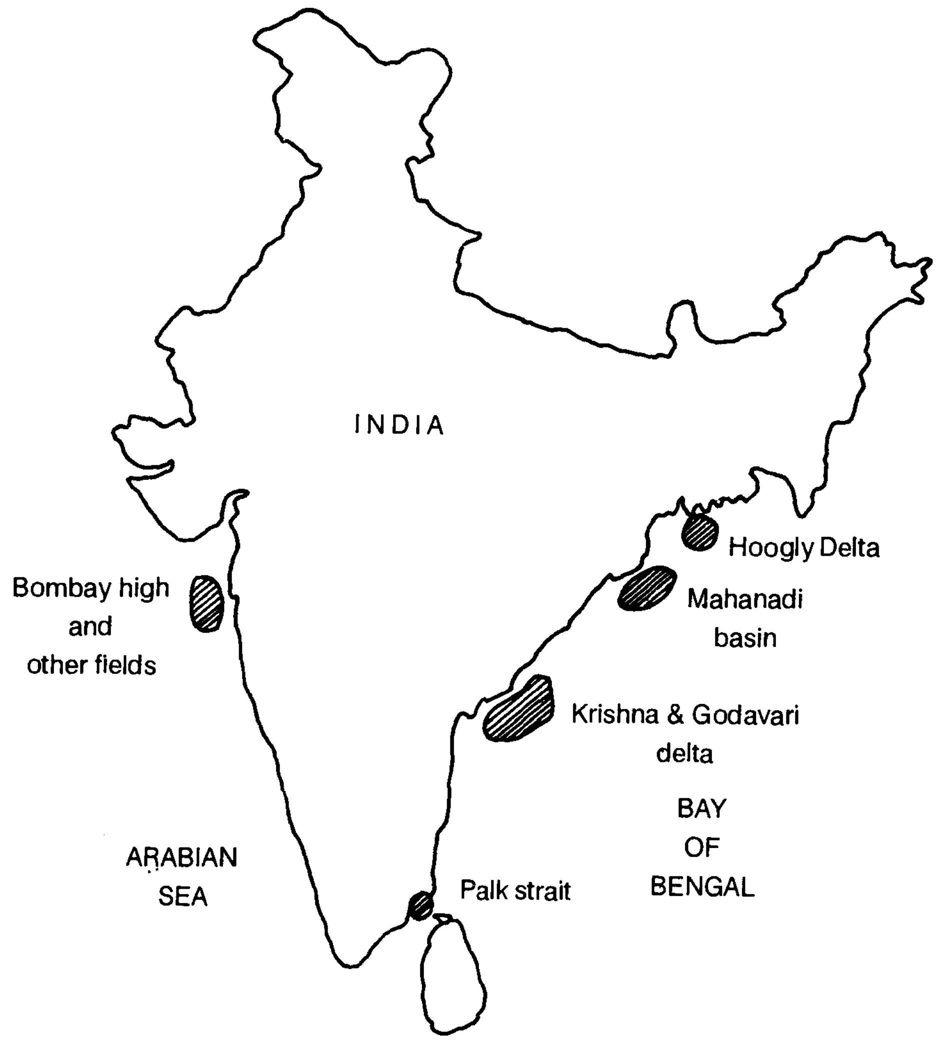

FIGURE 1. Sites Investigated by ONGC

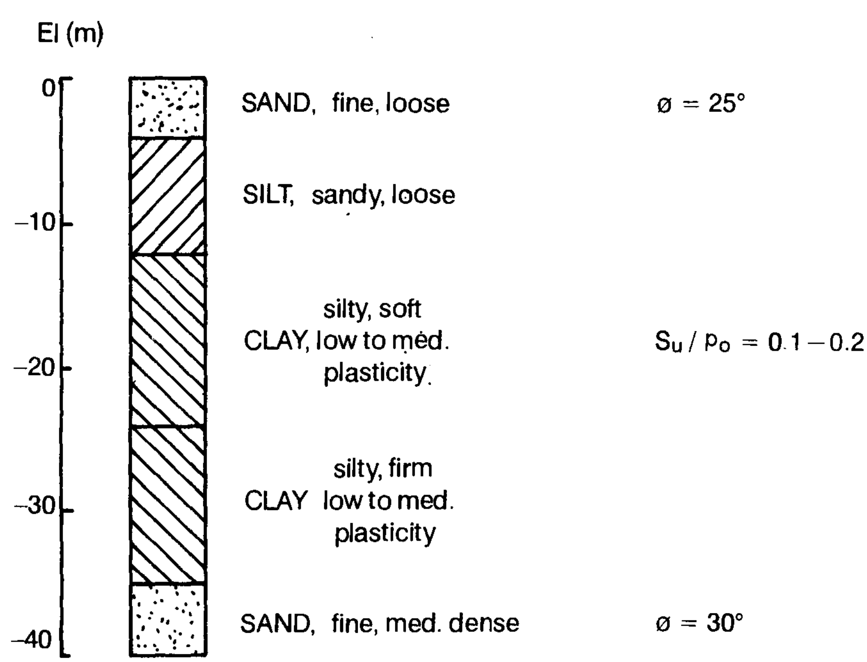

FIGURE 2. Profile Offshore Hoogly Delta

and Murthy (1968) report fine to very fine sand near the shore and near the outer part of the continental shelf, but in the middle part, they discovered medium to coarse sand.

NATURE AND DISTRIBUTION OF SOILS OFF-SHORE INDIA

What, then, are the kinds of soils that exist off-shore India and how are they distributed ? In the last 13/14 years, ONGC have commissioned the drilling of over 200 deep penetration bore holes. The bulk of them are in the Bombay-High region, but some have also been drilled at a number of places around the entire Indian coastline. These provide a wealth of information which has so far been inadequately studied primarily on account of the inaccessibility of the data since it is considered proprietary and confidential. From time to time, some data has been made available or has been published and a study of this reveals information and trends and these are summarized hereafter.

The five areas for which there is information, are shown in Fig. 1. In some areas only 2 or 3 bore holes have been drilled, whereas, in the region of Bombay-High and other fields there is information from more than 150 bore holes. Each area is considered in turn.

OFF-SHORE OF HOOGLY DELTA :

Fig. 2 presents an average profile evolved from very few bore holes drilled about 5 kms from the shore in water depth of about 14 m. Low to medium plasticity silty clay is the primary deposit at this site. The deposit is slightly underconsolidated and its strength varies from soft near the top to firm near the bottom. Traces of gas were found in the clay deposit and samples obtained from it expanded significantly upon retrieval.

OFF-SHORE OF MAHANADI BASIN :

From the data from a few bore holes drilled in water depths of about 100 m at a distance of about 20 kms from the coast one can deduce the average soil profile shown in Fig. 3 A very soft to soft clay layer, 2-6 m in thickness lies just below the mud line. The profile otherwise consists of a deposit of dense to very dense silica sand interrupted by a layer of variable thickness of stiff to very stiff, normally consolidated, medium plasticity clay which has gaseous zones in it.

OFF-SHORE OF GODAVARI AND KRISHNA BASINS :

At water depth of about 450 m, the soil profile is different just north of the Godavari Delta from that in front of the Godavari Delta, which is different from that in the zone between the Godavari and the Krishna Deltas and that in turn is again different from the soil profile in front of the Krishna Delta. In front of the deltas one encounters clays; in between them alternating layers of sand and clay and north of the Godavari Delta, one finds primarily

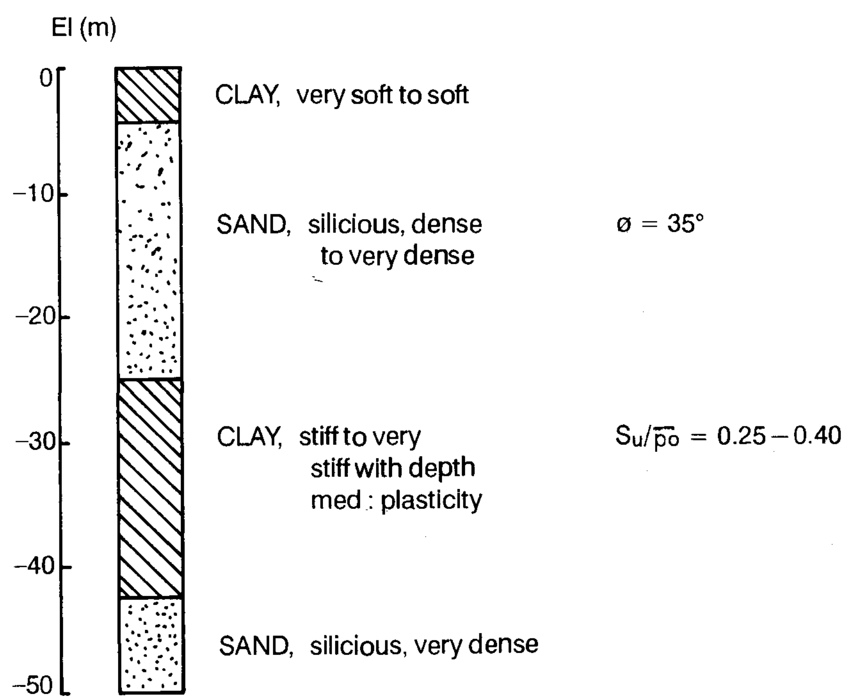

FIGURE 3. Profile Offshore Mahanadi Basin

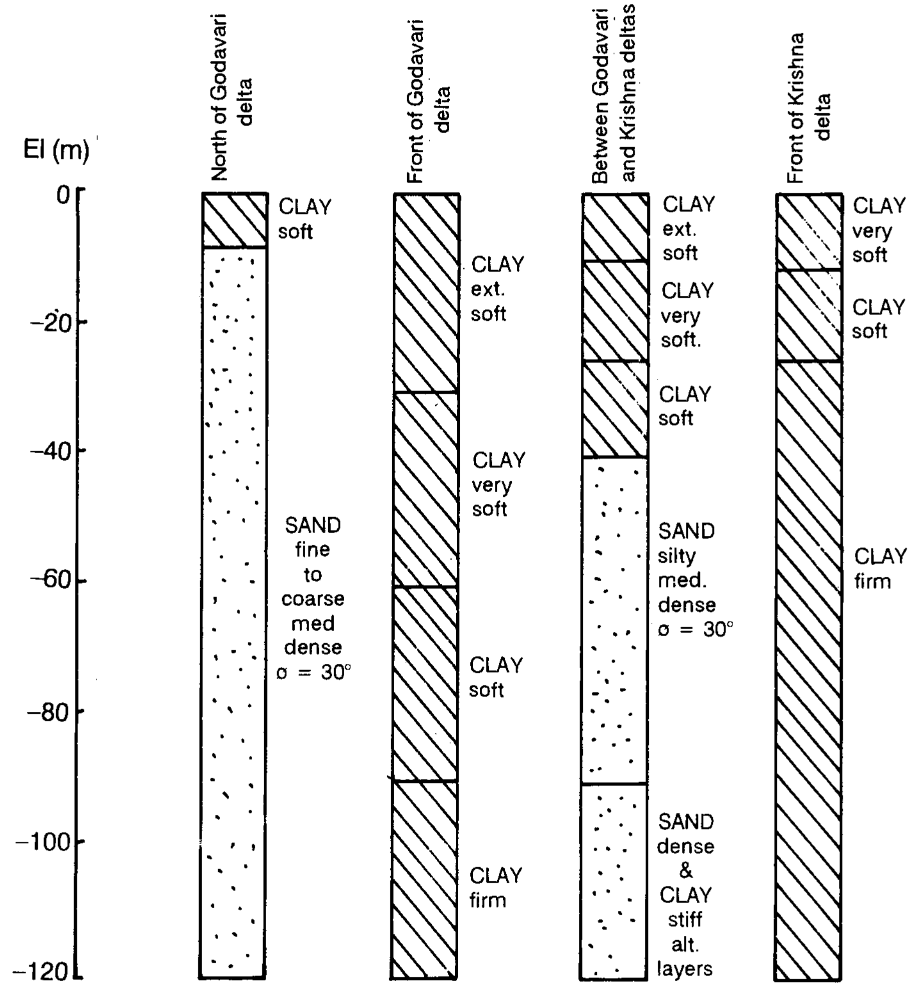

FIGURE 4. Profile Offshore Godavari and Krishna Deltas

sand. Fig. 4 depicts these profiles. Of considerable interest is the profile in front of the Godavari Delta which consists of clay which is very soft to soft down to a depth of almost 100 m and then it begins to get firm. The S_u/p'_o ratio, that is the ratio of undrained strength to the effective overburden pressure, for this clay ranges between 0.08 to 0.13 indicating a highly underconsolidated state. Gaseous zones were also observed in this clay.

PALK STRAIT

As one moves further south on the east coast, one encounters a marked change in the soil profile. At Palk Strait, in water depth of about 70 m, a few bore holes were drilled, 20 kms from the coast. The average profile is depicted in Fig. 5. Except for a thin soft clay layer near the surface, the site is characterised by high density, high strength, silty, sandy, calcareous clays, having $S_u/\bar{p}_o$ ratio in the range of 0.6 to 5.0 indicating a high degree of overconsolidation or more likely cementation.

BOMBAY-HIGH AND OTHER FIELDS :

The bulk of ONGC's investigation has been concentrated at Bombay-High and the nearby areas east of it that are designated as Bassein, Panna, Heera and Ratna. Each investigation further highlights the complexity of the soil profile in this region. One encounters a variety of soils, for example, calcareous sands, clays, calcareous clays, calcarenite and occasionally silica sands and even gravels. These soils are encountered with a wide range of strength. The clays especially near the mud line, are often very soft to soft; they become firm with depth and are usually normally consolidated. Both clays and sands are encountered with varying degrees of cementation. At great depths, sands are often so highly cemented that they are rock-like. Sand and clay layers usually alternate with depth at Bombay-High, but their thicknesses vary enormously. Since closely spaced bore holes reveal different profiles, it is impossile to predict the lateral extent of any layer. At the other nearby areas of Bassein,

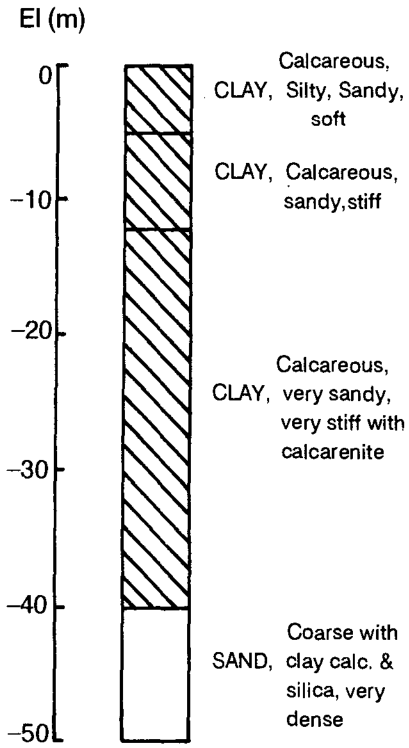

FIGURE 5. Profile at Palk Strait

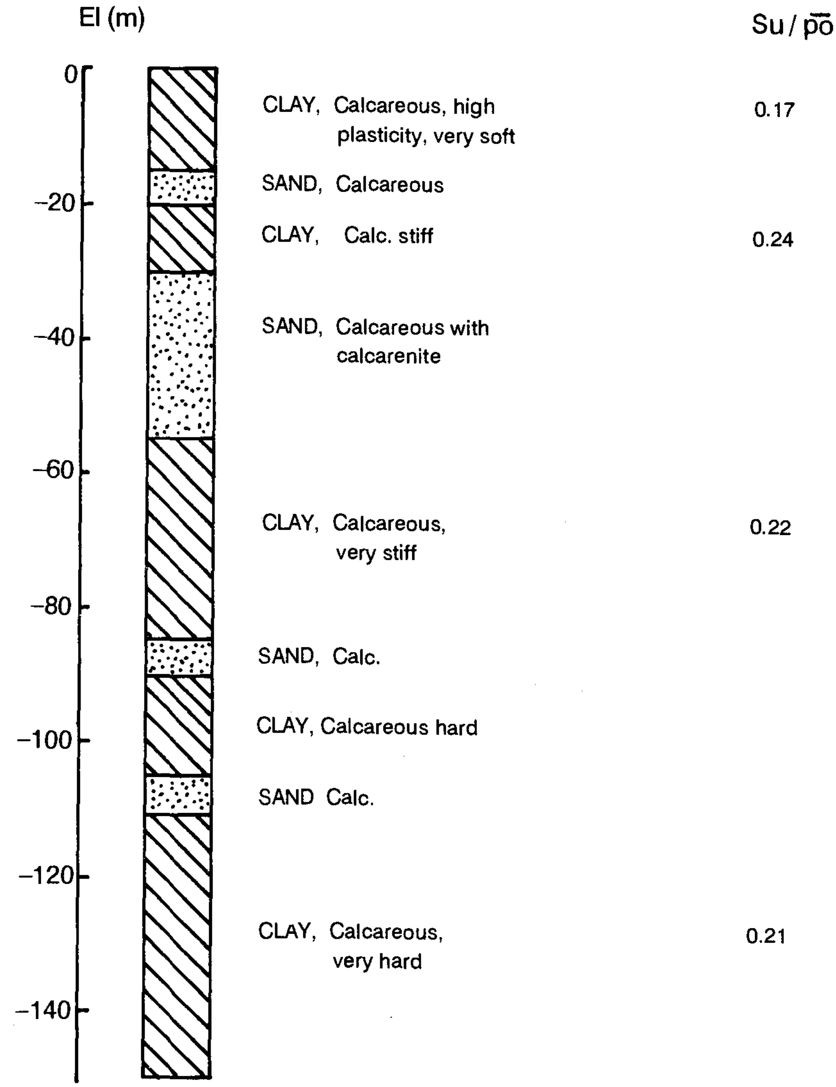

FIGURE 6. Profile at Bombay – High

Panna, Heera and Ratna, clays and calcareous clays dominate the profile more than sands. These clays appear to be normally consolidated. One connot present an average profile of this region, but only a typical one; Fig. 6 presents such a typical profile at Bombay-High.

From the infromation available, one ought not to generalize and yet without making some generalizations, it is not possible to develop any picture, howsoever vague, about the kinds of soils that exist on India's continental shelf. Let the following, therefore, be treated not as generalizations, but as sweeping generalizations :

1. Soils off the west coast have more carbonate content than those off the east coast.
2. Clays off the west coast are usually normally consolidated whereas many deposits off the east coast are underconsolidated and the rest usually normally consolidated.
3. Soil profiles off the west coast are complex alternating layers of different soil types whereas 3 or 4 distinct strata constitute the profile at many sites off the east coast.
4. Rock-like material is encountered off the west coast at not too great a depth whereas such material has not been encountered off the east coast.
5. The phenomenon of sample expansion due to presence of gaseous zones is not observed in samples from off the west coast, but is observed in samples from off the east coast.
6. The soils of the southern tip of India are calcareous, hard, overconsolidated and cemented.

One fears that too much has been stated in these six statements. It would be worthwhile to keep establishing the veracity or otherwise of these statements as more data from off-shore investigations accumulates.

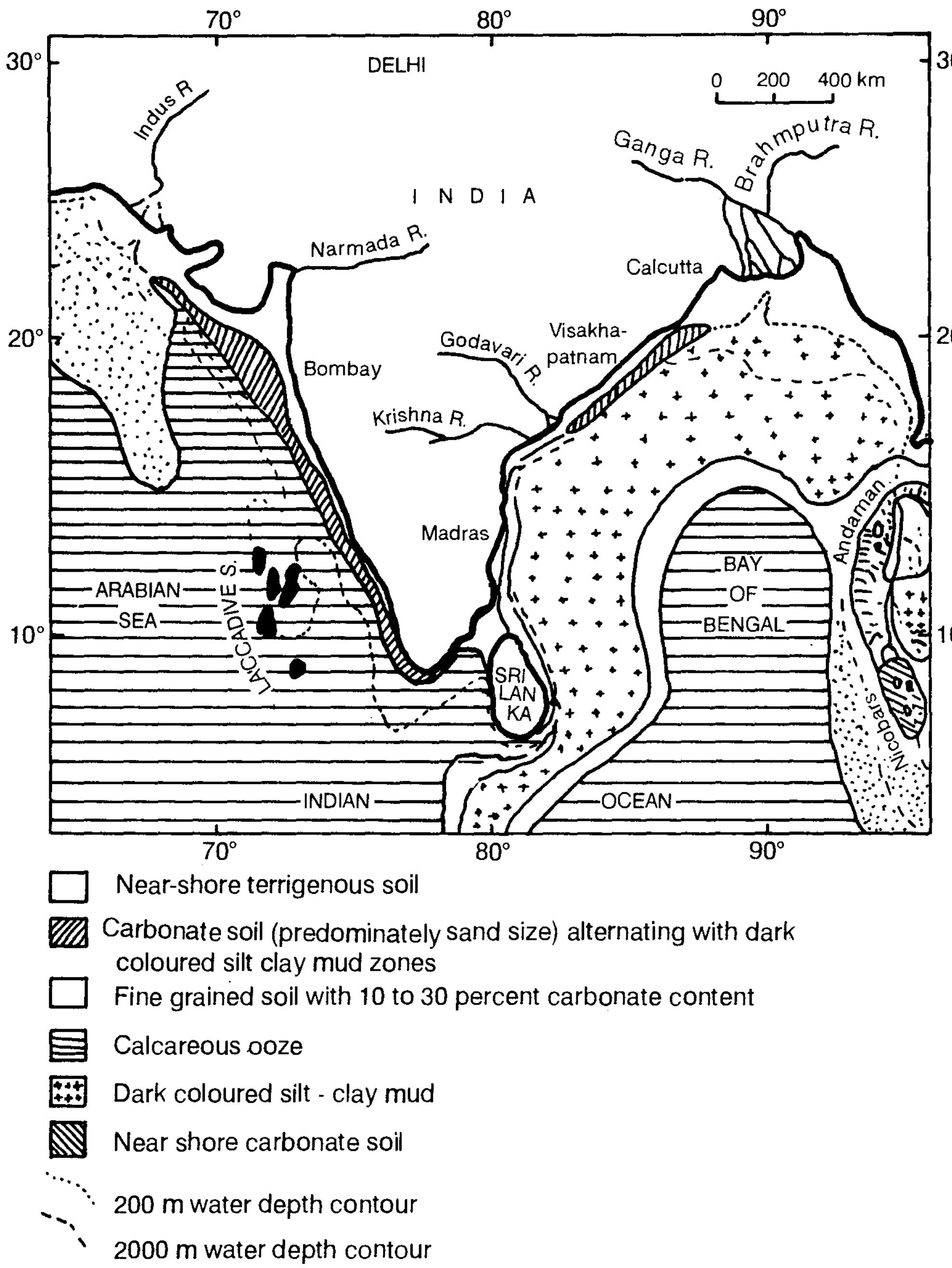

FIGURE. 7 Distribution of Soil Around India - After Datta et. al. (1982)

Apart from exploratory work commissioned by ONGC, one finds in literature studies on surface soils of the sea bed, for example, Subba Rao (1964), Nair and Pylee (1967), Schott (1967), Seetharamaswamy (1967), Hasmi and Nair (1976) as well as the atlas put out after the International Indian Ocean Expedition (1975). Datta et. al. (1982) have summarised this information in a simplified form as a map of surface sediments, see Fig. 7. Somewhat more detailed maps of surface sediments on the west coast and in the vicinity of Visakhapatnam are shown in Figs. 8 and 9. These attest to the complexity of the pattern of soils encountered when viewed on the lateral plain.

Apart from soils, the Indian off-shore environment is also characterised by the presence of Corals. Deshmukh et. al. (1985) have collected samples of Corals from numerous sites, see Fig. 10. Fig. 11 shows the presence of submerged and emerged Coral reefs in the Gulf of Kutch. Apart from such fringing reefs, there also exist in the Indian off-shore environment, barrier reefs and atolls, i.e., the other two land forms associated with Coral reefs.

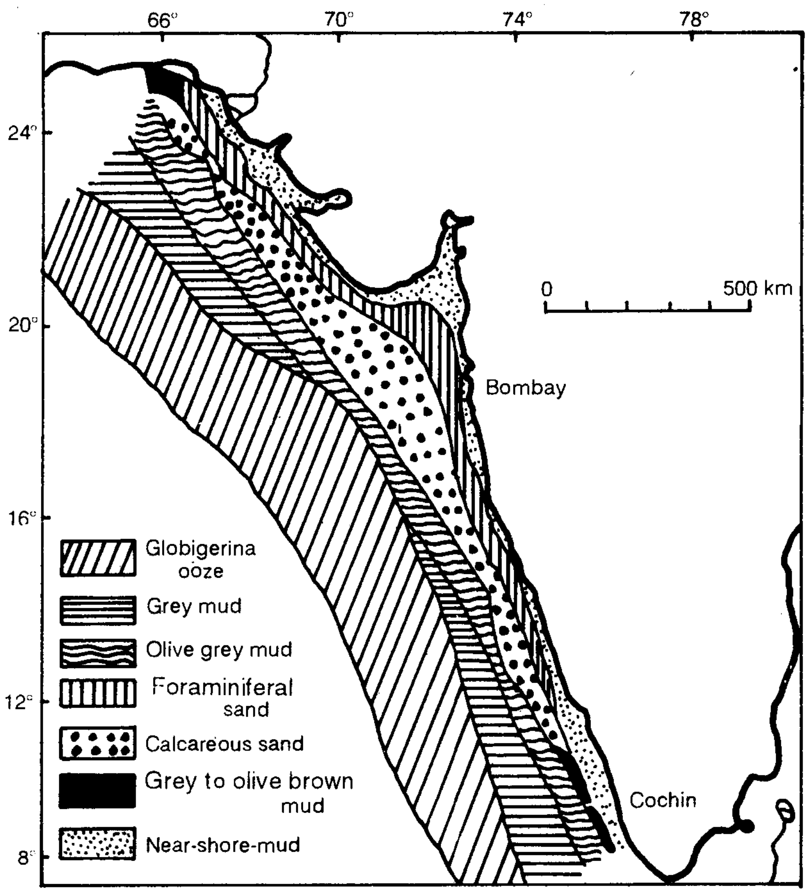

FIGURE 8. Distribution off Soils of the West Coast of India – After Schott (1967)

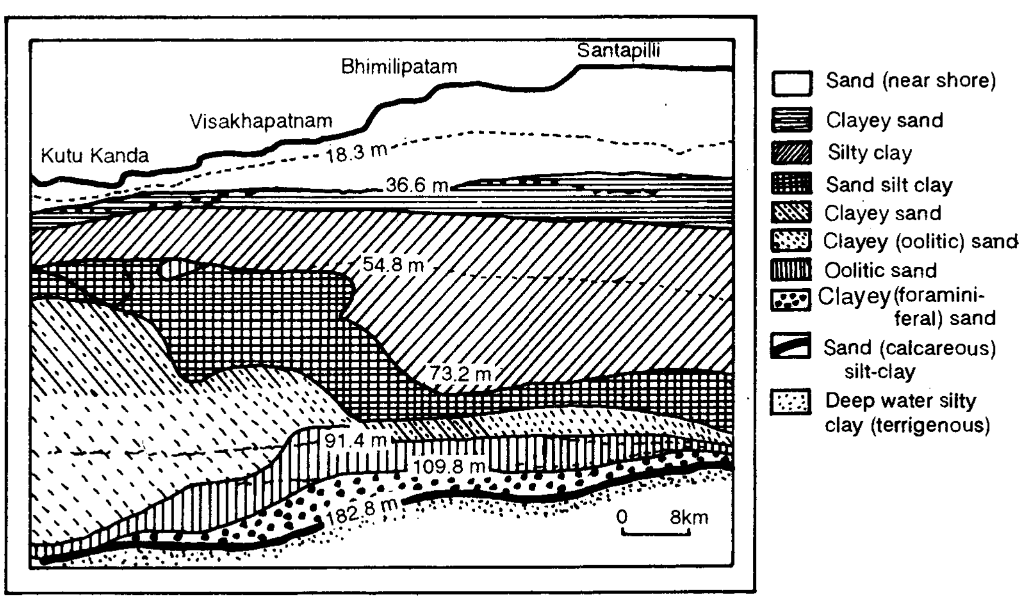

FIGURE 9. Distribution of Soils Off the East Coast of India Near Visakhapatnam – After Subba Rao (1964)

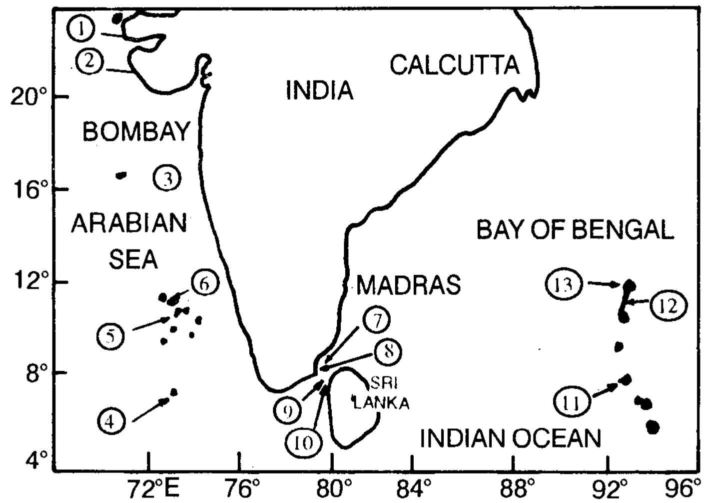

FIGURE 10. **Location of Corals in Indian Waters—After Deshmukh et. al. (1985 (1) Narara Bet, Gulf of Kutch, (2) Okha, Gulf of Kutch, (3) Malvan, Central West Coast, (4) Minicoy, Lakshadweep Islands. (5) Kavaratti, Lakshadweep Islands, (6) Kiltan, Lakshadweep Islands. (7) Mandapam, Palk Bay, (8) Pamban, Palk Bay, (9) Hare Island, Gulf of Mannar, (10) Krusadai Islands, Gulf of Mannar, (11) Car Nicobar, Nicobar Island (12) Maya Bunder, Andaman Islands. (13) Aerial Bay, Andaman Islands.**

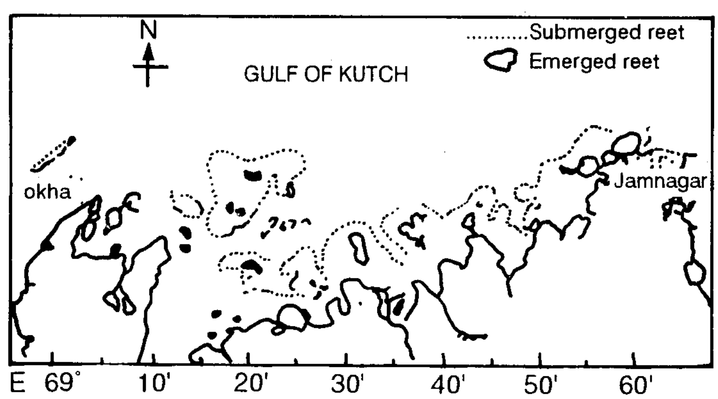

FIGURE 11. **Distribution of Coral Reefs in Gulf of Kutch—After Deshmukh et. al. (1984)**

NATURE & BEHAVIOUR OF CALCAREOUS DEPOSITS

From the discussion in the previous section, it is apparent that a major distinguishing feature of the soils encountred off-shore India, is the high calcareous content. It is a well-established fact that calcareous soils are encountered between latitudes 30 degrees north and 30 degrees south. Calcareous soils owe their origin to the accumulation of bioplastic debris, that is, the skeletal remains of biogenic population of the sea, both plants, for example, coralline algae and animals, for example, foraminifera etc. Apart form such skeletal deposits one also encounters non-skeletal deposits resulting perhaps from precipitation. In the Indian off-shore environment, one notes the presence of calcareous sands, of calcareous clays and of Corals. Each of these three has been the subject of detailed study at the Marine Geotechnology Laboratory at IIT Delhi. The nature and the behaviour of each is considered in turn hereafter.

CALCAREOUS SANDS

For investigating the nature and behaviour of calcareous sands, samples were obtained from Bombay-High from a depth of 50 m below the mud line, from surface deposits on the west coast of India and from surface deposits of coralline debris from an island in the Arabian Sea. Ottawa sand was used as a reference material for comparison purposes.

That calcareous sands are different from silica sands becomes apparent merely by looking at the grains

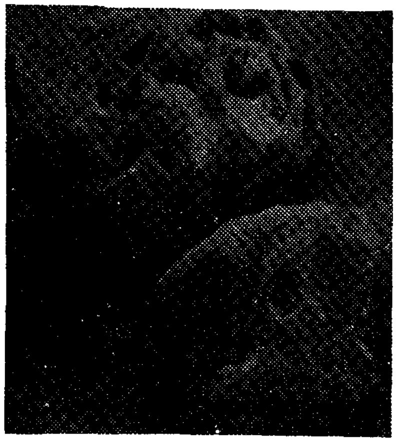

FIGURE 12
Calcareous SandGrain with Intraparticle Voids-After Datta(1980)

FIGURE 13
Calcareous SandGrain Platey in Shape - After Datta(1980)

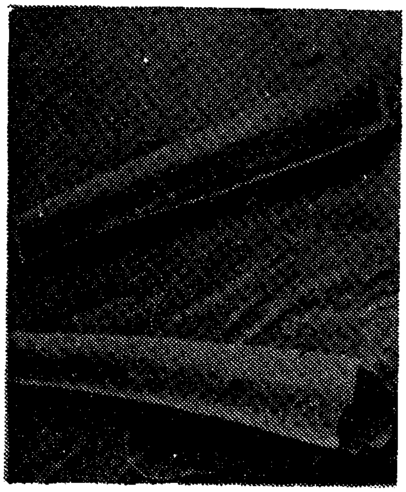

FIGURE 14
Calcareous SandGrain hollow Needless and Roled up Particle After Datta(1980)

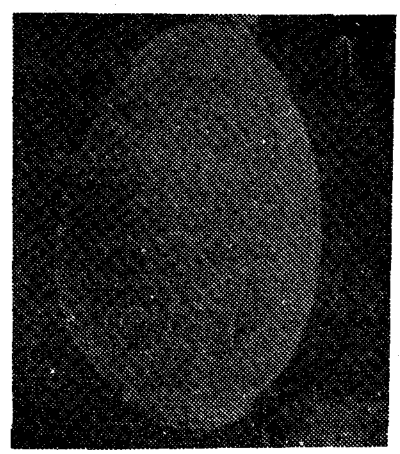

FIGURE 15
An Oolite Particle of Calcareous Sand-After Datta (1980)

of calcareous sand. One observes that these grains have intra-particle voids, see Fig. 12; they are often platey in shape, see Fig. 13 or are shaped as hollow needles or rolled up particles, see Fig. 14; some particles are, however, similar to particles of silica sand as is evident from Fig. 15 which is an oolite from the Bombay-High region.

The nature of these particles suggests that some of them will tend to crush when they are subjected to stresses and are likely to have different susceptibilities to crushing. In order that one may quantitatively express the extent of crushing, a Crushing Coefficient was defined as follows :

$$C_c = \frac{\text{\% of post stressed sand finer than } D_{10} \text{ of original sand}}{\text{\% of original sand finer than } D_{10} \text{ of original sand}}$$

$$C_c = \frac{\text{\% of post stressed sand finer than } D_{10} \text{ of original sand}}{10}$$

Fig. 16 depicts how C_c can be determined from the grain size distribution curves of the original sand and of that sand after it has been subjected to stress. A Crushing Coefficient of 1.0 indicates no crushing at all and as crushing increases so does the Crushing Coefficient. Fig. 17 depicts the Crushing Coefficient determined after subjecting 5 different sand samples to drained triaxial shear at different confining pressures in a plot of Crushing Coefficient vs. the confining stress. It is apparent from Fig. 17 that Ottawa sand, designated as T, does not crush

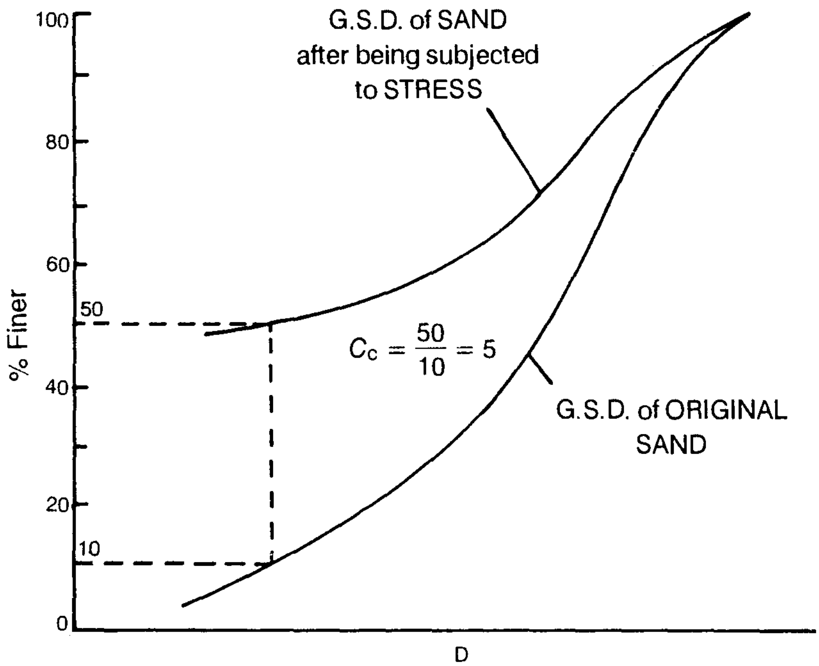

FIGURE.16 Method of Determining Crushing Coefficient - After Datta et. al. (1980)

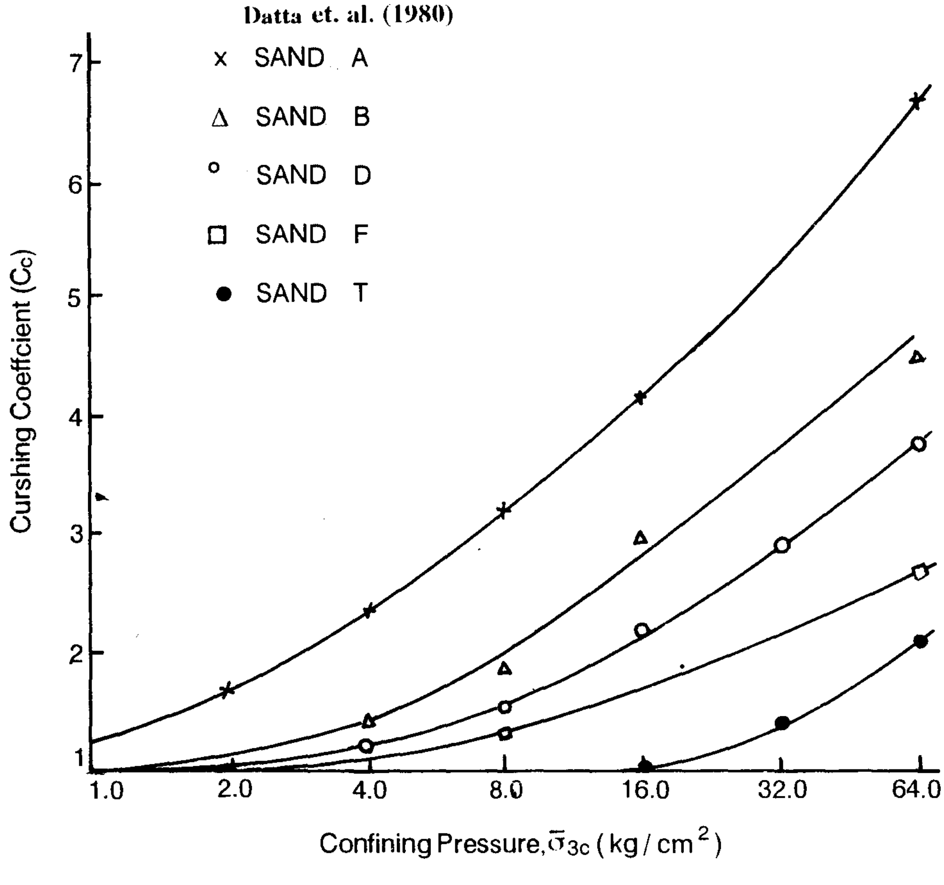

FIGURE 17. Effect of Confining Pressure on Post-Shear Crushing Coefficient in Drained Tests--After Datta et. al. (1980)

when sheared at confining stresses less than 16 Kg/cm². The 4 other curves for different calcareous sands, all exhibit much higher crushing. For convenience one can define the Susceptibility to Crushing, S_c, as the magnitude of the Crushing Coefficient after a sample has been subjected to drained shear at a confining stress of 64 Kg/cm². Table-1 presents the characteristics of the four calcareous sands that were subjected to study along with the characteristics of Ottawa sand, designated as T.

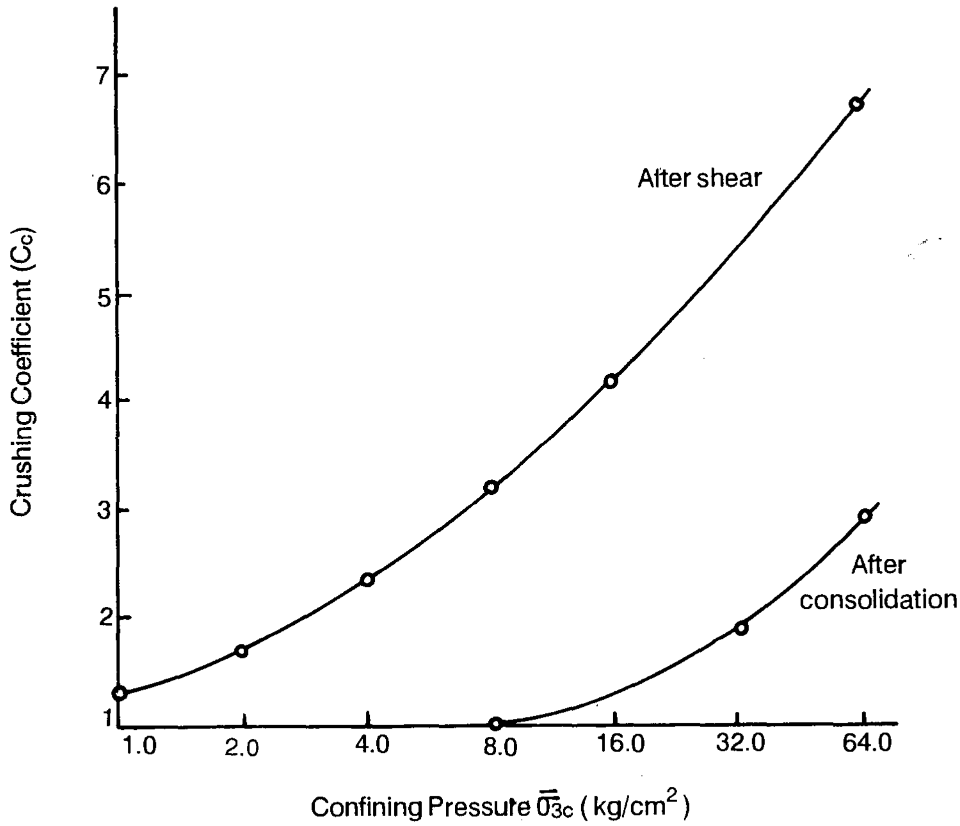

FIGURE 18. Effect of confining Pressure on Crushing Coefficient for Sand A After Consolidation and After Drained Shear—After Datta et. el. (1980)

Consider, first, the behaviour of these sands under drained loading. Fig. 18 presents the Crushing Coefficient of sand A subjected to only confining stress and also subjected first to confining and then shear stress upto failure, from which it is apparent that shear stresses induce much greater crushing than merely normal stresses. The stress-strain curves obtained for sand A when it is sheared under different confining stresses, are presented in Fig. 19; sand A, it will be recalled, has a very high Susceptibility to Crushing. From Fig. 19 it is apparent that as confining stress increases during drained shear, there is a reduction in the principal effective stress ratio, alteration of volume change behaviour from one of dilation to that of volume reduction, thereby changing the behaviour from that of a brittle material to one characteristic of a plastic material and that there is increase in the strain at failure. Similar tendency is observed even in Ottawa sand but only when it begins to crush at a confining stress of 64 Kg/cm^2 as is evident from Fig. 20. The influence of difference in the Susceptibility to Crushing on maximum principal effective stress ratio with confining stress is depicted in Fig. 21, where one notes that this ratio is constant for Ottawa sand until a high confining stress but is steadily reducing with confining stress for skeletal calcareous sands. That the maximum effective stress ratio reduces with confining stress, of course, implies that the failure envelope for calcareous sands is not a straight line but a concave downwards curve.

TABLE 1

CHARACTERISTICS OF SANDS STUDIED

Sand	Carbonate Content %	Origin	Particle Size	Coefficient of Uniformity	Susceptibility to Crushing S_c
A	92.2	Skeletal	Coarse	1.50	6.70
B	93.7	Skeletal	Coarse to medium	2.11	4.80
D	95.0	Skeletal	Medium to fine	1.53	3.75
F	90.1	Non-skeletal	Medium to fine	1.90	2.65
T	0.0	Terrestrial	Medium to coarse	1.33	2.10

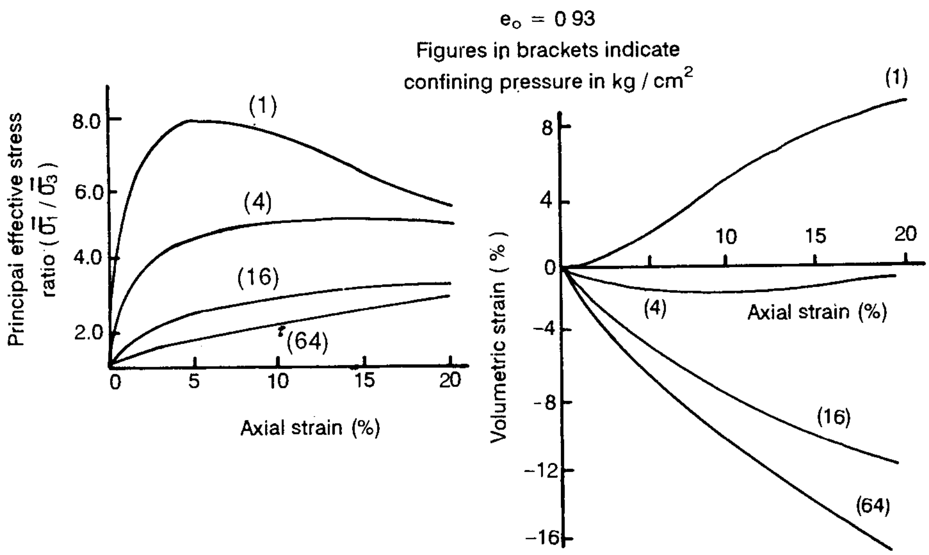

FIGURE 19. Stress-Strain-Volume Change Behaviour for Sand A—After Datta et. al. (1980)

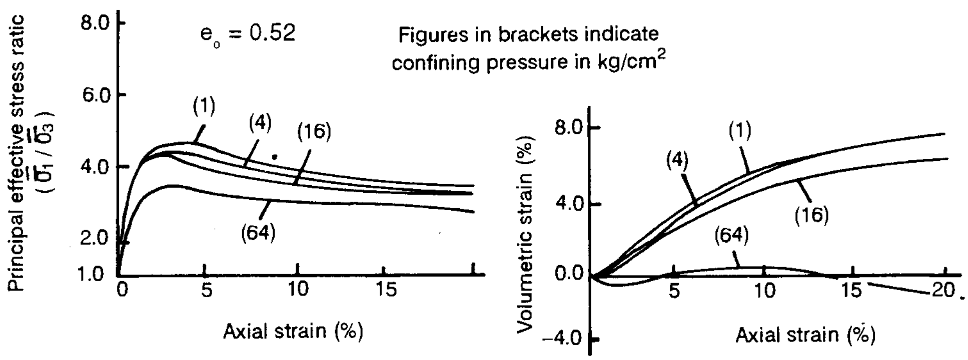

FIGURE 20. Stress-Strain-Volume Change Behaviour for Sand T—After Datta et. al. (1980)

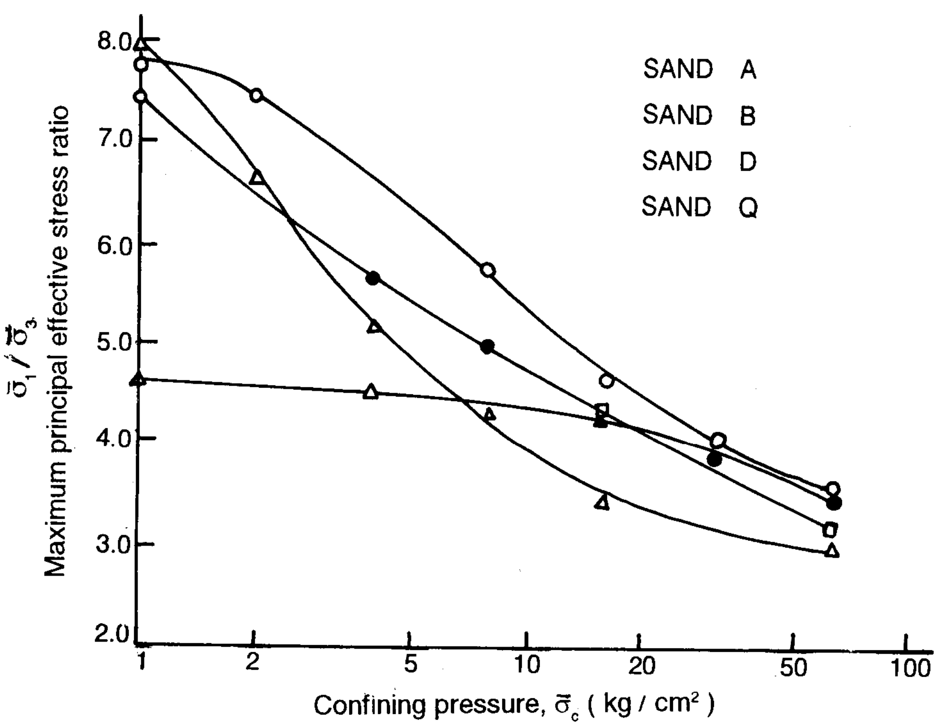

FIGURE 21. Influence of Confining Pressure on the Maximum Principal Effective Stress Ratio—After Datta et. al. (1980)

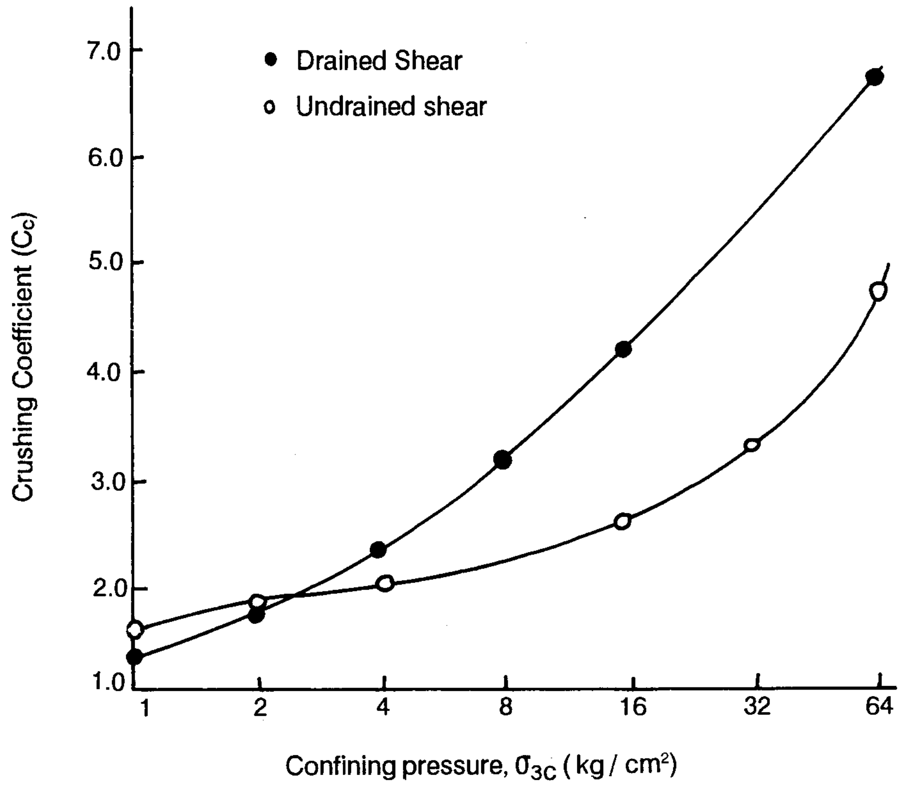

FIGURE 22. Variation of Crushing Coefficient with Confining Pressure for Sand A—After Datta et. al. (1979)

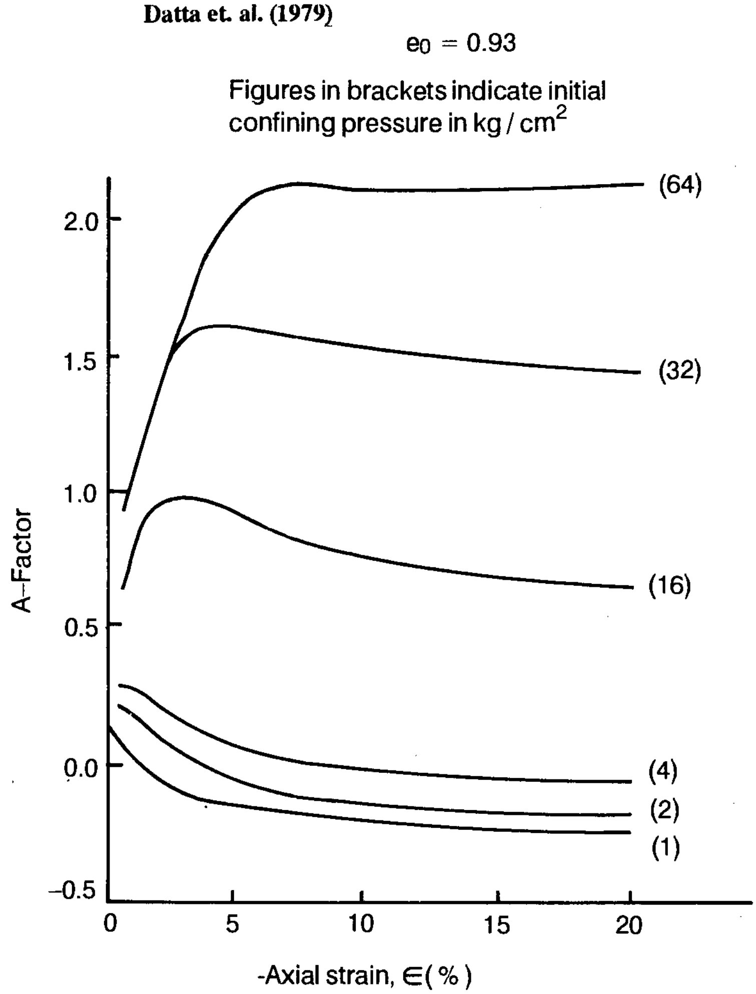

FIGURE 23. Variation of A—Factor with Strain for Sand A—After Datta et. al. (1979)

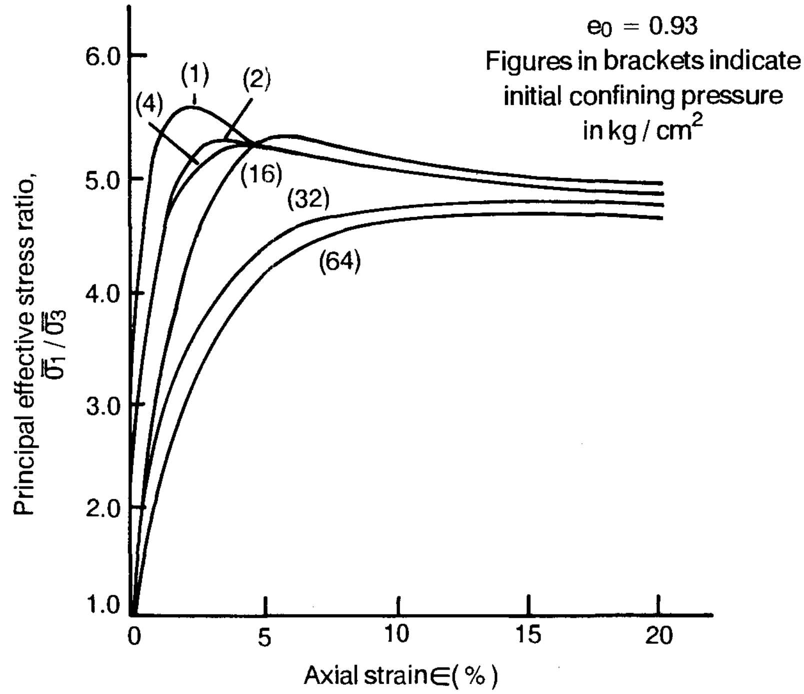

FIGURE 24. Variation of Principal Effective Stress Ratio with Strain for sand A—After Datta et. al. (1979)

Shearing under undrained conditions, when volume changes are not allowed to occur, is not as effective in causing particle crushing as shearing under drained conditions; this is evident from Fig. 22. The influence of particle crushing under undrained shear is reflected primarily in the pore water pressures that develop during shear. Fig. 23 shows how the A-factor for sand A changes from negative values at low confining stress to very high positive values at high confining stress. Apparently as particles crush, stress is transferred to the pore water. The principal effective stress ratio vs. axial strain relation is not significantly affected as is evident from Fig. 24. When samples are sheared at low confining stress, the effective stress path resembles the path for overconsolidated clays, see Fig. 25. At higher confining stresses, the shape changes to that of effective stress paths for normally consolidated soil indicating, again, a change in behaviour from that corresponding to a dilatent brittle material to that suggestive of a plastic material. This is, of course, consistent with the behaviour indicated earlier for the drained situation.

It has been pointed out that crushing induced by normal stresses is less than that produced by undrained shear which, in turn, is less than that produced under drained shear. An attempt was made to see if crushing could be related to the energy imparted to the soil when it is stressed. For sands A and D data presented in Fig. 26 indicates that indeed for each sand there is a unique relation between the Crushing Coefficient and the energy input. In soils with different Susceptibilities to Crushing, different energy input is obviously required to cause crushing of a comparable amount. Since sand A has higher Susceptibility to Crushing than sand D, one observes from Fig. 26 that the curve between Crushing Coefficient and energy input for sand A is located significantly above the curve for sand D.

Let C_{cr}, i.e., the Critical Crushing Coefficient, be defined as the Crushing Coefficient at which the soil behaviour is altered from dilatent to plastic, or more specifically, when in drained shear, the volume change

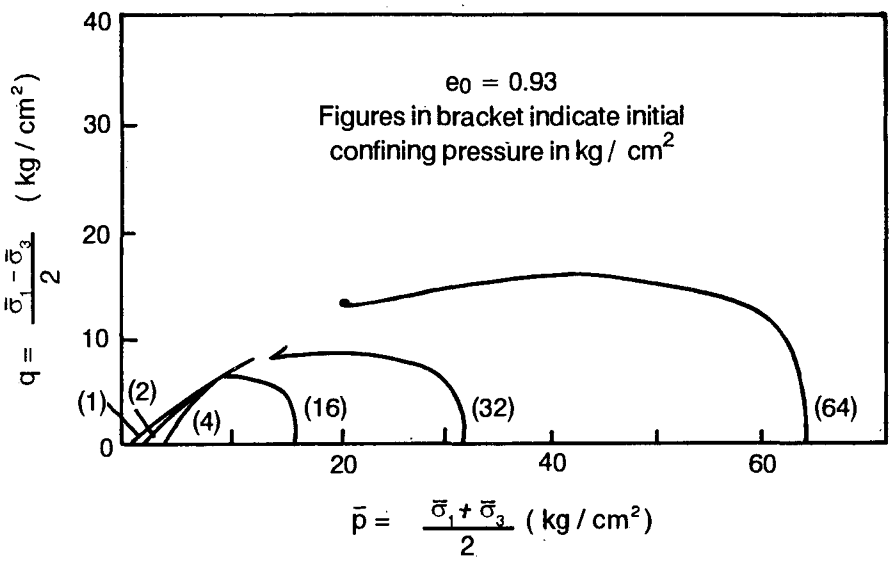

FIGURE 25. Effective Stress Paths for Sand A—After Datta et. al. (1979)

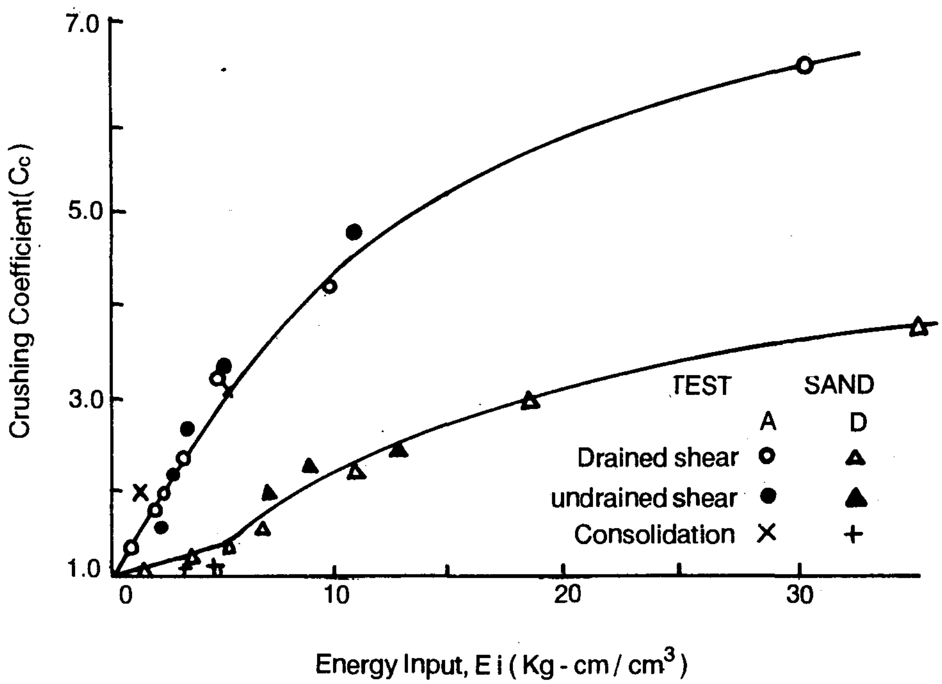

FIGURE 26. Variation of Crushing Coefficient with Energy Input—After Datta et. al. (1980a)

induced alters from volume increase to volume decrease and in undrained shear, when pore water pressure induced alters from negative to positive. Fig. 27 shows the relation of such a Critical Crushing Coefficient and the Susceptibility to Crushing for all the five sands tested both under drained and under undrained shear. One notes from this figure that the Critical Crushing Coefficient lies in a very narrow range from 1.4 to 2.5 and is independent of the Susceptibility to Crushing. The implication of this is, of course, that it requires a certain critical amount of crushing to alter soil behaviour. This crushing in sands with a high Susceptibility to Crushing occurs at low stresses whereas in sands with a low Susceptibility to Crushing occurs at very high stresses.

In order to be able to appreciate the behaviour of calcareous sands under stresses imposed during pile driving, a series of tests were conducted in which dense calcareous sands were subjected to a number of cycles of pulsating compressive stress at different stress levels. The stress level, τ/S_u, is defined as the shear stress applied by each pulse, τ, to the undrained strength of the sand sample under static loading. Fig. 28 (a) indicates the pore water pressure developed when the sample is subjected to different number of cycles of pulsating stress. One notes that even for sand at a very dense state in which under static stress negative pore water pressures

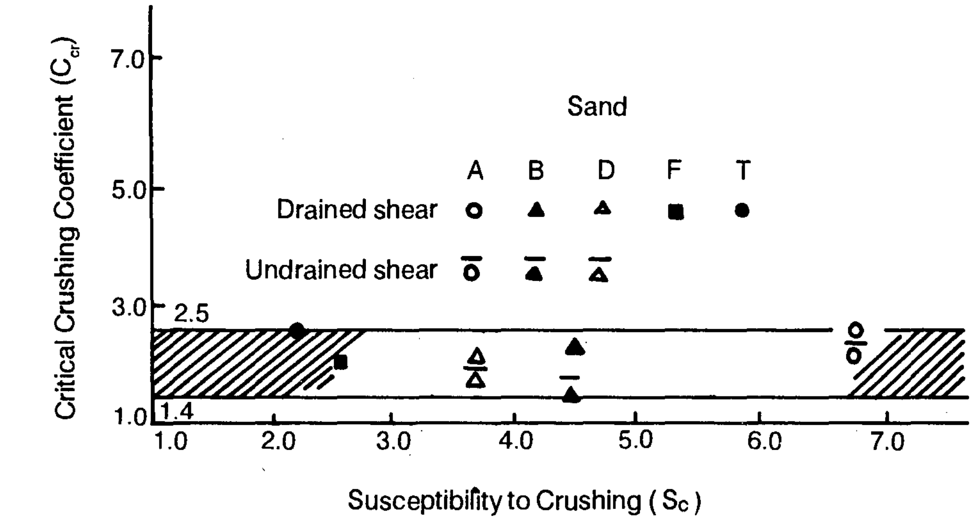

FIGURE 27. Relation Berween Critical Crushing Coefficient and Susceptibility to Crushing—After Datta et. al. (1980a)

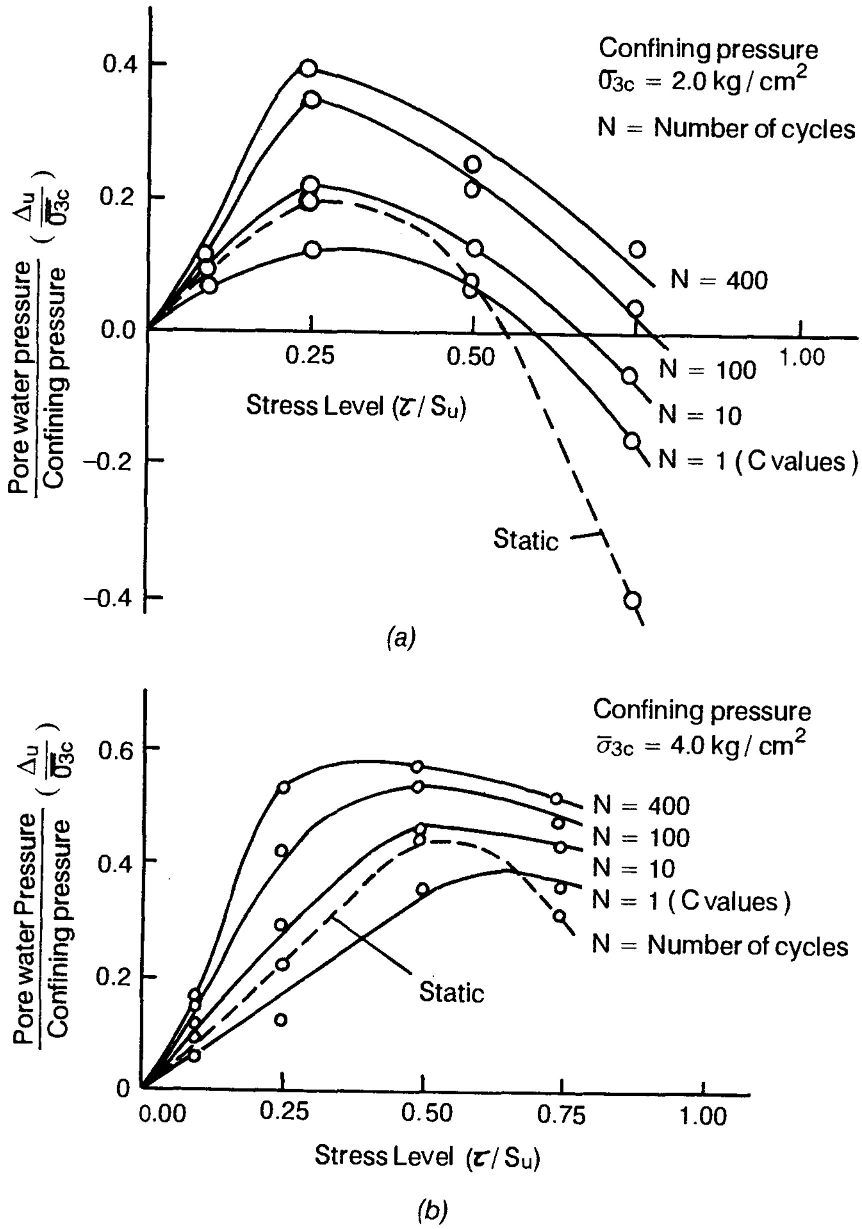

FIGURE 28. Variation of Induced Pore Water Pressure with Stress Level—After Datta et. al. (1980b)

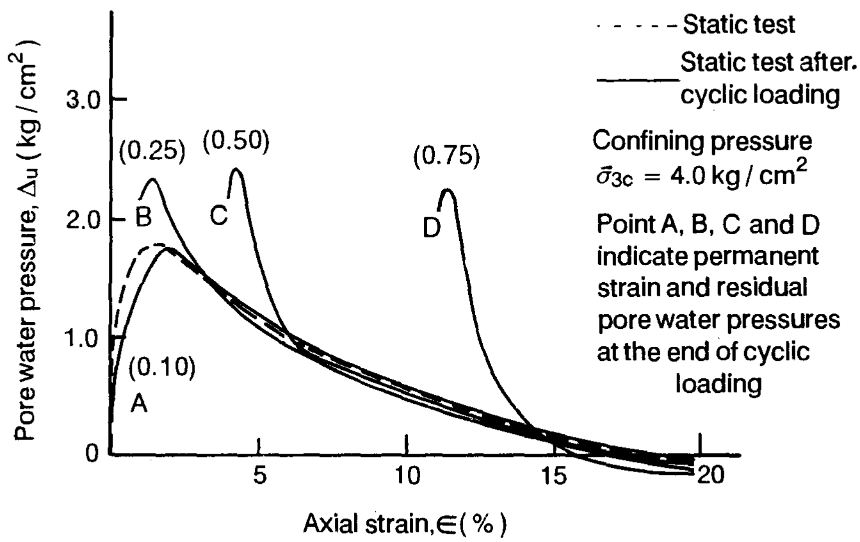

FIGURE 29. Static Pore Water Pressure Strain Behaviour After Cyclic Loading After datta et. al. (1980b)

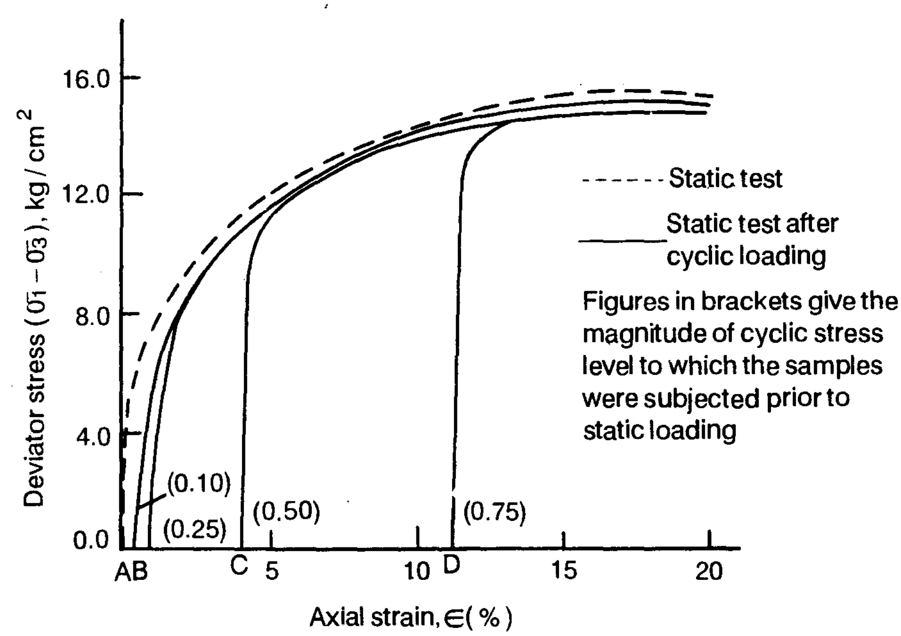

FIGURE 30. Static Stress-Strain Behaviour After Cyclic Loading—After Datta et. al. (1980b)

develop, pulsating stresses induce positive pore water pressures. The pore water pressures under static loading are quite similar to those induced by the first cycle of pulsating load. Thereafter, as the number of cycles of pulsating load increase, positive pore water pressures are induced and for 400 cycles the final pore water pressure values are positive for stress levels up to 0.75. Fig. 28 (b) is similar to Fig. 28 (a) except that the initial confining pressure is 4 Kg/cm² and that even under static loading pore water pressure induced is positive. The pore water pressure-axial strain relation when the sample is sheared under static conditions after cyclic loading is shown in Fig. 29, from which it is evident that the pore water pressure again joins up with the pore water pressure relation obtained under static loading alone. Similar behaviour is evident in Fig. 30 for the deviator stress-axial strain relation. It is apparent, therefore, that pulsating stresses generate positive pore water pressure which would reduce the effective stress. Under pile driving conditions one can, as such, expect relatively low resistance during pile driving but after the pile has been driven, the resistance of the pile would increase as pore water pressures dissipate; such observations have indeed been recorded during construction, see Agarwal et. al. (1979).

CALCAREOUS CLAYS

Calcareous Clays have been studied by Moore (1964), Buchan et. al. (1967), Einsele (1967), Kogler(1967), Kelly et. al. (1974), and since calcareous clays are different from other clays because they contain Calcium

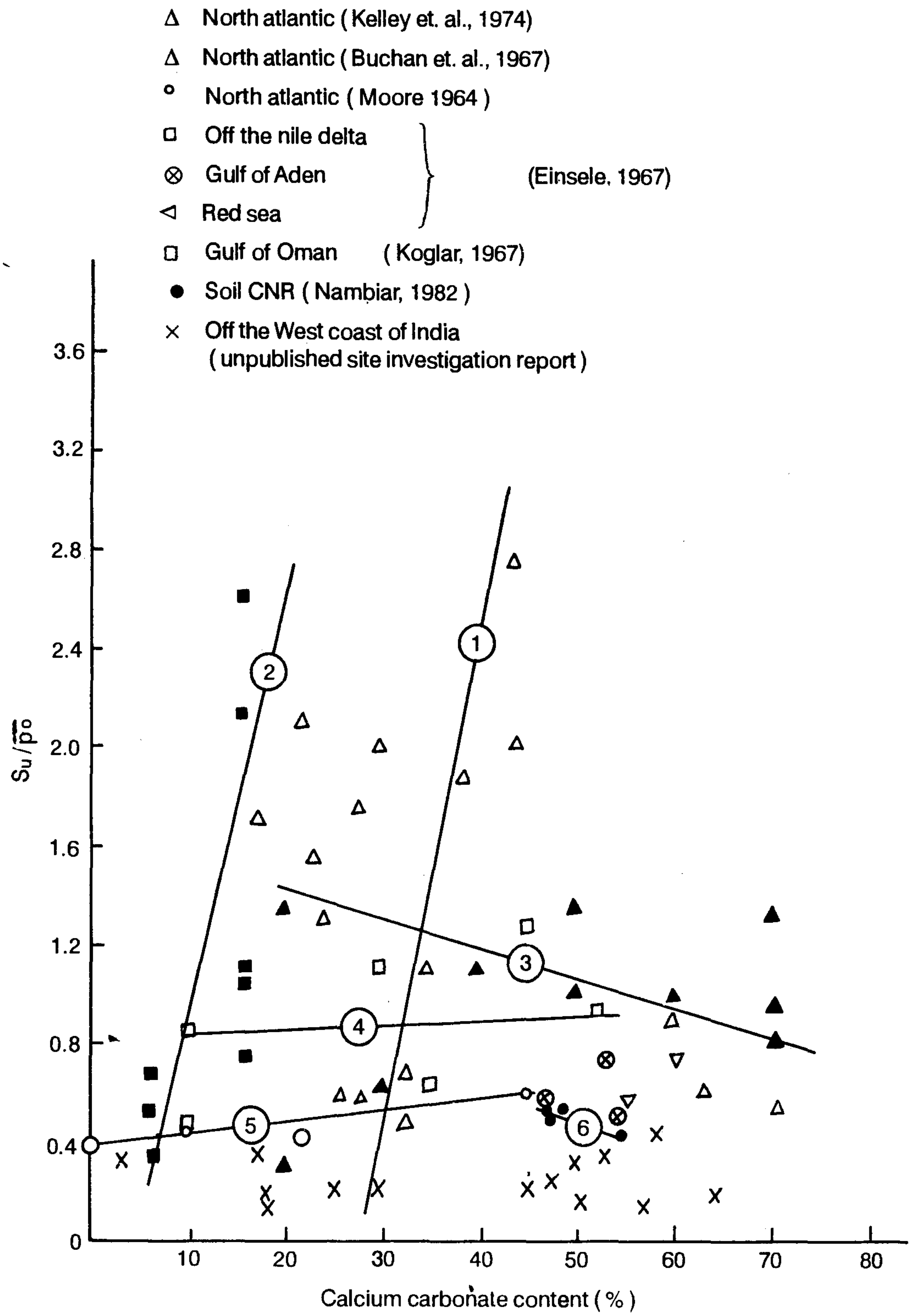

FIGURE 31. Variation of Su/ po with Calcium Carbonate Content—After Nambiar (1982)

Carbonate, these researchers attempted to correlate engineering behaviour of these clays with the Calcium Carbonate content. Such attempts, however, did not prove to be very successful, see, for example, Fig. 31, where the undrained strength over the effective overburden pressure ratio has been plotted vs. the Calcium Carbonate content and all one obtains is a mass of data showing little correlation.

Just as investigations on calcareous sand became meaningful, once it was recognized that it is not the content of Calcium Carbonate that is an important parameter, but the Susceptibility to Crushing of calcareous sand grains, studies on calcareous clay become meaningful when one notes, again, that it is not the Calcium Carbonate content that is the relevant parameter, but what is important is the form of existence of Calcium Carbonate in the clays.

Nambiar et. al. (1985) present a detailed investigation of a calcareous clay from off the west coast of India. This clay is encountered in a thick layer from about 5m to 30m below the mud-line. Its Calcium Carbonate content is of the order of 50 per cent. Scanning electron micrographs revealed that the Calcium Carbonate existed in the form of inert needles in the size range of silt and clay particles, see Figs. 32 & 33. Investigation of the index and engineering properties of this soil in its undisturbed state as well as after removal of Calcium Carbonate are presented in Table-2 and Table-3. A study of these two tables indicates that the removal of Calcium Carbonate makes only marginal changes in the properties of this clay, what in fact one would expect on account of removal of inert particles. In its undisturbed state, the clay is in a normally consolidated state and the presence of Calcium Carbonate is contributing essentially as would the presence of non-colloidal particles of silt and clay size.

In many clays Calcium Carbonate can exist in an another form, that is, as a cementing agent. Undisturbed samples of such material were not available for investigation. Nambiar (1982) reports a study in which Calcium Carbonate was deposited as a cementing agent in Kaolinite in the laboratory by sequentially permeating under small heads through a sample of Kaolinite, 1.35 M solution of Calcium Chloride for 2-3 weeks, followed by distilled water for one day, followed by 3-7 weeks of 1.32 M solution of Sodium Carbonate and finally followed by distilled water for one week. With this procedure, depending upon the time for which calcium Chloride and Sodium Carbonate solutions were permeated, Calcium Carbonate was deposited to the extent of 2.6 to 4.7 per cent. A study was then made to compare engineering behaviour of Kaolinite without any cementation and Kaolinite in

FIGURE 32

Calcareous Clay with Calcium Carbonate Evident as Needles After Nambiar (1982)

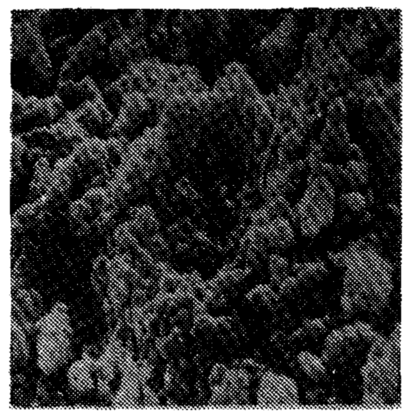

FIGURE 33

Calcareous Clay After Removal of Calcium Carbonate - Needles No Longer Present - After Nambiar (1982)

TABLE 2

INDEX PROPERTIES OF CALCAREOUS CLAY BEFORE AND AFTER REMOVEL OF $CaCo_3$

Sample Preparation Technique	Liquid Limit %	Plastic Limit %	Plasticity Index %	Shrinkage Limit %
Wetted from natural water content	92	45	47	--
Wetted after oven drying	76	39	37	36
Wetted after salt removal without oven dyring	90	45	45	--
Wetted after carbonate removal after oven drying	89	49	40	17

TABLE 3

ENGINEERING PROPERTIES OF CALCAREOUS CLAY IN ITS UNDISTRIBUTED STATE AND AFTER REMOVAL OF $CaCo_3$

	$CaCo_3$ Content	Compression Index, Cc	Swelling Index, Cs	A_f	c	ϕ deg.
Undistributed State	50%	0.8	0.17	0.9	0	35
After Removal of $CaCO_3$	0	0.6	0.10	0.7	0	33

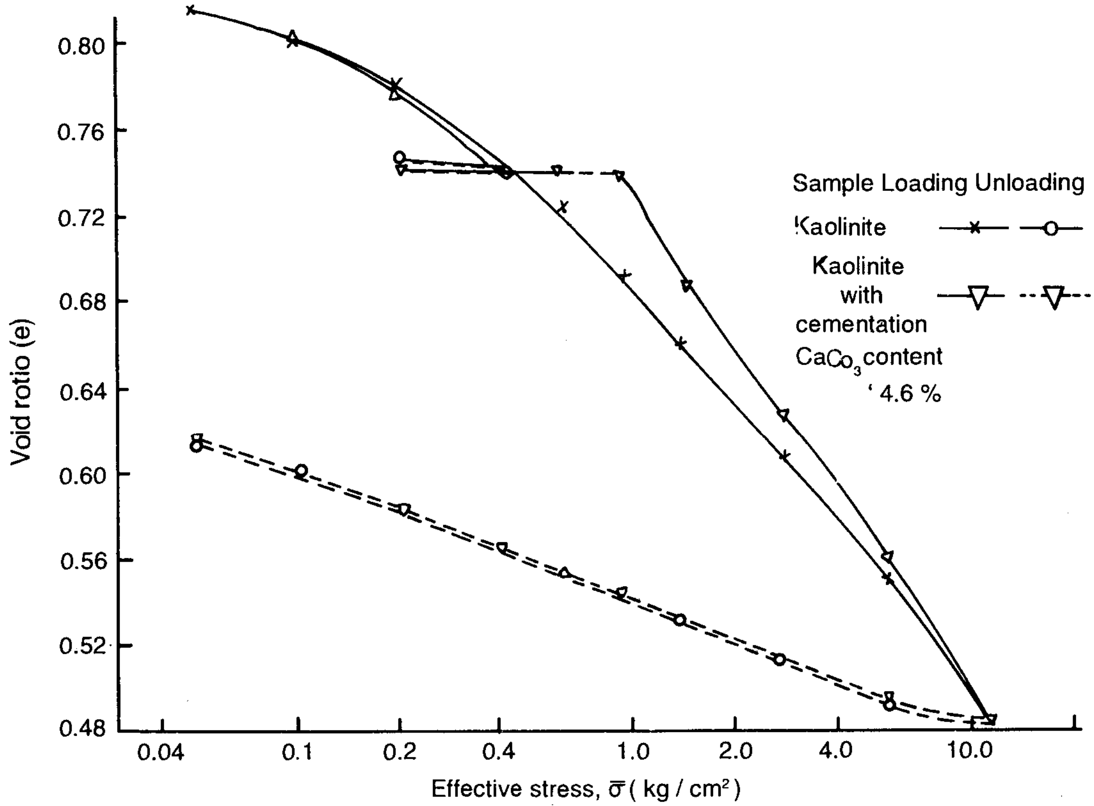

FIGURE 34. One-Dimensional Consolidation Curves for Kaolinite with and without cementation—After Nambiar (1982)

which 2.6 to 4.7 per cent of Calcium Carbonate had been deposited in the form of cementing agent. Fig. 34 shows the void ratio vs. effective stress relation for Kaolinite with and without cementation and Fig. 35 shows the ratio of apparent pre-consolidation pressure to actual pre-consolidation pressure that was observed on account of the cementing action arising from different amounts of Calcium Carbonate content deposited in the Kaolinite samples. From these two figures it is abundantly clear that the presence of very small quantity of Calcium Carbonate in the form of a cementing agent can markedly alter the engineeing behaviour of clays; something

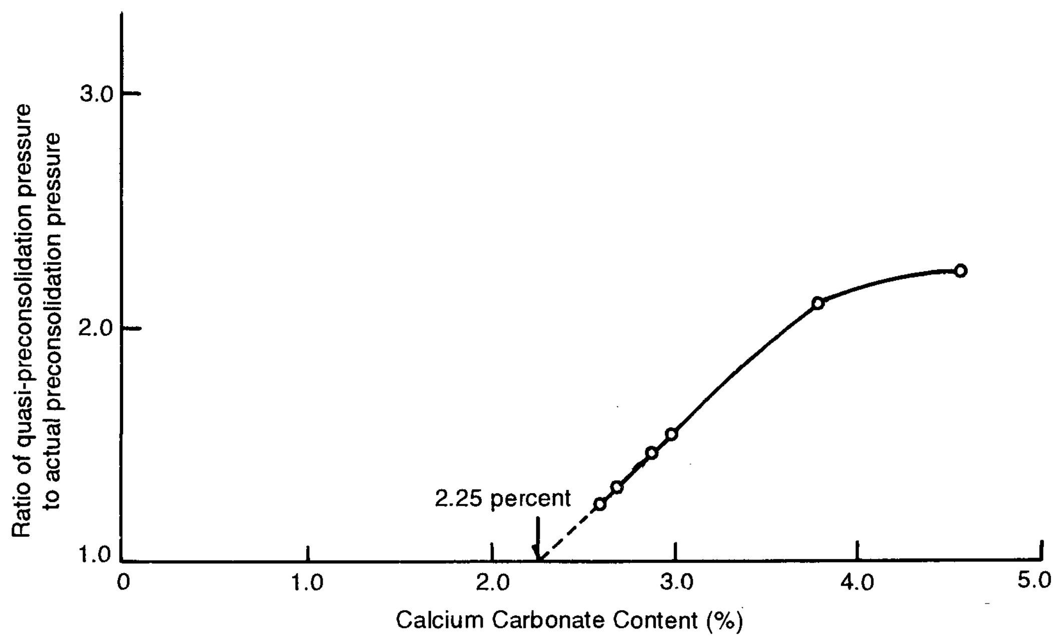

FIGURE 35. Variation of Ratio of Quasi-Preconsolidation Pressure to Actual Preconsolidation Pressure with Calcium Carbonate Content—After Nambiar (1982)

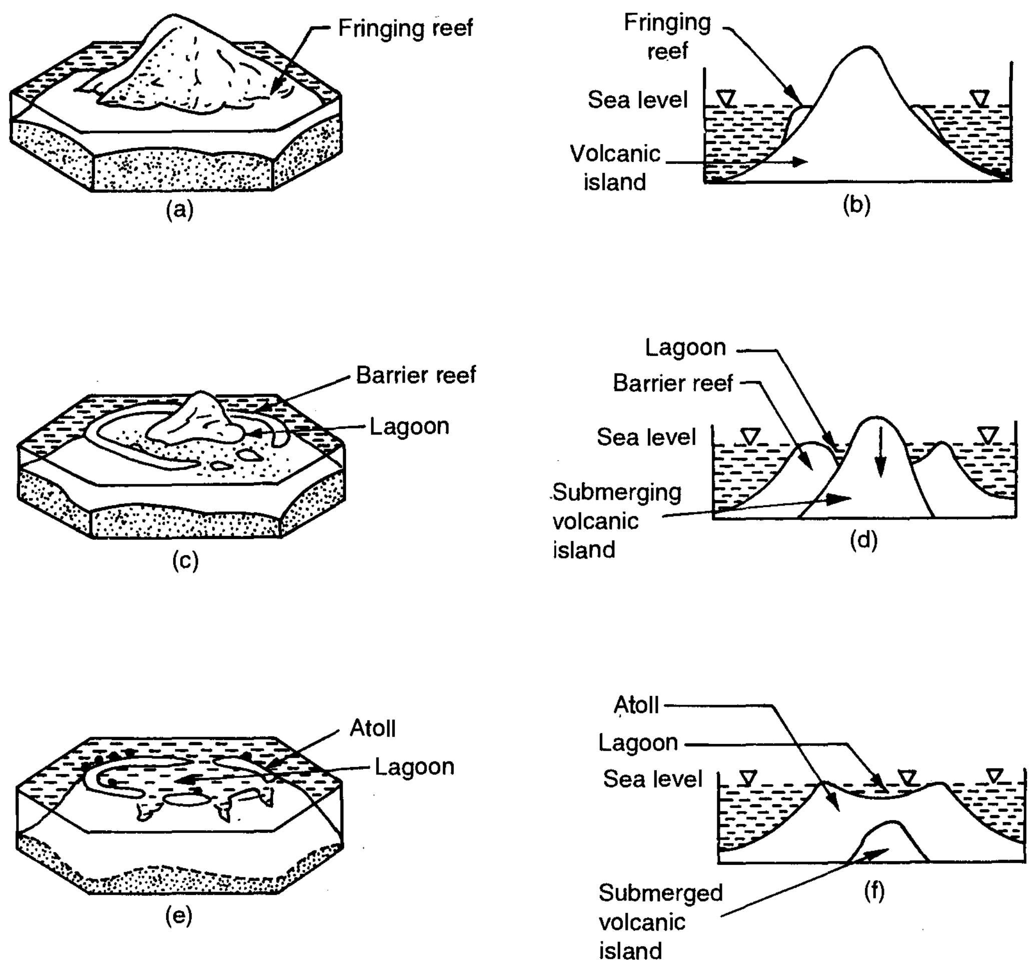

FIGURE 36. Origin of Coral Reef--After Darwin as quoted by Deshmukh (1983)

TABLE 4

PARAMETERS RELEVANT FOR ENGINEERING CLASSIFICATION OF CORALS

1.	Structure and Texture	4 Groups
2.	Degree of Lithification	Young; Old; Very old
3.	Type of Coral Reef	Continental; Oceanic
4.	Zonation of Coral Reef	Sheltered; Windward

which substantially larger quantities of Calcium Carbonate in an inert state are unable to accomplish. An understanding of calcareous clay is thus possible only if one first assesses the form of existence of Calcium Carbonate that is present in it.

CORALS

Corals have been extensively studied by biologists. Not only is the animal called Coral, but the house that this animal creates for itself to live in, which is what is normally seen and is Calcium Carbonate in content, is also called Coral. Coral, in the animal kingdom, comes under the category of invertebrates. Biogenetically, Corals are further divided into different classes,classes into sub-classes, sub-classes into order, order into suborder, sub-order into super family, super family into family, family into sub-family,sub-family into genus and finally, genus into species. Such a classification is, however, of no use from the view point of geotechnical engineering. Corals create land masses that are known as Coral reefs. It is on these reefs that civil engineers are often interested to construct. Not all Corals build reefs; those that do are known as reef building Corals. The land masses that are produced by reef building corals are known as fringing reef, barrier reef and atoll, as shown in Fig. 36. In India, as has been noted before, we have land masses of all the three kinds listed above.

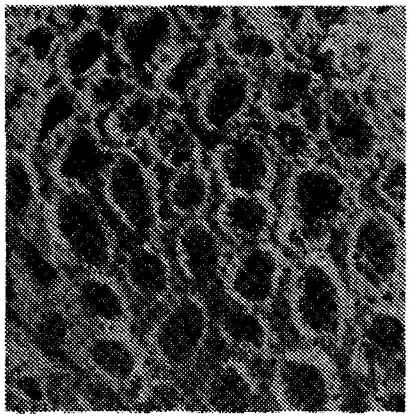

FIGURE 37 Group I Coral - After Deshmukh (1983)

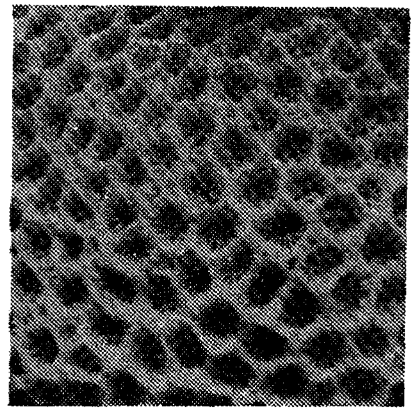

Figure 38 Group II coral - After Deshmukh (1983)

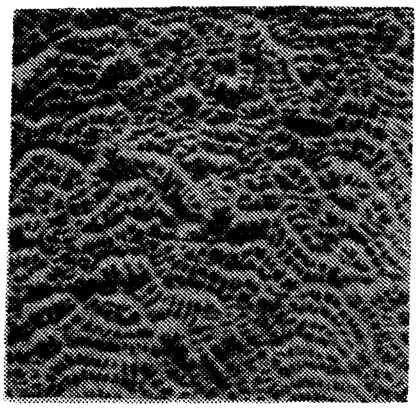

FIGURE 39 Group III Coral - After Deshmukh (1983)

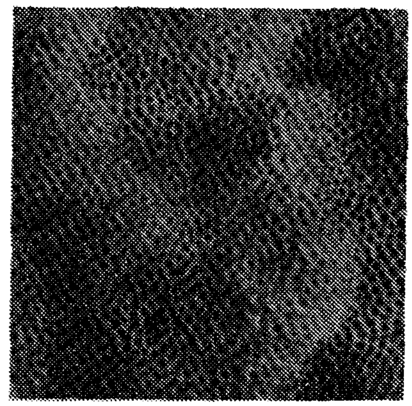

FIGURE 40 Group IV Coral - After Deshmukh (1983)

TABLE 5

GENERAL DESCRIPTION OF THE FOUR GROUPS OF CORALS

Group No	General Appearance	Texture	Corallite Diameter mm
I.	Contiguous circular, oval, or distorted corallites, with separate boundaries, corallite walls imperforate, imperforate calcareous material deposited in between the corallites, shape of corallite more or less cylindrical.	Coarse-Medium	3-18
II.	Contiguous polygonal corallites, with common boundaries, corallite walls imperforate, shape of corallites more or less prismatic	Medium	3-10
III.	Contiguous meandering corallites, with long common boundaries,corallite walls imperforate	Medium-Fine	2-4*
IV.	Contiguous polygonal corallites, with common boundaries (Microscopically the corallite walls are highly perforate)	Very fine	0.5-1.5

*Corallite diameter represents transverse distance between long boundaries of corallites, for a few uncommon varieties it varies between 10-15 mm.

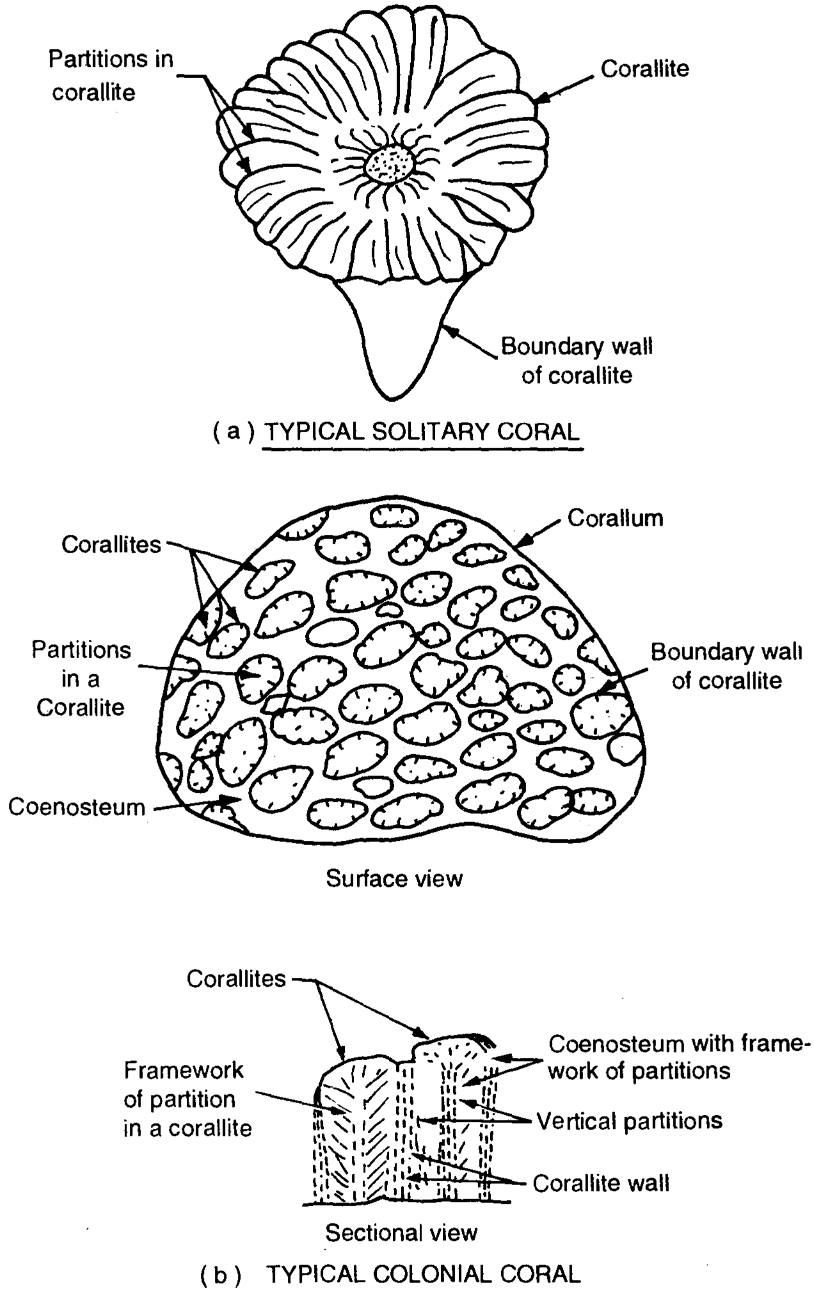

FIGURE 41. Schematic Representation of Solitary and Colonial Corals--After Deshmukh (1983)

In attempting to determine what parameters might be relevant in evolving an engineering classification for Corals, four factors were identified. These are tabulated in Table-4.

A visual study of a large variety of reef building Corals indicated that Corals could be grouped into four groups with identifiably different structures and textures. Figs. 37 to 40 show these four different structures. Before one can describe them and see the differences between them, it is necessary to take a look at a typical solitary and a typical colonial Coral which are shown in Fig. 41 where a number of terms which will be used hereafter, are indicated. Table-5 briefly describes the structure and texture of the 4 groups, designated as Group I, Group II, Group III and Group IV. Of particular significance is the fact that, whereas, the walls of the corallite of the first three groups are imperforate, that is, they do not have any holes in them, the walls of Group IV Corals are perforate. Figs. 42 & 43 show sections through Corals and these naturally suggest that Coral, as a rock, is likely to be anisotropic in its behaviour, Deshmukh et.al. (1983c).

Referring back to Table-4, after structure and texture, the next parameter, which was thought to be important from the view point of engineering behaviour of Corals, is the degree of lithification. When Corals are living and constructing their houses, the land mass is constantly in a state of growth. The spaces in between the

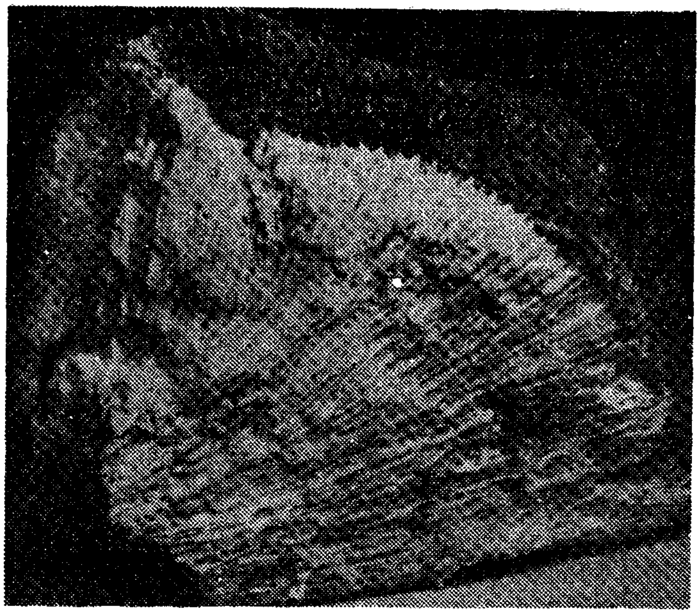

FIGURE 42 Section Through Coral - After Deshmukh (1983)

partitions in a corallite are open and clearly visible; such Coral may be described as being young in age. After the Coral animal abandons a Coral formation, with the passage of time, material is deposited in the open spaces within a corallite. This process of lithification reduces the porosity of the Coral. The process is a continuing one. In a very old Coral, there may be hardly any pores left, whereas, in an old Coral some pores may still be partially open. One may thus distinguish the various degrees of lithification broadly in terms of the age of the Coral which one may describe as young, old and very old. One can expect a young Coral with its large number of pores to behave more anisotropically than a very old Coral and also the very old Coral to be stronger than an otherwise similar young Coral.

The third parameter that is likely to be important from an engineering view point is the type of Coral reef as indicated in Table-4. A fringing reef is right next to the coast line, whereas, the barrier reef and the atoll are usually located in the ocean, far from the coast. The ocean environment is much more exposed and harsh than the environment along the coast. For something to survive in the ocean environment, it must be strong to withstand the impact of waves, the wind, the constant wetting and drying as well as the changes in temperature. It was thus hypothesized that Coral rock of oceanic reefs would be stronger, more resistant, less permeable etc.than Coral rock encountered in reefs that are close to the coast, that is, continental reefs.

The fourth and last parameter in Table-4 recognizes that Corals can be found either on the exposed windward side of a land mass or on the more protected side. Since the environmental conditions on the windward

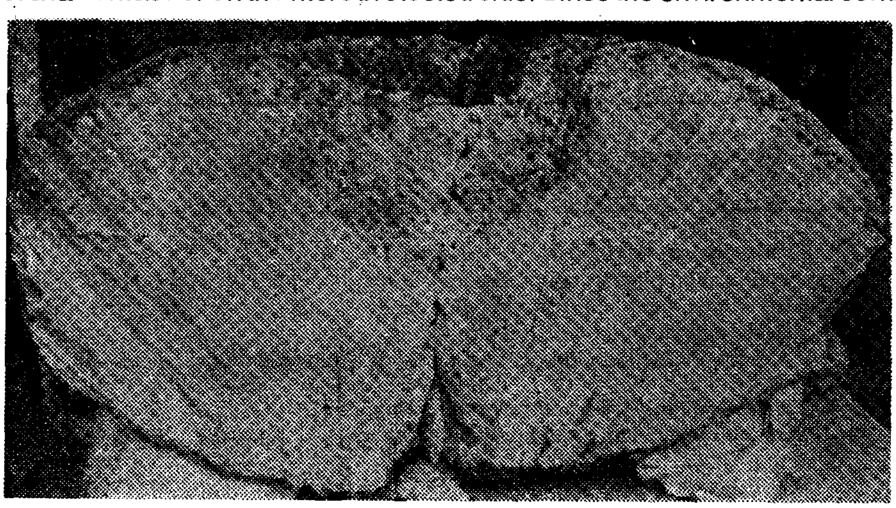

FIGURE 43 Section Through Coral - After Deshmukh (1983)

side would be much harsher than on the sheltered side, one would expect the Coral that survives the environment on the windward side to be more resistant than that situated in a sheltered environment.

That these four parameters are indeed the relevant ones from a geotechnical view point serving as the basic hypothesis, a large number of Corals were subjected to laboratory study which generally confirmed the relevance of each of these four parameters. For example, Fig. 44 shows the permeability vs. void ratio relation obtained from samples of Group II. The samples were obtained by drilling cores in 3 orthogonal directions from a block of Coral. In Fig. 44 one notes the relatively large spread in the values of permeability for any void ratio. This variation is on account of the anisotropic nature of the Coral material. Thus permeability determined in samples with the same void ratio but in different directions yields significantly different magnitudes.

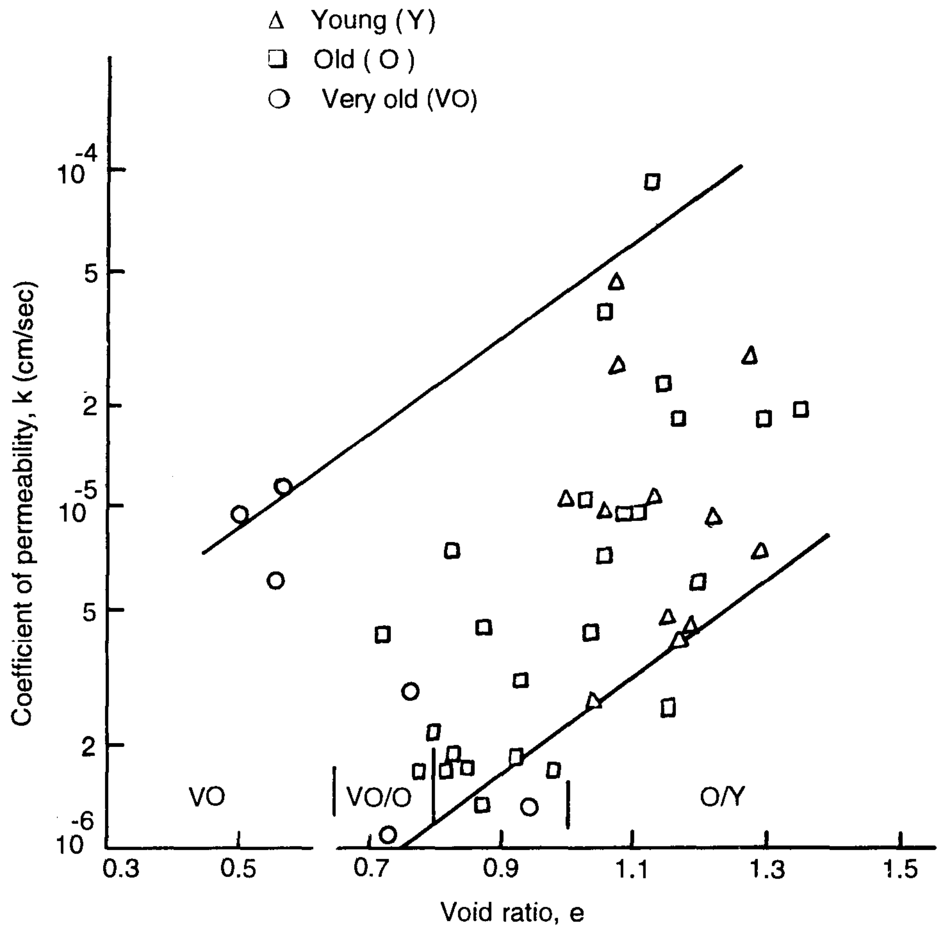

FIGURE 44. Variation of Coefficient of Permeability with Void Ratio for Rocks of Group-II--After Deshmukh (1983)

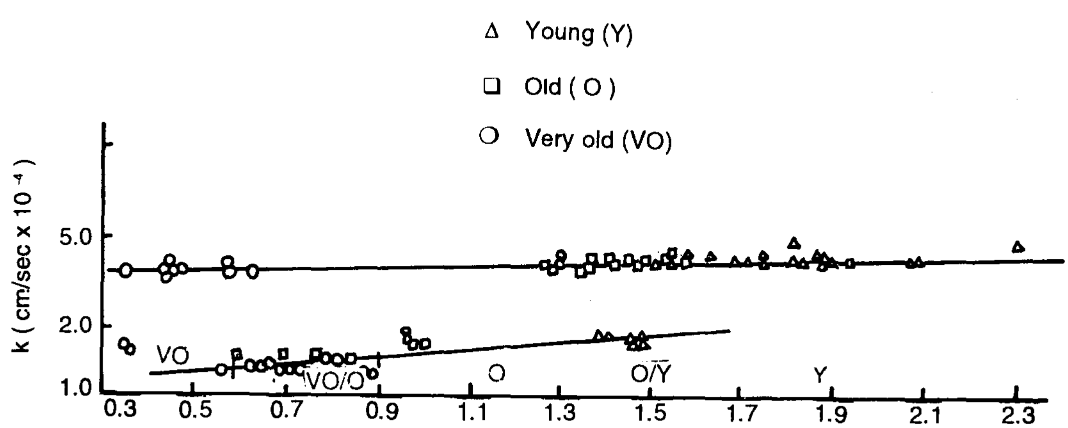

FIGURE 45. Variation of Coefficient of Permeability with Void Ratio for Rocks of Group-IV After Deshmukh (1983)

Fig. 45 indicates a similar plot of permeability vs. void ratio for Corals of Group IV and one notices the almost unique relation between permeability and void ratio. Apparently, there is little influence of anisotropy. That this is so is readily explained if one recalls that for Group IV Corals the corallite walls are perforate. These perforated walls ensure that water is able to travel with similar head-loss, both along and across these walls, thus making this Group relatively isotropic as far as the engineering property of permeability is concerned.

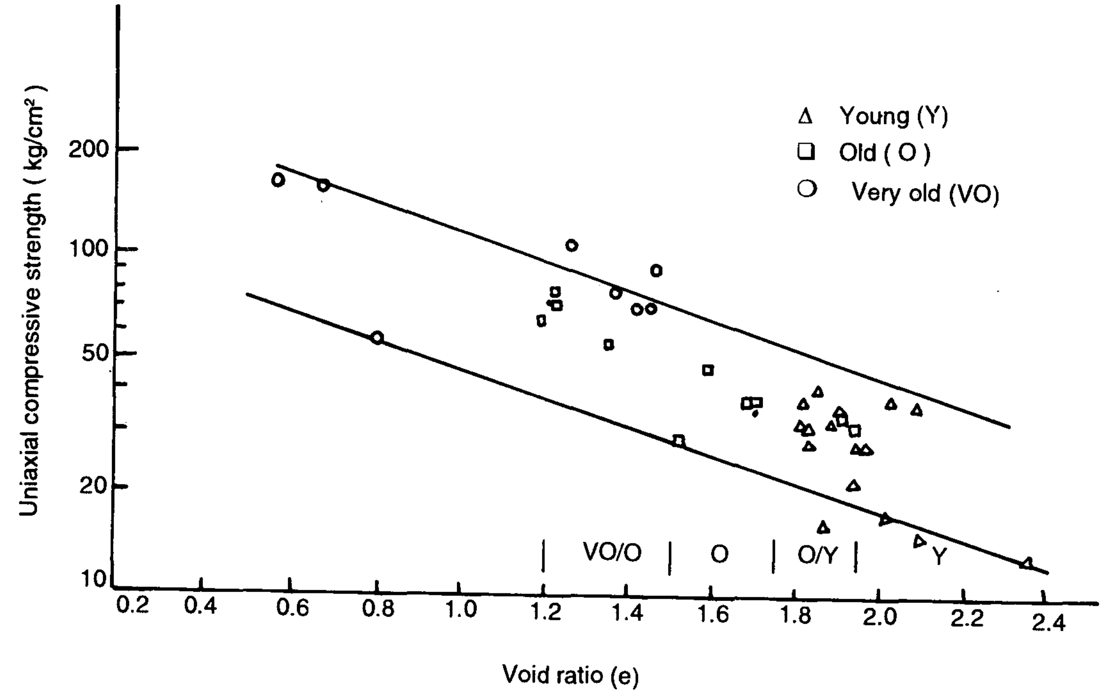

FIGURE 46. Variation of Uniaxial Compressive Strength with Void Ratio for Rocks of Group-I--After Deshmukh (1983)

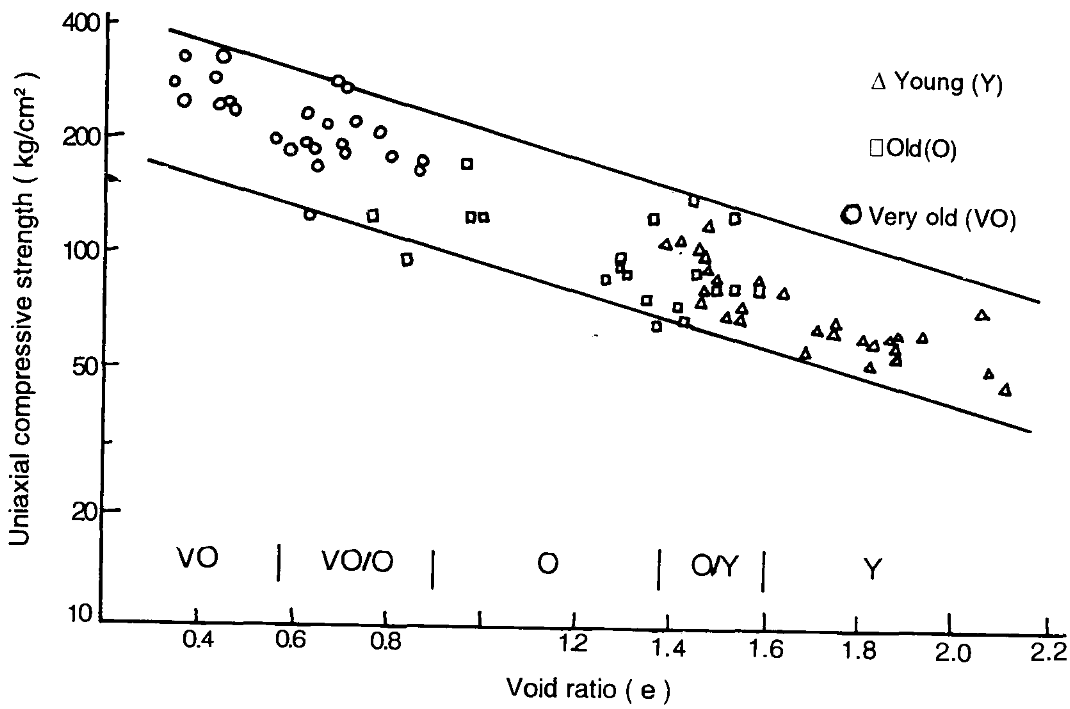

FIGURE 47. Variation of Uniaxial Compressive Strength with Void Ratio for Rocks of Group IV After Deshmukh (1983)

The uniaxial compressive strength vs. void ratio relation for Group I Corals is shown in Fig. 46 which indicates that the young Corals are not as strong as the very old ones. Fig. 47 indicates the same influence for Group IV Corals. One notes that in Figs. 46 & 47 the spread of data is quite similar indicating that the influence of anisotropy in Group 1 and Group IV Corals is also similar. The perforate walls of Group IV have little consequence in reducing the anisotropy of this Coral when the property under consideration is that of strength.

In Fig. 48 are shown four pairs of data. Corals in each pair are similar except that the data within the circle, is from Coral encountered in oceanic environment, and data within the triangle is from the continental environment. One notes that for each pair, the Coral in the oceanic environment is less permeable.

Similarly, data on uniaxial compressive strength for four pairs of Corals is presented in Fig. 49 in which for each pair the only difference is the location of the Coral on a reef. The open figures are for Coral from the windward side and the solid figures are for Corals from the sheltered side. It is apparent that the strength is higher on the windward side.

From a very large number of tests conducted, the range of values of permeability and uniaxial compressive strength for different groups of young Corals from continental reef and from the sheltered side were determined and are presented in Table. 6. By conducting permeability and unconfined compression tests on numerous other samples of different ages, obtained from different types of reefs and different locations within a reef and comparing the values obtained with the values given in Table-6, factors were determined which when multiplied with the values given in Table-6, would provide an estimate of the magnitude of permeability or unconfined compression strength of a Coral of age other than young, type other than continental and zone other than sheltered. The ranges of these multiplication factors are given in Table-7.

A procedure for geotechnically classifying a sample of Coral and obtaining a preliminary estimate of either its permeability or unconfined compression strength would thus be as follows, Deshmukh et. al. (1983a): the sample would first be visually inspected to determine its group with reference to Figs. 37-40 and Table-5; one would then inspect the state of openness of its pores to determine its age and designate it as young, old or very old; knowing the location from where the sample has been recovered, one would designate it as continental or oceanic as well as windward or sheltered; one would then use Tables-6 & 7 to obtain a very preliminary estimate of the permeability and unconfined compressioon strength of the sample. Such a procedure is clearly relevant and useful for the geotechnical engineer. It can be continuously refined as more data accumulates by altering the range of values of the two properties as given in Table-6 as well as the range of the multiplication factors given in Table-7.

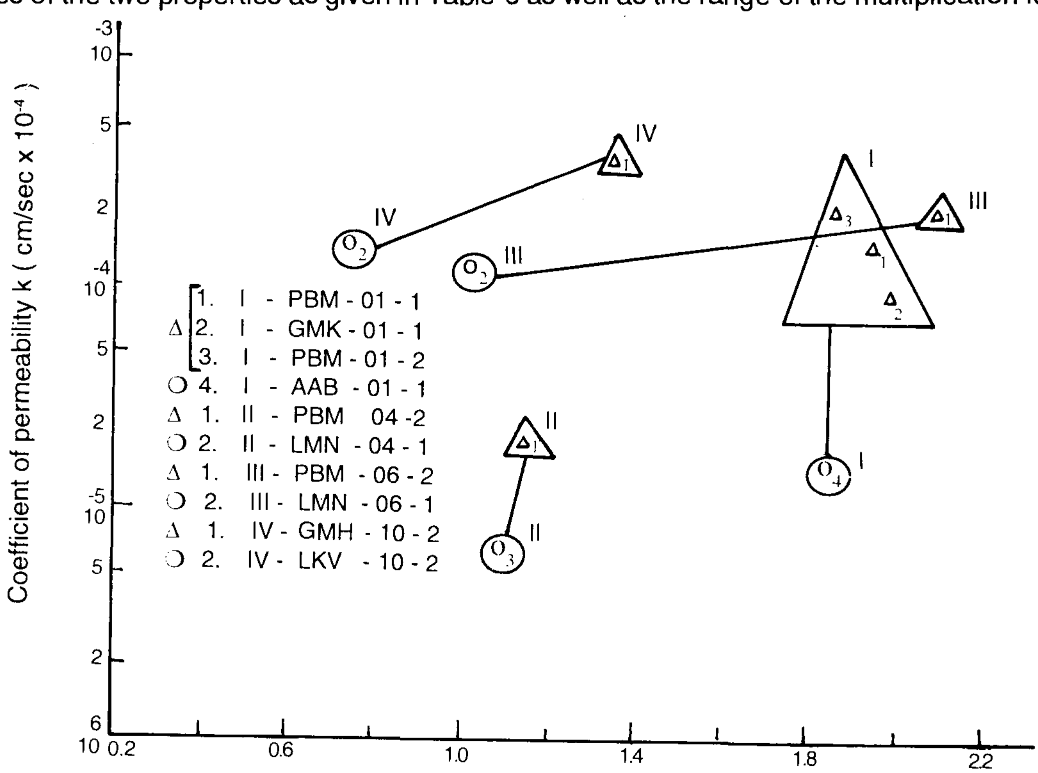

FIGURE 48. Variation of Permeability with Type of Reef-After Deshmukh (1983)

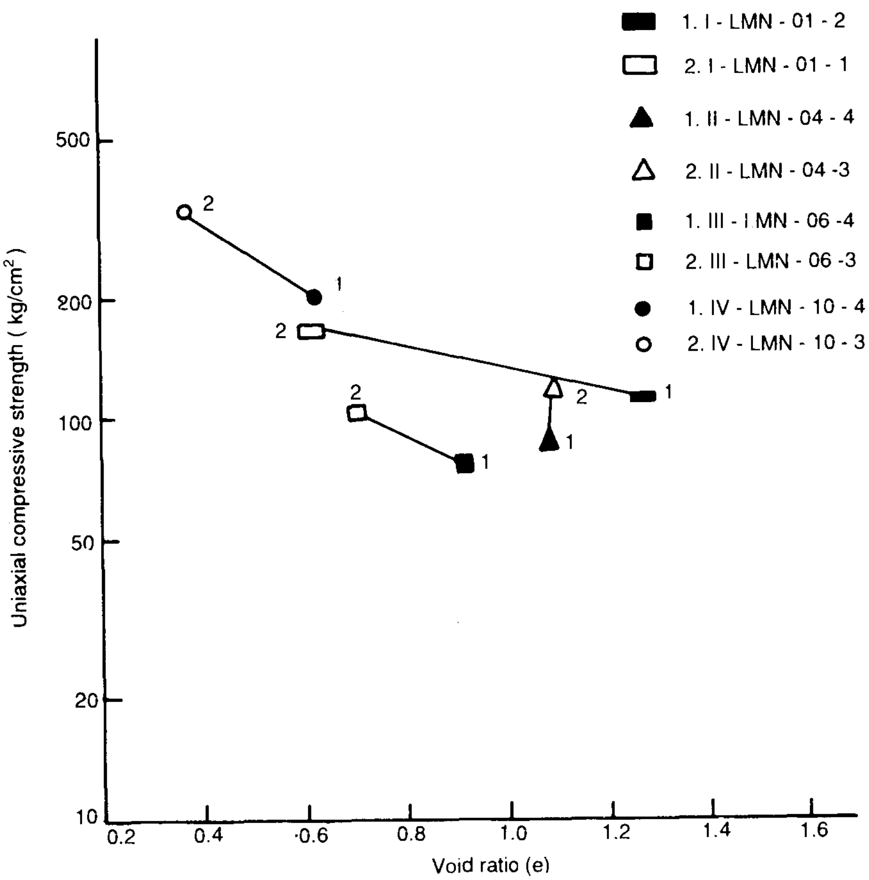

FIGURE 49. Variation of Uniaxial Compressive Strength with Zonation--After Deshmukh (1983)

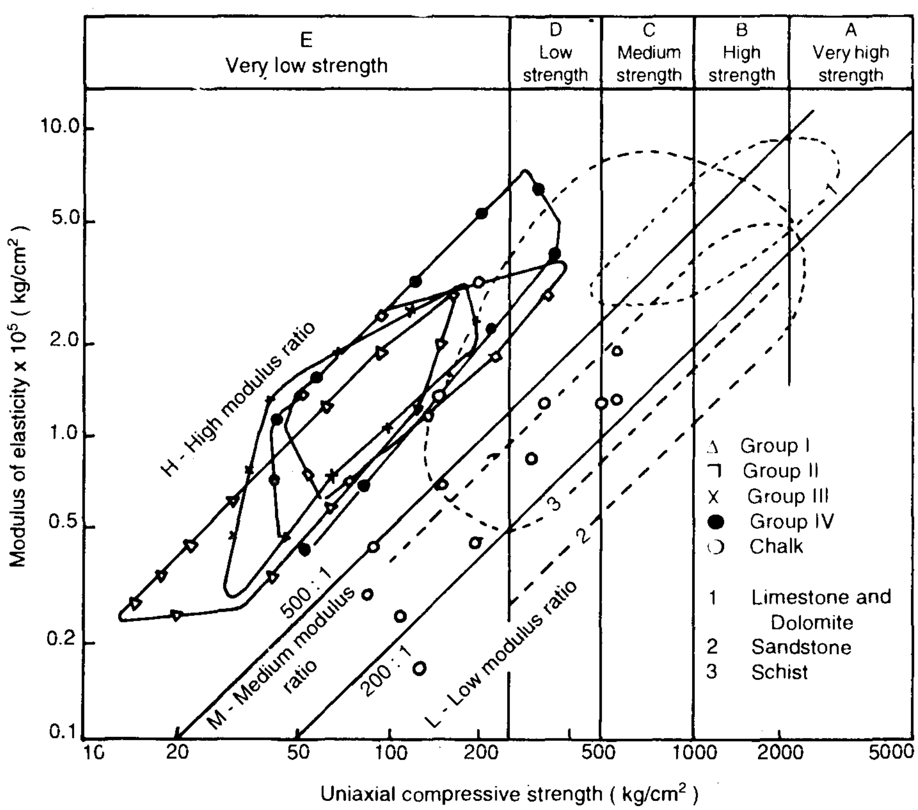

FIGURE 50. Relationship of Uniaxial Compressive Strength and Modulus of Elasticity-- After Deshmukh (1983)

TABLE 6

RANGE OF COEFFICIENT OF PERMEABILITY AND UNIAXIAL COMPRESSIVE STRENGTH FOR YOUNG SAMPLES OBTAINED FROM CONTINENTAL REEFS FROM SHELTERED SIDES

Group	Coefficient of Permeability, k 10^{-4} cm/sec	Uniaxial compressive Strength, ucs Kg/cm^2
I	1.0-3.0	15-25
II	0.04-0.9	50-100
III	1.0-4.0	25-50
IV	4.0-5.0	50-100

TABLE 7

MULTIPLICATION FACTORS TO BE USED IN CONJUNCTION WITH TABLE 6

Reef Type Zonation	Age of rock	Group I k	Group I ucs	Group II k	Group II ucs	Group III k	Group III ucs	Group IV k	Group IV ucs
Continental	Young	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Sheltered	Old	0.05-1.0	1.8	0.4	1.4	(0.5-1.0)	(1.4)	0.5-1.0	1.3
	Very Old	0.05-1.0	3.8	0.3	1.9	(0.5-1.0)	(1.8)	0.5-1.0	1.6
Continental	Young	0.1-1.0	1.6	1.0	1.1	1.0	1.3	1.0	1.1
Windward	Old	0.05-1.0	2.2	0.4	1.5	(0.5-1.0)	(1.8)	0.5-1.0	1.4
	Very Old	(0.05-1.0)	(4.6)	(0.3)	(2.4)	(0.5-1.0)	(2.3)	(0.5-1.0)	(1.9)
Oceanic	Young	(0.1-1.0)	(1.8)	1.0	1.4	0.5-1.0	1.4	0.5-1.0	1.3
Sheltered	Old	0.05-1.0	3.0	(0.4)	(1.9)	0.5-1.0	2.0	0.3-1.0	1.8
	Very Old	(0.05-1.0)	(6.0)	(0.3)	(3.5)	0.5-1.0	2.7	0.3-1.0	3.5
Oceanic	Young	0.1-1.0	2.0	1.0	1.5	0.5-1.0	1.5	0.5-1.0	1.5
Windward	Old	0.05-1.0	3.7	0.4	2.0	0.5-1.0	2.8	0.3-1.0	2.4
	Very Old	0.05-1.0	9.0	0.3	4.0	0.5-1.0	3.5	0.3-1.0	5.5

Data within parenthesis denotes extrapolated values.

One may summarize the result of this study by concluding that Coral is a rock that is anisotropic, although the extent of anisotrophy depends upon the property in question, further, that it has a permeability of the order of 10^{-4} cm/sec and that it can be classified as an EH rock, that is, one which has very low strength and high modulus ratio, see Fig. 50.

SOIL ENGINEERING PROBLEMS

An understanding of the nature, distribution and behaviour of soils off-shore India is, of course, a pre-requisite for solving geotechnical problems that arise in the off-shore environment. There is of course no shortage of problems that have been encountered during planning, design and installation of various structures offshore India. It is perhaps not inappropriate to list some of these problems and issues that have arisen and which need attention.

Site Investigation and Soil Behaviour :

(a) Improving soil sampling and testing procedures to account for stress release and gas coming out of solution.

(b) Developing statistical correlations, empirical guidelines and normalising procedures to obtain design soil parameters from even relatively poor samples.

(c) Improving and developing in-situ testing equipment, interpretation techniques and correlations with engineering properties of soils.

(d) Understanding the behaviour of submarine soils under cyclic loading.

(e) Understanding various sedimentation and transportation processes operative offshore India and predicting the state of consolidation of deposits undergoing continuous sedimentation.

Pile Supported Steel Jacket Type Platforms :

(a) Prediction of load carrying capacity of deep penetration piles (driven or grouted in drilled holes) in calcareous soils with or without cementation and in stratified deposits.

(b) Prediction of soil resistance during pile driving and soil set-up due to stoppages in driving.

(c) Prediction of static capacity of a pile from dynamic analysis of instrumented pile monitoring data obtained during driving.

Foundation Anchorage:

Prediction of holding capacity of various types of marine anchors in soft clays for ships, drilling rigs, lay-barges, single buoy moorings as well as compliant structures like guyed towers, tension leg platforms, etc.

Jack-Up Rigs:

(a) Prediction of penetration of "mats" or "spud-cans" of jack-up rigs into various types of soils and stratified deposits.

(b) Evaluation of likelihood of jack-up rig footings punching through a strong layer into the underlying weaker soil.

(c) Evaluation of soil resistance during pulling out of jack-up rig leg.

Submarine Pipelines :

(a) Determining resistance characteristics of pipeline-soil system along the route for each component of movement, i.e., axial, horizontal and vertical.

(b) Evaluation of initial settlement and lateral restraint due to soil friction and embedment upon installation by lay-barge method.

(c) Evaluation of longitudinal friction between pipe and the soil during laying by botton pull method.

(d) Evaluation of positional stability of the pipeline after installation to predict settlement or flotation of buried pipeline.

(e) Assessment of sea floor stability and liquefaction potential of soils due to earthquake and wave-induced loading.

(f) Determination of thermal conductivity of soils for predicting heat loss through buried submarine pipelines transporting oil.

This list can only be considered as illustrative rather than exhaustive and one must also not conclude from this list that lack of complete understanding of these and other issues is nolding up civil engineering activity on the sea bed. As has always been the situation,operational engineering decisions are constantly being taken and design and construction are proceeding as per need.

In the Marine Geotechnology Laboratory of I.I.T. Delhi, investigations have been carried out on many aspects indicated above. For example, on improving soil testing procedures-Datta et. al.(1986), Gulhati and Datta (1989); understanding the behaviour of submarine soils under cyclic loading Datta et.al. (1980b); understanding the state of consolidation of deposits undergoing continuous sedimentation Datta et.al. (1983); prediction of load carrying capacity of piles in calcareous soils Datta et. al.(1980c), Varadarajan and Bassi (1981); anchors in soft clays Datta and Gulhati (1985), Baba et. al. (1989), Datta et. al. (1990); evaluation of longitudinal friction between pipe and soil during laying by bottom pull method Gulhati et. al. (1978); evaluation of lateral stability of pipeline-Varadarajan and Singh (1981); evaluation of positional stability of pipeline-Gulhati et. al. (1978), Gulhati et. al. (1980), Gulhati et. al. (1981); determination of thermal conductivity-Rawat et. al. (1979), Juneja et. al. (1984).

In this Lecture, attention will be focussed on only three topics, which are :

1. Predicting the state of consolidation of deposits undergoing continuous sedimentation.
2. Predicting the amount of sinking of pipeline placed on the sea-bed.
3. Suitable anchors in soft soils.

Each is considered in turn.

STATE OF CONSOLIDATION IN DEPOSITS UNDERGOING CONTINUOUS SEDIMENTATION

The Mississippi river brings to the Gulf of Mexico, 550 m tons of sediment every year. The river Ganges and the Brahmaputra together bring 2500 m tons of sediment annually to the Bay of Bengal. This is 14% of the total sediment brought by all the rivers of the world into the sea. In the sedimentation process, when soil is deposited on the sea bed, the total stress in the deposit below increases, increasing the pore water pressure. Before this pore water pressure has a chance to dissipate, more soil is deposited and the total stress on account of additional soil induces more pore water pressure. It is, therefore, not difficult to comprehend why off-shore deposits in the vicinity of major deltas consist often of underconsolidated soil.

Noorany and Gizienski (1970) quote the work of Terzaghi and Olsson who had studied the mathematics of a deposit undergoing continuous sedimentation and concluded that the average degree of consolidation of a deposit undergoing steady rate of accumulation of soil decreases with time. From this conclusion Noorany and Gizienski (1970) deduced that in such a situation, with time, the effective stress will tend to zero and they further

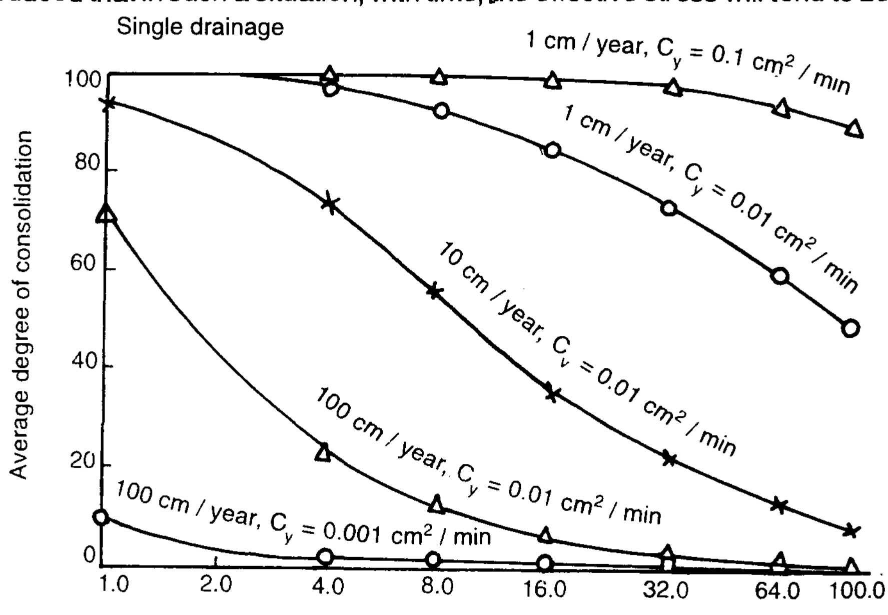

FIGURE 51. Variation of Average Degree of Consolidation--After Datta *et. al.* (1983)

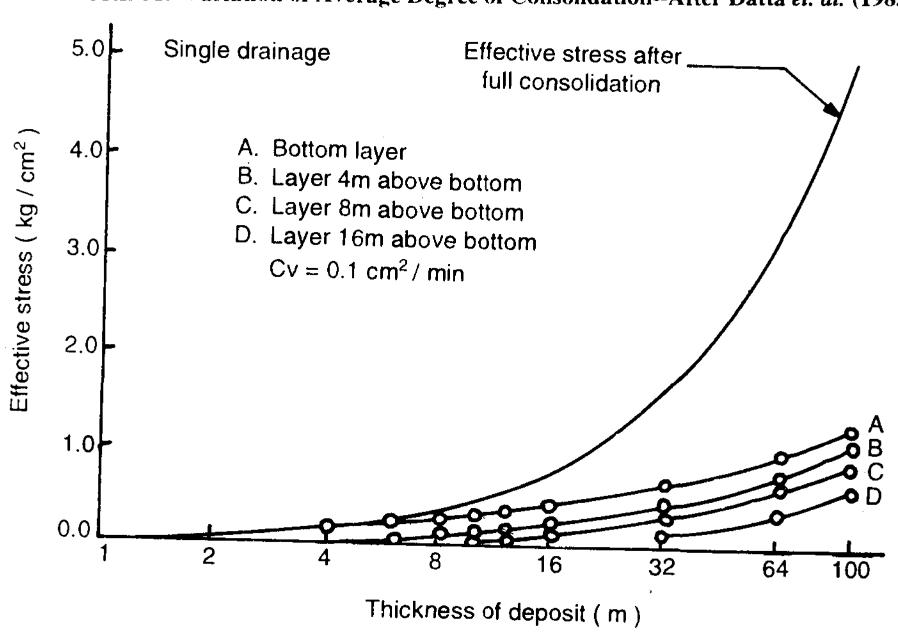

FIGURE 52. Variation of Effective Stress with Time at Different Locations-After Datta *et. al.* (1983)

deduced that in clayey soils that have not experienced any over-consolidation, this would imply that with time the strength will reduce to the value of the cohesion intercept in terms of effective stresses which is usually zero. In other words, the shear strength in a deposit undergoing continuous sedimentation will, according to them, reduce to zero with time. This is of course, a frightening proposition since it implies that one would not be able to locate any civil engineering structure on such a soil since the soil would have no capacity to withstand stresses.

Utilizing the finite difference formulation of the one dimensional consolidation process, Scott (1965), a computer programme was formulated to study the development of excess pore water pressure and effective stress with time and with depth as a deposit was built up, not by continuous deposition but through step loading, at a number of different rates using soils with different coefficients of consolidation, Datta et. al. (1983). The study confirmed, see Fig. 51, that the average degree of consolidation reduces with time as the thickness of the deposit increases for all combinations of rates of accumulation and coefficients of consolidation. This, however, does not imply that effective stress in the deposit will reduce to zero with time as is evident from a study of Figs. 52 and 53. Fig. 52 shows how the effective stress at various locations in the deposit increases with time as the thickness of the deposit increases. For a particular value of the coefficient of consolidation and at a given time, Fig. 53 indicates, how the effective stress increases with depth for different rates of accumulation. Clearly the deduction of Noorany and Gizienski (1970) was unfounded and one need not fear that effective stress in deposits offshore of major deltas will tend to zero; it will not only be positive but at all locations will increase with time.

AMOUNT OF SINKING OF PIPELINE PLACED ON THE SEA BED

So that submarine pipelines can function, it is essential that they have positional stability. If a pipeline is designed to rest on the sea bed, then it must not sink, nor move laterally and nor indeed should the soil below it move away, leaving the pipeline suspended. In the event, the pipeline has been designed such that it should remain below the mud-line, it is necessary that it remains there and neither sinks further nor rises up and surfaces at the sea bed.

In analysing the vertical stability of the pipeline, Reid (1951), Small et. al. (1971) and Ghazzaly and Lim (1975) have all considered the problem as being one of an object which continues to sink after being placed on a soil-water system until enough bearing capacity is mobilized to equal the applied bearing stress by the object and at that time the pipeline comes to rest with a safety factor of 1.0. Karal (1977) considered this problem using the theory of perfect plasticity.

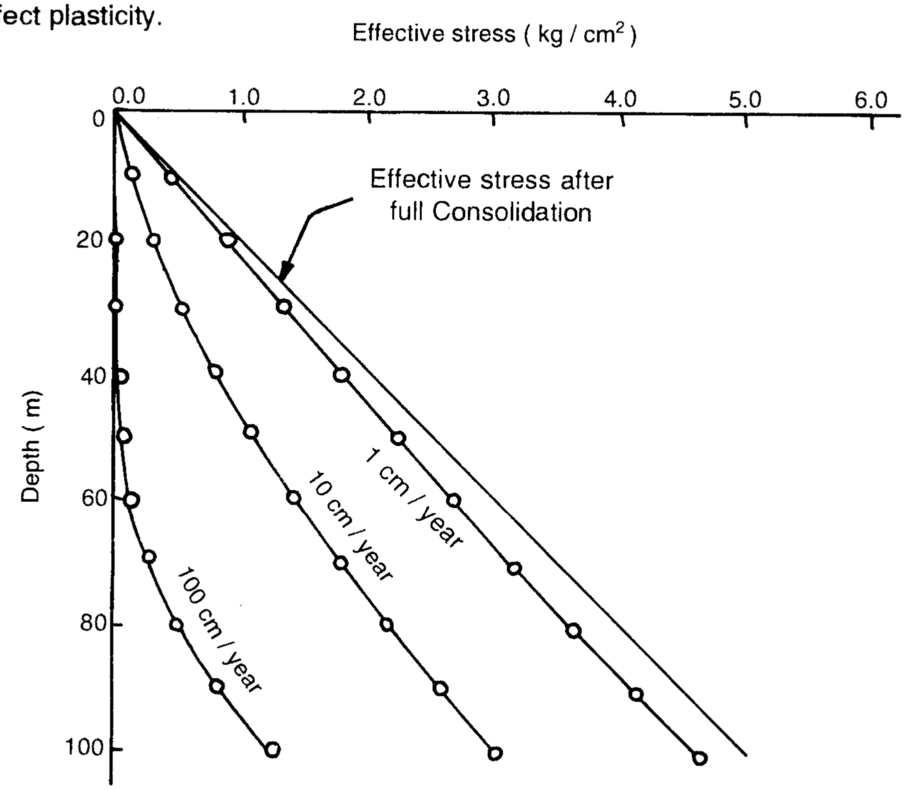

FIGURE 53. Variation of Effective Stress with Depth--After Datta *et. al.* (1983)

TABLE 8

PROPOSED THEORIES ON VERTICAL STABILITY OF SUBMARINE PIPELINES

Authors	Effective unit weight γ_e	Effective bearing Pressure*	Shear strength	Bearing Capacity factor
Reid (1951)	$\gamma_t/(1 + 100/\omega)$	W_s/D	Undrained strength	2
Small et.al (1971) for pipe axis above ground at a distance D_f	unit weight of water	$B = \frac{W_s/B}{\sqrt{D^2 - D^2_f}}$	Undrained strength	3
Small et.al.(1971) for pipe axis below ground at a distance D_f	unit weight of water	W_s/D	Undrained strength	Varying linearly from 3 to 7 as D_f/D varies from 0 to 4
Ghazzaly and Lim (1975)	total unit weight of soil, γ_t	W_s/D	Undrained strength	$\pi/2=1.57$
Ghazzaly and Lim (1975)	total unit weight of soil, γ_t	W_s/D	Yield stress	$\pi/2=1.57$

*To obtain submerged weight of pipeline W_s, subtract from weight of pipeline in air the term pipeline vol. multiplied by γ_e

ω= water content of soil

D= diameter

If this problem is considered as one of bearing capacity, then one needs to make assumptions on the following four aspects :

(a) What is the submerged weight of an object in soft saturated soil

(b) On what width is the submerged weight of the pipeline transferred to soil, that is, how is one to determine the stress imposed by the pipeline on the soil

(c) What is the shearing strength of soft saturated soil

(d) What is the bearing capacity factor

The nature of the assumptions made on these four aspects by Reid (1951), by Small et.al. (1971) and by Ghazzaly and Lim (1975) are summarised in Table-8. Studies reported by Gulhati et. al. (1980) and Gulhati et. al.(1981) indicate that experimental results compare well with a bearing capacity approach which makes these four assumptions in the following manner :

(a) The submerged weight of the pipeline be determined as its weight in air minus the influence of buoyancy recognizing that the buoyancy is produced not on account of the unit weight of water but on account of the total unit weight of the soil-water system.

(b) The stress imposed by the pipeline on the soil be determined

(i) as though the pipe load is transmitted on a width equal to half the chord width of the embedded part of the pipeline when the pipeline has sunk to a depth less than the radius of the pipeline,

(ii) as though the pipe load is transmitted on a width equal to the diameter of the pipeline when the pipeline has sunk to a depth equal to or more than the diameter of the pipeline, and

(iii) by linear interpolation between the radius and the diameter when the pipeline has sunk more than the radius but less than the diameter of the pipeline.

These assumptions have emerged from a study which attempted to determine a simple surface loading pattern which would result in stress contours in the soil which are similar to the stress contours produced when stress is transferred by the actual curved surface of the pipeline.

(c) The shear strength of the soil-water system be taken as its undrained strength determined from vane shear tests. For very soft soil, one may use large sized vanes for better precision.

(d) Bearing capacity factor be taken as suggested by Small et. al.(1971), that is, equal to 3 when the pipe embedment is less than radius of the pipeline and varying linearly from 3 to 7 as the depth to centre of the pipeline from the mud-line varies from zero to 4 times the diameter of the pipeline.

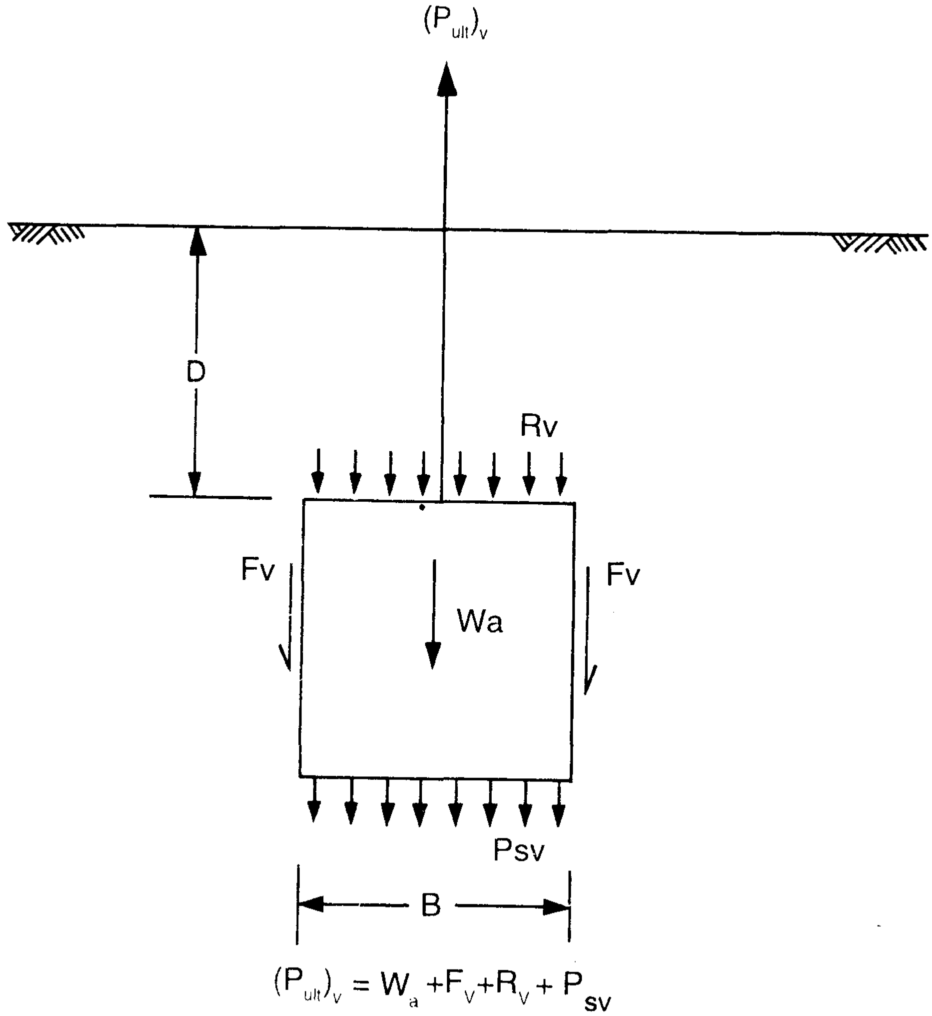

FIGURE 54. Forces Contributing to Breakout Capacity

ANCHORS IN SOFT SOILS

As has been noted before, off the east coast of India, submarine soils are soft, the continental shelf is narrow and there are indications that platforms may have to be located on the continental slope where water depth exceeds 200 m. In such a situation pile supported steel jacket type platforms being used on the west coast are usually uneconomical. Compliant structures like guyed towers and tension leg platforms are often more suitable. These compliant structures float on the water,can move laterally to a limited extent and are held in position by anchoring them to the sea-bed using a variety of anchors. Fig. 54 shows a gravity anchor and the forces that contribute to the breakout capacity $(P_{ult})_v$. These consist of the weight of the anchor, W_a, the resistance offered by the soil above the anchor, R_v, the friction that would be mobilised along the sides of the anchor, F_v, and perhaps a suction force, P_{sv}, that may develop at the base of the anchor when it is being pulled up. Fig. 55 shows the relevant forces for a plate anchor and a pile anchor. In the pile anchor the primary contribution is from side friction whereas in the plate anchor the contribution is from resistance of soil above the plate anchor and perhaps also from suction at the base. In order to undrstand the behaviour of the gravity anchor, a study is in progress in the Marine Geotechnology Laboratory of IIT Delhi in which initially experiments are being conducted on the plate anchor and the pile anchor. Investigation on plate anchors have now yielded useful results, some of which have been presented recently, see Baba et. al. (1989).

In order to be able to predict the behaviour of plate anchors, it is necessary that one determines the contribution to breakout force of the soil above the anchor as well as the contribution on account of suction below the anchor. In the ocean environment, one must recognise that it is not just the anchor behaviour under static load that is relevant, but one must also assess the influence of cyclic stresses which will be imposed on the anchor on account of loading from waves, currents, tides, etc.

There is reasonable agreement amongst researchers in relation to the amount of resistance that would be offered by the soil above the anchor and it is expressed as a breakout factor N_{bc}. Table-9 indicates the recommended breakout factor by various researchers. Its magnitude ranges between 9 and 10. Experimental investigations at IIT Delhi have confirmed this range of magnitude of the breakout factor.

There is, however, far too great a divergence of opinion as regards the contribution of the suction component. Table-10 indicates that the suggested range of the suction component varies from 1/2 of the undrained strength to as much as 7 times the undrained strength. Baba et. al (1989) conducted two sets of model studies on plate anchors. In the first set, suction was allowed to develop below the anchor, whereas, in the second set, suction was not allowed to develop. The test variable included soil prepared at 3 water contents and anchor pulled out at 4 different pull out velocities. Fig. 56 shows the brakout force with suction eliminated vs. the pull out velocity and one notes that the pull out velocity has no influence on breakout force. Fig. 57, obtained by a comparison of results of tests of set 1 and set 2 shows the percentage suction force developed below the anchor vs. the pull out velocity which clearly indicates that larger suction develops as the pull out velocity is increased.

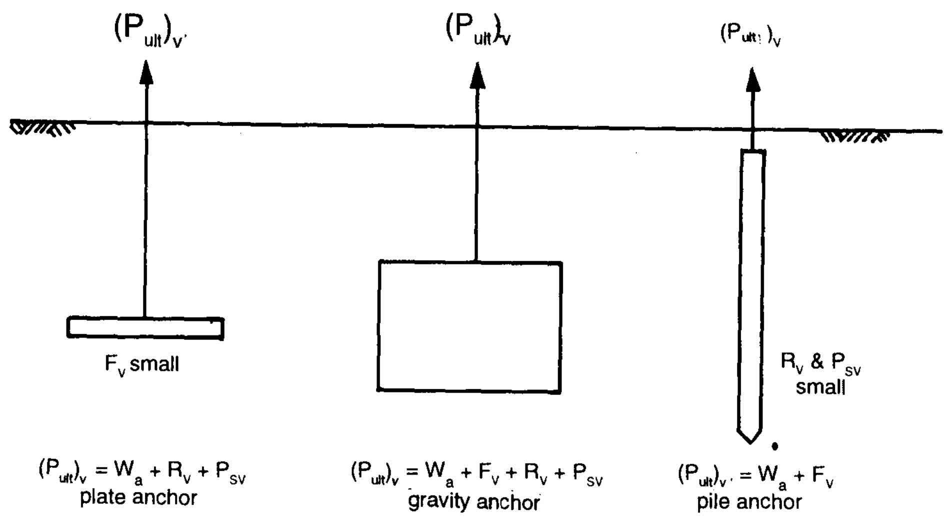

FIGURE 55. Influence of Shape of Anchor on Breakout Capacity

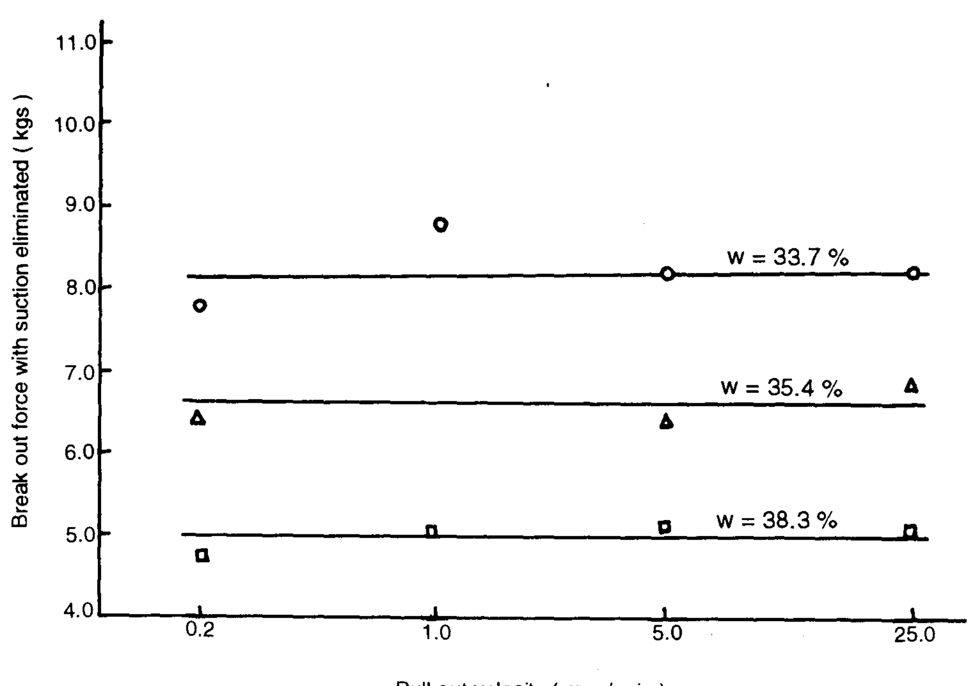

FIGURE 56. Breakout Force with Suction Eliminated not Influenced By Pull Out Velocity After Baba *et. al.* (1989)

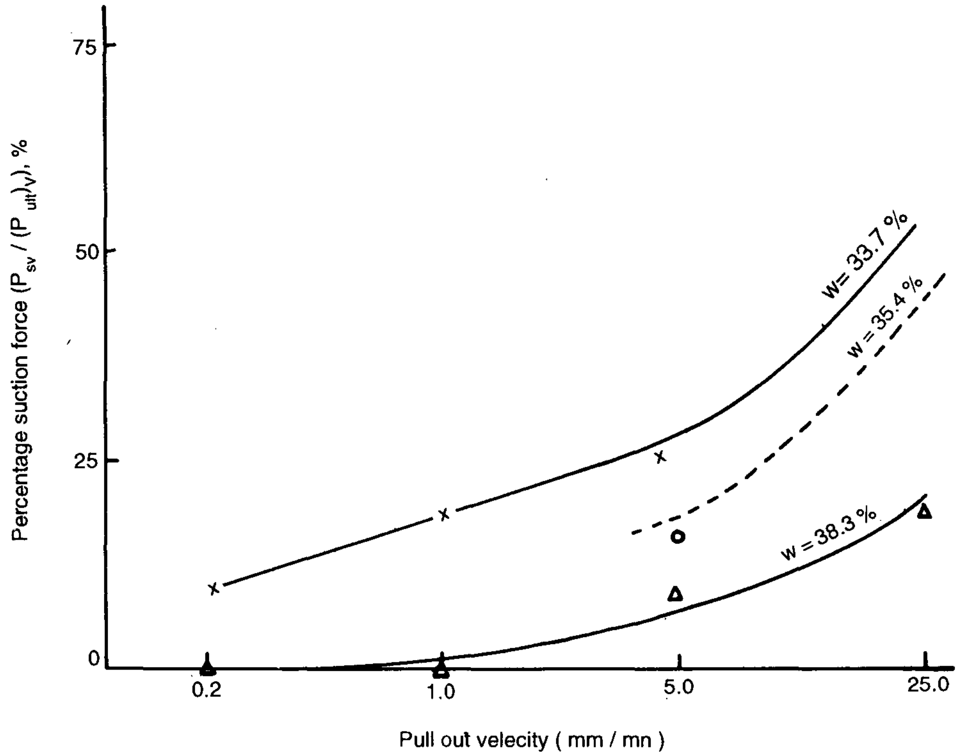

FIGURE 57. Increase in Suction Force with Increase in Pull out Velocity--After Baba *et. al.* (1989)

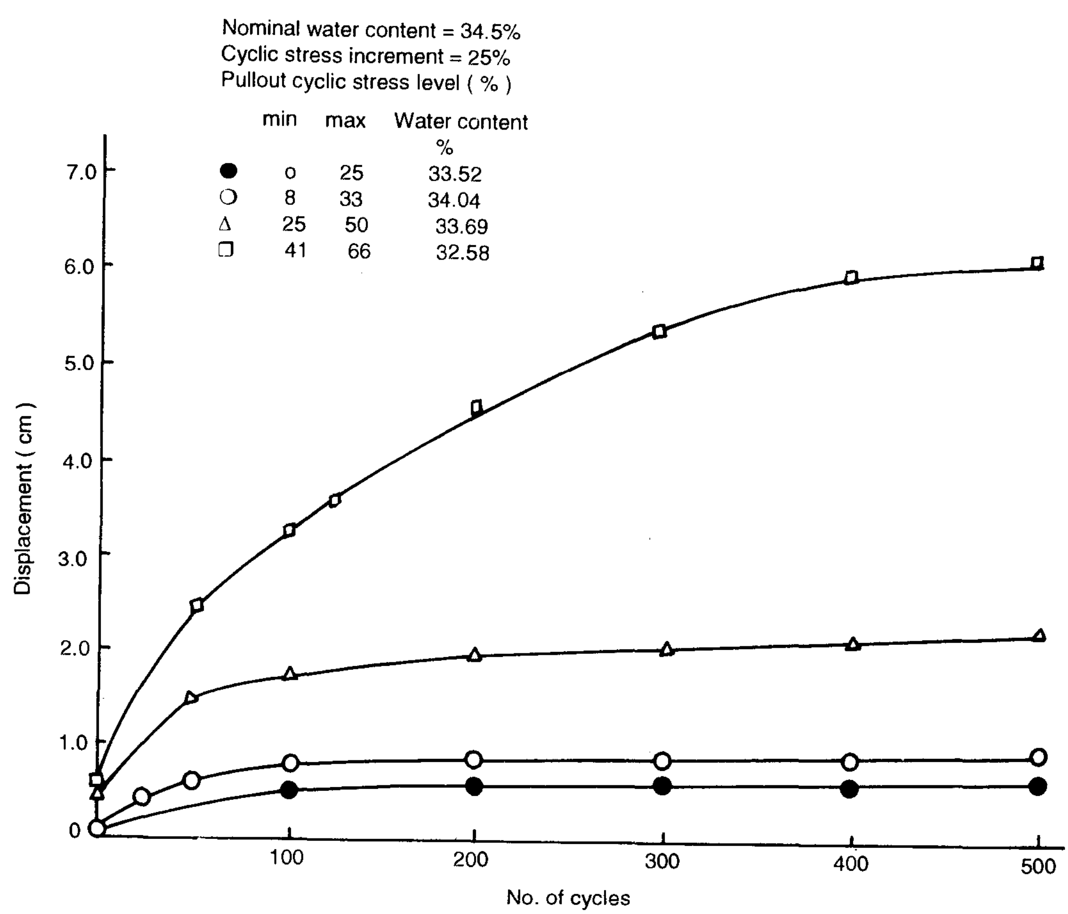

FIGURE 58. Movement of Plate Anchors with Number of Cycles for a Constant Cyclic Stress Increment--After Datta *et. al.* (1990)

TABLE 9

BREAKOUT THEORIES FOR ANCHORS IN CLAY

Investigator	Breakout Capacity Theory	D/B for Deep Anchor	N_{bc} for Deep Anchor
Vesic 1971	$S_u N_{bc}$	Soft Clay 2.0 Stiff Clay 5.0	9-10
Bemben et.al. 1973	$S_u N_{bc}$	4.0	9.0
Bemben and Kupferman 1975	$S_u N_{bc}$	4.5	9.4
Devie and Sutherland 1977	$S_u N_{bc}$	4.5	–
Das 1978	$S_u N_{bc}$	Soft Clay 3.0 Stiff Clay 7.0	9.0

TABLE 10

ESTIMATION OF P_{sv}

Researchers	P_{sv}/S_u
Bemben & Kupferman 1975	6
Nhiem 1975	5-7
Davie and Sutherland 1977	3
Le Tirant 1979	0.5

Table-11 presents the values of breakout factor due to suction, P_{sv}/S_u, for all the test variables studied from which one may conclude that suction is likely to develop only if the anchor is pulled out rapidly and that its magnitude diminishes as the water content of the soil increases. This conclusion has far reaching significance in relation to design of anchors for compliant structures in the off-shore environment where anchors are subjected not to strain-controlled pull out but rather to stress-controlled pull out and it is therefore, most unlikely that one can count on the suction mechanism to provide any significant contribution to the breakout capacity of the anchor.

Datta et.al. (1990) report the behaviour of plate anchors under cyclic loading. They conducted model tests on plate anchors in which the anchors were subjected to 500 cycles of different maximum cyclic stress levels as well as different cyclic stress increments and show that the amount of displacement on account of cyclic loading increases rapidly as the maximum cyclic stress level increases, see Fig. 58. For a maximum cyclic stress level, the amount of movement of the plate anchor is not significantly influenced by the magnitude of the cyclic stress increment as indicated in Fig. 59. From an analysis of the results they conclude that if a displacement of 20 percent of the anchor diameter is considered acceptable, plate anchors must be so designed that their static pull out capacity is at least 3 times the maximum cyclic stress level and if no movement under cyclic loading is to be tolerated, the maximum cyclic stress level would have to be kept well below 25 percent of the static pull out capacity.

TABLE 11

VALUES OF BREAKOUT FACTORS DUE TO SUCTION FORCE, P_{sv}/S_u, FROM STRAIN CONTROLLED TESTS

Breakout Time min	Pullout Velocity mm/min	w% 33.7	w% 35.4	w% 38.3
4	25	5.41	4.35	1.70
20	5	2.50	2.00	0.75
100	1	1.00	---	0.0
500	0.2	1.30	---	0.0

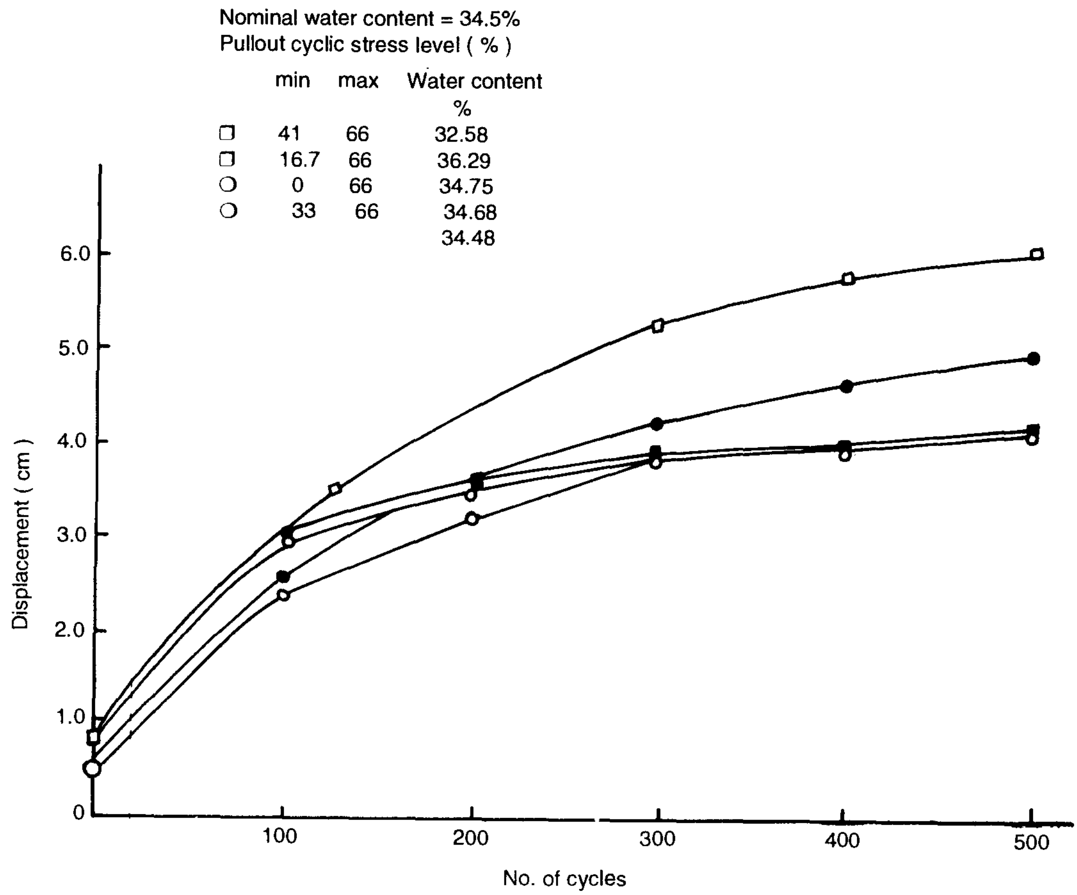

FIGURE 59. Movement of Plate Anchors with Number of Cycles for a Maximum Cyclic Stress Level of 66% After Datta *et. al* (1990)

CONCLUDING REMARKS

As was indicated at the outset, this Lecture was designed to generate interest in the soils and the soil engineering problems that exist off-shore of the Indian coast line. There is a large variety in these soils, in their state of consolidation, in the modifications they have suffered on account of the marine environment and they challenge us to understand their nature and their behaviour. There is even greater challenge in the soil engineering problems that are being encountered. Research has begun to address itself to these challenges. I hope this Lecture has succeeded in interesting some if not many to look beyond India's coast to India's off-shore environment.

ACKNOWLEDGEMENT

For many years research work has ceased being an individual effort and has become team work. I have been particularly fortunate in having been blessed with an excellent team of researchers. It gives me great pleasure to formally acknowledge them in the following few paragraphs.

The entire programme of research on off-shore soil mechanics began due to the optimistic enthusiasm of my colleague, Prof. G.V. Rao, and for many years we worked together. He has now moved on to a new frontier-Geosynthetics.

Perhaps, the maximum contribution to this research effort has come from my former student, then scholar and now colleague, Dr. Manoj Datta. His thought process is not just systematic, but enormously creative and his attention to detail and the perspective he has acquired over the years, is unmatched.

The bulk of the research work has been carried out in the form of Ph.D. theses, M. Tech. theses, and even B. Tech. projects. Dr. Datta's doctoral work on calcareous sands, Dr. M.R.M. Nambiar's work on calcareous clays and Dr.A.M. Deshmukh's work on Corals forms the back-bone of this Lecture and it gives me great pleasure to note that each of them is now independently pursuing further research in off-shore soil mechanics. Mr. Baleshwar Singh, Mr. N.U. Khan and Mr.Raghunath Dass are currently engaged in doctoral research.

The list of M. Tech. students is much longer and includes Mr. Amarjit Singh, Mr. S.J. J. Bari, Mr. R. S. Bassi, Mr. T. K. Chatterjee, Mr. R. D. Gupta, Mr.S. K. Gupta, Mr. Harsh Vardhan, Mr. P.M. Joshi, Late Mr. P. L. Juneja, Mr. K. K. Kampany, Mr. N. Kumar, Mr. Rajendra Prasad, Mr. Y. N. Ramamurthy, Mr. Ranjit Singh, Mr. K. Ravi Sundram, Maj. H.S. Sethi, Mr. S. P. Sharma, Mr. S. N. K. Sultania.

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Delhi is far away from the ocean environment. We are deeply indebted to Dr. S. L. Agarwal and Dr. P. C. Rawat of EIL, to Mr. S.K.A. Rao and Dr. P.K. Pant of ONGC, to Prof. Adrian Richards, Prof. R. C. Chaney, Dr. B. McClelland and to many others who made it possible for us to get close to the ocean, to the samples from the ocean and to the problems in that environment.

In establishing the Marine Geotechnology Laboratory at IIT Delhi, we have received financial support from research projects sponsored by EIL and ONGC as well as in-house support from IIT Delhi itself. In this endeavour, we have had the privilege of continuous guidance from my colleague, Prof. T. Ramamurthy and support from other members of the Soils Division, that is, Dr. K.K. Gupta, Dr. R. Kaniraj, Dr. J. M. Kate, Dr. K. S. Rao, Prof. K. G. Sharma and Prof. A. Varadarajan. Prof. B. S. Satija from Govt. Engg. College, Jabalpur worked in the laboratory for one year.

Dr. Rawat and Dr. Datta reviewed drafts of this paper and made very valuable suggestions.

To all mentioned above and to many who have not been mentioned, I wish to express my gratitude.

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