Towards Excellence in Geotechnical Engineering Practice in India*

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PREAMBLE

It is indeed a great privilege and honour to have been invited to deliver the IGS lecture 2002. Having spent in Academics over the last 41 years, the choice of the topic of the lecture namely "Towards Excellence in Geotechnical Engineering Practice in the Country" needs a bit of explanation.

Starting very early as a research student at the University of Karlsruhe in Germany, I had the unique privilege of associating myself with a number of projects involving foundation consultancy. The independent consultancy opportunity came at the Banaras University in 1968, probably one of the first of its kind for that University. Later starting from 1970 at IIT Madras very many challenging and interesting foundation problems have been referred to our group, which continued very intensively over a 25 years period. While I was at IIT Delhi between 1995 and 2000, my involvement continued with some of the major foundation consultancies, with the assistance of the colleague at IIT Delhi. The same is continuing at IIT Madras since 2000.

Consultancy services have been extended for a variety of projects all over the country and some abroad. The notable ones include Fertilizer plants, Power stations, Harbour structures involving almost all the major ports in the country, and industrial structures, involving every aspect of geotechnical engineering practice. I would also like to mention my long association with Larsen & Toubro, ECC Construction group as a retainer consultant to them for over 25 years.

India set itself the target of becoming a developed nation by the year 2020. The planned GDP growth rate for the 10th plan is 8% per year. A major focus of the efforts immediately and in the next decades is on infrastructure by way of highways, ports, airports, power, water, housing, sanitation and so on. The Geotechnical Engineers have a very important role to play in this challenging task. The geotechnical engineering practice in the country has to be on par with the best in the world. This prompted me to choose the title as given above.

As I started preparing the lecture, very quickly I realized that the subject is too vast and one has to be selective both in terms of the topics covered as well as the examples (case studies). Understandably I have chosen examples from my own experience. I would also like to add that the examples have been presented in a way to convey a message. There is no intention whatsoever to point out any special merits of myself and my colleagues or any deficiencies in any of the other parties involved.

Finally I want to pre-acknowledge the contributions of a large number of organisations and individuals from the client side and also my own colleagues both at IIT Madras and IIT Delhi and other institutions.

GEOTECHNICAL INVESTIGATIONS

General

High quality geotechnical investigaations are the very foundation for excellance in practice. Detailed specifications for the geotechnical investigations are available in our Indian standards (IS1892, IS2720). However

Quality of Investigations

INDIA	WORLD STANDARD
Generally Poor Quality of the Equipment	Highly Sophisticated and Mechanized Equipment
Calyx Drilling Technique	Continuous Core Sampling (in soils as well)
SPT Equipment Unchanged over the Years (unreliable)	SPT Equipment With Blow Energy Directly on Top of the Sampler
Conventional Static Cone Penetration Equipment	Static Cone Test with Electric Cone
Very recently few companies have electric cone	

the range of practices vary widely in India. In terms of field investigations, the most primitive equipments to fairly sophisticated equipments are in use. Unfortunately in majority of the cases the practice is nearer to the primitive one. Similarly in the laboratory testing also the practices widely vary. There is very little standardization and accreditation so far.

The result of these sub-standard practices is that in a large number of projects in which the author was involved, there where substantial differences between actual soil profiles and the profiles that are available at the time of design/as a part of the tender specifications.

Some of these are illustrated in Figs.1 to 11.

Main Problem

In Sufficient Appreciation with Regard to the Quality and Importance of Soil Investigation at all Levels. The responsibility for this squarely rests on the Geotechnical community of the country and is a major failure on our part.
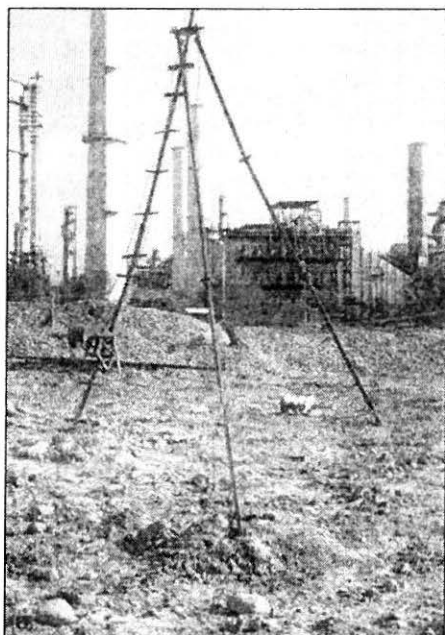

FIGURE 1 : Flimsy, Non-standard Equipment, Improper Testing Procedures

FIGURE 2 : Crude Method, No Safety Measures, No Assurance of Vertical
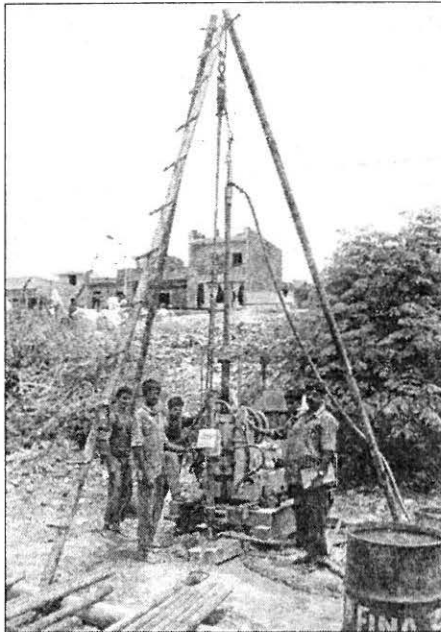Application of Load

FIGURE 3 : No Safety Measures, Bare Foot, No Helmets

FIGURE 4 : Auger Boring with Hand Operated Mud Pump

FIGURE 5 : Safety ?

FIGURE 6 : Cone Penetration Equipment

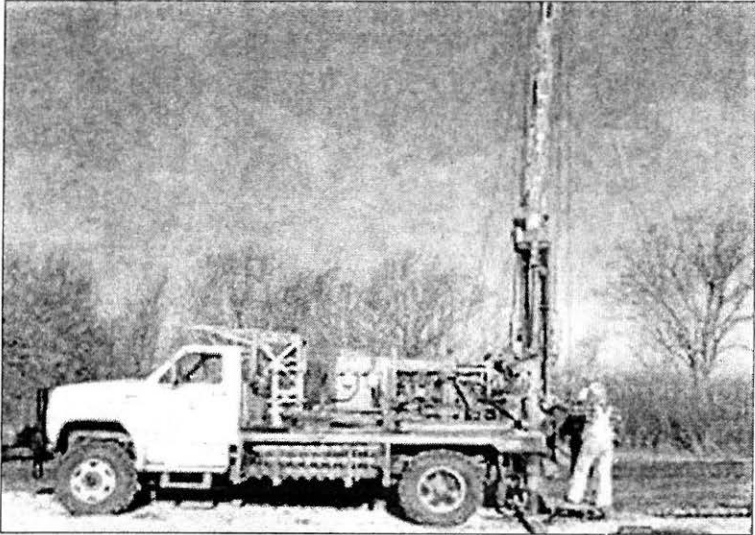FIGURE 7 : Improper Design of Loading Arrangement

FIGURE 8 : Truck Mounted Boring Equipment of High Standard

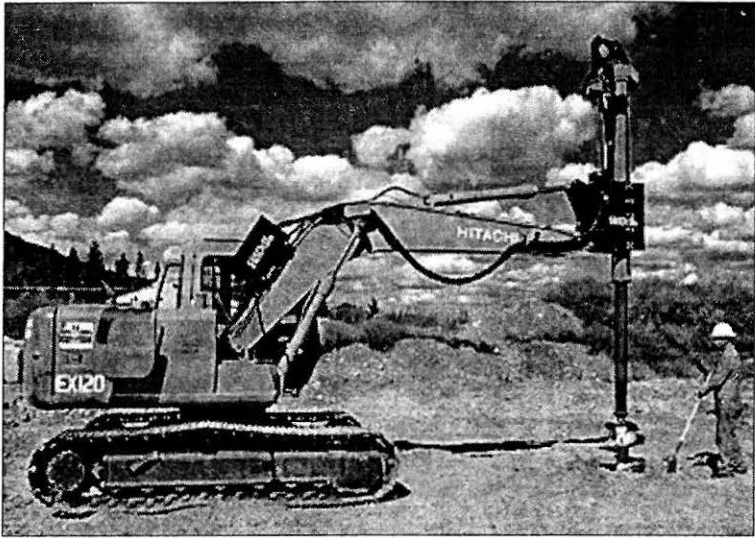FIGURE 9 : Auger Boring with All Necessary Safety Measures
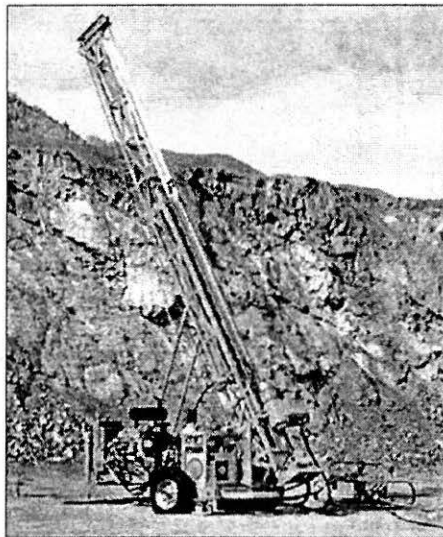
FIGURE 10 : Auger Boring

FIGURE 11 : Inclined Boring

Problems Arising Out of Deviation in Soil Profiles During Project Execution

It is a standard practice in India to stipulate in the tender for a project, that the information provided in the tender on subsoil profiles and soil characteristics is for general information only and the owner is not responsible for the correctness of these information. It is further stated that the contractor should satisfy himself about the correctness of the information and carry out, if desired, an independent soil investigation at the site before his offer for construction /execution of foundation and structures is given.

This stipulation, is supposed to safeguard the owner to avoid any disputes, if the soil strata during execution is found to be deviating from what is given in the tender document.

On the other hand from the contractor's point of view it is impossible for him to carry out an independent soil investigation within the limited time that is available between issue of tender documents and the time of submission of technical bids. Also it is expensive. However, for fear of the offer not considered by the Tender Committee, the bidder blindly accepts the stipulation in the tender as given above.

It has been noticed by the author that, in a large number of projects, there are substantial differences between actual soil profiles and the profiles that are presented in the tender document. The reasons for such variations are:

1. Inadequate soil investigation in the beginning (too few boreholes), last minute shift in the layouts with insufficient investigation in the new location, etc.

2. Improper investigation, testing and interpretation by the boring contractor/soil consultant.

Consequences of such deviations are obvious to be commented upon. Thus, there is an urgent need to find a satisfactory solution to these alarming problems. The author is of the opinion that the prime responsibility for the correctness of the soil investigation should rest foremost with the company carrying out the soil investigation and laboratory testing. They should be made responsible of any significant deviations found in the soil profiles and test results during subsequent soil investigation/execution of the projects. Any extra claims and damages that may arise out of the incorrect information in the soil investigation should be the responsibility of the party entrusted with the soil investigations. However to take care of the natural variations in the soil profiles at a given site, one has to work out some norms for permissible

deviations. It is reiterated that the variation to a very great extent will be minimized if standard practices are followed during the soil investigations, which unfortunately is not the case quite often.

For illustration two examples are given:

(1)   Road over bridge (ROB) near Madras (Fig.12 and Fig.13)

(2)   Petro chemical complex near Madras (Fig.14)

Figure 12 shows (case1) a comparison of two boreholes for a Road over bridges. The bridge has 5 spans of 10.7 m, with earthen embankments on either side. As per tender Borehole I, top two layers extending upto 7 m depth have SPT (N) values between 12 to 16 and the soil report recommended an allowable bearing pressure of 150 kN/m^2 for pier foundation at 2m depth below ground level. Accordingly, the piers were constructed on shallow foundation. When foundations for four piers were completed and the fifth was under construction, the approach earth embankment had a settlement of 2 m with corresponding heaving up of the soil by 1.5 m (Fig.13).

A Confirmation soil investigation was carried out and the average soil profile obtained is shown in Borehole II of Fig.12. This reveals that except
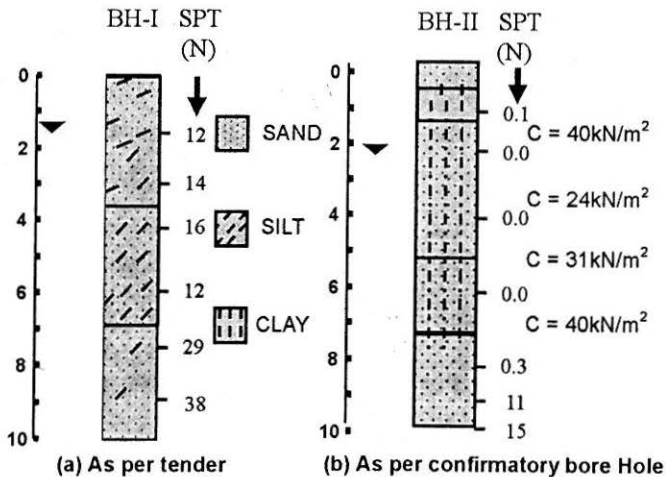
...... 'C' Values from Field Vane Shear Test

FIGURE 12 : Comparison of Bore Profile and SPT Values (Railway Bridge near Madras)
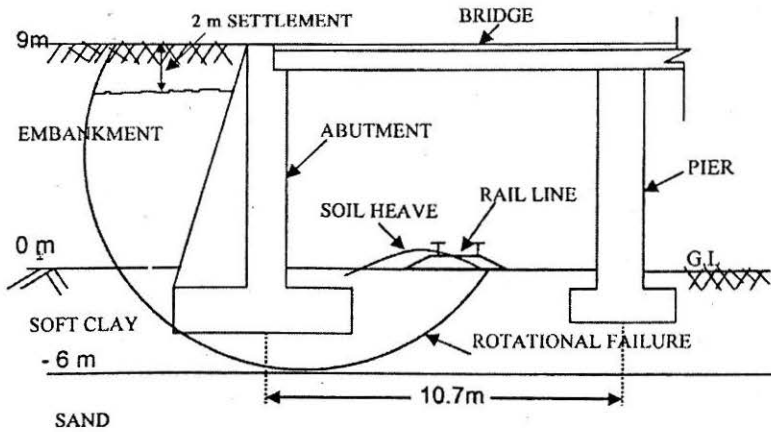
FIGURE 13 : Cross Section of Road Ove Bridge (Schematic Diagram)

top 1 to 1.5 m layer of sandy clay, the soil is soft marine clay extending up
to about 8 m below ground level. The rehabilitation measures included piles
installed around the shallow foundations and integrated with the foundation.
There was considerable delay in completion of the project and of course
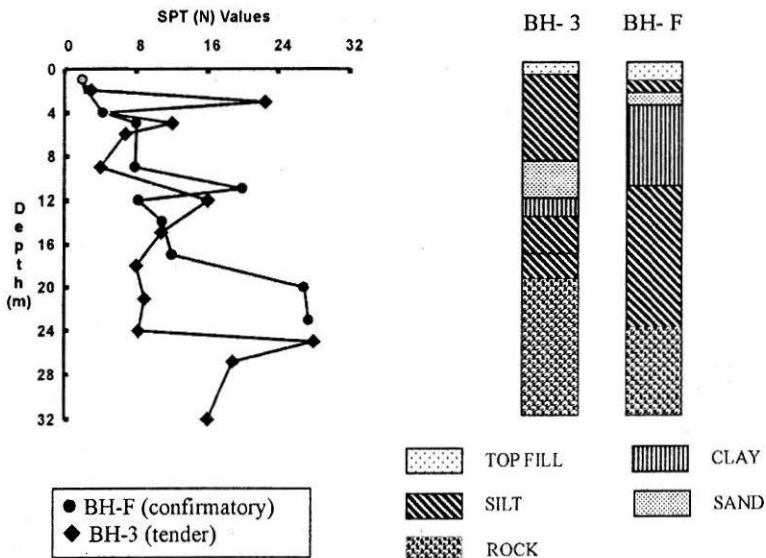additional costs and disputes as well.

**FIGURE 14 : Comparison of Bore Profile and SPT Values (Perto-Chemical
Complex near madras)**

Figure 14 show a comparison of two boreholes (Case 2), made just 3m apart from each other for conformation for the petrochemical plant. Twelve such confirmatory boreholes carried out showed consistently that the SPT (N) values reported in original soil report are higher and pile depths have to be around 25 m as against 20 m originally recommended. A test pile as per the original recommendation has also failed to take the design load. The confirmatory soil investigations have saved a major disaster. The pile penetration depths decided on the confirmatory boreholes resulted in large increase in piling quantity.

CHOICES OF APPROPRIATE FOUNDATIONS AND EXECUTION

An optimum foundation design should ensure technical adequacy, cost effectiveness and ease of execution. Achieving this is easily said than done. There are many variables including insufficient information as well as inaccurate information at the time of design, variations in strata and changes in project requirement during execution.

To illustrate some of these aspects the following case studies are presented:

1.   Foundation Problems on Soft Clay for a Fertiliser Plant

2.   Fertilizer Plant in Gangetic Belt – Possibilities of Optimum Design

Foundation Problems on Soft Clay For a Fertiliser Plant

Introduction

The fertiliser plant is situated near Kakinada town on the east coast of India. The site is within 1000 m distance from the coast with average ground elevations between 1 m to 2 m above the mean sea level.

The site area is more or less flat ground covering an area of about 1.75 km2. The soil strata comprises of recent alluvlum deposits in an alternating continental and shallow marine environment and formed by the sediments of river Godavari along with sand blown by wind.

The average soil strata consists of top layer of dense fine sand 4 to 8 m thick followed by soft marine clay up to a depth of 14 to 20 m below ground level. Below soft clay, a very stiff clay layer is observed up to the explored depth of 34 m. while both the top sand layer and the stiff clay layer are competent to support shallow foundations and deep foundations

respectively, the presence of soft clay layer between the two created following foundation problems which had to be seriously considered in designing foundations for various units of the plant:

1. Due to high compressibility under the height 1.2 m site fill, of soft clay deposits, the sand layer above also settles. In case of pile foundation these settlements will cause larger negative drag on piles both from the soft clay and sand layer. The magnitude of negative drag estimated from theory considerably reduced the pile capacity (up to 50%) and thus increases the number of piles.

2. For structures like bulk storage silos where large areas are loaded with uniformly distributed load of high magnitude, the stability of the soft clay layer under deep seated failure / lateral flow has to be checked. The lateral flow in soft clay layer may result in large settlement of the floor and also exert lateral forces on neighboring deep foundations.

Soil Characteristics at Site

Extensive subsoil investigations were carried out in view of the large variations observed in the strata conditions. These comprised of 95 bore holes 15 m to 35 m deep and 20 static cone penetration tests up to 20 m to 30 m depth. The strata, in general, consist of three layers, namely; top sand layer followed by soft marine clay layer and a very stiff clay layer. However, there are large variations in the thickness and density /consistency of the layers from place to place. A typical profile is shown in Fig.15 along with penetration results. Fine sand with thickness varying from 3 to 8 m. The ground water table is fluctuating in this layer between 0.5 m to 1.3 m above mean sea level.
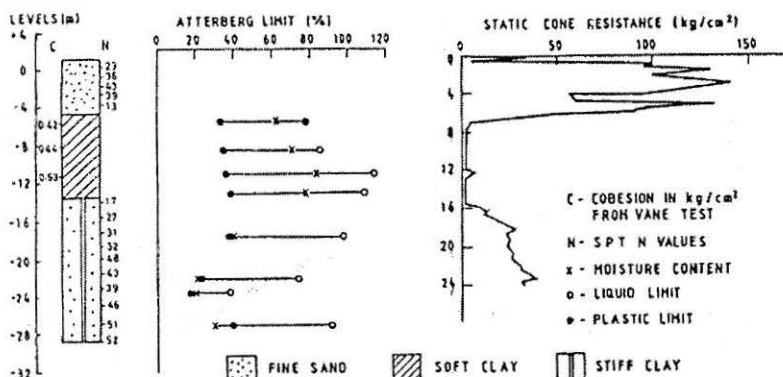

FIGURE 15 : Typical Soil Profile and Cone Penetration Results

Soft Clay Layer

This is grey marine clay with silt seams and lenses. Its thickness varies between 6 m and 18 m. The average properties are:

Natural moisture conten	:	50 - 80%
Liquid limit	:	70 - 100%
Plastic limit	:	20 - 40%
Modulus of compressibility (ES)	:	10 - 30 kg/cm^2
Coefficient of consolidation (CV)	:	2×10^{-4} - 4×10^{-4} cm^2/sec
Unconfined compression strength	:	0.2 - 0.4 kg/cm^2
Field vane shear strength	:	0.2 - 0.6 kg/cm^2

Stiff Clay Layer

Stiff to very stiff clay layer occurs below soft clay at elevation of 14 to 24 m below the ground level. The clay is fissured due to very stiff consistency of the layer and the low moisture content, undisturbed sampling in the layer was very difficult. At the time of extracting from sampling tube the samples were cracking. The average properties of the layer are:

Natural moisture conten	:	50 - 80%
Standard Penetration Value (N)	:	20 - 69
Natural Moisture content	:	25 - 40%
Liquid limit	:	70 - 100%
Plastic limit	:	20 - 40%
Consistency Index	:	1 - 1.2

Ground Water

The ground water has very aggressive chemicals. The average chloride content is 10 – 30 gms/litre against normal permissible limit of 1 gm/litre for reinforced cement concrete works. The sulphate content is also high varying between 2 – 4 gms/litre. Thus the ground water required special considerations like use of sulphate resistance cement, minimum 50 mm cover to the reinforcement etc, for all foundation works.

Settlements Arising out of Site Fill

The general ground level at the site was to be raised by about 1.2 m by placing a fill. The surcharge, due to the weight of the fill, of about 2 t/m^2 will be transmitted to all the layers below in full without any distribution.

This 2 t/m^2 load has certain implications with regard to the foundations, because of the very soft clay layer below the top sand. The main point is to what extent this increase in load will result in settlement of the clay layer.

Conventionally (as per the present-day design practices), the settlement, 's' is calculated as follows:

$$s = m_v \, \Delta\sigma \, H$$

where
$$m_v = \text{coefficient of volume decrease}$$
$$\Delta\sigma = \text{increase in vertical stress}$$
$$H = \text{thickness of layer}$$

Based on the consolidation test results the average m_v value for the soft clay can be taken as 0.05 cm^2/kg. The thickness of soft clay layer varies between 6 to 18 m. Taking the average thickness as 12 m, settlements s = 12 cm. Therefore depending on the location the settlement will vary between 6cm to 18 cm. However, according to the theory, it takes a long time for these settlements to occur. For a 12 m thick layer with an average coefficient of consolidation, $C_v = 2 \times 10^{-4}$ cm^2/sec, time required for 90%, 50% and 10% settlement is 194, 45 and 2 years respectively.

The situation will be drastically different due to the presence of silt seams/lenses and it is impossible to quantify their effect. Qualitatively the settlements would take place much faster.

Yet another aspect to be considered is the limitation of the consolidation theory with regard to its applicability for cases of low stress while using in highly plastic clays of the present type. These clays have what is known as a threshold or stagnation gradient and for gradient below this value, there will be no consolidation and consequently no settlements.

Foundations Adopted

These are:

i)   All heavy and settlement sensitive structures are supported on pile

foundation. The negative drag to be considered was estimated for different situations. Also, the precast concrete piles adopted were applied with a bitumen coat in the top two layers, which very effectively reduced the negative drag.

ii)   All light structures, not sensitive to settlements are supported on shallow foundations. Though the settlements estimated are high, most of these settlements will be uniform (due to compression of the soft clay layer). Also the rate of settlement will be low.

iii)   For structures having distributed surcharge load over large area (e.g. silos, water storages, etc.), stability is achieved by providing peripheral bunds to reduce the shear stresses on soft clay.

Pile Foundations

While adopting the pile foundations, the following aspects have been considered:

(a)   Any settlement of the soft clay layer either under the fill or due to any other reason would result in significant negative friction (drag) on the pile both from the soft clay as well as from the top sand layer.

(b)   To make the foundation economical, it would be desirable to reduce the negative drag by appropriate means.

(c)   The support for the pile has to come from the stiff clay layer.

Two alternative pile types have been considered:

        a)   Precast driven piles

        b)   Bored cast-in-situ piles

From the point of handling and driving capabilities available, the optimum dimensions for precast pile were 40 cm × 40 cm square, 22 m long.

For a final decision with regard to the pile type, full scale tests were carried out up to failure on 6 test piles, 3 each of the above two types. The piles were installed in three different locations in pairs of bored and precast driven piles in close proximity. All the piles were 22 m long except pile P1, which was 19.5 m long. The equivalent diameter for the bored pile was 45 cm.
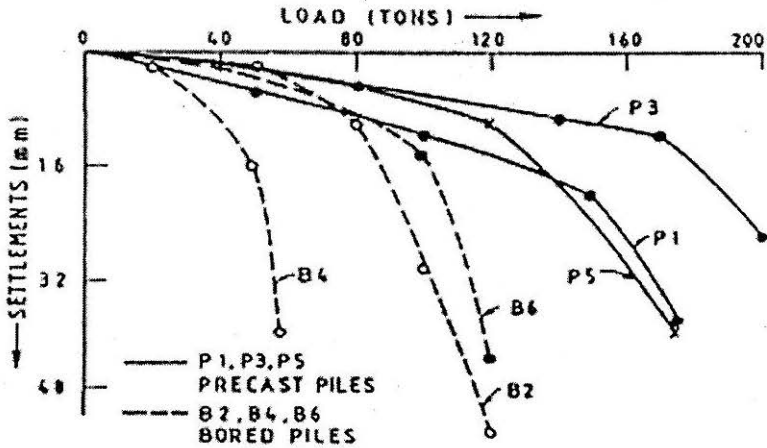
FIGURE 16 : Results of Pile Load Tests

Bored cast-in-situ pile were installed by regular bailer boring method and concreting with a tremie pipe. Precast piles were installed first by water jetting in the top sand layer and subsequently driving with a drop hammer of 4–5 t-m energy per below. The final settlement observed was about 20 mm for last 10 blows.

The loading was applied by regular kentledge arrangement. As the top sand layer was expected to settle, no resistance from this layer was considered in pile design. To simulate this condition during load test, an annular gap was created in the sand layer around the pile by placing an oversized casing up to the soft clay layer. The gap between pile and the casing was filled with bentonite slurry.

The results of all six load tests are shown in Fig.16. As can be seen, the precast piles behaved better and carried 40–80% more load compared to board cast-in-situ type. The results are analysed in Table 1 wherein the allowable load on each pile is worked out as per Indian Standard (IS 2911). Finally precast of driven piles with design load of 70–100 tons were adopted for the following reasons:

i)   For comparable dimensions, the precast pile has about 50 to 80% higher structural capacity than a bored pile.

ii)   Precast pile offers better protection to reinforcement than bored pile. This is particularly important under the present saline ground water conditions.

Table 1 : Working Load in Tons as per IS:2911

Pile No.(1) (1)	2/3 of load at 12 mm settlement (2)	Failure Load (3)	Half of Failure Load (4)	Working load (smaller of 2 and 4) (5)
P1	73	160	80	73 (100)*
B2	55	108	54	54
P3	110	194	97	97
B4	32	54	27	27
P5	85	164	82	82
B6	65	120	60	60

* The pile length is 19.5 m, and failure load extrapolated for 22 m length is 100 tons.

iii)   The jetting up to 12 m will reduce the friction over this depth and consequently the effects of negative drag.

iv)   In case of precast piles, the negative drag can be reduced by applying a slip layer of bitumen.

v)   The reduction factors required for group action in driven piles are generally less than in bored piles.

Bored piles are not preferred for this site for the following reasons:

i)   Boring operations through stiff fissured clay will result in softening of this layer, thereby limiting the load carrying capacity of piles.

ii)   In case of bored piles, there is no possibility of applying any slip layer and to reduce the negative drag.

Anticipated Negative Drag

It is very difficult to estimate the negative drag which depends on many factors like:

(a)   magnitude and rate of settlement of the strata,
(b)   settlement of the piles,
(c)   method of pile installation,
(d)   the presence of slip layer, if any, and its long term behaviour,
(e)   whether it is an isolated pile,
(f)   the time lapsed between the placement of site fill and pile installation etc.

Finally the negative drag for an isolated pile in the present case has been estimated as 40 tons, based on theoretical considerations, load tests on short piles resting on soft clay and also instrumented piles where the load distribution with depth has been measured in a test.

For proper choice of the bitumen coating (material and thickness), method of application and their effect in reducing the drag, model tests were carried out in laboratory in sand-bed. Model piles adopted were precast concrete 10 cm diameter and 50 cm long. The piles were applied with coatings of three different grades and in three different thicknesses. The ditumen coast was applied by a brush after heating at 50°C. The results showed 80 to 90 % reduction in frictional value (Kandasamy, 1979). Based on this study 3 mm thick coating of SAE 80 grade bitumen was adopted and 50% reduction in drag force was assumed for pile design.

Finally, following drag values were adopted in pile design:

i)   For isolated pile, piles in a small pile group and peripheral piles in case of large pile group (e.g. a storage tank foundation) the negative drag considered is 20 tons;

ii) For interior piles in a large group the maximum drag is restricted to the weight of the fill on equivalent area of pile which works out to 7 to 8 tons.

About 20 members of initial pile load tests subsequently have confirmed the predictions of the pile behaviour.

Foundation for Lightly Loaded Structures

The top 3 to 8 m thick sand layer (except few locations where the thickness is less than 3 m or missing) is a good bearing stratum. For economy and speed in construction, full advantage of this layer was taken by adopting shallow foundations for light structures which are not sensitive to settlements. Depending on total thickness of top sand layer plus site fill available below the foundation, the maximum width of foundation was restricted to 25% of the sand thickness to ensure no significant stresses on the soft clay layer. Thus, the settlements arising out of low stress transferred to soft clay will be less and of uniform nature. The bearing capacity adopted was 15 t/m^2 for strip and 20 t/m^2 for individual rectangular foundations. The settlements expected are within 25 to 40 mm.

Problems Associated with Storage Structures

The plant consists of few structures having high surface loading (6 to

15 t/m²) spread over a large area (40 m to over 100 m). These are Urea Silo, Bagging Plant, Water Storages, etc. In view of the large lateral dimensions, most of the surfaces load will hardly and distribution. Therefore following points have to be considered for safety of foundations:

1.   Bearing capacity failure at the soft clay level.

2.   Side slip circle failure along edge of the structure.

3.   Possibility of lateral flow in soft clay and its effect on neighbouring deep foundations.

One straight forward solution is to provide pile foundation for the entire floor area. But in view of the large area, this would be too expensive and also time consuming. Also, large settlements can be permitted for such storage flooring with minor repair for economy. To assess the problems associated with large area fills, a trial fill has been made

Observation on Trial Fill

The 5 m high trial fill measuring 30 m × 30 m at ground level and 15 m × 15 m at 5 m height was instrumented with two inclinometer tubes and five settlement gauges as shown in Fig.17. The stresses transferred at the top of the soft clay layer are theoretically worked out and shown in Fig.17. As can be seen, the stress at the centre is equal to the total intensity of 5 m
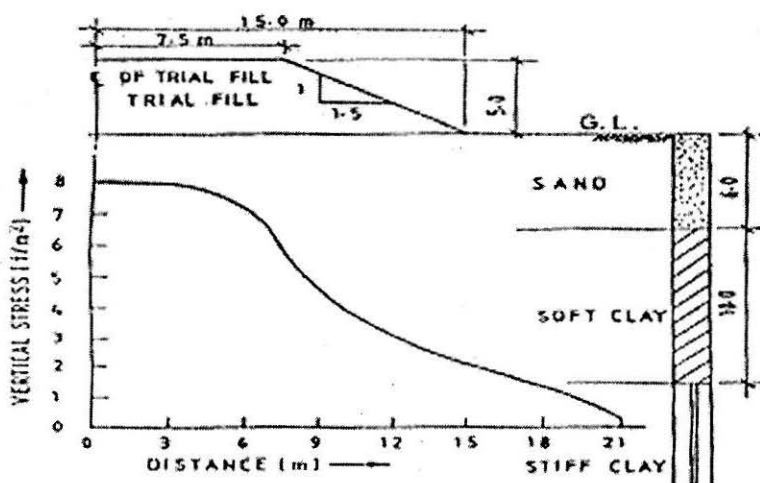

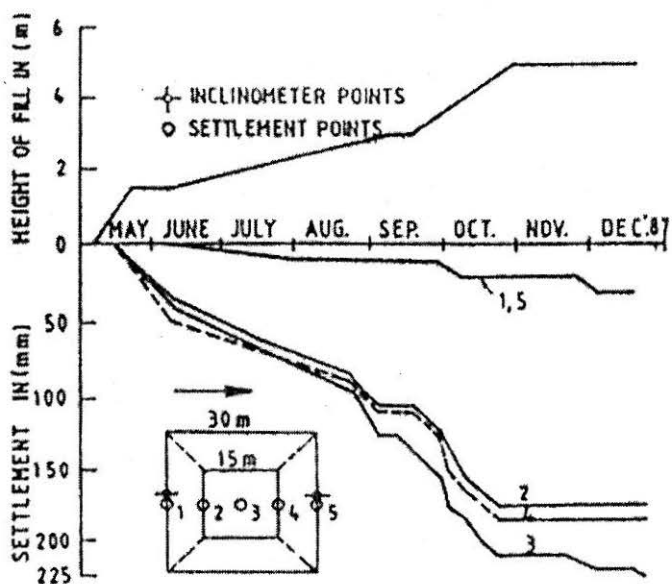FIGURE 17 : Vertical Stress Distribution below Trial Fill on Top of Soft Clay

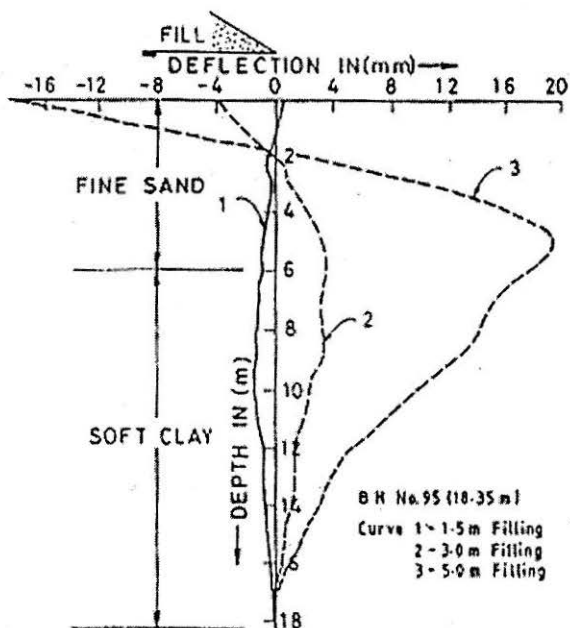**FIGURE 18 : Settlement Below Trial Fill**

FIGURE 19 : Horizontal Displacements Measured under Toe of the Trial Fill

high fill (about 8.5 t/m^2) which reduces along the sloping surface as shown in the figure.

The resulting settlements measured by settlement gauges and lateral movements measured by the inclinometer are shown in Fig.18 and Fig.19 respectively. As can be seen the maximum settlement observed is 225 mm with settlement almost stabilizing over a period of six months only. Maximum lateral movement observed by inclinometer (Fig.19) is also within 20 mm.

The back calculated soil modulus works out to 40 kg/cm^2, and as the settlement increases the value of modulus will reduce.

Summary and Conclusions

At a fertiliser plant site on east coast of India the average soil profile consists of 6 m thick top sand layer followed by 8 m of soft clay overlaying very stiff fissured clay. This stratification posed several foundation problems including those arising out of settlements from the soft clay layer under a 1.2 m thick fill necessary for raising the general ground level. Studies carried out included detailed soil investigation, full scale load test on 3 pairs of precast and bored cast-in-situ piles to arrive at the final choice of the pile and a 5 m high trial fill to study the behaviour of the soft clay under area loading. These studies resulted in economical and adequate foundations, which are as follows:

i)   For heavily loaded structures and structures sensitive to settlements, precast driven piles were adopted. To reduce negative drag, a layer of bitumen coat of SAE-80 grade has been successfully used.

ii)   For structures lightly loaded and not sensitive to settlements, shallow foundations have been adopted taking advantage of top sand layer. This resulted in considerable economies.

iii)   It is very difficult to predict the behaviour of soft clay layer under surface loadings. A trial fill was of a great help in assessing this behaviour. Monitoring the settlements of the structures is highly desirable.

Fertilizer Plant in Gangetic Belt – Optimum Design Possibilities

The soil strata at site is essentially silty sand with low N values (< 10) up to 10 m depth, N > 20 beyond 20 m depth and N–values ranging between 10 to 20 for the layer between 10 to 20 m depth. There was high water table with threat of Liquefaction during Earthquake.

Phase-I Foundation Design

Considering all the aspects it was decided to provide RCC piles, driven cast in situ, 400 mm diameter for most of the structures. In addition sand compaction piles 2 to 3 rows around RCC piles were provided to overcome the problem of liquefaction during earthquakes envisaged. The pile capacity chosen are axial vertical load 50 tonnes (downward), uplift 5 tonnes. Horizontal capacity is 2.5 tonnes. The above resulted in the following total requirement:

16000 RCC piles and 32000 sand compaction piles. Installation of such a large number of piles became a major issue. With the capacity available in the country at the time it was estimated that the time required for installing the RCC piles and compaction piles will be 6 months more than what was originally envisaged for the project. This six months delay would result in an un-acceptable time over run and financial losses. This prompted the owner to review the foundation design and the same was entrusted to the author and his colleagues.

Revised Spacing for the Sand Compaction Piles

In original design the compaction piles spacing of 3D was chosen based on trial compaction with two different spacing in a triangular pattern. These spacings were 5D and 3D. The results have shown that while 3D spacing gave the desired improvement in N-values, the improvement with 5D spacing is not adequate.

Additional trial compaction was carried out with the spacing of 4D. The results have clearly shown that the 4D spacing is adequate to obtain required densification as measured by N-values. This has enabled an increase in the spacing of compaction piles from 3D to 4D. As the number of piles required are inversely proportional to the spare of the required spacing it has resulted in over all reduction of compaction piles by about 50% i.e. from 32000 compaction piles to 16000.

Pile Capacities: Review of Pile Load Test Results and Additional Tests

The results of the all the available pile load tests were reviewed and some additional tests were carried out particularly relating to the uplift capacity. This enabled increasing the pile capacities as given below.

Revised pile capacities:

Vertical downward – 65 tonnes instead of 50 tonnes original

TRIAL AREA	C OF CO-ORDINATES		L (m)	B (m)
PRILL TOWER	754N	300E	9.60	7.80
BENFIELD AREA	715N	100E	12.90	10.20
COMPRESSOR HOUSE AREA	733N	20E	10.80	9.40

TRIAL AREA	SPACING 'S'
PRILL TOWER	1.50
BENFIELD AREA	2.15
COMPRESSOR HOUSE AREA	1.80

LEGEND

X SPT-PRECOMPACTION;   • = SPT-PRECOMPACTION

■ = SCPT-PRECOMPACTION;   ● = SCPT-PRECOMPACTION

▶ = DCPT-PRECOMPACTION;   ∨
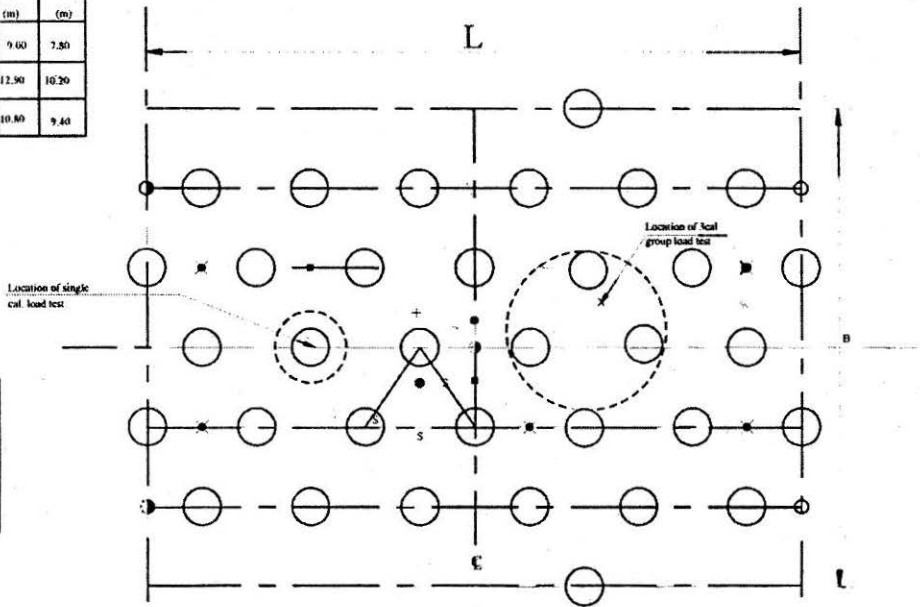
LOCATION OF PRE & POST COMPACTION
TESTS IN A TEST AREA

Location of single cal. load test

Location of 5cal group load test

FIGURE 20 : Location of Pre and Post Compaction Tests in a Test Area

Uplift capacity   —   25 tonnes instead of 5 tonnes original (the pile
                          could not be pulled out at a vertical load of 50
                          tonnes)

Lateral capacity   —   3.5 tonnes as against original 2.5 tonnes

The revised capacities have reduced the requirement of RCC piles by 40% i.e from 16000 piles to 9400. The pile foundations were executed as per the revised design.

Observation of the Performance of the Foundation

Because of the optimum design that was adopted for the plant all the structures were closely observed and the performance of the foundation was fully adequate and satisfactory.

Phase-II of the Plant

Subsequently it was decided to double the capacity of the plant by adding a plant similar to the Phase I plant in the neighbourhood. This time the alternative of vibro-stone columns for ground improvement were considered in detail in place of the RCC piles and sand compaction piles. Extensive field trials work carried out with different spacing of vibro stone columns of 960 mm diameter. A large number of load test were carried out on single columns and a groups of columns. Several footing tests were also conducted and/envisaged for confirmation, during execution.

Trial Tests Using Vibro-Stone Columns

In order to verify the improvement achievable, three test plots measuring approximately 10 m × 10 m each have been selected in prill tower location, benefield area and compressor house. Each of these areas is treated with 960 mm diameter vibro stone columns up to 11 m depth installed in a triangular pattern with center to center spacing of 15 m, 2.15 m and 1.8 m respectively. The general individual layout for pre and post compaction test locations are shown in Fig.20. The Vibrofloat Columns were installed by M/s. Cemindia Co., Ltd., Calcutta and the procedure is given below.

Installation Procedure

In this technique stone columns are formed by Vibrofloat equipment. Vibrofloat a mechanised large high frequency vibrator (Vibroneedle) is attached to a string of follow on tubes of required length. The whole assembly is then hung from a crawler crane and placed vertically over a

stone column point laid out earlier. High pressure water passed through the assembly emanating as water jets at the bottom aided with the vibration of vibrofloat needle allow the total assembly to sink into the ground and thus a hole will be formed. After reaching the envisaged depth, the bottom water jet is stopped and the side water jet alone is operated. The stone backfill is poured through the annular space and compaction of stone metal will be done in lifts with the vibroneedle aided with effective vibration (generally kept between 1600 to 1800 rpm). For effective compaction, the increase in hydraulic pressure must be of the order 17 to 20 kg/cm^2. The backfilling and compaction is continued until the column is formed up to the ground level.

Pre and Post Compaction Test Results

Pre and Post density tests were carried out in all the three test plots. For each test plot, two boreholes along with SPTs were carried out on virgin soil and after treatment. In addition, two SCPT's and five DCPT's have also been carried out for each test plot. One single stone column footing load test and one test on three stone columns group were conducted after installation
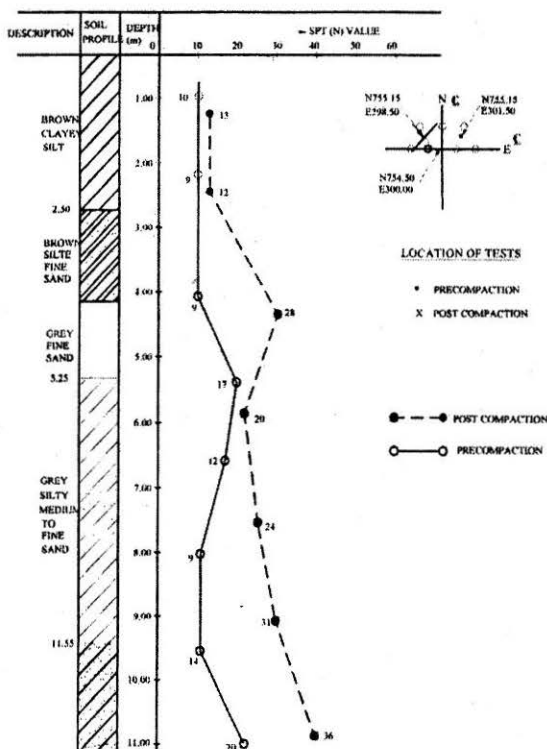

FIGURE 21 : Comparison of Pre and Post SPT in Prilling Tower Area

of stone columns. The footing size for single columns and three column groups were chosen considering the individual stone column and its contributory area of soil surrounding it. The locations of various tests carried out is shown in Fig.20.

The results are analysed and discussed in detail:

Standard Penetration Test (SPT)

The test were carried out as per IS 2131. The SPT was conducted at depth interval of 1.5 m or at depth where the strata changed. The test is carried by driving a standard split spoon by means of 65 kg hammer with a 75 cm free fall.

The 'N' values given in Fig.21 for prill tower for both pre and post compaction. The approximate improvement in N values are given in the Table 2.

Static Cone Penetration Test (SCPT)

The test were carried out as per IS 4968, Part III. From the test depth, a 10 cm^2 area cone is pushed for a depth of 10 cm and cone resistance noted. Thereafter both jacket and cone are pushed together for another 10 cm and thereby combined resistance of jacket plus cone is noted. This procedure is repeated till the depth of 11 m is reached. The cone resistance values are plotted with respect to depth as shown in Fig.22. The test results confirm the increase in cone resistance with depth after stone column installation.

Table 2 : Approximate Improvement in N Values

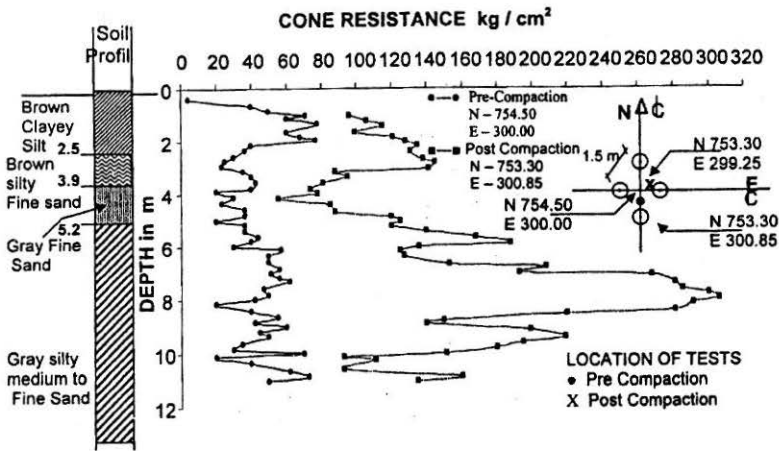| Area	Depth	Layer	% Increase in 'N' Value	'N' After Treatment
Prill Tower	0 - 2.5	Silty Clay	30	13
	2.5 - 11	Silty Sand	17 - 120	20 - 36
Benefield Area	0 - 2.3	Silty Clay	81	20
	2.3 - 11	Silty Sand	36 - 123	16 - 34
Compressor House	0 - 3.5	Silty Clay	01	11
	3.5 - 11	Silty Sand	60 - 400	24 - 46

**FIGURE 22 : Comparison of SPT Results in Prill Tower Area
(Spacing 1.50 m)**

Dynamic Cone Penetration Test (DCPT)

The tests were conducted as per IS:4968, Part III by driving a standard size cone (62.5 mm dia) provided with vents for flow of bentonite slurry and attached to a string of drill rods. A continuous flow of bentonite slurry is pumped through the rods and the cone to avoid friction between the rods and the soil. The number of blows, required for every 0.30 M penetration of cone is being recorded.
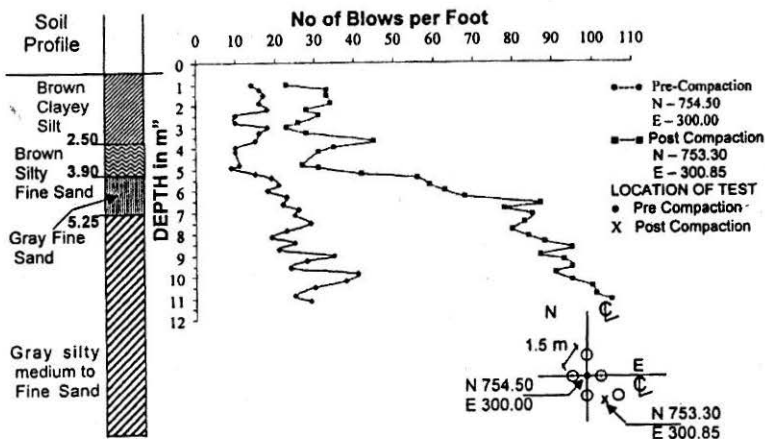
**FIGURE 23 : Comparison of DCPT Test Results in Prill Tower Area
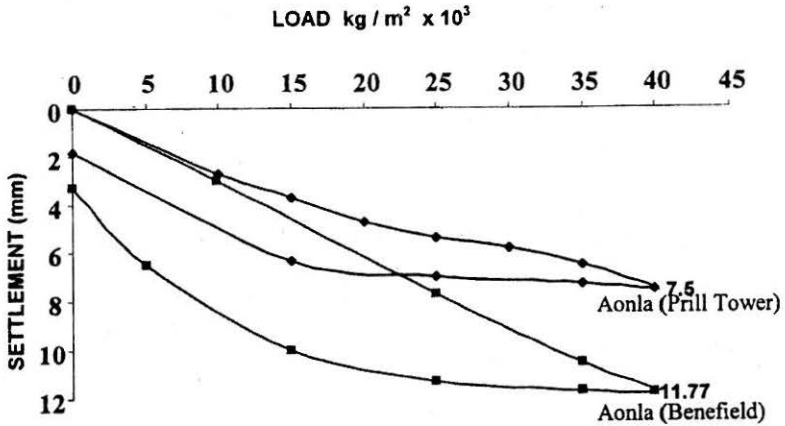(Spacing 1.50 m)**

LOAD kg / m² x 10³

FIGURE 24 : Load Settlement Curves for Single Column Load Test

The DCPT values (N_{cbr}) are plotted with reference to depth for prill tower as shown in Fig.23. The results confirm the increase in value of Ncbr with depth after stone column installation.

Footing test on single and group of columns

The single as well as 3-column group tests were conducted for 40 T/m² and 25 T/m² respectively. A circular R.C.C. footing having an area equal to the area represented by the columns with chosen center to center spacing was placed over the columns as shown in Fig.20. The results are shown in Figs.24 and 25.
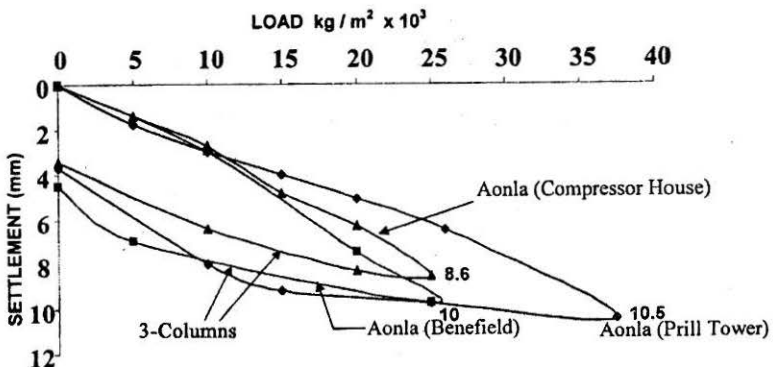
LOAD kg / m² x 10³

FIGURE 25 : Load Settlement Curves for Single Column and 3-Column
Group Load Test

As can be seen, in each case, the settlements are in the range of 6 to 10 mm only and the results are satisfactory.

Finally 960 mm diameter vibro stone columns with varied spacings of 2D, 2.25D and 2.5D were adopted depending on the loading intensity and permissible settlements. This has resulted in substantial savings in time and cost. Subsequent observations during the operation of the Phase II of the plant is confirmed the fully satisfactory behaviour of the foundations.

BERTHING SRUCTURES

Introduction

From foundation angle berthing structures can be classified as

    (a)   Gravity Structures
    (b)   Pile Structures

For any gravity structures we may have either a block wall or a caisson filled with sand. In case of pile structures, they may consist of sheet pile or diaphragm wall with tie-back or open type pile structure or a combination of both. Some typical arrangements of various type structures are given in Fig.26.

Berthing structures are subjected to vertical and horizontal loads. The combination of these forces will result in vertical forces and moments at the sea bed to be suitably transferred to the soil/sea bed

Foundations

Gravity Foundations

Gravity structures essentially achieve stability by means of their large deadweight due to huge dimensions. Gravity foundations are possible only with strong seabed strata. The vertical loads are transferred as the soil reaction at the base, horizontal forces as base shear and moments as non uniform base pressure at the interface between foundation and soil.

The Geotechnical problems involved are:

(a)   Installation of the structure
(b)   Stability under the effect of its weight and repeated loads
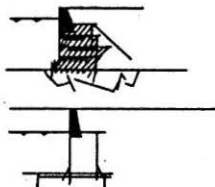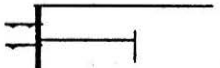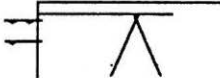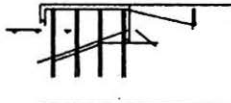(c)   Settlement of soil below
(d)   Scour around the foundation
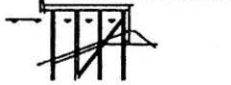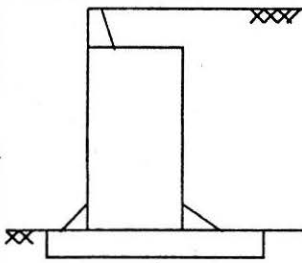
	TYPE OF STRUCTURE	
VERTICAL FACE		BLOCK WALL
		CAISSON
		SHEET PILE WALL WITH TIE ROD
		SHEET PILE WALL WITH RAKER PILES
OPEN		OPEN PILED STRUCTURE
		OPEN PILED STRUCTURE WITH RELIEVING PLATFORM
		CAISSON

FIGURE 26 : Various Types of Arrangements

Pile Foundation

The forces are transferred to the soil by way of axial forces, horizontal forces and moments at the sea bed level. There are fairly well established procedures to design piles for given set of forces and sea bed conditions. A variety of piles are possible, the most commonly adopted in India are:

(a) Large diameter bored piles   750 mm to 1500 mm dia

(b)   Precast prestressed pipe piles   800 mm to 1000 mm dia
                                               wall thickness 150 to 200 mm

(c)   Precast Concrete piles   400 to 450 mm square

Case Studies

Here 5 case studies are presented each typical in its own way. These are:

1.   Oil Jetty at Tuticorin Port
2.   Fertiliser Berth at Paradeep Port
3.   3rd Cargo Berth at Paradeep
4.   7th Cargo Berth at Kandla
5.   Earth Quake Damage and Repair of Berths at Kandla Port

Oil Jetty at Tuticorin Port

A piled berthing structure was being built in Tuticorin for the first time in the year 1979. In the contract it was envisaged that the piles will be taken 2.5 m in to the rock; however during execution in the first 3 piles, the pile depths were as high as 22 m into the rock. The contractors contention was that the rock was very soft and unless the piles are taken to that depth the safety of the berth is not ensured. Accepting such a large depth for the piles would have meant an extra cost of 75 to 80% compared to the original estimate. At this point of time the assistance of the author was sought by the port authorities. The presentation is based on the experiences gained in the process of study, investigation and a final solution to the problem. The pile depths finally adopted varied between about 4 m in case of approach jetty and about 8 m in case of berthing dolphins.

For the jetty in 11 m water depth, large diameter bored piles (800 mm and 1200 mm dia) have been provided. The founding stratum is a weak calcareous sand stone. The details of the procedure adopted to arrive at pile embedment depths and the results of full scale load test on 800 mm diameter piles are presented. The problems associated with investigations and design procedures in weak rocks are briefly considered and discussed.

Piles in Weak Rock - Introduction

The increasing use of high capacity piles has resulted in increased attention to design problems for piles in weak rocks. Most of the deposits are weathered to certain extent and in such cases the point resistance alone is not sufficient for high capacity piles. In such cases some more loads is usually transferred to the rock by means of shaft adhesion/friction achieved

by socketing piles into such rock. Specific design procedures for these cases are not available and often designs are done using some empirical methods or local standards of practice which are mostly conservative.

From available literature, it seems possible to establish approximate empirical relationships between SPT values and the skin friction, end bearing resistance and modulus of deformation for piles. These relationships will be specific to each type of rock because ultimate strength values are sensitive to the effects of installation, in addition to the condition of the rock (extent of weathering, spacing of discontinuities, etc.).

Investigation method that can be adopted for a particular case depends on many factors like, rock condition, accessibility, diameter of shaft, availability of equipment's, time, budget, etc. Few of the investigation methods which are generally used in week rocks are very briefly described indicating their suitability and limitations

Core Boring

Rock cores of varying diameter are extracted by diamond core drilling methods. Although the rock cores are only relatively small samples of the rock and do not clearly indicate the spacing of the rock defects etc., it does give some understanding of the rock. Such rock cores are then tested for crushing strength. Using empirical relationship shaft adhesion/friction and ultimate end bearing pressure can be determined. In some cases sample preparation may not be possible, and in such cases the point load strength index may be used.

When the rock is highly weathered, the core recovery will be very poor and the largest core may not be sufficient for crushing strength test. Using the latest developed techniques like Triple Tube Retractor Core Barrels, it may be possible to obtain a good quality core even for extremely weathered formation. However enough experience for this type of technique is not yet available in the country (India)

Standard Penetration Test (SPT)

Though it is a crude test, in addition to simplicity, it has the advantage of SPT values being related to a large body of practical experience. In hard strata where 30 cm penetration of the spoon is not possible the results are extrapolated. This sometimes gives misleading information. Since the blows for driving the sampler into the subsoil are applied above ground level, the length and horizontal rigidity of the extension rods play an important role. When working at relatively great depths a large portion of the impact energy may be absorbed by horizontal vibration giving higher N values.

Design Procedures

In most of the methods available, shear strength of the rock is arrived at either based on broad rock classification or based on some in situ tests like standard penetration test. The accuracy in design depends on engineering judgment in selecting the shear strength of rock. Since safety is of paramount importance than economy in major projects, the design is usually made on a conservative side.

Method based on SPT (N) values

From the N values, the shear strength parameters are obtained and allowable load on pile is calculated using the formula:

$$Q_{allow} = q_a \left(\pi D^2/4 \right) + \alpha C_s \pi DL$$

Though the method may have some times serious limitations in view of the limitations of the correlation itself and also the N values (already discussed), in certain cases, where more precise measurements are not possible (for one reason or other), may prove as a useful design tool.

Method based on German Standard DIN 4014

Based on large number of load tests on large diameter bored piles in Germany and all over the world, certain values are recommended for pile base pressure for different types of rocks. Side friction is to be taken as 10% of the base pressure. As per this code the resulting settlements will be of the order of 10 mm. Minimum depth of embedment is specified as 0.5 times the diameter of pile. For normal loading condition a factor of safety of 2.0 is recommended. The only limitation of the method is that the values are grouped on very broad classification of rock weathering. It is very difficult to arrive at ultimate base pressure from these values.

The Oil Jetty

The Structure

Figure 27 shows the layout plan of the structure. Berthing dolphins on its either side are designed to resist the horizontal impact when the ship berths. They are provided with 2.6 m thick deck supported on 1200 mm dia piles in 4 m × 6 m grid. Two cat walks are provided for approaching the berthing dolphins.
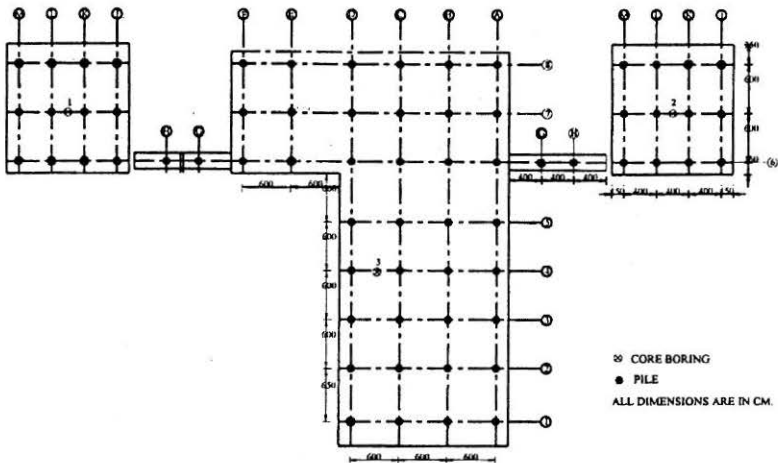
FIGURE 27 : Layout Plan of the Oil Jetty at Tuticorin Port

The substrata condition at site

The location of various borings carried out is shown in Fig.27. The bore logs are almost similar. The borings were done using a Callex drill. The core recoveries are very poor varying between 0 and 20 percent. The coring times required were on an average 1½ to 2 hours per meter length of coring. This clearly shows that the rock at site is weak and unless very careful core boring techniques are employed, it is not possible to obtain good cores.

In view of the above it was decided to carry out continuous standard penetration tests in pile bores B2 and C2 under construction at that time. The average N values obtained are between 200 to 220. In most cases the N values have been extrapolated on the basis of the penetration achieved mostly for 50 to 100 blows. Hence the results are on the conservative side. Incidentally this was the most reliable information available on the strength characteristics of the strata. As per the classification of British Standard CP 2004, the rock stratum at site falls under the category moderately weak to weak rock.

The samples from the SPT tubes were carefully examined. In general the samples give the appearance of a crushed weak rock. The calcium carbonate content of these samples varies mostly between 20 to 40 percent. The residue is essentially quartz sand. Further it was noted that pile borings have been advanced only by means of heavy chiseling. On basis of all the above investigation results, it was concluded that the substrata at site is essentially a moderately weak to weak calcareous sandstone.
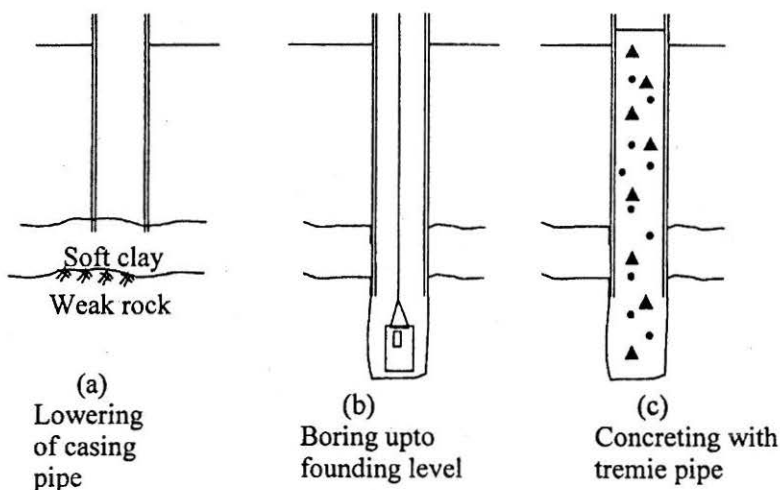
FIGURE 28 : Pile Construction Procedure (Oil Jetty)

Method of Pile construction

The jetty is supported on bored cast-in-situ piles. For construction, first a 6 mm thick steel pipe of required diameter (800 mm or 1200 mm) is lowered up to the firm bed level. The bore advanced by means of chiseling and bailing out the broken material. The reinforcing cage is then lowered and concreting carried out using a tremie pipe (Fig.28). All these operations were carried out using steel gantry specially erected for this purpose.

Pile Embedment Depths

On the basis of the available data on substrata conditions at site and a comprehensive review of all available information on large diameter bored piles and published literature, the following 2 methods have been adopted to arrive at the pile depths/pile load capacities.

Method I – Method based on SPT values

The procedure is described earlier. For an N value of 200, the shear strength is 180 t/m^2. The ratio of side friction to base pressure is taken as 1:9. This is somewhat on the conservative side.

Method II – Method based on German standard DIN 4014

This method also has been described earlier. On a conservative side the calcareous sandstone at site is classified as highly weathered (last category of
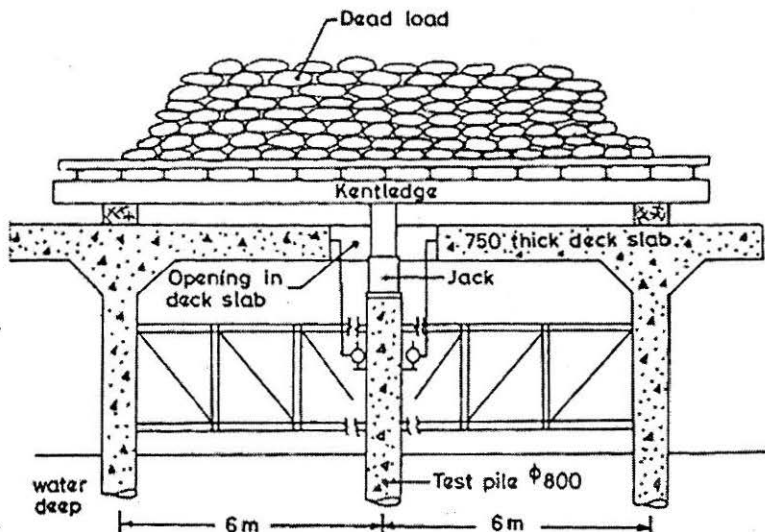
FIGURE 29 : Load Test Arrangement at Oil Jetty

rock) and the ultimate base pressure is taken as 300 t/m^2

The maximum load on any pile of the approach jetty is 165 tons. For this load the pile embedment required, is nominal (end bearing alone is sufficient to carry the load) as per method I and 1.3 m as per method II. Finally, in the absence of actual pile load tests and previous experience a conservative approach has been taken and the criterion adopted is to give an additional factor of safety of at least 1.5. The depth required is provided below the liner bottom, as the contact between the line and the irregular rock surface is uncertain. The adopted founding level for the piles of the approach jetty is −18.0 m, which is approximately 4 m below the liner bottom.

Pile Load Test

One of the working piles (B7) of the jetty head was tested under vertical loads to check the design adequacy. The pile details are:

$$\begin{aligned}
\text{Founding level} \quad &= \quad -17.88 \text{ m,} \\
\text{Bottom of liner} \quad &= \quad -14.17 \text{ m,} \\
\text{Pile embedment} \quad &= \quad 3.71 \text{ m.}
\end{aligned}$$

The pile was tested up to 1½ times the maximum designed load (165 t), i.e. upto 250 tonnes. The load was applied by means of a hydraulic jack reacting against a loaded platform (Fig.29). The pile was tested under cyclic
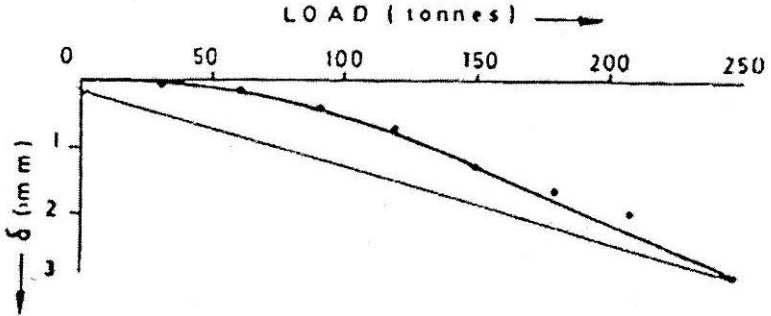
FIGURE 30 : Load Settlement Curve for Pile B7

load as given in IS:2911 Part I. The load was applied in 30 tonnes increment. The maximum load of 250 tonnes was maintained for 24 hours. Fig.30 shows the load settlement curve for the pile. The test results amply prove the adequacy of the pile embedment depth adopted. Thus the insistence of the contractor to take the piles much deeper was totally wrong.
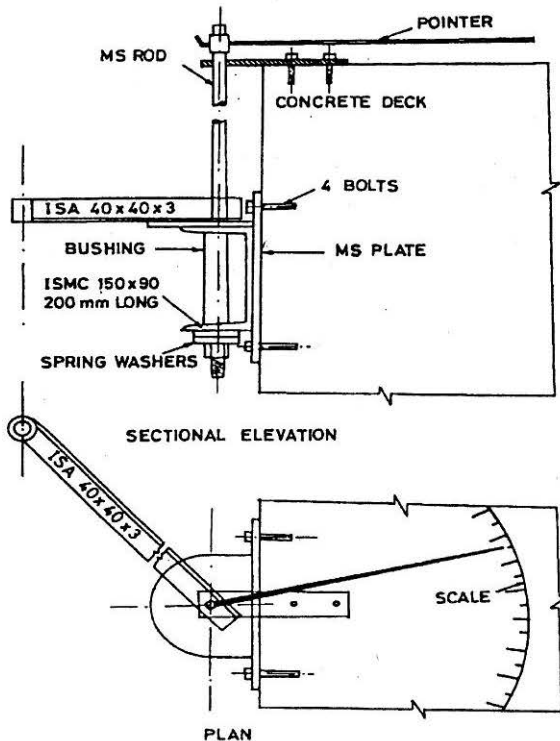

FIGURE 31 : Gauge for Measuring Fender Compression

Monitoring of Jetty under Berthing

Initially horizontal load test was planned for one of the working piles. Since the piles are standing in 11 m water depth, in the absence of fixity at top level, they will behave like a cantilever under the test load. The maximum test load that can be permitted for allowable bending stresses (in the absence of vertical load) is only 5 tons where as the design lateral load is about 17 tons. It was therefore decided not to conduct the test on individual pile, but instead to monitor the behavior of the pile group (entire berthing dolphin) during actual berthing operation to obtain the load displacement behavior. The berthing force was obtained by measuring the compression of the fenders and using the fender compression characteristics curve supplied by the manufacturer. The fender compressions were measured using a special type of mechanical gauges (Fig.31). The observations were made during berthing of four different ships. The displacements of the jetty were also determined by analysis using SAP programme. The point of fixity was assumed at rock level itself. The maximum observed displacement is only 15 mm against allowable displacement of 100mm.Even under the design-berthing load of 200 tons, the extrapolated value of displacement is within the allowable limit. Thus the pile was found adequate to resist the berthing loads.

Based on these results, the pile penetration for the adjacent Coal Jetty under construction at that time was optimised.

Design of Fertilizer Berth

The fertiliser berth at Paradip port as originally designed is shown in Fig.32. During execution there were some difficulties; in particular handling of the reinforcement cage weighing about 24 tonnes pre panel became a major problem. At that point of time Paradip port decided for a review of the design and entrusted the same to IIT Madras. The IIT Madras team felt that the combination of large diameter vertical piles, which are connected together by a rigid deck, and a deadman diaphragm wall with a long tie-rod which is essentially flexible is not a good solution. This arrangement might result in substantial portion of the horizontal forces being transferred to the vertical piles, as against the assumption of the original designer that the vertical piles will take only the vertical loads and all the horizontal forces will be transferred through the tie rod system to the deadman diaphragm wall.

Alternative Schemes

After a preliminary analysis of several combinations, finally the following alternative schemes were analysed in detail
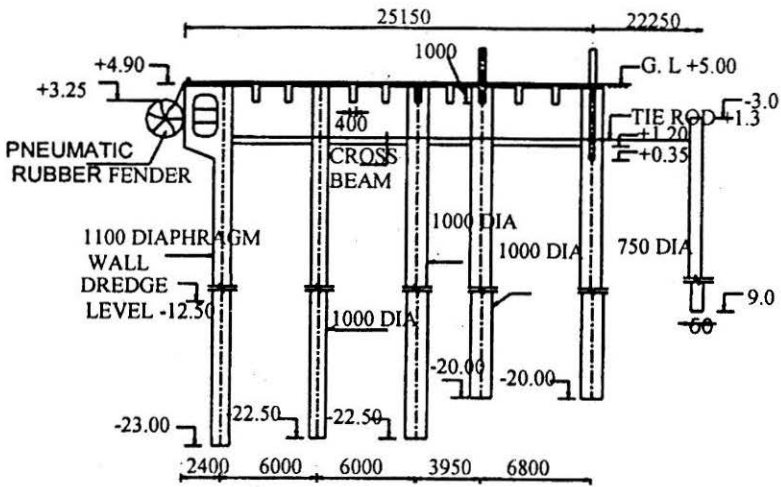
FIGURE 32 : Vertical Piles with Tie as per the Original Design

1.   Vertical piles with tie as per the original design (Fig.32)

2.   Modified design using only vertical piles and with increased pile
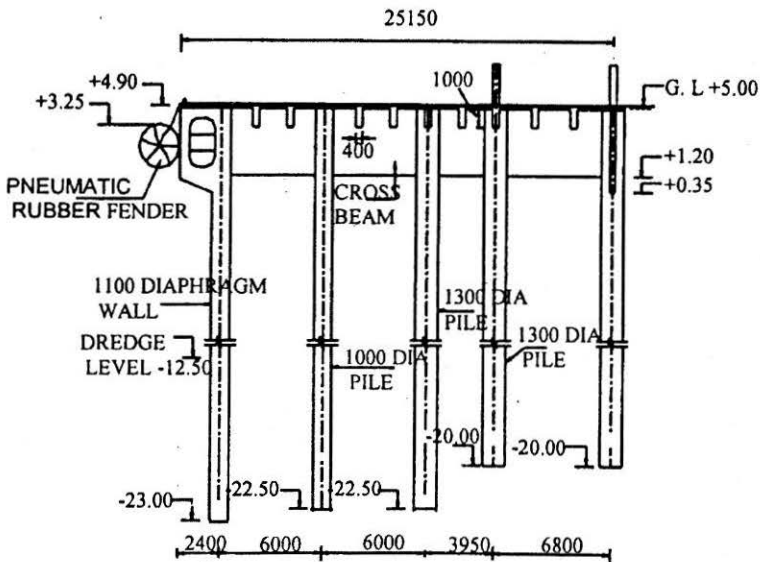   diameters (Fig.33).

FIGURE 33 : Modified Design using only Vertical Piles and with Increased
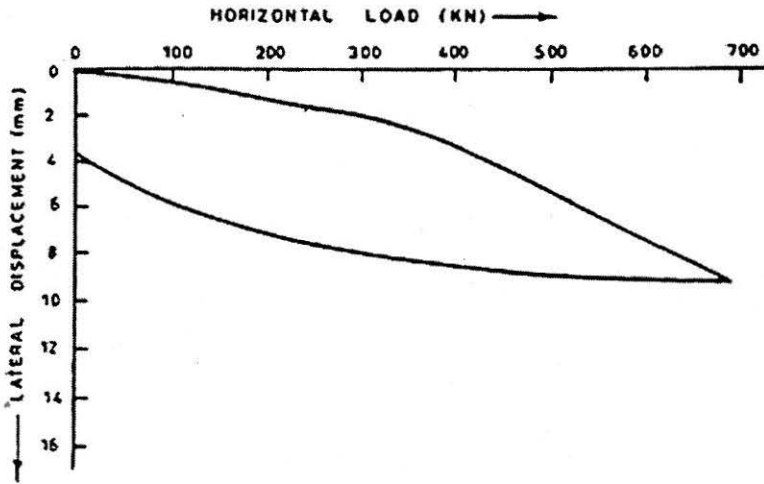Pile Diameters

HORIZONTAL LOAD (KN) ⟶

FIGURE 34 : Lateral Load Test on 1300 mm Diameter Pile

The results of the study indicate that

1.   The tie back system is flexible compared with the 3.85 m deep beam connected to three large-diameter vertical piles and hence the deadman is loaded only after substantial lateral load, i.e. about 70% is transferred to vertical piles.

2.   If the vertical-pile diameters of third and fourth rows are increased to 1300 mm and 750 mm, respectively, the piles alone are capable of taking the entire horizontal load.

Lateral Load Test

To confirm the lateral load capacity of the piles, load tests were carried out on a 1300 mm diameter working pile up to its design capacity of 700 kN. The test results Fig.34 show that the lateral displacement under 700 kN is about 10 mm.

Based on the studies, the berth was designed eliminating the use of anchor rods and the deadman diaphragm wall and using 1,300 mm diameter vertical piles only, which reduced the cost considerably (by about 30%) and speeded up the work (8 months saving in construction time). The berth was successfully commissioned and is in operation now.

Finally the berth was built as per the IIT design without the tie rod

and the deadman diaphragm wall. This has proved to be very economical, easy to built and nearly six months of construction was saved. The cost saving has over 30%.

3rd Cargo Berth at Paradeep

The general cargo berth at Paradeep Port situated in the East Coast of India consists of a diaphragm wall anchored to deadman diaphragm wall by tie-rods. In addition, the diaphragm wall is connected to two rows of large diameter vertical piles by a rigid deck system. The deck system being rigid and tie-rods flexible, the lateral loads are shared between the piles and the deadman diaphragm wall. To assess the load sharing, forces in the tie-rods were measured before and after dredging. The results obtained were compared with those of a finite element analysis of the structural system using beam element for the diaphragm walls, piles, and beams, and boundary elements for the soil medium. The measured tie-rod forces compared very well with the analytical result and confirmed that a substantial portion of the lateral load was transferred to the piles. Based on the studies, the design of another berth in the vicinity was modified to retain the vertical piles and eliminate the deadman diaphragm wall and tie-rods, resulting in substantial savings in cost and time.

Introduction

The cross-section of the berth is shown in Fig.35. The analysis is carried out for the load case of active earth pressure on a diaphragm wall assuming the deadman as both rigid and flexible. The three tie-rods connecting the 4m diaphragm wall panel are idealized using spring elements if the deadman is assumed to be rigid or a truss element if the deadman is assumed to be flexible. The active earth pressure is calculated as nodal loads at 1m intervals. The soil springs are also assumed at 1 m intervals below the dredge level of −11.5 m for the diaphragm wall and +3.0 m for the deadman. Due to lateral displacement, there will be an active zone behind the diaphragm wall.

Analysis

SAP IV was used for the analysis of the system. The results shows that when the deadman is assumed to be rigid, the lateral loads transferred to the beam and the three tie-rods are 198 kN and 578 kN respectively. The lateral displacement in the diaphragm wall at anchor level is 5.9 mm and at the centerline of the deck is 5.5 mm.

When the diaphragm is assumed to be flexible, the lateral load transferred to the beam and the tie-rods is 249 kN and 391 kN, respectively.
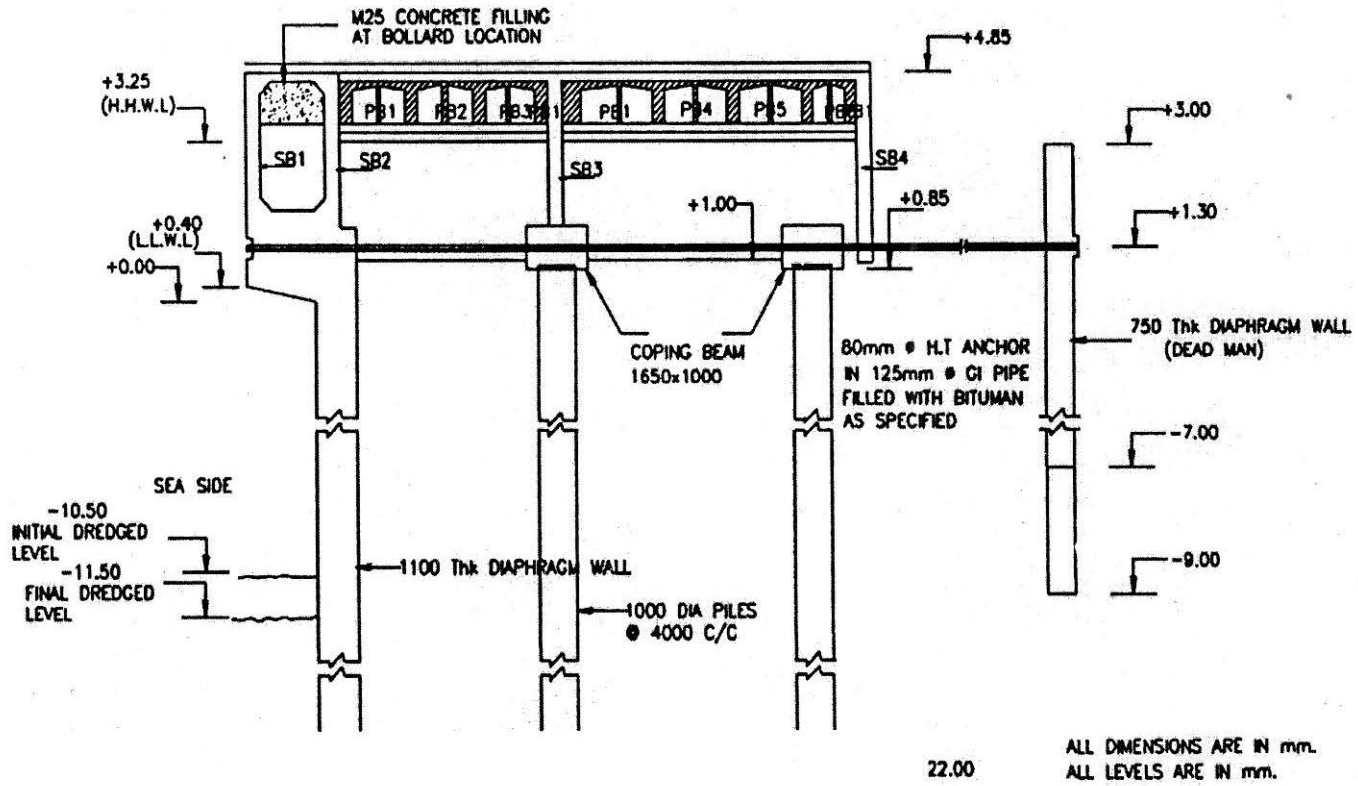
M25 CONCRETE FILLING
AT BOLLARD LOCATION

+4.85

+3.25
(H.H.W.L)

+3.00

PB1   PB2   PB3 PB1   PB1   PB4   PB5   PB6

SB1   SB2   SB3   SB4

+1.00   +0.85

+1.30

+0.40
(L.L.W.L)

+0.00

COPING BEAM
1650x1000

80mm ∅ H.T ANCHOR
IN 125mm ∅ CI PIPE
FILLED WITH BITUMAN
AS SPECIFIED

750 Thk DIAPHRAGM WALL
(DEAD MAN)

SEA SIDE

-7.00

-10.50
INITIAL DREDGED
LEVEL

-11.50
FINAL DREDGED
LEVEL

1100 Thk DIAPHRAGM WALL

1000 DIA PILES
∅ 4000 C/C

-9.00

22.00

ALL DIMENSIONS ARE IN mm.
ALL LEVELS ARE IN mm.

FIGURE 35 : Typical Cross Section of Third General Cargo Berth

Table 3 : Summary of Tensions in Tie-Rod and Beam

	Analytical			Experiment		
	Per Designer	Assuming deadman to be rigid	Assuming deadman to be flexible	Load Cell 1	Load Cell 2	Load Cell 3
Tension in tie-rod in kN	600	193	130	125	129	73
Tension in beam in kN	0	198	249	—	—	—

The lateral displacement in the diaphragm wall at tie level is 7.05 mm and at the centerline of the deck is 6.81 mm. The lateral displacement at tie level in the deadman is 4.29 mm. The results are summarized in the Table 3.

Load-Cell Design Criteria

Tie rods at three locations were selected for installation of strain-gauge-type load cells for force measurements Fig.36a. The following are the requirements for the load cell to be designed:

1.   Maximum load 600 kN.

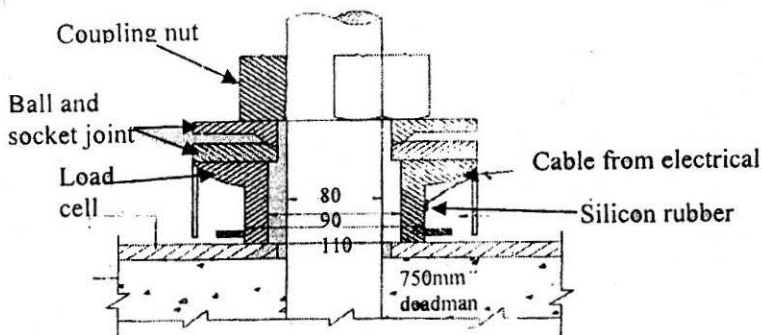2.   Arrangements for axial load transfer to the sensing element.
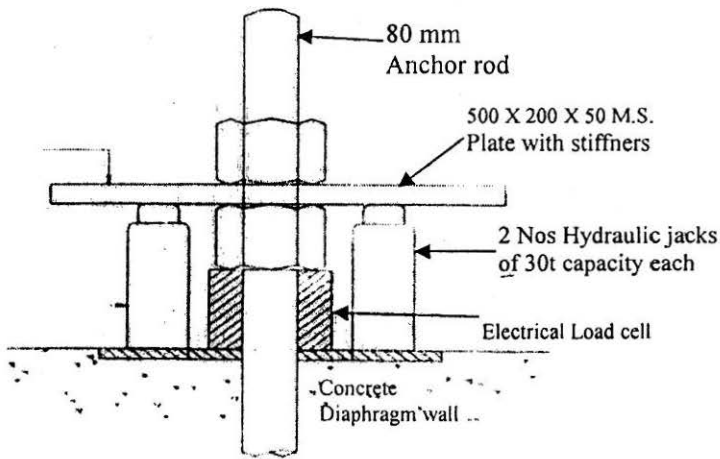

FIGURE 36a : Load Cell

FIGURE 36b : Testing Arrangement

3. Arrangement for moisture proofing of the gauge

4. Load cell must allow the tie rod to pass through it so that the load on
    the cell will be compressive

To ensure the axial load transfer, the load cell was designed to have
two separate pieces with spherical seating, one for load transfer and the
other for sensing. Calibration was done by applying known loads on the
cells in a compression testing machine up to 600 kN and 20% more. The
three load cells have a sensitivity of about 300 N.

Pretension in Anchor Rod

As part of the construction scheme, pretensioning of all the anchor
rods was done. Two hydraulic jacks were used to give pretension of about
30 kN (read from pressure gauge) Fig.36b. The Anchor nut introduced before
pretensioning was hand-tightened initially and then a pipe wrench was used
to tighten it further. After this, a check nut was introduced and tightened.
Load cell was fixed in position in October, 1984, and pretensioning was
measured using a strain-measuring unit, DMD 20. The measured load was
once again verified by releasing the load on the cell with the help of
hydraulic jacks and observing the corresponding pressure-gauge reading.

Tie-Rod Forces after Dredging

The tie-rod forces ware again measured in February 1985, after
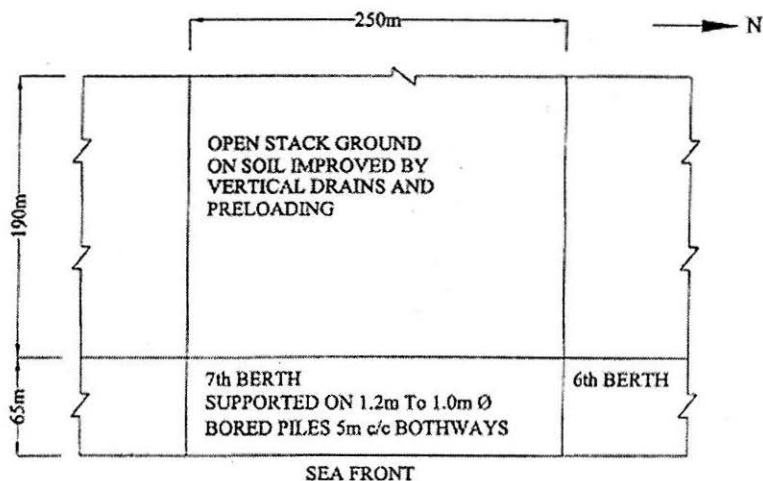dredging to the required depth of 11.5 m was completed. During the

FIGURE 37 : Layout of Cargo Berth

measurements, there was 0.6 m water above the load cell and continuous pumping was required for measurements. The forces in the three tie-rods are 125 kN, 129 kN and 73 kN respectively, and are compared well with the tension of 130 kN obtained assuming the deadman diaphragm to be flexible in the analytical study.

7th Cargo Berth – Kandla

The berth measures 250 m × 65 m. Apart from this, a measured 250 m × 190 m was to be prepared behind the berth to store the cargo upto a loading intensity of 200 kN/m². Schematic layout of the berthing structure is shown in Fig.37.

Typical cross section of the berth is shown in Fig.38. The natural sea bed level at the berthing face prior to construction was −1 to −5 m. after construction, the area in front of the berth was required to be dreaded to a level of −10.7.

Soil Conditions

Typical soil profile at the berth location is shown in Fig.39. As can be seen, from −5 m to −15 m level exists a thick soft clay deposit having SPT(N) = 0. This is followed by a dense sand layer upto −20 m and a stiff clay layer upto −25 m. beyond −25 m, the strata is dense sand upto the explored depth of 40 m.
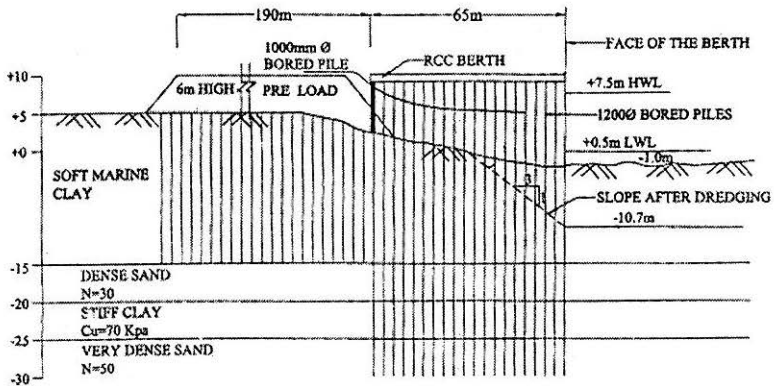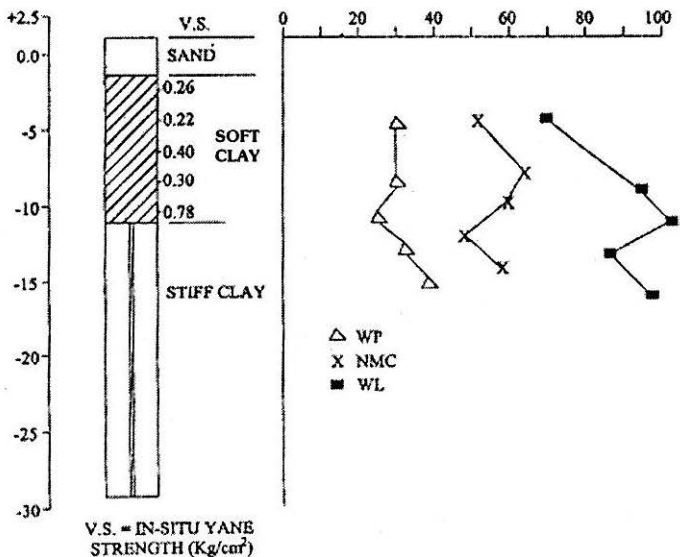
FIGURE 38 : Typical Section of Cargo Berth

FIGURE 39 : Typical Soil Profile – Kandla Port

Construction problem

1. After completing the berth structure, when the area in the front is dredged to −10.7 m level, the soft clay does not have adequate stability to stand a vertical cut. In the event of any lateral movement in the soft clay layer, due to such instability, large lateral forces can be transmitted to the RCC piles. The RCC piles become very expensive if such lateral forces are to be considered in the design.

2.   The high tide level is +7.5 m against ground level of +5 m. This means, during high tide the area gets in undulated by the sea water and therefore construction of RCC piles is difficult. Any fill placed to raise the ground level will further endanger the instability of the slope.

3.   The large area behind (250 m × 190 m) required ground improvement by preloading with the soil sill of 6 m height. The height of such fill can create further instability.

Possible Methods of Constructions

Following methods are generally adopted for construction of piled berth

1.   Installation of piles by end-on method. In this method, piling is initiated from the coast side and a moving steel gantry is supported on the completed pile. The platform keeps moving towards the sea more and more piles are installed. In this method all the forces of piling equipment is transferred to a deeper layer without stressing the soft clay layer. However, the initial cost of fabrication of the moving gantry is high and the progress is also slow.

2   The area of piling is raised to avoid inundation during high tide. Once the area is raised, the normal piling tripods can be used to carryout the pile. In this method more number of pilling rigs can be mobilized the work fast. However the method has following disadvantages.

   a)   Large amount of soil filling is required under the berth as well as in front of the berth, part of which has to be subsequently removed while dredging. In addition to cost of filling/dredging this requires more construction time.

   b)   The weight of the soil fill can trigger slope instability and damage the piles when dredging is carried out in the front to −10.7 m level.

Out of the above two methods, the second method was adopted for construction.

Slope Failure

During construction, when 50% of the piles were installed, s deep seated slope stability failure took place as shown in Fig.40.

This resulted in damage to large number of piles. Some of these piles got displaced as much as 1 m. The damaged piles were not capable of
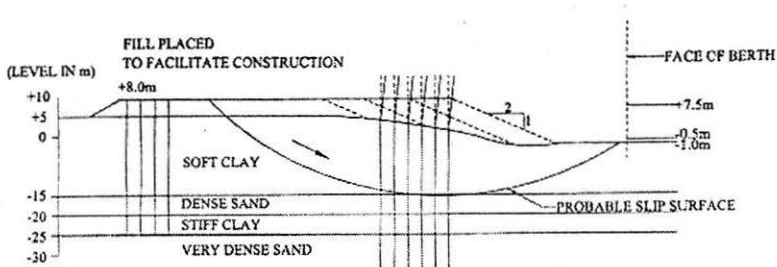
FIGURE 40 : Slip Surface

carrying the vertical/horizontal load for which they were designed. Due to this, it became necessary to install additional piles to replace the damaged piles. The remedial measures were very expensive and delayed the completion of the berth significantly.

Eight piles had lateral displacements at top of over 2.5 m clearly indicating that the piles were fully broken. 30 piles had displacements between 50 to 100 cms; 65 piles displacements of 30 to 50 cms and 100 piles top displacements of less than 30 cm. Extensive pile integrity tests have been carried out and the results have clearly shown structural damage (plastic hinge at deeper levels). Theoretical analysis has shown that for the given pile diameters and the soil situation, top displacements upto 30 cm would still be acceptable, as this extent of displacement would not cause structural damage to piles. Therefore all piles with top displacement of more than 30 cm were ignored and additional piles have been put in place in their position. As additional measures the following has been done.

a) Increase in steel in connecting beams and top slab to get a better integration of the entire structural system

b) Dredging was carried out in stages with careful monitoring of the lateral of the deck displacements

c) Part of the soil below the deck was removed by water jetting to flatten the slope.

d) Careful monitoring of slope below the berths once in three months to ensure no damage to the piles.

Thus the berth was successfully constructed and made operational. This berth with stood the January 2001 earthquake very well while some of the older berths in the same location have been subjected to partial damage, which is discussed in the next session

Earth Quake Damage and Repair of Berths at Kandla Port

The earthquake of magnitude M.6.9, which occurred in January 2001, has caused great destruction including at Kandla, 40 km south of the epicenter. The local conditions of the town and the damages to a variety of structures are briefly described. The results of the investigations of moderately damaged cargo berths, the analysis of failure and the solution for rehabilitation are presented. The situation of the Kandla Harbour offers a unique opportunity of a full scale test. Further research work is indicated.

The Quake and the Damages

A devastating earthquake of magnitude M 6.9 stuck Bhuj, Gujarat, Western India on 26th January 2001, the most powerful one to strike India in more than a half century. The effect was extremely severe. 1700 villages were destroyed, 400,000 houses damaged, 17,000 persons died, more than 150,000 injured, 200,000 homeless and estimated loss of property over US $ 6 billion (Rao, 2001).

The salient features of the quake are: It lasted for 90 seconds, there were more than 150 aftershocks and the focal point is at a shallow depth of 18.2 km.

Several of the heavy damages are more or less due to soil and foundation failure. This specially applies to the damages in Kandla, a port town situated about 40 km southwest from the epicenter.

The subsoil of the flatland consists of marine sediments with saline pore water. Typical soil profiles show in 5 to 15 m depth medium-dense gravely sand under a soft loamy marsh (plastic silty clay of soft to stiff consistency). Fifty years ago a filling consisting of loamy sand (thickness of 2 to 4 m) was placed in the city area and was compacted at a few spots only. Constructions of one and up to four floor buildings were mostly founded on footing and strip foundations. In the port area liquid storage tanks and multi-storied structures were mainly founded in gravely sand on vertical and raker piles or granular columns. The shore constructions for Berthing and loading platforms, and roadways are built in concrete. The first berths, built fifty years ago are founded on precast concrete spun piles driven in the sand layer. Subsequent berths are on deeper and larger diameter bored piles. Storage structures are founded on raker pile systems.

About 120 two to four storeyed buildings on shallow foundations have strongly tilted and collapsed which caused the loss of approximately 1000 human life. In the port area tanks remained intact, but the soil next to them

sagged several decimetres causing damage to pipelines. Superstructures founded landwards of the port area on vertical piles also remained intact. The soil around them sagged. The storage structures near the berths founded on raker piles are more or less strongly damaged. The heads of the vertical piles below some of the berths are slightly damaged. On the other hand the raker piles are strongly damaged. On the traffic area of the wharfage, cracks appeared parallel to the shoreline. Occasionally mud volcanoes with diameters of up to 0.2 metre came out of these cracks after the earthquake. Even months after the earthquake the cracks still grew in width. The soil behind the traffic area founded on piles sagged about 2 to 3 decimetres immediately after the earthquake. The following months it sagged once again by the same amount.

Damages to the Berthing Structures at the Kandla Port

Kandla is a major and important port handling 18% of total port traffic of India. The main damage was to warehouses, some buildings and some of the berthing structures. The lateral ground movement resulting from movement of the soft clay slopes below the berths has significantly contributed to the damage of the berthing structures. Surface cracks, 2 cm and 5 cm wide developed parallel to the water front.

The over view performance of the berths (Raju, 2001):

Cargo berths 1 to 5	– moderate damages
Cargo berths 6, 7, 8	– no damages
Oil jetty 1	– considerable damage
Oil jetty 2, 3, 4	– fully intact
Approach to maintenance jetty is under construction	- damage to free standing Piles

Cargo Berths 1 to 5

These were built in early 50's and have an overall length of 1200 m and a width of 23 m. Immediately next to them are transit sheds of width 43 m. The arrangement is shown in Fig.41.

The berth itself is built on vertical piles 500 mm in dia precast hollow spun piles driven to –14.6 m level. They are tied together with tie beams at +3.4 m level and connected at this level to large number of raker piles behind which support the transit sheds.
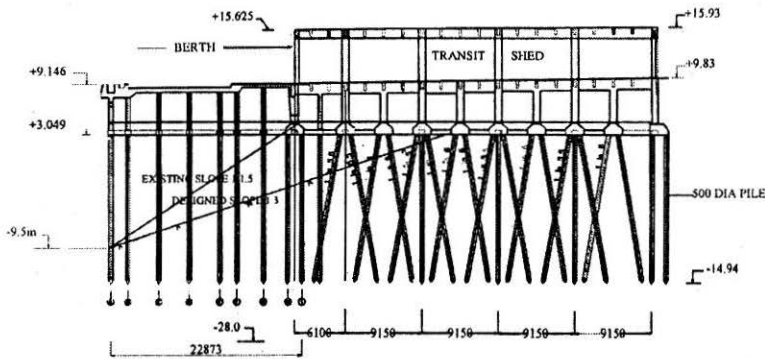
FIGURE 41 : Typical Cross Section of Cargo Berths 1 – 5

For the design of the berth it has been assumed that the horizontal forces due to mooring and berthing and due to earthquake will be transferred to the raker piles behind and vertical piles will only take care of dead loads and live loads on the deck. Each berth has 10 blocks; each 23 m × 23 m.

The slope below the berth is designed to be stable by itself with an inclination of 3 H to 1 V. However because of siltation and difficulty of dredging below the berth, the average slope at present is as steep as 1.5 H to 1 V.

The Damages

The damage was essentially to the piles. The damage assessment was done by a close inspection of the piles, below the berths. It was possible to see closely all the vertical piles and one row of the raker piles. Horizontal cracks are seen in 50% of the vertical piles. In case of raker piles they are found to be sheared/crushed at the junction with the tie beam.

Unfortunately it is not possible to see the raker piles beyond the first row. Adjacent to berth number '1', there is small berth (called 'o' berth) with a number of raker piles, which are clearly visible. Most of these have been sheared. Therefore there is an apprehension that substantial number of raker piles behind the berths 1 to 5 could have been damaged.

The damage to vertical piles is in the form of cracks upto 1 mm in width, 10 to 30 cm below the tie beam, in about 1500 piles out of the total 3000 piles.
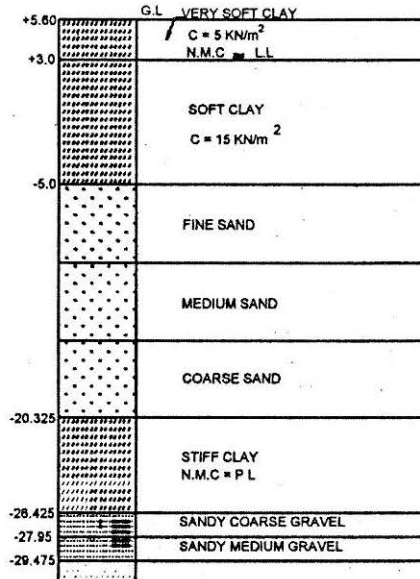
FIGURE 42 : Soil Profile for Berth 1 – 4

Soil Profile

There are considerable variations in the soil profile. Typical soil profile for berth 1 to 4 is shown in Fig.42. It consists of a very soft to soft top clay layer of about 10 m thick with 'c' value of 5 to 15 kN/m^2 followed by 15 m thick sand layer over lying a stiff clay layer of about 6 m thickness, followed by sand layers. For berth 5 the clay layer is nearly 20 m thick followed by only a 2 m thick sand layer.

Reasons for Failure

As mentioned earlier the slope below the berth was originally designed to be stable by itself with 3 H to 1 V. However the present slope is 1.5 H to 1 V, which is not stable. This means that the slope is being supported by piles through dowel action, which in turn means that the soil is exerting lateral pressure on the piles. Under seismic conditions the instability of the slope will increase, which in turn exerts higher horizontal forces on piles.

Therefore it can be concluded that the damage to the piles is due to the combination of the lateral pressure on the pile from the soil and seismic force due to the mass of the deck and the stored materials on it. Apparently the raker piles were damaged first under the initial seismic activity as they
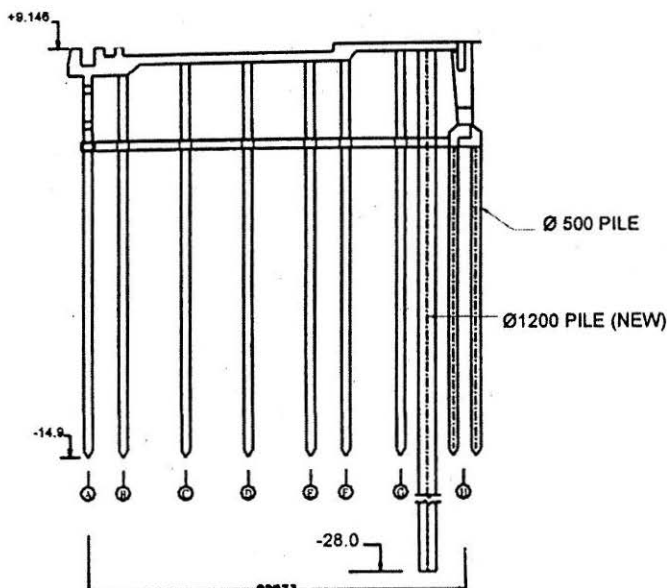
FIGURE 43 : Cross Section with New Piles

attract most of the horizontal forces. Subsequently, the horizontal forces are taken by the vertical piles for which they have not been designed. In addition the slope instability resulted in lateral soil pressures on the vertical piles.

Restoration of Berths

After analyzing different options, it was decided to provide a row of vertical piles of 1200 mm dia, RCC bored with 2% steel, with toe level of −30 m installed in the centre of the outer most span (away from the water front).

Figure 43 shows the cross section with the additional pile. The additional 1200 mm dia pile will take about 50% of the horizontal forces on the berth. Similarly it will take about 70% of the lateral force exerted by the soil on the piles due to slope instability.

Repair of the cracked portion of the old piles

These piles are provided with 50 mm micro concrete (Renderoc UW) cover on the outer face over a length of 1 m from the top of the pile.

# GROUND IMPROVEMENT – VIBRO COMPACTION AND VIBRO REPLACEMENT

Ground improvement has been in vogue for optimum foundation solutions for a very long time globally. Vibro compaction, initially called vibro floatation, has been there since early 1920s and has been very widely used for compacting sand deposits both on land and underwater. For sand compaction on land this has been used in India, though to a limited extent. In parallel we have been using in India sand compaction piles and also compaction using a vibratory needle with a vibrator mounted on the top of it. These were essentially used to density the sand to overcome the problems of liquefaction and very rarely to use the compacted ground to support spread foundations as an alternative to pile foundation.

In case of soft ground stone columns are in use in the country for some time. Initially these columns, called as rammed stone columns, were installed by making a bore hole first filling them with stones and then ramming them. There have been some successes and also some failures. There have been few instances of installation of vibro stone columns in soft ground for supporting tank foundations. There again some failures by way of excessive settlements have been noticed with all this an impression is created that stone columns in soft ground is not a solution without problems.

Internationally the techniques of vibro compaction and vibro replacement (vibro stone columns) have made great progress. In many cases they have been successfully used all over the world as an alternative to pile foundation. In situations like high seismicity resulting in very large horizontal forces, gravity foundations on treated grounds are also more advantages as against a pile foundation. Very often the cost of these types of foundation is only about 50% of the conventional solution of a pile foundation. Internationally the success of vibro compaction and vibro replacement is mainly due to the great developments that have taken place in the design and manufacture of the vibrators, as well as the methodologies of monitoring the vibro compaction and vibro replacement resulting in high reliability. Also these techniques are very efficient and installation can take place very fast.

Earlier I have already presented the successful use of vibro stone columns for the IFFCO Anola Fertilizer project Phase II. Below I give two more very recent examples from India

1. Vibro compaction of sand below the foundations of a breakwater

2. Vibro replacement foundation for large LNG storage tanks of 100 m diameter with loading intensities of 20 tonnes/m^2 near Hazira on the west coast.

U. S. STANDARD SIEVE SIZES
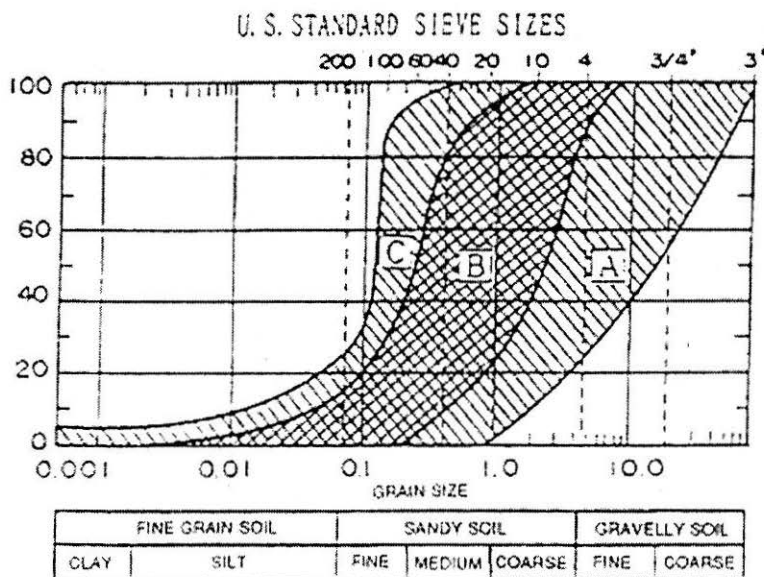
Three main points are illustrated in the above figure.

1. The vibroflotation method is most effective in loose sand with gradation in zone B ($-D_{20}$ = 0.1 to 1mm, D_{50} = 0.3 to 2.5 mm)
2. Compaction by vibroflotation becomes very difficult in sand with gradation in zone C, and
3. Compaction is possible for soils with a portion of the gradation curve passing through zone C. Penetration of the vibroflot becomes difficult in relatively dense sand, cemented sand, and gravel in zone A.

FIGURE 44a : Gradation Appropriate for Vibro Flotation / Vibro Compaction Method

(**Source** : " Hand Book on Liquefaction Remediation of Reclaimed Land" by Port and Harbour Research Institute, Ministry of Transport, Japan, Published by A.A. Balkema)

Gradation of Suitable for Vibro Compaction / Vibro Replacement

Based on extensive field experience the gradation of sands suitable for vibro-compaction have been variously reported in literature and universally accepted. The information from 3 different sources are presented in Figs.44a, 44b and 44c.

Offshore Vibro Compaction of Sand below Foundation of a Breakwater

(Source: CD Rom of M/s Keller, Overseas Division Offenbach Kaiserleistraße 44 Postfach 10 06 64, D-63006 Offenbach)
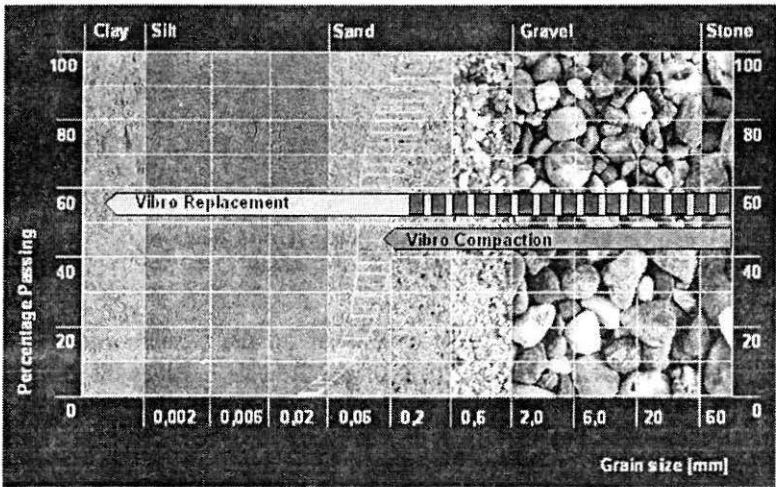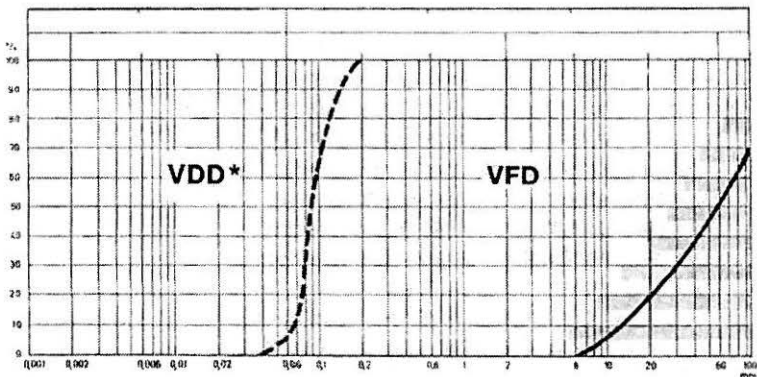
FIGURE 44b :   Gradation Appropriate for Vibro Flotation / Vibro Compaction Method
(Source : "Keller Grundbau GmbH", Head office and Overseas Division, Germany)

Note   :   Vibro compaction suitable for $D_{20} > 0.109$ mm and $D_{50} > 0.116$ mm
Vibro compaction ideal for $D_{20} > 0.144$ mm and $D_{50} > 0.24$ mm

Gradation border lines for the improvement
of soils with BAUER deep vibrators

*Shear strength $20 < c_u < 100$ kN/m^2
$(400 < c_u < 2'000$ psf$)$

VDD   - Vibro Displacement Densification
VFD   - Vibro Flotation Densification / Vibro Compaction

FIGURE 44c : Gradation Appropriate For Vibro Flotation / Vibro Compaction Method
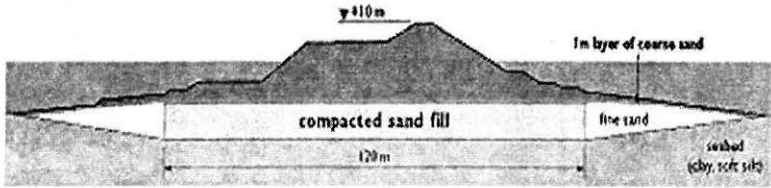(Source : Bauer Spezialtiefbau GmbH, Germany)

FIGURE 45 : Break Water Cross Section

A major port is being built at Karwar on the western coast of India, 100 km south of Goa. The first phase of the projects involves the construction of two breakwaters having a base width of 120 m and total length of about 5 km as shown in Fig.45

The Soil and Improvement Scheme

The natural soil, consisting of clay and soft silt has been removed from the seabed by dredging and the trench refilled with imported sand. Soil improvement was carried out in order to achieve the following targets:

- uniform soil condition
- reduction of settlements
- minimising of long-term settlement and creep
- safety against liquefaction

The Offshore Site Works

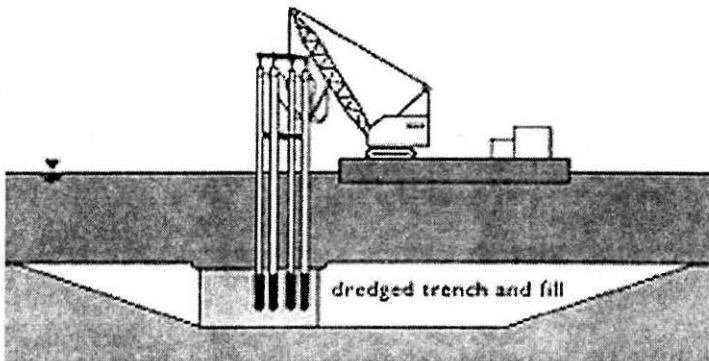The method selected was Keller's deep vibro Compaction Technology.

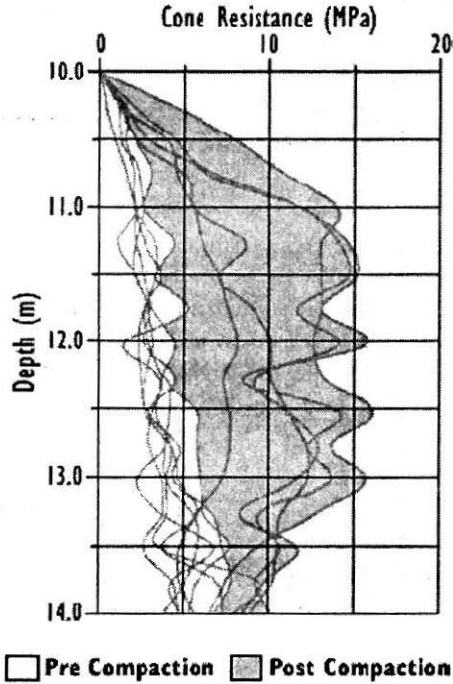FIGURE 46 : General Arrangement of Vibro Equipment

FIGURE 47 : Results of Pre and Post Compaction CPT

Four S-300 vibrators, extended to 18 m and held 3m apart by a specially constructed steel frame, were suspended from a crane to compact the imported sand fill to a depth of 3 m to 4 m under a water depth of 10 m. The vibro equipment with generators, pumps and workshop etc. were installed on a 50 m × 20 m pontoon equipped with a 120-ton crane. With the 4-vibrator arrangement an area of 36 m is covered with one compaction operation. The general arrangement of the Vibro Equipment is shown in Fig.46. A daily (24h) production of 2000 m^2 to 4000 m^2 area was achieved.

The Quality Control

- Positioning of the compaction points by GPS (Global Positioning System),

- During compaction process by Keller M4 Recorder, monitoring depth, power consumption and time,

- Pre-and post compaction CPT with seabed – CPT-machine as shown Fig.47.

Vibro Replacement Foundations for Large LNG Storage Tanks on the West Coast

(Source: Keller (M) Sdn. Bhd. 35, Lorong B. Kampung Pakar Batu 5, Jalan Sungei Besi, 57100 Kuala Lumpur, Malaysia)

Introduction

A project site in the West Coast of India has proposed to construct two LNG storage tanks. The subsoil stratum in the proposed tank area was loose silty sand up to a depth of 11m below existing ground level. The silty sand with varying water table due to the high tidal variation and the severity of earth quake in this region lead the designer to consider two different types of foundations.

1) Pile foundation and

2) Vibroreplacement (Stone Columns)

The above two methods were analysed, the vibroreplacement method was comparatively cheap and the construction time was less, and the same was selected. The Vibroreplacement was executed for two LNG tanks of about 47,136-lin.m. The job started on 6th May and finished on 14th August including 12 trial column to fix the basic parameter for the soil improvement like Spacing of column and Quality control parameters.

Soil Strata and Improvement Works

The strata consist of silty sand up to a depth 10 m to 11 m below ground level. Below this strata Medium dense sand with small layer of clay of 0.2 to 0.3 m exist between 11 m to 16 m. Following this layer dense sand layer exist. The soil profile is given in Fig.48.

Based on the design calculation spacing of 2.4 m × 2.4 m square grid pattern was obtained. The depth of stone column was 16 m below existing ground level. Later the design parameter was confirmed after conducting the trial test.

The Minimum cone resistance value after the installation of stone column, before compaction and after stone column is given in Fig.49.

Site Operation

Vibrator with 15 m extension tube was hung on a 75 T crane with boom length of 27 m was placed over the selected point. After starting the
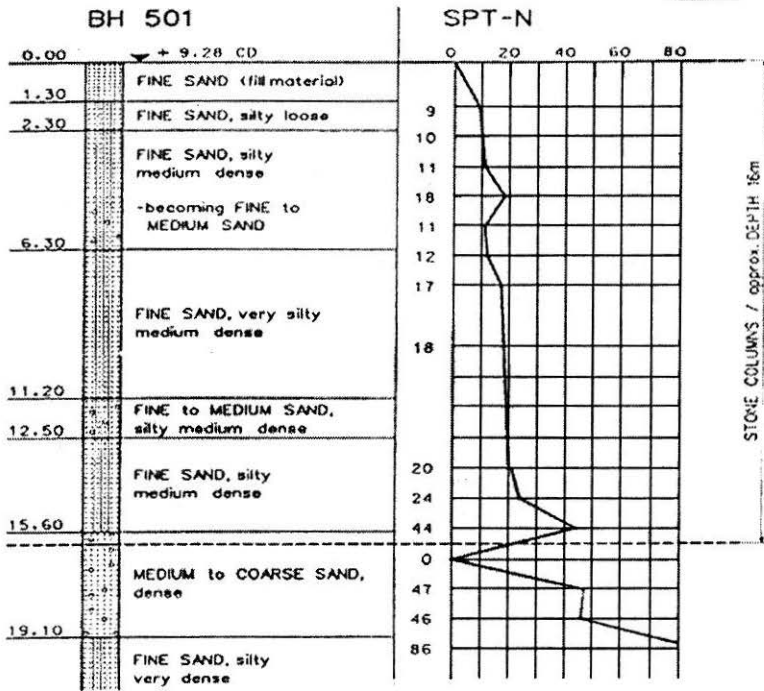
GRID   Spacing of grid 2.4 m x 2.4 m

Stone Column Ø 1.0m

FIGURE 48 : Soil Profile

motor and water jet, the vibrator was lowered into the ground. The oscillating vibrator with its extension tubes sank rapidly into the soil under its own weight. When the designed depth of 16 m is reached, the annular space is ready for the building of the stone column. The constant water flow was maintained to ensure that the stone backfill was washed to the tip of the vibrator and the soft in situ soils are flushed out. The vibrator was slowly withdrawn in steps of 0.7 to 1.0 m and the stone fall to the tip of the vibrator. The vibrator was then lowered back into the hole between 0.70 to 0.80 m thereby creating a 0.2 to 0.3 m length of stone column. The action of the vibrator compresses stone radially into the surrounding soil and also compacts the stone in the annular space thus formed a stone column of nominal diameter of 1.0 m to 1.1 m. The constructed column diameter was determined by gauging the stone consumption during installation. This compaction procedure continued till the full length of the stone column has
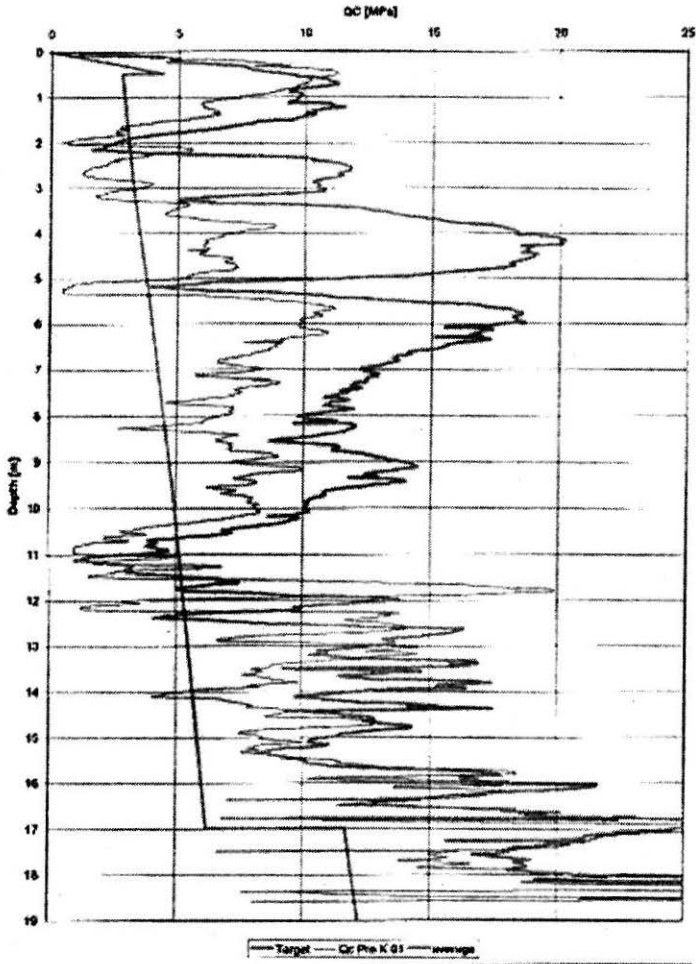
FIGURE 49 : Cone Resistance Values

been constructed. The view of vibro setup with crane is given in Fig.50.

Quality Control

The quality of each stone column was checked using an automatic recording device. This instrument yields a computer record of the installation process in a continuous graphical mode, plotting depth versus time and power consumption versus time. The information provided includes:

- Stone column reference number
- Date of installation

FIGURE 50 : View of Vibro Setup with Crane

FIGURE 51 : Typical Computer Printout

- Start and finish times of installation
- Period required for installation
- Power consumption during penetration
- Power consumption during compaction
- Maximum power consumption
- Maximum depth

These printouts were used to check the quality control during the project. Typical computer print out is given in Fig.51.

The degree of improvement (increased shear strength) achieved by the vibro-replacement process was checked by conducting series of Electric cone penetration tests. The CPT curve showing all the detail is given in Fig.52.

The cross sectional view & exposed view of stone column after excavation are given in Fig.52. The view of tank area after excavation is given in Fig.53.
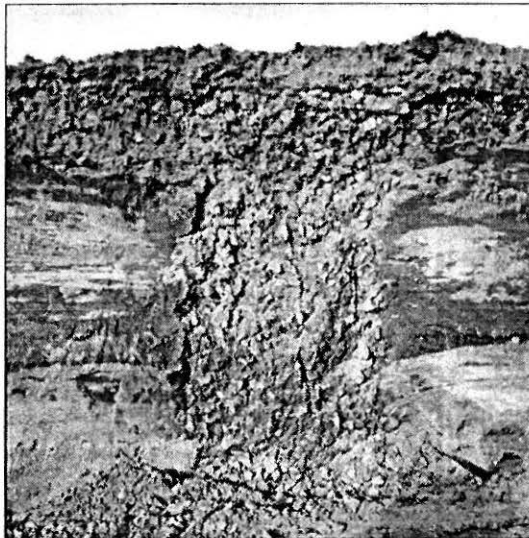

FIGURE 52 : Cross Sectional View and Exposed View of Stone Column
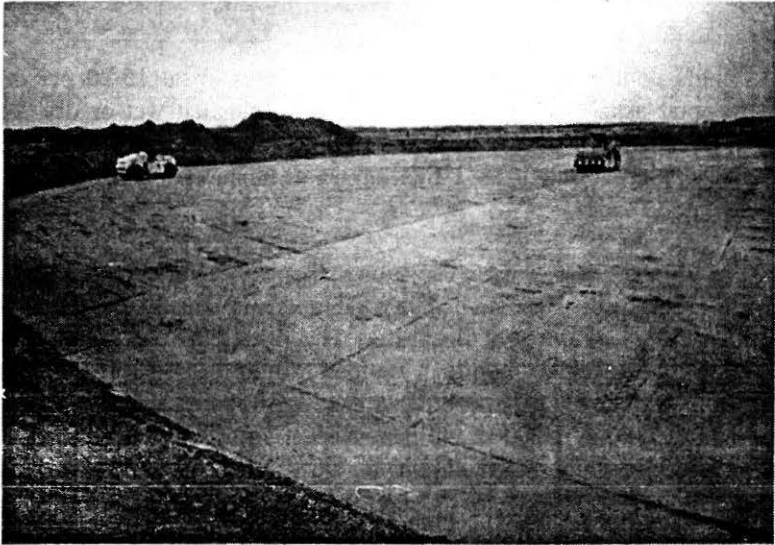after Excavation

FIGURE 53 : View of Tank Area after Excavation

FLY ASH AND MANAGEMENT OF ASH PONDS

Introduction

Thermal energy is chief source of energy in India at present and it will continue to be so in near future. Large quantities of fly ash generated by thermal power plants occupies precious agricultural land for deposition. Currently over 100 million tons of ash is produced in the country.

Through the Fly Ash Technology Mission of Department of Science and Technology the ash utilisation has increased from 2% of 60 million tonnes about 5 years back (i.e 1.2 million tonnes) to 20% of 100 million tonnes i.e 20 million tonnes in 2001. While this is a remarkable achievement, still we have to find ways and means of safely depositing 80 million tonnes of ash every year in the country.

The Vision

The ultimate goal of ash disposal should obviously be complete utilisation of the ash. What are the options?

It is better to look at the new power stations and existing power stations separately.

New Power Stations

(a)   Handle bottom ash separately (this constitutes about 15-20 per cent of the total ash). It is like find sand, can be handled wet or dry. This is an excellent fill material for embankments, below roads, etc. One can also explore using it as a fine aggregate in mortar and concrete (just like river sand).

(b)   The chimney ash, which constitutes about 80-85 per cent of the total ash, should also be handled dry. This ash is usually collected in 3-5 silos and is of different finenesses. The finest part can readily be used in portland cement. Further options would be brick making, mine filling, filling up of low lying areas, etc. Finally, the excess material has to be safely handled and deposited in ash dump/ash ponds, meeting all the environmental requirements.

Existing Power Stations

There are large tracks of land covered by pond ash. The following needs to be done urgently:

(a)   The safety of ash ponds against dyke failure and flow of ash slurry has to be ensured under all circumstances, including under earthquake loading.

(b)   The areas, no more in use, need to be rehabilitated for use as recreational areas, for afforestation or even for industrial and residential purposes.

Dry Handling of Ash

As per the guidelines of Ministry of Power, all future power plants are likely to use the dry system of collection, transportation and disposal of ash. One of the major benefits of the dry handling system is that bottom ash and flyash can be collected and stored separately.

Bottom ash forms 15 to 20 percent of the total ash produced at thermal power plants. Is a coarse grained material which is free draining in nature and exhibits excellent engineering properties such as high strength, high permeability and low compressibility. The entire production of bottom ash can be easily used in construction of embankments, fills, base and sub-base course of roads, dykes of ash-ponds and several other similar applications.

Flyash forms 80 to 85 percent of the total ash produced. The finest

Table 4 : Utilisation of Flyash

Area	Utility
Earthworks	Embankments, Backfills, Highways –Base and sub-base, Soil Stabilisation, Structural Fills.
Building Materials	Pozzolana Cement, Bricks, Blocks, Slabs and Wall Panels, Lightweight Aggregate, Concrete, Grouts.
Agricultural	Soil Conditioning, Manufacture of Fertilizers.

fraction of the flyash can be readily used in the manufacture of PPC and the remaining portion can be utilised effectively in the building material industry for production of bricks, blocks, concrete and other related products (Table 4). It is estimated that cement production will touch the 100 million tonnes per annum mark by the year 2000. A 20 percent substitution in cement and related products would imply a utilisation of 20 million tonnes of flyash annually.

The unutilised portion of ash can be used for filling low-lying areas or for back filling of mines to minimise the impact of unutilised ash on land.

Dry handling systems, thus appear to be well suited towards giving necessary thrust to higher utilisation of ash and reducing the impact of unutilised ash on land use patterns. Dry handling systems, however, require special air pollution control, measures in the areas where ash stored in the form of ash mounds because of dust emission problems.

Wet Handling of Ash

Of the 80 off thermal power plants in the country, 80 percent use the wet system of collection, transportation and disposal of ash. Water forms a very convenient medium for transporting ash in the form of slurry as well as for spreading the ash in ash ponds through the depositional process. However the wet system of disposal has two major shortcomings.

(a) At most power plants, the bottom ash and flyash are mixed together before transporting the ash in the form of slurry. Consequently bottom ash and fly ash accumulate together in an ash – pond in a mixed state and they are not available in separate forms for utilisation. As a result the advantage of use of bottom ash, which has a high utility value, is lost in the wet system in comparison to the dry system. Some sorting of ash into its coarse and fine fraction is reported at the ash-pond with the coarse ash settling at the inflow point and the fine

ash accumulating at the outflow point. However, this sorting is not complete and hence leads to hesitancy in commercial use of the material

(b)   The wet disposal system requires large quantities of water, If the water is not recirculated. In addition, pollution of surface water by suspended fine ash and of ground water by leaching of contaminants are two major concerns relating to environmental damage in wet disposal systems.

The immediate technology imperatives appear to be the conversion of the wet systems to dry systems of handling, to enable maximum utilisation of ash. However, as an intermediate step, two measures can be taken which would not be as capital intensive as a complete conversion of the system but would yield some benefits:

(i)   Use of separate pipelines to transport bottom ash and flyash and use of separate ash-ponds for their storage will allow full utilisation of bottom ash by the construction industry. A 100% utilisation of bottom ash would result in 15 to 20% reduction in storage required for ash. Studies, including those at UT Delhi, clearly indicate that separate handling of bottom and flyash is feasible.

(ii) Recirculation of the water used for transportation and disposal of ash, by using properly designed decanting facilities, would ensure that requirements of water are reduced significantly and the possibility of surface water pollution are eliminated completely.

The above two measures would result in significant improvements in utilisation of ash as well as environment-friendly disposal of ash.

Safe Management of Ash-Ponds

With most of the thermal power plants facing difficulty in acquiring additional land for expansion of ash disposal areas, the tendency is towards increasing the height of existing facilities. The emphasis at existing ash ponds is on the following:

(a)   Increasing the height of dykes using ash
(b)   Improving effectiveness of decanting systems
(c)   Minimizing environmental impact
(d)   Reclaiming old ash ponds (after closure)

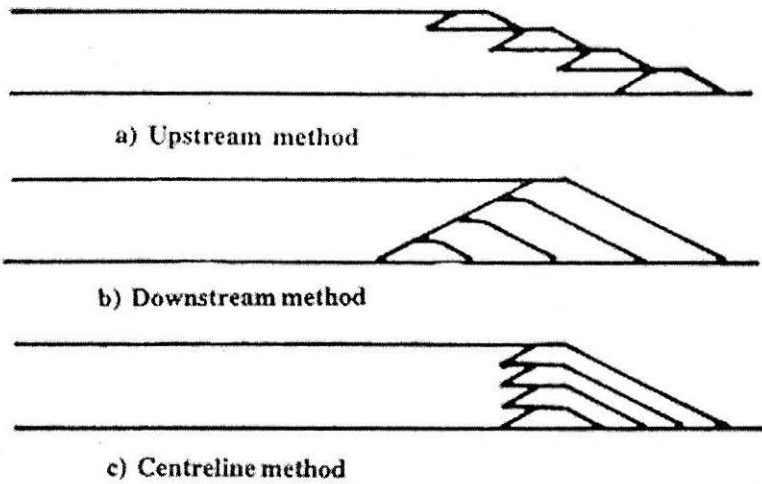Technology options pertaining to each aspect are discussed hereafter.

a) Upstream method

b) Downstream method

c) Centreline method

FIGURE 54 : Methods of Raising Dyke Height

Raising the Heights of Dykes

The use of ash to raise the height of dykes is now well accepted. One can use the following alternatives while constructing the dykes:

(i)   Compacted ash with earth cover/outer shell;
(ii)   Alternative layers of compacted ash and locally available soil; and
(iii) Ash mixed with soil

The first method usually offers the maximum economy and the greatest ease in construction.

Three techniques (Dayal, 1987) can be used for raising the height of dikes, namely the upstream technique, downstream technique and centreline technique (Fig.54). The upstream technique is usually preferred because (a) it requires minimal earth work; (b) the height can be raised incrementally in stages; and (c) dust control measures can be adopted on the outer face of the embankment as the height increases.

The desirable aspects of downstream and centreline techniques are greater compaction control and hence better stability and steeper outer slopes. However, the upstream method is usually the most economical and hence adopted most often. In the upstream method, the embankments are constructed progressively on soft ash deposit. Therefore, special measures for proper drainage of the ash deposit are critical for stability of upstream
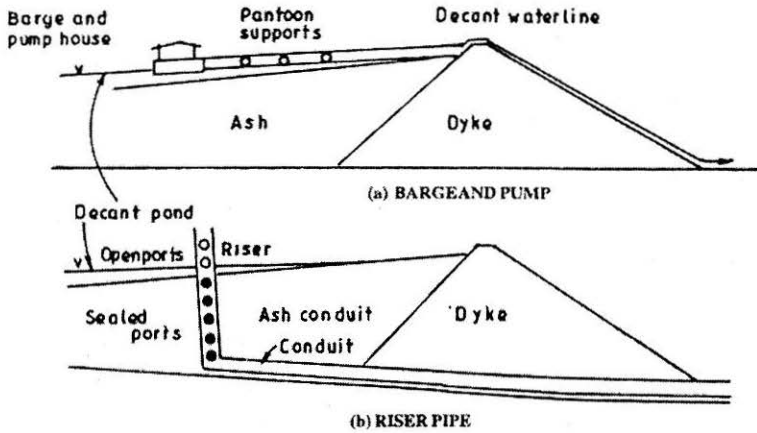
FIGURE 55 : Methods of Decanting Water

embankments and long term monitoring of movements and pore water pressures are considered essential, specially in high siesmicity areas.

Decanting Systems

When ash is deposited in the form of slurry, most of the coarse ash particles settle down in the ashpond near the inflow point. The finer particles are carried further to the ponded water where they eventually sediment out. Water is then decanted for discharge to nearby water bodies. If sufficient time of sedimentation is not available, some fine ash will be carried out in the suspended state with the decanted water.

(i)   The decanting systems used are: (a) overflow weirs; (b) pumps or syphons (fixed or mounted on barge); and (c) vertical concrete risers with intake ports (Fig.55). Floating

Systems offer the flexibility of relocation within the pond and hence better control on decanting arrangements. Nevertheless most decanting systems are observed to allow some sediment discharge during high flow periods and this is a major cause of concern to environmentalists. The use of secondary sedimentation structures such as settling ponds, filter lagoons, or filter shafts is considered desirable and this in an area, which needs greater research inputs in the immediate future.

Minimising Environmental Impact

The impact of ash-ponds near thermal plants on the local environments

are usually stated to be the following (Thakre, 1995):

(a)   Unattractive visual appearance over a long period of time corresponding
       to the operative life;

(b)   Gradual change in landform from flat or low lying in the beginning to
       be elevated or mound shaped in the end;

(c)   Dust emissions from dried flat surface during hot dry weather;

(d)   Overflow of fine ash from the decant pond to nearby water bodies
       during period of high precipitation/ surface run off;

(e)   Leaching of trace elements, in particular heavy metals, into surface
       water and ground water; and

(f)   Accumulation of heavy metals in soils and plants around ash ponds

Vegetative growth on the slopes of dykes of ash ponds can reduce the visual impact; however vegetation cannot be provided on the surface during the active life of an ashpond. Long-term implication of change in landform e.g. change in drainage patterns and change in future land use plan of the area, should be considered at the initial stage to arrive at the final height of the ash-pond.

Dust emissions during the active life of an ash pond can be minimised by using continuous water sprinkling systems as well as by providing a vegetative buffer zone adjacent to the dyke to mitigate dust nuisance. A final earth cover, at the end of the operating period of the pond, with appropriate plantation is considered essential for prevention of dust emissions on a long-term basis at ash-ponds.

To prevent contamination of surface waters in water bodies adjacent to ash ponds, it is desirable to use secondary sedimentation systems such as settling ponds, filter lagoons etc. in addition to Primary sedimentation within ash-pond.

The leachability of heavy metals from ash is well documented. However the concentration of leached ions is not likely to have significant impact on nearly surface water bodies where periodic high flows will prevent accumulation of leached ions is not likely to have significant impact on nearly surface water bodies where periodic high flows will prevent accumulation of leaches contaminants. The influence of leached trace elements on ground water quality is an area of major concern because of the possibility of progressive accumulation of leached trace elements into ground
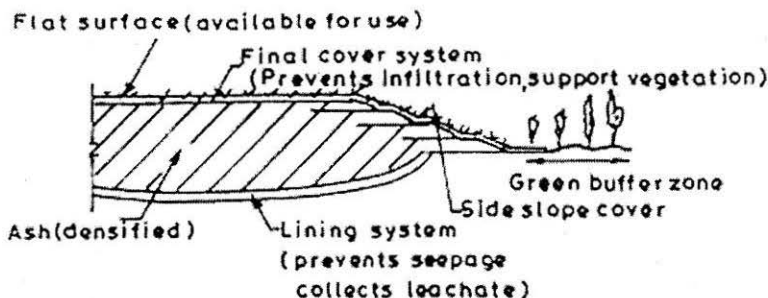
FIGURE 56 : Engineered Closure of an Ash Pond

water beneath an ash pond. Data pertaining to ground water contamination is scanty and more studies are needed in this direction.

The possibility of accumulation of heavy metals in soils and plants around ash-ponds through leachate or air borne ash cannot be ruled out. However, effective control of dust emissions and leachate from ash ponds will ensure that the concentrations remain within acceptable limits (Fig.56).

Reclamation of Old Ash Ponds

Reclamation of old and abandoned ash-ponds is often necessary in densely populated areas where land availability may be scarce and the ash pond area may be needed for recreational, industrial or commercial purposes. One can construct structures on abandoned ash ponds by using long piles to transmit load through the loose ash deposit to firm soil beneath the ground. However, when the thickness of the ash deposit is large, other techniques can be used more economically for in-situ densification of large areas of the deposit (Gandhi, 1996) these includes:

(i)   Vacuum dewatering;
(ii)   Compaction by vibration / vibroflotation; and
(iii)   Compaction by blasting

In addition, use of admixtures, geogrids and stone columns beneath loaded areas can also be undertaken. Available literature suggests that pond ash can be competed in a manner similar to loose cohesionless soil deposits. The use of the above techniques not only increases the capability of the ash deposit to support higher loading, but also precludes the possibility of liquefaction of such deposits in earthquakes prone areas. In addition the above techniques also helps in improving the stability of embankments constructed by the upstream method of construction.

Recent Experience with Utilisation of Flyash

a) *IOCL Bottling Plant near Delhi*

Indian oil corporation has a 75 acres plot for construction of LPG Bottling Plant near Madanpur, New Delhi, about 3-4 kms away from ash pond of Badanpur Thermal Power Station (BTPS). Since the plant site is low lying land, first job to be undertaken involved land development i.e. raising level of 32 acres out of about 2-3 metres. This involved executing 3,70,000 m^3 of earthwork within a total period of 6 months. Since Delhi area is facing acute shortage of earth due to large-scale construction activity, such large quantum of earth was not available within economic leads. However, option to use pond ash was a novel alternative as only conventional material i.e earth had been utilised for such applications so far. The job was very successfully completed to meet the required specification of compaction.

b) Approaches for number of flyovers in Delhi have been constructed using flyash as structural fill. Fly-ash can be compacted satisfactorily with varying moisture content. It is therefore not difficult to carryout approach construction even during rains. Due to high angle of friction, flyash can be used effectively as a reinforced earth wall, which results in large saving in the cost of conventional RCC retaining walls for the approaches. All the flyovers built in Delhi recently using flyash have been found to behave very satisfactorily.

c) A major road embankment for a road connecting Delhi and Noida has been built using fly-ash. The embankment has a height of 9 m and length exceeding 2 km. The fly-ash was obtained from the nearby power plant. To protect the surface erosion, a 1 m thick earth cover has been provided. Detailed specifications for construction and monitoring during construction were carried out by Central Road Research Institute, Delhi.

d) Construction of Ash Dyke

Number of thermal power plants are now utilising flyash itself as a construction material for the dyke of the ashpond. Korba thermal power station of NTPC was the first to raise the ash dyke utilising flyash. Subsequently number of thermal power plants at Vijayawada, NALCO in Orissa, Tuticorin in Tamilnadu etc., have used flyash for ash dyke construction. One of the major problems in its use in ash dyke is to prevent the surface erosion. This is taken care by providing 1 m thick earth cover. Also due to high permeability of flyash material, internal drains in the ash dyke are essential.

Conclusions

Various technological options for converting the flyash, presently a waste material, into a useful material are considered what is given are only indicators, and there may be many other options which need to be looked into in greater details. Based on the current levels of information and available technology, the preferred options seem to be:

(i)   Separate handling of bottom ash, which constitutes about 15-20 per cent of the total ash, either hydraulically or dry. This is a value product similar to fine sand and, hence, an asset for use in embankments, roads and as fill material. One can also explore its use as a fine aggregate in mortar and concrete.

(ii)   With new power stations, the chimney ash, which is already being collected in different silos, should be handled dry. The finest portion can readily be used in conjunction with cement with an estimated user potential of about 20 per cent of the total ash in cement and cement related products.

(iii)   With existing power stations, efforts should be made to convert wet ash handling systems to dry handling systems in the shortest possible time. In the intervening period, there is a need to improve techniques of handling ash and also management of the ash ponds.

(iv)   Large tracts of land are under ash in the form of ash ponds. There is an urgent need to ensure that these ponds are safe from breach of ash dykes and flowing out of ash slurry under all conditions, including earthquakes. Further, efforts should be made to utilise these ash ponds for recreational and agricultural purposes and habitation.

CONCLUDING REMARKS

As a geotechnical community we have done quite well in India. However our potential is much higher. While we have shown that some of our activities in the country have been on par with the best in the world, this excellence has to spread to every aspect of geotechnical activity on a much broader front.

What then is required to achieve the excellence I have been taking about? It is for each one of us to ponder and it is not difficult to find the answers. Based on my experience, I recommend the following:

1.   A commitment to excellence from each one of us, geotechnical

engineers, present and not present here.

2. A positive attitude to continuously learn and to accept change for the better.

3. Partnership and team work among all concerned: owners, consultants and contractors

4. Code of ethics for everyone to follow such as,

    i)   To act on the basis of a well informed conscience

    ii) To act in the interest of the community and for common good

    iii) To be scrupulously honest in the discharge of out duties as engineers

    iv) To utilise our knowledge of engineering in the service of our mother land and humanity.

    v)   Finally to remember our fundamental duties as enshrined in our Constitution, Article 51A, Part IVA, Fundamental Duties: To strive towards excellence in all spheres of individual and collective activity, so that the nation constantly rises to higher level of endeavour and achievement.

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