03 Jun 1988, 10:30 am - 5:30 pm

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Proceedings: Second International Conference on Case Histories in Geotechnical Engineering, June 1-5, 1988, St. Louis, Mo., Paper No. 5.37

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SYNOPSIS : The paper presents case histories of performance of foundations where stone columns were provided, alongwith relevant data regarding structural systems, soil conditions, construction methods and field control criteria. A wide range of applications are included comprising stone columns for area treatment and stone column in small and large groups for isolated footings, pipe pedestals and bridge abutments. In some of the cases design load exceeded the estimated yield load over a part of the stone column length yet collapse did not occur because the soil stress around the stone column increased as more load was passed on to the soil when yield stress was exceeded. There was also the benefit of drainage afforded by the stone columns. Load test data are furnished to substantiate the design approach which takes into consideration the strengthening of the soil annulus around the stone column resulting from compaction and subsequent consolidation.

1.0 INTRODUCTION

1.1 The case histories presented herein report experience of stone column applications for projects of where a total of over 15,000 stone columns were used. The cases include early applications during 1972-76 when theories and design approaches were in the process of evolution. Cases reported include projects where the behaviour matched the postulation, as well as instances when the behaviour was not as expected. The paper begins with a brief review of design approach and theories currently in use. In a further section authors' design approach is summarised and its theoretical basis is explained. This is followed by presentation of case histories with relevant information regarding the soil characteristics, estimated loads and observed performance with regard to settlements and yield loads. A comparison of estimated yield loads and actual loads as well as estimated and observed settlements are furnished for structures which have performed generally as expected. Explanations are furnished of the possible causes of observed deviations from anticipated performance. Salient features of observed behaviour are then summarised in concluding section. Towards the end of the paper, suggestions are made as to further studies and observations needed to clarify some of the unresolved issues and further optimising stone column systems by judicious use of soil reinforcements in the upper 'critical' zones and by providing sand pads with reinforcing fabric layers for minimising differential settlements.

1.2 When the elastic-plastic model and the unit cell approach is used for design of stone columns major uncertainities exist regarding the estimation of yield load. Attempts have been made to estimate the yield loads by adaptation of the cavity expansion theory (Mitchell, 1981), or using the passive earth pressure simulation for a two dimentional case (Van Impe, 1987). A compilation of the results of single column load tests is presented in Section 4.0, and the test results are compared with estimated yield loads. An approximate estimate of the stone column deformation modulus can be made from load test data. Summary of single load test for cyclic loading of 7 day duration are presented to provide an indication of stone column deformation under sustained load.

1.3 The case histories have been grouped to cover various types of applications as detailed below :

- In the first category are included the isolated group of stone columns which are subject to considerable drag loads. There is reason to believe that the stone columns have been during construction or initial surcharge loading, subjected to loads exceeding yield loads calculated according to conventional methods such as Mitchell's adaptation of the cavity expansion theory (Mitchell, 1981). The critical stage of loading for such stone columns is at the end of construction during surcharge loading. It is postulated herein that the additional settlements due to live loads or service loads would be small and would well be within the tolerance of the structure. The case histories substantiate this posulation. Among the cases reported are foundations for steam and ammonia pipelines or large water pipelines. There has been no sign whatsoever of damage by differential settlement in the cases reported, after several years of operation.
- In the second group of cases are included the groups of stone columns supporting rigid structures such as box abutments of major bridges. Here again performance experience of several years and as well observations during recent construction substantiates the design approach.
- In the third group of cases are included embankments and flexible structures subject to area loads and strip loads. The case histories provided an opportunity to verify the design method for a case of soils with significant preconsolidation pressure as well as a pipeline under construction in an area with very soft underconsolidated clays.
- The fourth group of cases covers stone columns in the 'elastic' range such as tank foundations where a conservative basis of design was adopted. The estimated settlement in one case is in fair agreement with the theory of Van Impe, while in another case, the peculiar behaviour of ground treated with 'floating' stone column is discussed to bring out the limitations of the method of analysis used.

2.0 REVEIW OF THEORIES AND DESIGN METHODS

2.1 Theoretical approaches for design of stone columns can be grouped into three categories as described below :

- Analysing the stone column soil system as a 'composite' material where the load shared by the stone column is dependant on the relative values of deformability of the stone column and the surrounding soil. Conventional elastic solution can be used, if care is taken by limiting the load to ensure that the stone column does not yield. The problem then reduces to evaluation of design parameters defining the load deformation behaviour of the stone column and the soil.

In the second method an elastic-plastic stone column behaviour is postulated. The load sharing between soil and stone columns in a unit cell consisting of the stone column and surrounding soil could be based on elastic solutions until yield occurs. Thereafter the stone column load would be limited to the 'yield' value, thereafter the load shared by the soil would be estimated by use of equilibrium relations. Theories such as Vesic's cavity expansion theory may be used to estimate the yield load. Alternatively other modes of failure would be considered and passive pressure theories can be applied by resorting to a two dimensional simulation (Van Impe, 1987).

The third is a semi empirical approach. The stone column system behaviour is postulated in terms of the replacement factor i.e. the ratio of the area of the stone column to the area of the ground treated (α). The relation between the replacement factor, α , and the settlement ratio which is defined as the ratio of settlements of treated and untreated ground β is based on design curves established from past experience on large scale tests.

2.2 The stone column cylindrical element of compacted granular material usually is in a 'high dilatant' condition; high values of angle of internal friction are therefore realised in practice. However, designers often fail to take into consideration the influence of construction methods. It is difficult to model the stone column soil interaction analytically as the soil surrounding the stone column has a complex stress history. It is first subjected to a release of stress while boring. This is followed by recompaction and build up of radial stresses; these stresses could be of a high order depending on the level of compactive effort and consumption of stone and sand. When stone columns are installed through tubes provided with dispensable shoes, the initial release of stress is avoided. An annulus of soil in the immediate vicinity of the stone column-soil interface gains strength as consolidation takes place after installation. The extent of gain varies according to the distance from the soil-stone column interface and is also dependant on the consumption of the stone and the corresponding lateral displacement. There is a radical change in the stress conditions starting from the initial Ko state where the direction of the major principal stress is vertical to a final axisymmetrical state of stress where the maximum principal stress is in a horizontal radial direction. The authors believe that a precise theoretical assessment of the consequences of these stress changes is not feasible. It is also very difficult to verify the theoretical postulations by observation as instrumentation of the zone of interest would be difficult because of the disturbance caused during the installation of the stone columns. One must therefore rely on semi empirical methods and use of load tests to evaluate the parameters used.

2.3 It must also be noted that the stone column behaviour at different elevations would not be the same. In view of the benefit of the increased overburden stress and the availability of comparatively stronger soils at the lower levels, elastic behaviour is often realised in the lower part of stone columns whereas yield generally occurs in the upper layers. Quite often the weak layer immediately below the drying crust or the compacted granular soil pad is 'critical'. It is the authors' contention that, in an optimised design of stone columns, the existence of a zone where yield occurs must be allowed for. The designers must therefore address themselves to be task of analysing the consequences of stone columns 'yield'. In the lower layers where yield does not occur elastic theories can very well be used. Too much refinement in elastic analysis is generally not required since the settlement in the optimised system mainly arises, from the soil stone column deformation in the 'plastic' zone.

2.4 An important aspect of the stone column behaviour in the plastic zone is the contribution made by the stress increase in the soil surrounding the stone column due to the process of load sharing. A conclusion that emerges from an examination of the stone column 'unit cell' behaviour is that there is no hazard of collapse. The stone column progressively gains strength as more load is passed on to the soil. A design which allows for the possibility of 'yield' of stone column in a part of its length is therefore not subject to hazard of collapse and progressive failure, provided that the lateral loads are small. There is however the hazard of stone column failure in sensitive soil. A design approach which relies on the above postulations based on unit cell theory, is not recommended for sensitive clays.

3.0 DESIGN APPROACH ADOPTED IN THE CASES REVIEWED

3.1 The proposed design approach is based on an initial categorisation of the soil zones into elastic and the plastic zones (K.R. Datye, 1982) (Ref. Fig. 1). The stone columns share of load is estimated by using equilibrium methods and a preliminary evaluation is made of the hazard of the stone column yield in different layers considering the insitu undrained strength and overburden pressure in different layers. This preliminary estimate of 'yield' load is verified by load tests. The realised capacity of the stone column columns (or yield load) has generally been significantly higher than the values estimated according to the parameters suggested by Mitchell (1981). Soil deformation in the elastic zone were estimated by treating the stone column as a compressible pile. The uncertainty in the estimation of stone column settlements in the elastic zone generally arises out of the difficulties in evaluation of the deformation parameters of the soil and the elastic constants for the stone columns. This aspect is discussed further in para 3.5.

3.2 The deformations in the plastic zone are estimated by considering the unit cell wherein the sectional area of the stone column is worked out by examination of the consumption record and it is presumed that the volume of the compacted stone and sand would be about 80% of the total of the loose volumes of sand and stone placed in the stone column.

3.3 Construction Methods : The rammed stone columns were installed in cased bore holes after removing the soil and compacting the stone and sand by ramming as casing was extracted progressively. A gap graded mixture has been used where the maximum size of sand is limited to 5 mm and the minimum size of stone is 25 mm. The sand forms a slurry and works its way into the voids in the stone during compaction when the stone and sand are placed in hore holes full of water. Even if stone and sand was placed in alternate layers thorough mixing has been achieved in practice as verified by inspection of stone columns after excavation and dewatering. Field control was exercised by measuring the consumption and observing the 'set'. The 'set' was defined as the penetration for 25 blows with specified fall of a rammer of specified weight. The installation details are described in Datye & Nagaraju (1981) and the of consequences of installation methods are discussed in Datye & Nagaraju (1984).

3.4 Experience has brought out very clearly the advantages derived from the gain of strength in the soil annulus surrounding the stone column. The maximum vertical stress in the stone column has been generally found to exceed 50 times Cu as against the postulated value of 25 times Cu according to Mitchell (1981).

3.5 Deformation Modulus of the Stone Column : The deformability of the stone column material in situ depends on the material characteristics, gradation and the compactive efforts used in forming the column. If a well graded material is used which is not liable to get crushed at the particle contacts, stone column modulii should match the characteristics of a well compacted rockfill or a dense sand. In the cases reported, highly dilatant and dense material was produced by using a gap graded mixture and observations showed that the voids in the crushed zone were entirely filled by the sand and since a very coarse stone in the size range from 25 to 75 mm constituted of hard angular fragments of sound rock was used, remarkably dense stone columns were formed. Particle breakdown, if any, actually contributed to improvement of gradation. In the load test for the first cycle of loading stone column generally reveals very low compressibility corresponding to an elastic modulus of 20,000 to 60,000 t/m^2. After a few cycles of loading and after allowing for consolidation for 7 days at working load, the modulus was reduced and the deformation was increased by factor of about 3. The designs are based on estimated modulus of 8000 t/m^2, which is close to the values suggested by Mitchell. It should be noted that the design values recommended take into consideration the likely increase of the deformation of the stone column due to the consolidation of the soil in the radial direction in the soil annulus surrounding the stone column. The actual performance of the stone columns in groups is expected to be better than single column as the opportunity for lateral deformation gets restricted due to increase in the vertical stress in the soil as the stone column deforms in the plastic range and a greater share of load is passed on the soil.

3.6 Soil Modulus : The relevant parameters of the soil constituting the unit cell is the drained oedometric modulus. It is usually adequate to estimate Pc from shear strength measurements, on the basis of Cu/pc relations and coefficient of compressibility from laboratory consolidation tests.

3.7 Elastic Analysis : It is the authors' view that conventional methods of analysis for compressible piles or composite material constituting the unit cell are adequate. Too much refinement in the analysis of the elastic settlement is not usually required in practice, since consolidation is very rapid and rectifications or modifications can usually be carried out after a short period of observation.

3.8 Time of Consolidation : The actual time of consolidation of the stone column system has been found to be very short (usually less than 2 weeks). This is due to several factors described below :

- There appears to be no smear effect presumably due to remoulding of the soil near the stone column interface and a thorough mixing of the sand and soil.
- There is reason to believe that hydro fracturing occurs due to the high radial stresses developed during compaction and this would increase the horizontal coefficient of consolidation.

4.0 SINGLE STONE COLUMN BEHAVIOUR

4.1 The single stone column behaviour was interpreted on the basis of an elastic-plastic model where a single column load test which is essentially similar to a pile load test was used (See Fig. 2). The load is transferred by means of a cylindrical loading element to the top of the stone column situated at a depth of about 1.2 m or more depending on the soil condition. By using a smooth sided cylinder coated with bitumen the friction is minimised and it is presumed that the entire load in the single column test is transferred to the top of the stone column. The test results are interpreted in a conventional way as in pile load tests and the yield stress is worked out by dividing the yield load by the area of the stone column estimated from the consumption data. It is presumed that the stone column cross sectional area corresponds to the average net volume per metre in the zone of interest and the net volume of the compacted sand mixture is taken to be 0.8 x the volume of sand plus stone consumed during the installation of the stone column as measured in boxes or bins. A parameter F'sc was derived as per following equation.

σ_v = Cu F'sc

where σ_v = yield stress in the stone column

Cu = undrained cohesion

4.2 Mitchell has proposed, based on the Vesic's cavity expansion theory a value of 25 for F'sc. But in the cases presented, the F'sc values turned out to be in excess of 40 and were in fact often in the range of 50-60. (See Table 1). The data compiled in table pertain to 1830 mm diameter piepline, Sion Koliwada; 2345 mm dia pipeline, Kasheli; and stone column installations in Mangalore Chemicals & Fertilisers. Low values are only for very weak soils Cu $\sim$ 0.6 - 0.7 t/m^2.

4.3 'E' value for the stone columns are estimated from individual stone column load test results. The 'E' value for the first cycle (immediate) loading of the stone column (undrained modulus) is as high as 50,000-70,000 t/m^2 while for sustained loading (7 days loading) the 'E' value is about 7000 t/m^2. The data are presented in Table 1. The estimation of 'E' are very approximate and it gives only order of magnitude. The E value has been very much in the range as suggested by Mitchell (1981) i.e. 4000-7000 t/m^2.

4.4 The settlement magnitude for the different foundations are as follows :

Untreated ground	500-100 mm	- 10 m thick layer of clay
Stone column treated ground	50 - 100 mm	- 10 m stone column length

Considering an 'E' value of 8000 t/m^2, strain in the stone column at yield would be 0.6%, which will cause a settlement of 60 mm for a 10 m long stone column. This shows that the settlement of the stone column treated ground would be in the range as mentioned above.

5.0 ISOLATED GROUPS OF STONE COLUMNS

In the cases presented below, the footings bear on stone columns covered by a pad of granular soil 0.6-1.0 m thick. The small groups of stone column thus installed are observed to have performed well. Total load including estimated drag forces were of an order suggested by Broms (1979). The stone columns would have yielded over a part of its length. Even under heavy loads the stone column system did not show any signs of collapse and the settlement is very small at the end of construction (of the pipeline/pipe racks).

5.1 Pipe Rack at IFFCO, Kandla (1972)

5.1.1 IFFCO Kandla has constructed pipe racks in 1972. The footings of the pipe rack were supported on 750 mm dia 10 m long stone columns.

5.1.2 Subsoil profile and characteristics are presented in Fig. 3.

5.1.3 Stone column design : The stone columns were designed to carry 20 t sustained load and 30 t as short term maximum load. The footings were treated as rigid pile caps, and each footing was supported on 6 stone columns.

Loads on one footing were as follows :

Drag load (upper bound)	120 t
D.L. + L.L.	150 t
Preload (surcharge)	123 t
Total	393 t

With 50% of Drag load (lower bound) total = 333 t.
Drag load was calculated according to Broms guidelines - Broms (1979).

Therefore each stone column was subjected to estimated vertical load of 55t - 65 t (nearly 2.5 times the load given by Mitchell's "25Cu" criteria).

5.1.4 Behaviour of the Stone Column Foundation : The final settlement observed were 15 mm (maximum) to 6 mm (minimum) whereas that of untreated ground would have been of the order of 800 mm. Also the time involved in removing, placing the preload, installation of sand drains etc. would have been considerable. With the piles, the major problem would be the drag caused by settlement of surrounding soil.

5.2 2345 mm diameter Pipeline at Kasheli near Bombay

5.2.1 The pipeline is constructed from Anjru Diping to Majiwade in Thane Dist. The Pipeline alignment is nearly 8 km long and crosses several creeks. The pipeline is divided into 3 sections as follows :

Section 1 Majiwade to RT4 (underground pipeline)
Section 2 RT4 to C-point (piepline above ground)
Section 3 C1-point to G-point (pipeline above ground and over bridge III)

Typical subsoil profiles and soil characteristics are presented in Figure 4.

5.2.2 Stone Column Foundation : The pipeline is supported in concrete pedestals with base area 4.2 m x 2.1 m through which 70 T piepline load is transferred to the ground. Each pedestal bears on 6 rammed stone columns. Nominal diameter of the stone columns is 750 mm. For compaction control first 'set' was taken at a depth of 4 m above tip level and for each 3.5 m additional length one 'set' was taken. The set criteria was for 1.5 T rammer and 1.5 m fall penetration should be less than 20 mm for 25 blows. Approach embankments for the bridges were provided with area treatment with stone columns. Average length of stone columns were as follows :

Section 1 3 - 7 m
Section 2 4 - 8.5 m
Section 3 4 - 7 m

For these sections average diameter of stone column 800 - 1000 mm, Cu of the soil = 1 t/m^2.

5.2.3 Load on each footing was as follows :

Dead load of pipe		13 t
Pedestal load		21 t
Drag load		112 t
	Total	146 t* at the end of construction
Additional water load		45 t
Final load		191 t** at the ime of commissioning the pipeline.

* Therefore load/column was 25 t.

** At the time of commissioning the pipeline, load per column was 32 t.

5.2.4 Settlement Monitoring : A few pedestals were selected and provided with magnetic settlement markers, surface settlement markers (plate type) and piezometers to monitor the settlements of clay layers and ground. In almost all the cases, the settlement markers were installed when 50% of the loading (i.e. embankment construction) was over. Settlement recorded was 30 mm. Since Δ p is very small, the settlement at the time of commissioning the pipelines would not be of any consequence (Ref. Annexure 1).

5.2.5 Consumption data were analysed considering 80% of the loose volume dumped in. For the stone column gap graded materials were used comprising of stone column of size 25 mm to 75 mm and sand of maximum particle size 5 mm in the ratio of 5:2.

5.2.6 If the stone column would have been designed using Mitchell's '25 Cu' criteria (with F.O.S. = 1) the number of stone columns would have been as large as 16 as against 6 columns provided, which show no sign of yielding.

6.0 LARGE GROUP OF STONE COLUMNS

6.1 Belapur Bridge Abutment (1975)

Stone columns were used for treatment of foundations of the abutment of a major highway bridge near Bombay. The brdige is over a creek and has spans of 50 m, designed to carry a 70 t tracked vehicle. Approach to the bridge is an embankment on ground treated with 40 mm sand drains. The box abutment rests on 37 Nos. of 750 mm diameter rammed stone column with spacing 1.7 m. The design capacity of the stone column is 25 t, the stipulated yield value of 40 t was confirmed by load tests. The bridge deck rests on caisson foundations. Soil characteristics are as exhibited in the figure 5. The load intensity at the base of the abutment is 12 to 14 t/m^2, considering an embankment height of 6.7 m having unit weight of 1.8 t/m^2. The actual load may be higher due to drag forces. Settlement of the virgin soil under this load would be of the order of 1.75 m. Settlement as observed after 7 years of construction is of the order of 8 cm (accuracy ± 5 mm), considering that the deck slab of the bridge and box abutment were constructed to same elevation. This settlement is less than 5% of the settlement of the virgin ground under comparable load. Considering load intensity of 14 t/m^2 and plan area of box abutment 4.5 x 12 m, the estimated load is 756 t and a total drag of 338 t calculated as per Broms' guidelines (Broms, 1979). The load per stone column was almost equal to the design load since 37 stone columns were provided. Alternatives such as piles, preloading were examined and rejected. Piles would have been subject to heavy drag forces and lateral loads due to the deformation of the soil. Preload would have required longer time for stage loading and would have interfered with the construction of the abutment. The structural performance of the abutment is satisfactory except for a minor crack due to an unsatisfactory junction detail. The settlement is stabilised and there is no noticeable settlement in the last 5 years.

6.2 Kasheli Box Abutment

This box abutment was supported on 49 m x No. of 750 mm diameter stone column spaced 1.3 m c/c. The box abutment was designed to carry two pipeline of diameters 2345 mm and 3100 mm on either side and a road in the middle maximum stress intensity at the foundation level is 25 t/m^2. A compacted sand pad of 400 mm thickness is provided between the box abutment footing and stone column top. The soil profile is exhibited in Fig. 4. Over a period of more than 1 year the box abutment has not shown any significant settlement (i.e. observed settlement is less than 50 mm).

7.0 AREA TREATMENT WITH STONE COLUMNS

As against the individual footing i.e. small group of stone columns, the large group of stone column is capable of reaching much more load because each unit cell of stone column bears more load due to, radial confinement of stone columns. In large group the loads also turned out to be very small. The cases falling under this category are reported below.

7.1 Large Scale Test Plot at Bhandup near Bombay (1982)

7.1.1 Construction of lagoons for treatment of sewage is contemplated near Bhandup, a north-east suburb of Bombay. Use of stone columns is foreseen for improvement of the ground for the embankment of the lagoon and foundations of various structures. An instrumented test embankment was constructed to verify the design approach and specifically the following aspects :-

- Use of single column load test data for evaluating the yield load parameters for large stone column groups.
- Estimates of increase in stone column capacity for stone columns in large groups subjected to area loads.
- Use of equilibrium methods for estimating the stress in the soil in stone column treated ground and corresponding values of settlement.
- Verification of the efficacy of stone columns as load relievers for ensuring overall stability of embankments on soft clay.

7.1.2 The subsurface profile and soil characteristics are exhibited in Fig. 6. The relevant data are summarised bleow :

Cu = 0.7 t/m^2 Cc/1+eo = 0.23
Depth of clay 4.8 m

7.1.3 The stone column layout was selected to provide a factor of safety of 1.4 with respect to overall stability. At the designed height of embankment, the yield capacity of stone column was exceeded so that a considerable part of the applied load was shared by the soil. Shear deformations were expected along with significant lateral movements near the toe of the embankment.

7.1.4 Following Instruments where provided for monitoring

- Overflow type as well as magnetic type buried settlement markers.
- Casagrande piezometers placed below the centre of the embankment.
- Lateral displacement markers installed near the toe of the fill to indicate horizontal movements.

The scheme of instrumentation is exhibited in Fig. 7.

7.1.5 Single column load test results for the rammed stone columns show that the column did not yield even when the vertical stress intensity in the column had reached 45 Cu.

7.1.6 Several settlement markers were installed, many of liquid level markers did not function due to damage of the tubing by rodents. However, 5 markers functioned satisfactorily. Maximum observed settlement varied from 90 mm to 125 mm giving an average settlement of 112 mm at the centre of the loaded area. The observed settlements in an adjoining sand drain test plot were analysed to verify the design values of parameter Cc/1+eo & Pc. Based on these verified parameters the settlement of untreated ground at the load imposed on the stone column test plot was estimated as 450 mm. A ratio of settlement of stone column treated ground to settlement of untreated soil was 0.25. The stone columns arrangement below the embankment has an area replacement ratio ($1/\alpha$) of 4.4.

7.1.7 The relation between $1/\alpha$ and β are computed by the equilibrium method allowing for the stone column yield. Curves accounting for the effect of preconsolidation pressure are also shown in Datye & Nagaraju (1984). Observed settlements are quite close to values estimated by equilibrium theory even after making some allowance for the probable error in estimation of the tributary are for the stone column groups and the corresponding imposed load on the unit cells.

7.1.8 Lateral displacements were monitored by rigid stakes. The top stiff descicated clay and compacted general fill was isolated by installing the marker in 60 cm diameter RCC pipe. Horizontal movements of the marker were measured by a theodolite and the tilt was read by placing a tiltmeter on top of the marker. Lateral displacements were computed from tilt readings and theodolite readings. Even for 6 m height of fill were small which confirmed that factor of safety against shear failure was sufficiently large. Movements were accentuated only after excavating for a depth of 1.5 m below ground level near the toe even then the lateral movements were smaller than an adjoining plot with 5 m fill load on untreated ground.

7.2 Stone Column for 1830 mm dia Pipeline, Sion-Koliwada, Bombay

7.2.1 The water pipeline at Sion Koliwada is supported on the ground treated with an arrangement of stone columns consisting of two rows at a spacing of 4 m along the rows and 2 m between the rows and a line of stone column in the centre at spacing of 4 m c/c.

7.2.2 Very soft underconsolidated clays deposits (4 m thick), having a Cu of 0.6 t/m^2, was encountered here through which stone columns were installed bearing on dense murrum strata. Final compaction 'set' criteria were followed for the stone column installation as discussed earlier.

7.2.3 Settlement monitoring data are not available at the time of writing this paper. However, when the data are available, they will provide good basis for the stone column design approach discussed earlier.

8.0 STONE COLUMNS IN ELASTIC RANGE

For critical structures such as storage tanks storing hazardous liquids where large settlements could not be tolerated, the large groups of stone columns were designed conservatively. In these cases a 2.0 m thick compacted sand pad has been provided for load dispersion.

8.1 Phosphoric Acid Tanks at Mangalore Chemicals & Fertilisers, Mangalore

8.1.1 Subsoil profile is presented in the Fig. 8.

8.1.2 Tank Data :

No. of tanks	= 2
Dia. of the tanks	= 23 m
Storage height	= 13 m
Design load intensity at the tank bottom	= 22 t/m^2

8.1.3 Stone Column Foundation :

Type of stone column	= Vibro-float
Nominal dia. of stone column	= 750 mm
Total No. of stone columns	= 780 (for the two tanks)
Spacing of stone column	= 1.3 m c/c in triangular grid
Load intensity at the top of stone column	= 14.43 t/m^2
Load on the unit cell	= 19.3 t

While the individual stone columns were subjected to load test, settlement under 35 t load was 6-15 mm.

8.1.4 When the tanks were subjected to hydrotest they have shown settlements of 39 mm and 55 mm.

8.1.5 Considering an average sectional area of 0.785 m^2 and Cu value of 1 t/m^2 the stone column capacity would be 20 t according to Mitchell (1981) and with a factor of safety of 3 times the number stone columns would be required.

8.1.6 Settlement calculations are presented in Annexure 2.

8.2 Phos Acid Tanks at IFFCO, Kandla

8.2.1 Tank Data :

No. of tanks	= 2
Dia. of tanks	= 28 m
Storage height	= 10 m
Design load intensity at the bottom of the tank	= 20 t/m^2

8.2.2 The subsoil characteristics are same as presented in Fig. 3.

	Initial values	Values after preloading
Sand fill	= 2.0 m	
Soft clay (3.0 m thick) Cu	= 1 t/m^2	3.2 t/m^2
Clayey sand (5.0 m thick) Cu	= 3 t/m^2	4.5 t/m^2

Dense sand layer below clayey sand layers.

8.2.3 Area Treatment : Sand drains were provided in the entire area and the area was preloaded upto half the maximum design stress. 1 m of the top soft clay was removed and filled with compacted sand and thereafter 750 mm rammed stone columns were installed. The tank was constructed on a sand pad of total 5 m thickness.

8.2.4 Testing of the Tanks : The tanks were tested by filling water and then with acids. The settlement recorded are as follows :

	Water load (10 t/m^2)		Phos acid load 18 t/m^2		Total	
	Max (mm)	Min (mm)	Max (mm)	Min (mm)	Max (mm)	Min (mm)
North tank	60	48	142	74	202	122
South tank	49	42	118	80	167	122

8.3 Settlements observed versus computed

8.3.1 Results of computations are summarised in Annexure 2.

8.3.2 For M.C.F. computed settlements are close to the values computed by Van Impe method.

8.3.3 In Kandla settlement would be very small as by equilibrium method considering that the soil was preloaded prior to stone column installation and stone column yield value was 30 t. The large actual settlement can be attributed to settlement of compressible soil layers below stone column tip. The stone columns tend to concentrate the load increasing the intensity on the soil layer below the tip. The effective preload intensity in the lower layers is singificantly less than the applied load on the surface. These two factors seem to have contributed settlement beyond the estimated values.

9.0 CONCLUSIONS

9.1 The case histories bring out clearly the advantages of using a simple theory based on a one dimensional analysis where the soil-stone column interaction is simulated by unit cell consisting of a cylindrical element of compacted granular soil placed within a soil annulus. A simple linear elastic model of stone column behaviour and equilibrium relations have been used successfully to predict behaviour of ground improved with stone columns.

9.2 Precise modelling of the soil-stone column interaction is not feasible in view of the complications arising from a complex stress path followed by soil elements around the stone column involving relaxation from an initial Ko state to a stress condition with high radial horizontal stresses. Rotation of the direction of major principal stress, variation of stress conditions with distance from the soil stone column interface further complicates the analysis. The benefit of axial symmetry could be taken by adopting the unit cell simulation. The reliability of prediction very much depends on accuracy in estimation of the odeometric modulus of the soil, determination of sectional area of the compacted stone column insitu and estimation of the yield load as well as deformation modulus of the stone columns. If the stone column section is determined from consumption records and elastic modulae in the range of values suggested by Mitchell are used, conventional elastic solution provide an adequate basis for estimation of settlements for stone column systems working in the elastic range.

9.3 A major element of uncertainty lies in the estimation of the yield stress for the stone column. This parameter is strongly influenced by construction methods. For the cases reported it has been found that the actual stone column capacities realised are about twice the values estimated according to Mitchell's parameters based on cavity expansion theory (Mitchell, 1981).

9.4 The case histories substantiate the author's postulation that the stone column in a unit cell would not collapse as the increased load transferred to the soil after local yield occurs, improves the capacity of the stone column. The case histories substantiate the author's view that factor of safety in the range of 4-3 is not necessary. Number of stone columns required can very well be determined by taking the unit cell capacity to be equal to yield loads measured in single column load test.

9.5 The case history demonstrates conclusively that there is no hazard of long term increase in the deformation by progressive enlargement of the stone column as the surrounding soil consolidates. The long term or the sustained load modulii are in the range of values proposed by Mitchell (1981) eventhough the loads for each unit cell are 2 to 3 times the capacities based on Mitchell's parameters. Several years of post construction performance observations over a range of soil condition bring out clearly the merit of an observation based design approach where the design parameters were evaluated by conducting single column load tests. Suggested elastic-plastic model provides a good assessment of the settlements when yield occurs over a significant part of the stone column length when replacement ratio is high. However, when stone column behaviour is in elastic range as for cases when replacement ratio is in excess of 0.4 the Van Impe model would provide a better prediction.

9.6 As the stone columns also function as a drain and the consolidation of the surrounding soil takes place in a very short period, adverse consequences of deviation from the postulated behaviour can be taken care of by controlled stage loading and use of surchage loads. In initial stages of loading observations of settlements and pore pressures by magnetic plate settlement markers and porous tube piezometers located at various elevation could be adequate for verification of stone column behaviour. By choice of suitable structural system, structural damage can be avoided and therefore too great a refinement in estimation of settlement is not necessary. The case studies have brought out the importance of the influence of construction method. Successful application of the suggested design approach is dependant on establishing field procedure and compaction control criteria in the initial stages of work by observing consumption of stone followed by use of load tests to verify the estimates of yield loads and deformation modulii.

9.7 Suggestions for Further Research

There is a need to take up systematic investigations of unit cell behaviour by laboratory large scale odeometer tests and mathematical modelling to verify the postulation made herein regarding the absence of hazard of collapse of stone columns subjected to loads exceeding the estimated yield loads. More systematic investigation is also needed of the gain in strength of the soil annulus surrounding the stone column and the corresponding increase in the yield load. This could best be done by studying the soil behaviour in large odeometers tests and establishing the relationship between the post consolidation water contents and the state of stress in the soil annulus for given initial void ratio Pc and Cc values. The field observation of moisture content in similar soil after installation of stone column and the reduction in the moisture condition as compared to initial conditions will help to substantiate the postulated unit cell behaviour.

More observations from large loaded area and as well as small groups with magnetic settlement markers porous tube piezometers and soil moisture content measurement before and after consolidation will help to verify the zone categorisation and postulated load settlement behaviour.

There is a need to standardise the load test procedure. Use of load cells would help to establish the actual stress developed in the stone column and by using loading elements of various lengths the yield load at various levels can be determined.

By use of soil reinforcement for the granular fill the hazard of premature collapse of the stone column in the critical zone can be minimised, thereby increasing the capacity of stone column. There is also the prospect of using sand mats reinforced with geotextiles which would minimise differential settlement. In many practical applications stage loading could then be used in the place of preloading to take care of differential settlement.

ACKNOWLEDGEMENTS

The authors would express their appreciation of the co-operation of Indian Farmers Fertiliser Co-op. Ltd. & Municipal Corporation of Greater Bombay in load testing observation and in accepting an observation based approach especially in the earlier stages of the development of the stone columns. Discussions with Prof. Van Impe and Prof. Juran have been helpful in clarifying the issues. The encouraging observations of Prof. Broms on authors papers during early stages of the development have been a great value in optimising stone column designs.

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Table - 1 : SINGLE STONE COLUMN BEHAVIOUR

Site	Stone Column No.	Column Length (m)	Consumption (%)	Average Diameter (m)	Max. Load (t)	Total settlement under max. load immediate (mm)	Total settlement under max. load at 24 hrs. (mm)	Cu t/m^2	E (t/m^2) 24 hrs.	E (t/m^2) 7 days	σ_v/Cu = F'sc	Remarks
2345 mm pipeline, Kasheli, Bombay	SC-1	4.53	186	1.060	40	18.885	36.060	1	17560	3850	45.33	Stone column not yielded
	SC-5	8.10	196	1.090	40	12.590	20.970	1	26455	11306	42.87	Stone column not yielded
	RT2-C/ TB-208	7.25	120	0.822	40.51	9.100	16.100	1	62099	21681	76.33	Stone column not yielded
	SC-4	3.18	215	1.140	40	8.630	14.280	1	17803	6231	39.2	Stone column not yielded
	SC-2	4.00	174	1.030	40	6.290	7.540	1	38405	18974	48.03	Stone column not yielded
						24 hrs. settlement	7 days settlement					
1830 mm Pipeline, Bombay	1554-C	3.325	259	1.062	30	65	105	0.6	1732	1073	33.88	Stone column yielded
	1066-C	8.540	237	1.016	30	73	75§	0.6	4329	2809	37.02	Stone column yielded
	564-E	4.480	156	0.824	20.13	48	61¶	0.6	3523	2028	37.77	Stone column yielded
	1698-C	4.700	215	1.100	30.2	70	79*	0.6	2135	1291	31.79	Stone column yielded
MCF Acid Tanks	996	9.25	124	1.00	35.5	15.0	-	1	32000	12800¥	63	Stone column not yielded
	810	8.5	118	0.99	35.5	12.2	-	1	31500	12600¥	66	Stone column not yielded
	881	9.0	127	1.02	35	9.1	-	1	44050	17620¥	62	Stone column not yielded
	682	7.75	134	1.04	36	5.9	-	1	36500	14600¥	59	Stone column not yielded

§ Measured under 20 t
¶ Measured under 14.73 t
* Measured under load of 20.62 t
¥ Assuming $E_{2\ hrs.}/E_{7\ days} = 2.5$

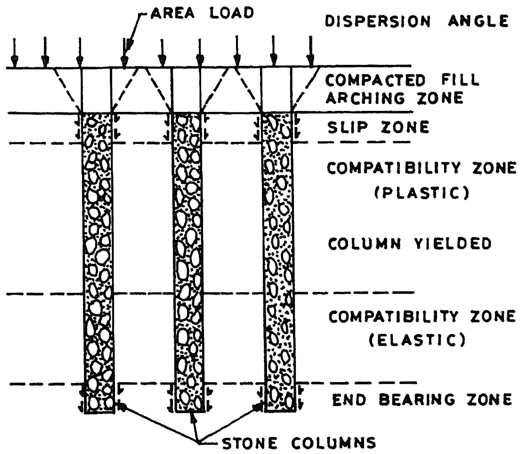

Fig. 1 : ZONE CATEGORIZATION

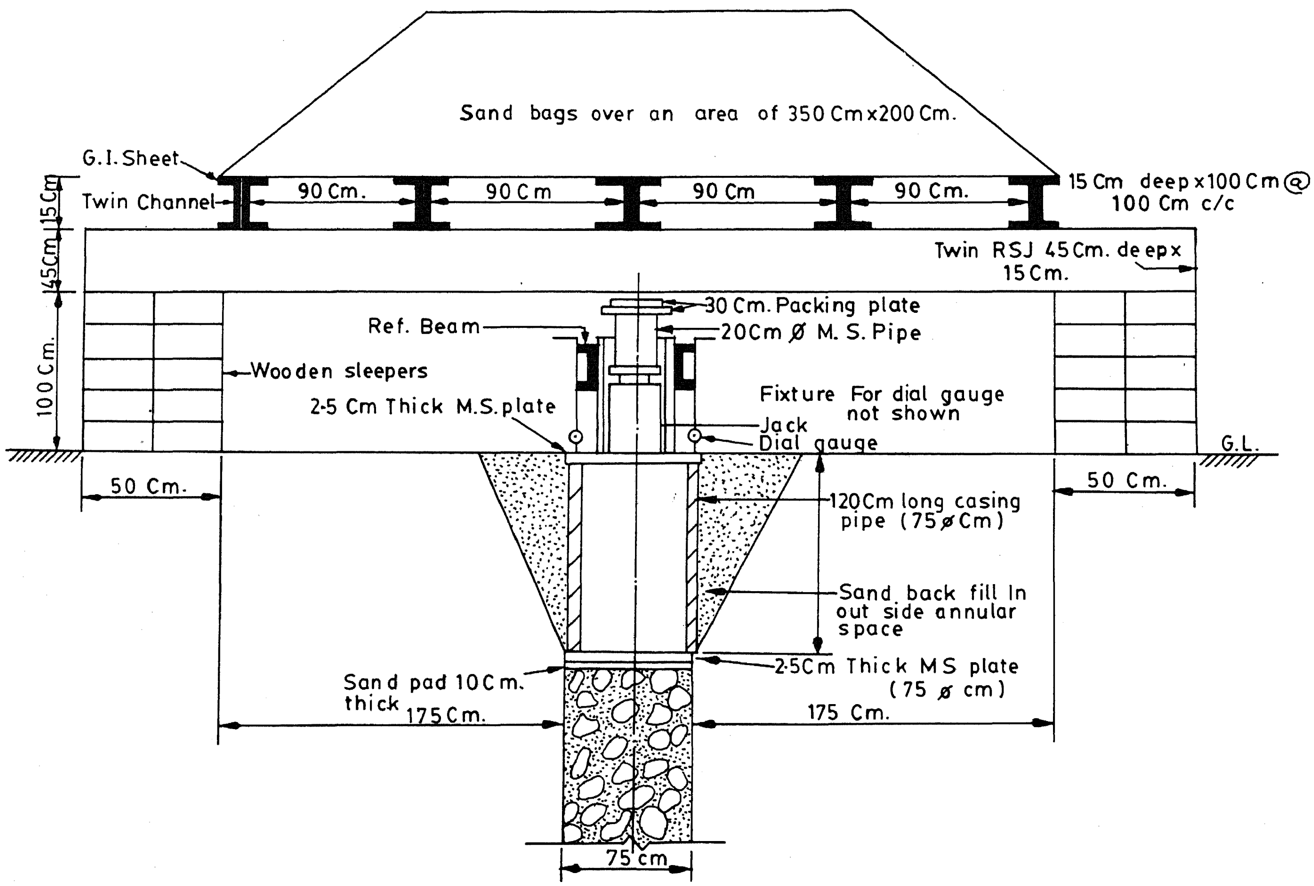

Fig. 2 : SINGLE STONE COLUMN LOAD TEST ARRANGEMENT

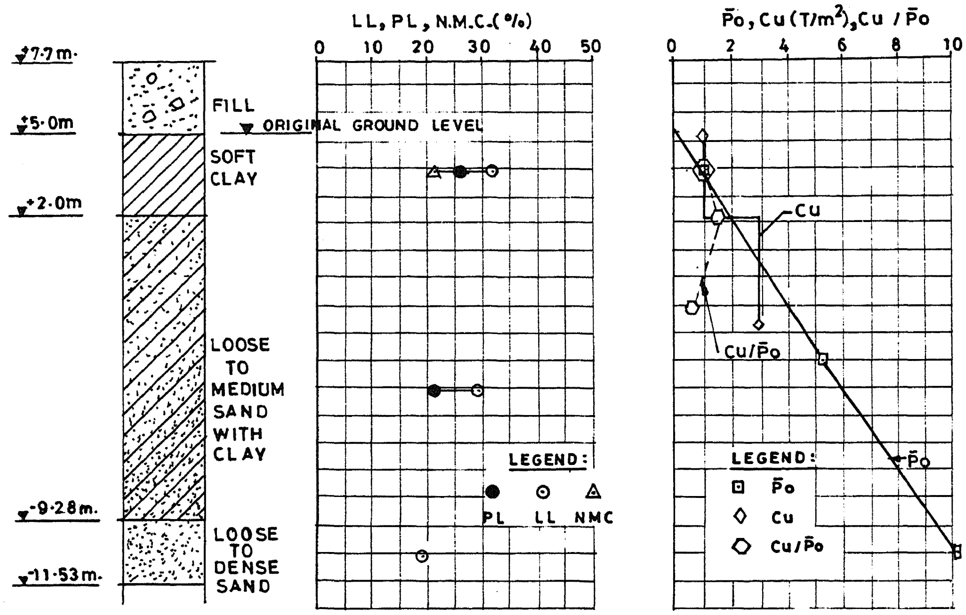

Fig. 3 : SUBSOIL CONDITIONS AT KANDLA

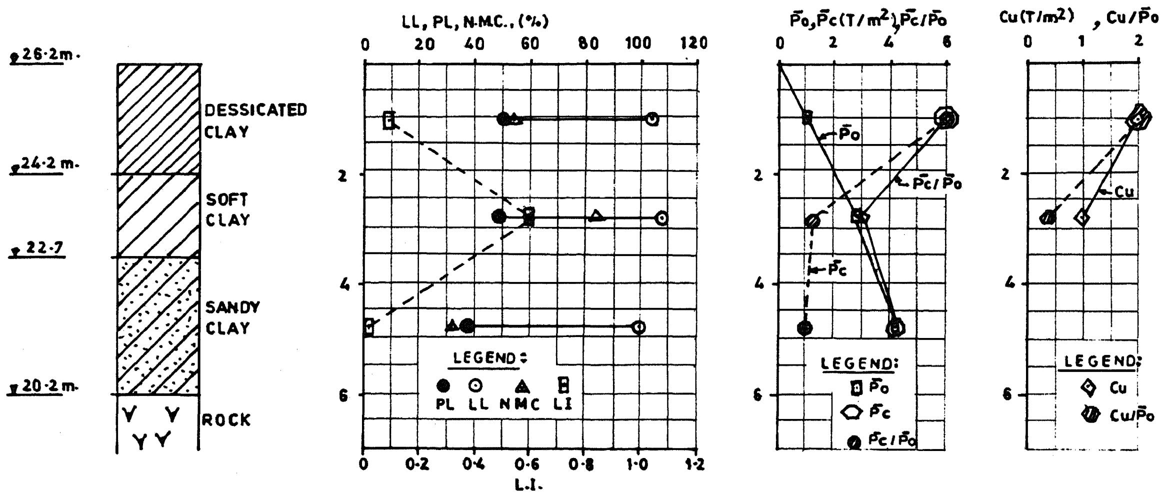

Fig. 4 : SUBSOIL CONDITIONS AT KASHELI

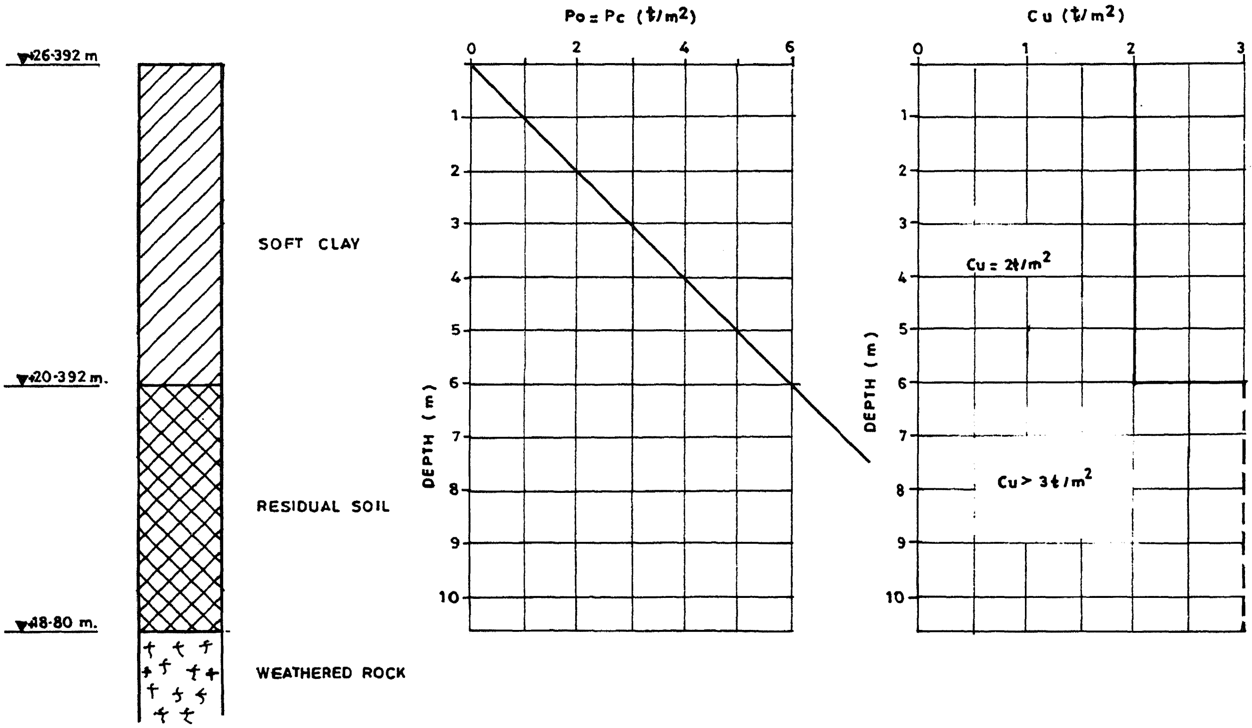

Fig. 5 : SUBSOIL PROFILE FOR BELAPUR ABUTMENT SIDE

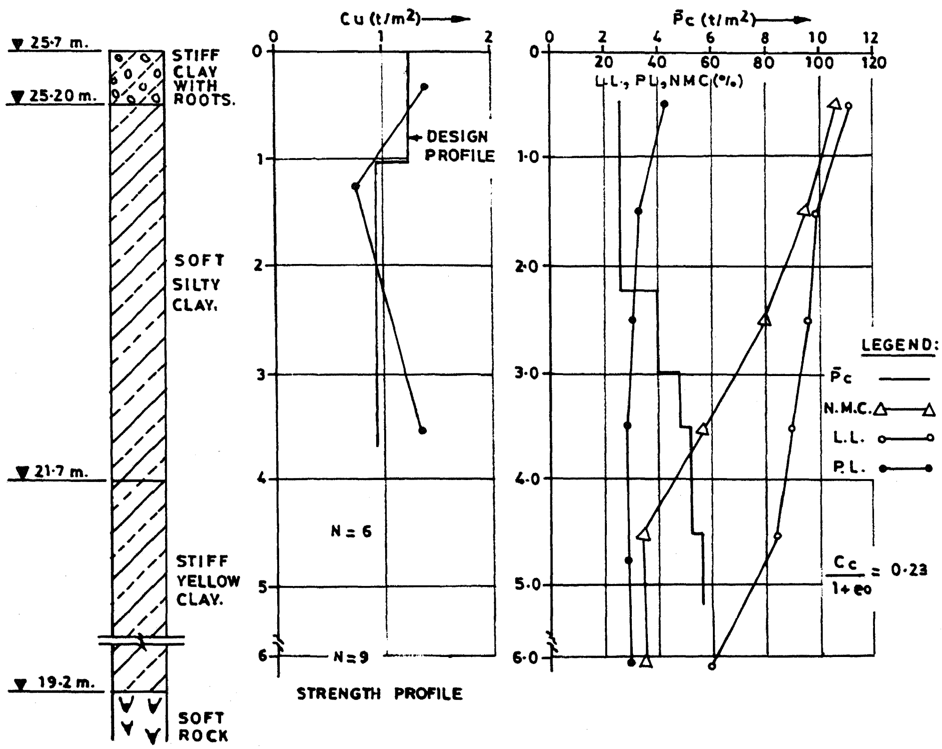

Fig. 6 : SUBSOIL CONDITIONS AT BHANDUP TEST PLOT

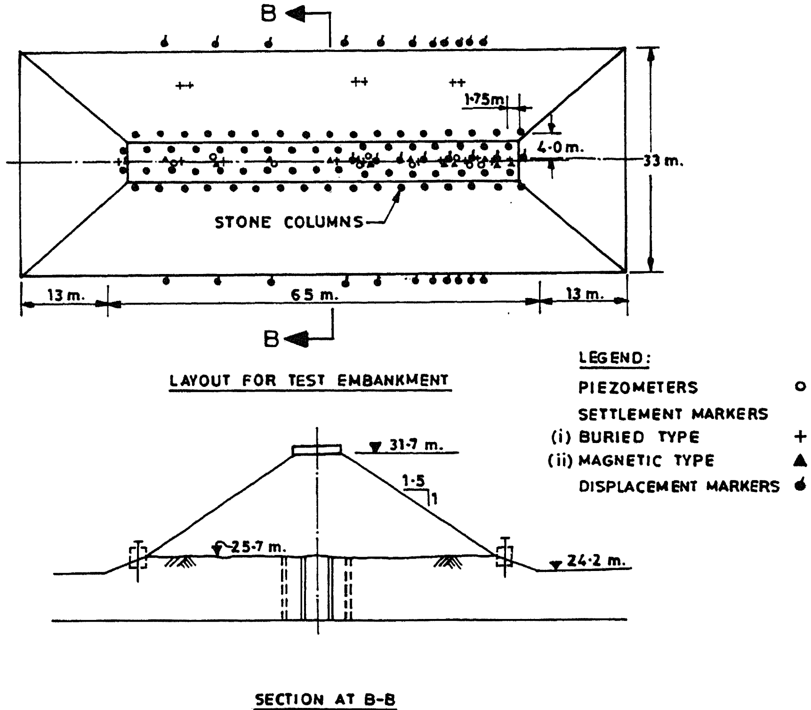

Fig. 7 : EMBANKMENT & INSTRUMENTATION DETAILS AT BHANDUP TEST PLOT

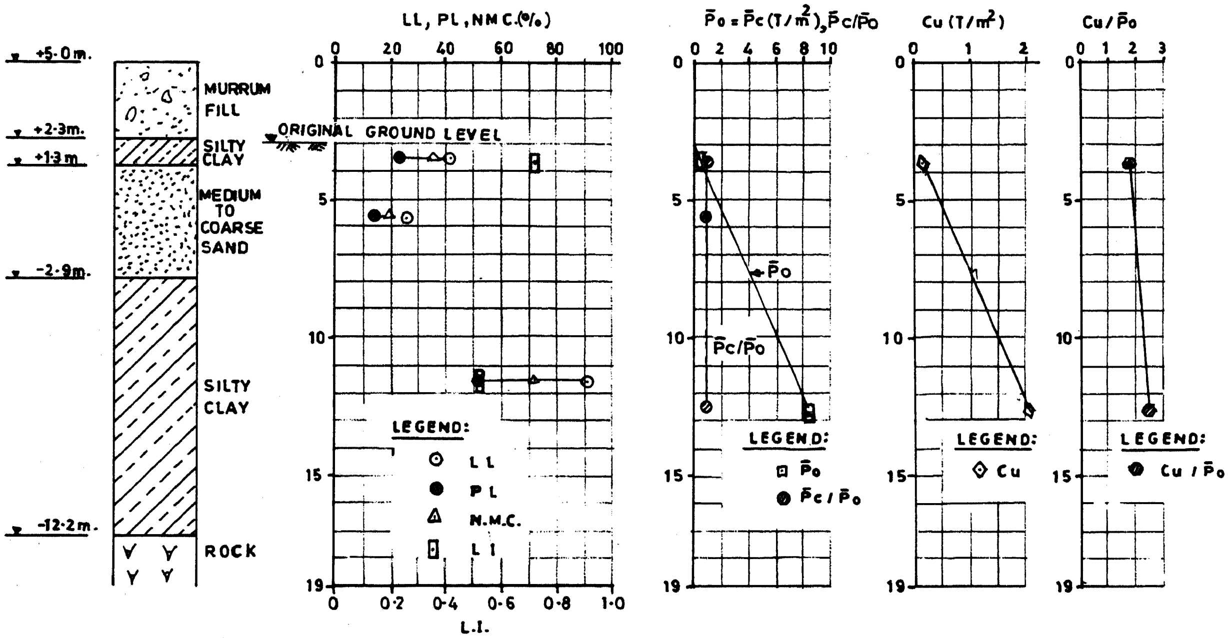

Fig. 8 : SUBSOIL CONDITIONS AT M.C.F. MANGALORE PORT AREA

Annexure - 1 : SETTLEMENT CALCULATIONS FOR KASHELI PIPELINE

Layer	Thickness (mm)	Cu t/m^2	$\frac{Cc}{1+eo}$	P_c t/m^2	P_o t/m^2	Settlement calculation of untreated ground				Settlement calculation using Van Impe's model	
						Δp_1* t/m^2	Settlement (mm)	Δp_2** t/m^2	Settlement (mm)	Under Δp_1*	Under Δp_2**
Desiccated Clay	2000	2	0.3	6.0	1.00	7.66	142	10.0	157		
Soft clay	1500	1	0.23	3.0	2.75	7.03	177	9.2	207		
Sandy clay	2500	-	-	4.25	4.25	5.29	222	6.9	261		
	Total Settlement						541		625	325	375

* due to Dead load of pipe + pedestal load + drag load.

** due to Dead load of pipe + pedestal load + drag load + water load

NOTE : 1) As per Van Impe's model 50 mm of settlement will occur after placing water load. It may be noted that in actual practice Cu of the soil will increase after first stage of loading after consolidation (since sufficient time was available between two stages during construction) resulting in increase in Pc value. Therefore settlement would be far less than the calculated ones.

2) Observed settlement during last half of the first stage loading was 30 mm.

Annexure - 2 : SETTLEMENT CALCULATIONS FOR MCF, MANGALORE

Tank diameter = 23 m

Storage height = 13 m

Stress at the tank bottom = 22 t/m^2

Stress at the stone column top level = 16 t/m^2

(There is a compacted sand fill below tank bottom and above stone column top)

Layer	Thickness (mm)	Settlement calculation of untreated ground						Settlement with stone column (Van Impe's model) (mm)
		$\frac{Cc}{1+eo}$	Cu (t/m^2)	P_c (t/m^2)	P_o (t/m^2)	ΔP (t/m^2)	Settlement (mm)	
Silty clay	1000	0.10	0.10	0.5	0.5	15.951	137	12
Medium to coarse sand	4200	Settlement not taken into account due to negligible contribution						
Silty clay	9300	0.10	2.1	8.17	8.17	14.20	407	42
Total Settlement (mm)							544	54

NOTE : Observed settlement = 55 m under maximum acid load