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SYNOPSIS

Two pilot tests were conducted on two different areas to evaluate the effects of compaction columns on the engineering properties of a sand-clay layered soil profile. In the first field test, 120 stone columns were constructed using vibro replacement and casing-ramming techiques, in a equilateral triangular array with different spacing. In the second field test, 20 stone columns were constructed using casing-ramming technique, in a equilateral triangular array spaced 3m c/c.
In order to evaluate, the stone columns effectiveness and effects, both a detailed in-situ testing and instrumentation programs were planned to be carried out in five different stages. Results obtained from field tests suggest that compaction columns improve the engineering properties of both granular and cohesive soils.

INTRODUCTION

Subsidence associated with oil production in the east coast of Lake Maracaibo in Western Venezuela was detected in the late 1920's (Murria and Jaeger, 1992). The geomorphology of the area (low, swampy lands) prompted the need to protect inhabitants and oil industry installations from lake waters by means of a coastal dike. To date the coastal protection system consists of 47 km coastal dikes, 59 km of inner (diversion) dikes, 490 km of drainage channels and 28 pumping stations.

Western Venezuela is an area of moderate seismicity. Recent evaluation of the behavior of the coastal dikes under earthquake shaking have indicated the possibility of dike failure due to liquefaction of the dike foundation soils. Mitigative measures consisting, basically, of downstream (landside) berms - and upstream (lakeside) additional riprap have been implemented. Post seismic stability analyses of different sections along the dike indicate required width of berms ranging between 10 and 85m. Although berm size in the vicinity of the dikes is restrained or restricted in some areas by the presence of communities and/or oil facilities. Based on those restrictions, it was decided to limit the width of the berms to 40m and to improve the shallow soil layers underneath a limited length of the berm.

The soil improvement method chosen has been stone compaction piles see Fig. 1. In order to evaluate the stone columns effectiveness and effects, two pilot field tests were performed. The site selected for the first field test is located around the station S40 + 00, in section 3A of the Bachaquero dike see Fig. 2. It covers a rectangular area of approximately 1100 m^2. The site was chosen because it is in very low risk area, located away from any community or oil installation, and because the soil stratigraphy was considered similar to that of the areas where this methodology might be applied. The objectives of the Bachaquero-COLM pilot test were as follows (Sgambatti, Echezuria and Blanco, 1989):

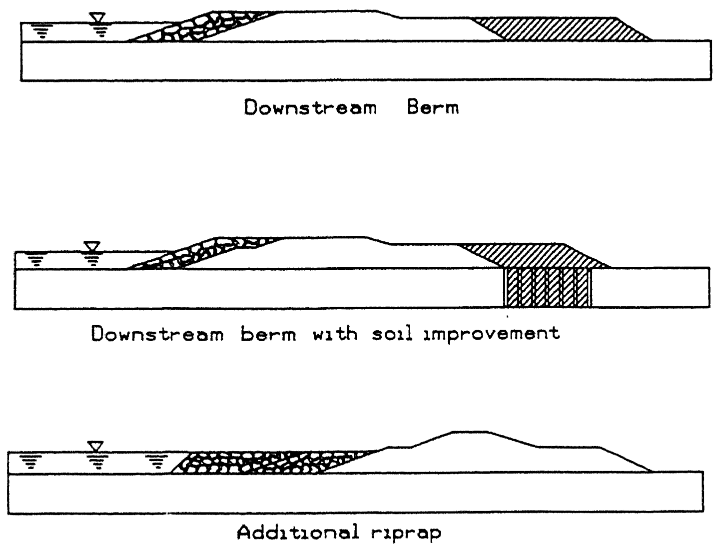


Fig. 1 MITIGATION MEASURES

(1) To evaluate the influence of the compaction piles on the soil geotechnical properties,

(2) To compare the effectiveness achieved by two different compaction pile construction methods: vibroreplacement and casing-ramming,

(3) To assess the influence of the spacing between columns,

(4) To study the effect of stone column installation on the strength of clays and organic soils,

(5) To evaluate the pore water pressure behavior variation during stone column construction and

(6) To evaluate the effect of time.

The site selected for the second field test is located towards the northern part of Lagunillas dike (See Fig. 2).

The main reason to choose this area was the presence of high plasticity clay strata in the site subsoil.

The design of this pilot test was based upon experience acquired from the Bachaquero pilot test. The objectives of the Lagunillas pilot test were the same as Bachaquero; and in addition it was desired to evaluate the effect of stone column installation on the strength of high plasticity clays.

Evaluation of soil site conditions at different stages of the field tests was accomplished using standard penetration tests (SPT), cone penetration tests (CPT), in situ vane shear tests (VST), laboratory tests and seismic shear wave velocity determinations. Pore water pressure build-up during the construction of the piles was monitored by means of piezometers installed at different depths. Additionally movement within the soil mass was monitored by inclinometers placed conveniently in the test area.

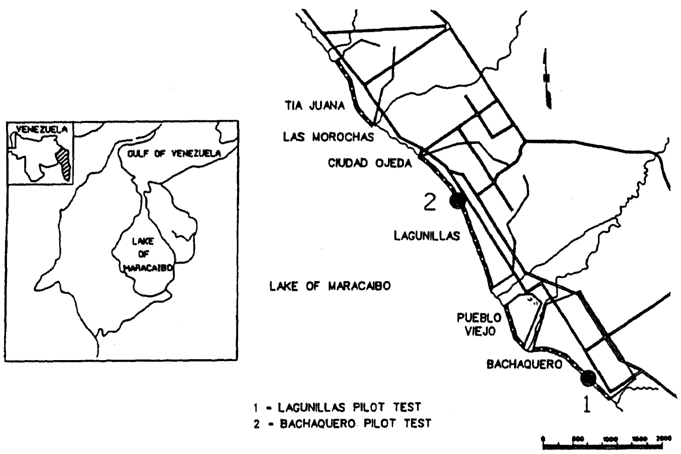


Fig. 2 FIELD TESTS LOCATION

BACHAQUERO FIELD TEST

The effect of strengthening the foundation soil by means of compaction piles depends on many factors (Datye and Nagajaru, 1981) including:

a) method of construction and characteristics of the equipment,
b) in-situ soil conditions, eg soil type, gradation and fines content, relative density, state of stresses and soil structure,
c) the actual construction procedure and,
d) the grid pattern and spacing between columns.

When programming the Bachaquero-COLM field test, some of these influencing factors were taken into account to permit later comparisons on the relative effectiveness. Two different construction techniques were utilized: vibroreplacement and casing driving and ramming. To evaluate the effect of the proximity of the columns, spacings of 2 m c/c; 2,5m c/c; 3m c/c and 4,0 m c/c were chosen. Concerning the grid pattern, an equilateral triangular array was decided as being considered the most efficient and economical.

In order to evaluate all those aspects regarding stone column effectiveness and effects, both a detailed testing program and an instrumentation arrangement were planned to be completed in five different stages:

1) stage 0: prior to any construction or soil treatment, 2) stage 1: after the construction of a 3 m height embankment, which was used as a working platform, 3) stage 2: immediately after column construction, 4) stage 3: two months after column construction and 5) stage 4: eight months after column construction.

A total of one hundred and twenty compaction piles were constructed by means of the two different construction methods. Half of them by vibroreplacement and the other half by ramming compaction. All the columns were constructed down to a depth of 16 m bearing on a firm silty clay. The nominal diameter varied between 0,55 m and 0,60 m. Actual diameters went up to as much as 1,20 m, within the very soft clay stratum.

During the construction of the compaction piles, most of the instrumentation became badly damaged or useless. Moreover, the inclinometer tubes were broken and filled with soil, indicating very large movements of the soil mass.

Site stratigraphy

Table 1, below, summarizes the major soil strata encountered at the test site, and some engineering properties.

TABLE 1. Summary of site stratigraphy

STRAUM	DEPTH (m) FROM	TO	SOIL DESCRIPTION	NSPT	SU (Kg/cm^2)	qc (Kg/cm^2)
1	0	4-4.40	Very silty yellowinsh brown to dark greenish gray loose fine sand coarse very sandy non plastic silt, with inclusions of soft silty clay layers, locally organic.	13	-	50
II	4-4.40	8.20-8.70	Very soft to soft dark gray low plasticity silty clay and clayey silt	1-2	0.22	4
III	8.20-8.70	9.10	Medium dense to dense yellowish to dark gray silty fine sand, which thins or pinchs out towards the south-wertern part of the area.	3-7	-	80
IV	9.10-10.0	13+	Soft to firm dark silty gray clay and clayey silt	2	0.24	5

Construction Methods

Vibroreplaccment Method

The vibro-replacement method is used to improve cohesive soils containing more than 18% passing # 200 U.S. standard sieve (Brown, 1976). The equipment used is the vibroflot which is sunk into the ground under its own weight, assisted by water or air jets as a flushing medium until it reaches the predetermined depth. The method can be used either with the wet or dry process. In the wet process, a hole is formed in the ground by jetting a vibroflot down to the desired depth with water. When the vibroflot is withdrawn, it leaves a borehole of greater diameter than the vibrator. The uncased hole is flushed out and filled in stages with 12 mm-75 mm size imported gravel. The densification is provided by an electrically or hydraulically actuated vibrator.

Casing driving and ramming method

This method uses the energy delivered by a falling hammer to drive an open-end hollow casing into the soil. The hole created by the lateral displacement of the soil, as the sleeve advances to the desired depth, begins to be filled with the backfill material when the final depth is reached and the casing is progressively retrieved. The literature consulted so far (Datye and Nagajaru, 1981. Ghazali and Khan, 1986) indicates that there are not serious restrictions for its aplication to any type of soil, although it is not totally certain for cohesive soils.

Results

Soil penetration resistance

Comparisons were planned to be based on SPT and CPT results. However, CPT testing was interrupted due to equipment problems. The only available comparable CPT data show that the vibroreplacement method increases penetration resistance of the granular strata even at 4 m c/c column separation. Penetration resistance in the clayey material does not seem to change. On the other hand, the results corresponding to the casing driving and ramming method indicate that even columns spaced 4 m c/c increased the penetration resistance of both cohesive and granular materials. The relative increment in the granular soil was in the range from 100 to 250% and the cohesive soils improved their penetration resistance in a range from 25% to 50%.

The data obtained from Nspt values, comparing blow count previous to soil treatment with those obtained approximately two months after completing column construction, indicate a considerable increase in penetration resistance in the area treated with the driving and ramming method whereas the results show a moderate improvement in Nspt values in the area where the vibroreplacement method was used, for the same 2 m c/c column spacing.

Shear strength from field vane tests

The data obtained from in-situ vane tests show that both construction methods increase soil shear strength. Registered values for both methods are very similar for either peak or residual strengths. However, the scatter in the vibroreplacement area is much larger.

It is also worth mentioning that, either peak or residual strength values after treatment tend to be lower around 6 m to 7 m and near 10.5 m to 11.5 m depths for the two construction techniques. Strength values in the vibroreplacement area are lower than those in the casing driving and ramming area. This may be a consequence of the level of consolidation reached by the clay at the time of performing the field vane tests. In fact, at these two depths ranges, layers of silt with organic content are found, which are separated by a silty sand layer. Shear vane results in the vibroreplacement area indicate that strength values are larger in the zones adjacent to this sand stratum for both organic silt layers.

The improvement achieved in the cohesive soils treated with both methods was in a order from 100% to 400%.

LAGUNILLAS PILOT TEST

The Lagunillas field test involved: (a) determination of the original site soil conditions, (b) installation of the instrumentation directed to monitor pore water pressures, ground acceleration and ground deformation, (c) construction of twenty stone columns by the casing driving and hammering technique, and (d) determination of the "after columns construction" soil conditions.

At the Lagunillas field test both, the casing driving and ramming method of construction and the equilateral triangular array, of 3m spacing c/c, were selected as they resulted the most efficient in the Bachaquero field test. The columns were constructed down to a depth of 13.5m, bearing on a firm high plasticity clay. The field work was programmed to be performed in three phases, as follows:

1) Determination of the "before" soil conditions and installation of the instrumentation,
2) Construction of compaction piles and monitoring of the construction process and
3) Determination of the "after" soil conditions.

Site Stratigraphy

The site stratigraphy consists of alternate layers of granular, organic and cohesive soils. Minor textural and color variations and inclusions of other soil types within each generalized stratum are present.

Major strata of the site stratigraphy are summarized in Table 2 and the properties of the strata in Table 3.

TABLE.2 Summary of site stratigraphy

Stratum	Depth (m) From	Depth (m) To	Soil Description
I	3.35	6.50	Medium dense, very sandy plastic silt (MNP)
II	6.50	7.95	Medium compact, low plasticity silt (ML)
III	7.95	12.15	Very soft to soft, low plasticity silty clay (CL)
IV	12.15	14.40	Stiff, high plasticity clay (CH)

TABLE.3 Summary of the average soil properties of the site.

Stratum	Passing # 200	Gs	Nspt	qc (MN/m2)	qc/Nspt	LL	Ip	Su (t/m2)
I	65%	2.65	17	11	0.75	-	-	-
II	87%	2.68	6	4	0.67	28	6	-
III	98%	2.77	1	0.5	0.5	55	28	2.70
IV	84%	2.81	15	4	0.27	24	8	4.18

Test program

Soil improvement due to column construction was evaluated through penetration tests (SPT and CPT), in-situ vane shear tests (VST) and laboratory tests conducted on undisturbed samples. All these in-situ tests have been conducted through the time in four stages as follows: inmediately after columns construction (3-7 days), one month later, three months later and eleven months later. The in-situ tests program is shown in figure 3 and table 4. Results from laboratory tests are not present in this paper.

Geotechnical instrumentation

The pore pressure behaviour before, during and after the columns contruction were monitored by electrical and vibrating wire piezometers at different depths.

The surface movements were monitored through bench marks. The location of the instrumentation is shown in Fig. 3 and detail given in Table 4.

Backfill material

Crushed stone was used to construct compaction columns at the Lagunillas pilot section. Results of sieve analyses on this material are summarized below:

Material Type	D10	D20	D50	Brown Suitability Number
Crushed stone	13	15	22	0.22

The suitability number was defined by Brown (1976) on the basis of case histories to evaluate the appropriateness of the backfill material to be used. Table 5 indicates the criteria for evaluating backfill material based on grain size distribution.

TABLE.5 Backfill evaluation criteria

SUITABILITY NUMBER	0-10	10-20	20-30	30-50	50
RATING	EXCELLENT	GOOD	FAIR	POOR	UNSUITABLE

Results

Backfill Consumption

The average backfill material consumption per stratum is shown below. A volume reduction of 0.8 has been considered, to take into account the effect of the hammering. The average diameter in the granular strata varies between 0.70 to 0.80m, and between 0.80 to 1.10m in the cohesive strata.

Average consumption (m3)/m

STRATUM I	STRATUM II	STRATUM III	STRATUM IV
0.32	0.45	0.59	0.50

Pore pressure build up

The construction sequence has an effect on the generation and dissipation of pore water pressures. The relative increments with respect to the pressure before constructing the column have an irregular pattern, which is believed to be a consequence of partial dissipation and redistribution of pore pressures due to the construction sequence. The construction sequence is showns in Fig. 4.

Figs. 5 through 7 illustrate the effect of the construction sequence on the behaviour of the pore pressure, normalized by the hydrostatic pressure, as well the effective stress normalized by the hydrostatic pressure. The initial value of pore pressure in each figure corresponds to that before starting column construction.

SYMBOLS DESCRIPTION	P0	P1 Day 3-7	P2 Day 30	P3 Day 90	P4 Day 320
BORING AND WASHING FOR SHELBIES SAMPLES	2	2	-	-	2
C.P.T.	5	5	-	2	2
S.P.T. / V.S.T.	1 / 3	2 / 9	1 / 3	2 / 6	2 / 6
PERMEABILITY	2	2			
CROSS HOLE	1				
OPEN PIPE PIEZOMETERS	8				
DYNAMICS PIEZOMETERS	3				
ELECTRIC PIEZOMETERS	4				
ACELEROMETERS	10				
TILTMETERS	4				
HORIZONTAL-VERTICAL CONTROL DISPLACEMENTS	22				
HIDRAULIC FRACTURE TEST					3

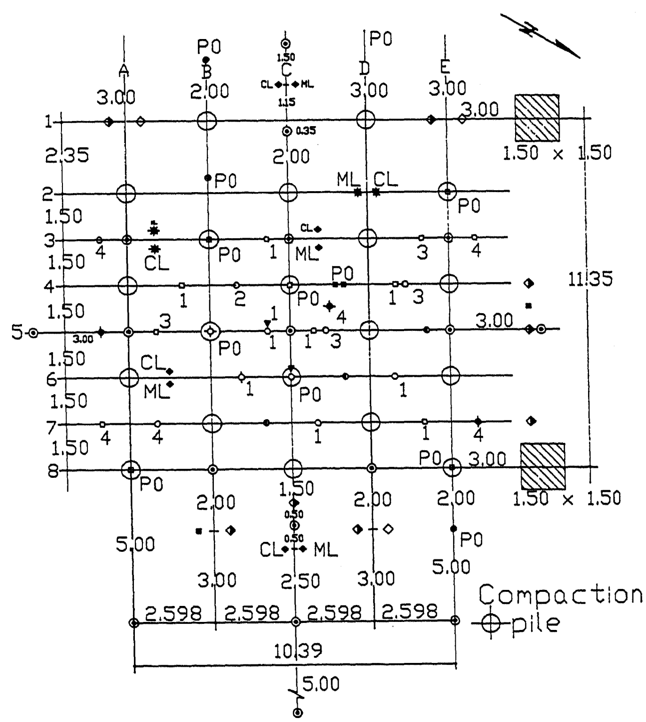


Fig. 3 LAGUNILLAS PILOT TEST LAYOUT

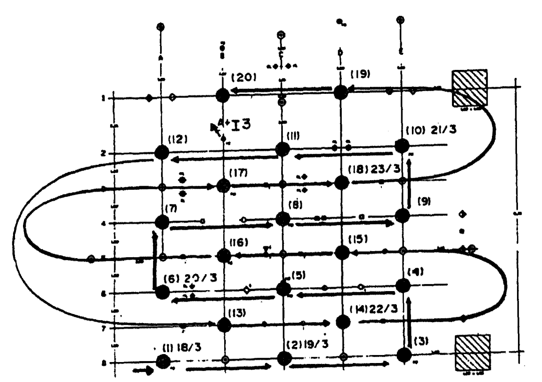


Fig. 4 COMPACTION COLUMNS CONSTRUCTION SEQUENCE

Fig. 5 shows the increase in Stratum III of the absolute pore pressure due to the construction of columns 1 to 6.

After construction of column 7, the pore water pressure gradually dissipated as the distance from the column to the piezometer increased. During the construction of the inner columns pore water pressure increased as the distance to the piezometer decreased. The curve of the absolute pore pressure normalized by the hydrostatic pressure shows the increments of the pore pressure.

Figures 6 and 7 illustrate the effect of the construction sequence, moving away and approaching to the electric piezometers, located in the first stratum.

Soil penetration resistance

Fig. 8 illustrates the corrected average values of Nspt for stages zero and 4. Fig 9 presents the trend of the cone resistance (qc) for different stages. These figures indicate a considerable increase in penetration resistance. This increment ranges in the first stratum between 25% and 300%, with the largest values toward the bottom of the layer. The increment in the stratum II ranges between 30% and 300% with the largest values toward the upper portion of the layer. Only slight increments in penetration resistance were obtained in cohesive soils. Nonetheless, penetration resistance tests are not sensitive to evaluate strength changes in cohesive soils.

Shear strength from field vane tests in cohesive soils

During the Lagunillas study, a series of in situ vane tests were conducted to monitor the variation in Su with time after instalation. In order to interpret the results, it was necessary to examine general trends rather than specific details since the soils at the test section vary considerably across the area as the laboratory and CPT tests reported.

The increase of peak shear strength values range between 30% and 60%. The residual shear strength values presents an increment between 40% and 100%. Fig. 10 shows the trends of the field vane tests results.

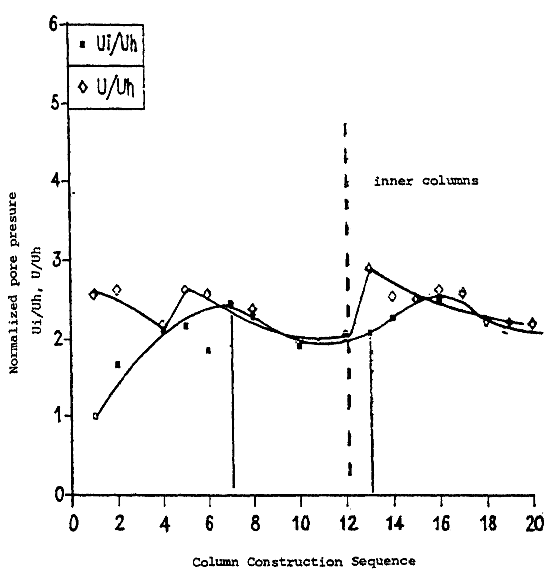


Fig. 5. PORE PRESSURE VARIATION IN LAYER 3 (CL) DURING COLUMN CONSTRUCTION

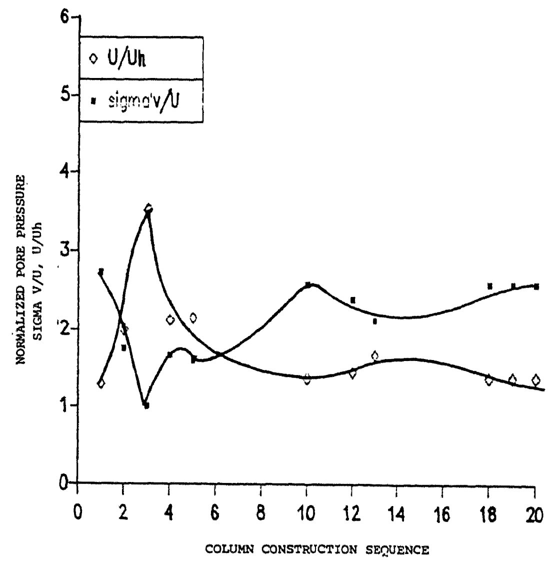


Fig. 6. PORE PRESSURE VARIATION IN LAYER 1(MNP) DURING COLUMN CONSTRUCTION

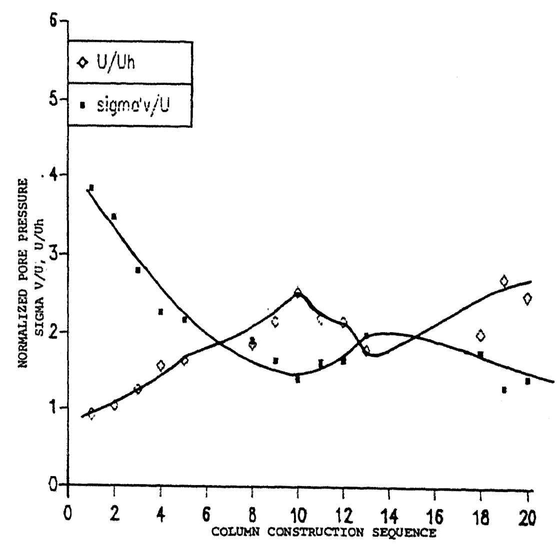


Fig. 7. PORE PRESSURE VARIATION IN LAYER 1(MNP) DURING COLUMN CONSTRUCTION

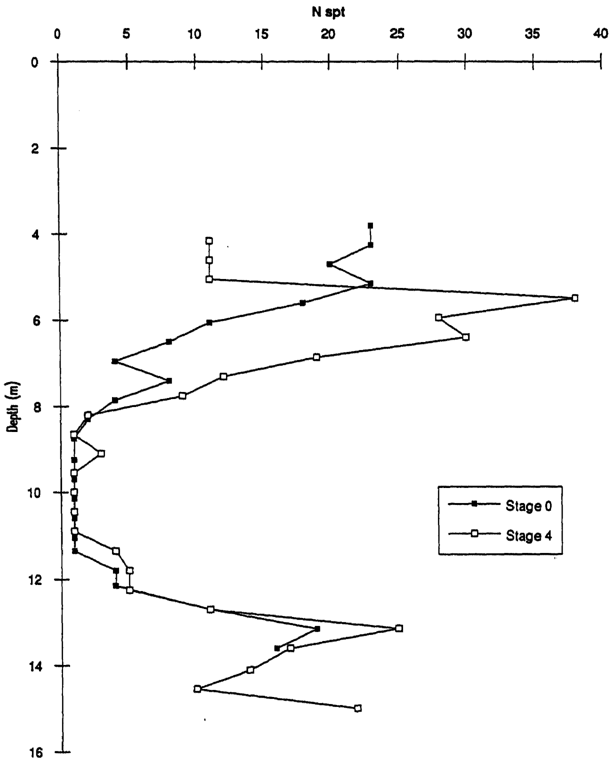


Fig. 8 .Before and After of installation of stone columns

campaigns until construction stopped, with à total displacement of 7.6 cm. A height decrease was monitored later on with a total displacement of 15.6 cm see Fig. 13.

It was, therefore, shown that eight months after the construction, berm vertical displacement had not stopped.

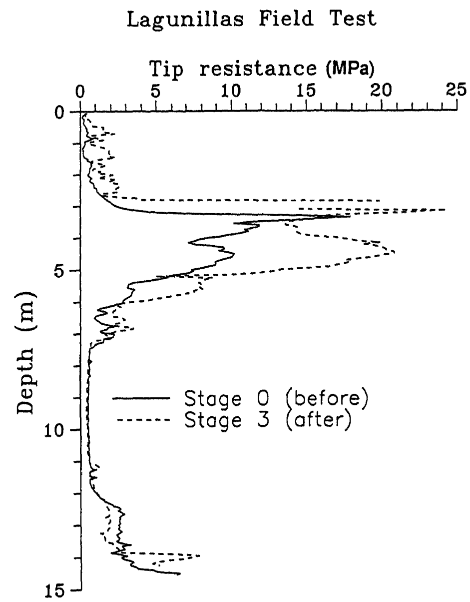


Fig. 9 Cone profiles before and after improvement

Monitoring of ground deformation

Three control points were placed away from the test site in order to guarantee their stability, based on previous results from the Bachaquero test. Twenty two object points conformed the survey network, conveniently spread within the field test in such a way that they could provide a complete picture of the ground deformation (vertical and horizontal) due to pile construction see Fig. 11.

Horizontal displacements

Field data were reduced to compute adjusted coordinates of object and control points. This adjustment provided displacement vectors of each point and its associated error ellipse at 95% confidence level. Significant vectors of these points are shown in Fig. 12. The results shown by these vectors indicate the trend of the body displacement due to pile construction. The maximum horizontal displacement measured was of the order of 7.0 cm. As expected, some points returned to their initial position once the piles were constructed, thus confirming the elastic component of ground deformation.

Vertical displacements

A total of 17 campaigns of high precision first order levelling were performed using two bench marks as controls located nearby the test section. The reduced height differences from the field measurements were introduced in a graphic computer file for continuous evaluation. An increment of height was obtained in the first five

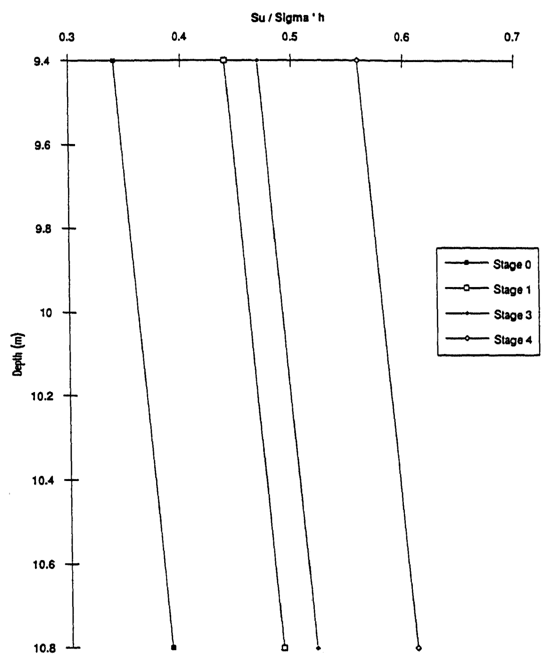


Fig. 10 . STRATUM III - UNDRAINED SHEAR STRENGHT TREND

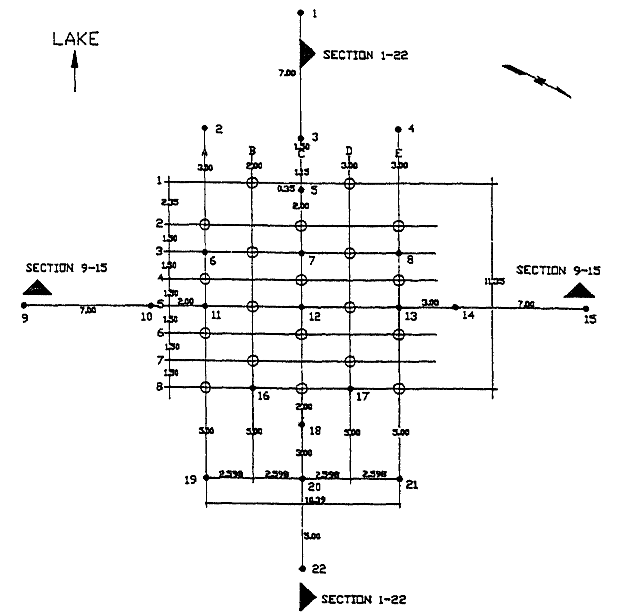
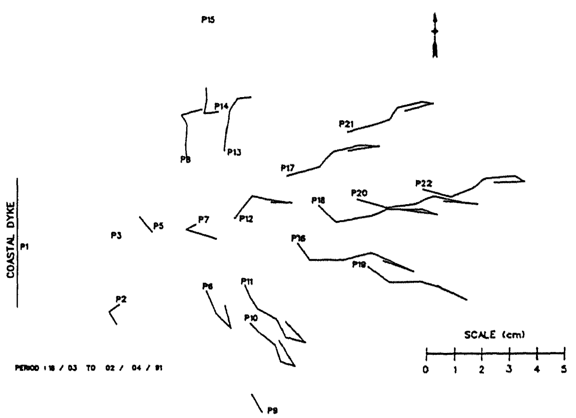


Fig. 11 OBJECT POINTS LOCATION

Fig. 12 HORIZONTAL DISPLACEMENTS OF VECTORS



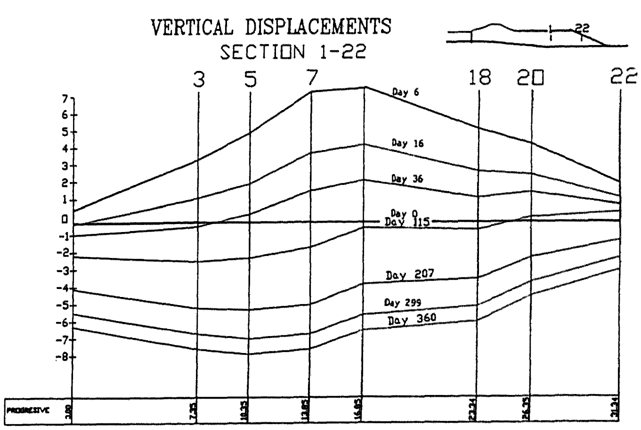


Fig. 13 VERTICAL DISPLACEMENTS

CONCLUSIONS

- In the Bachaquero field test the casing driving and ramming tecnique showed the most effectiveness.
- The geotechnical instrumentation design of Lagunillas field test was based on experience obtained from Bachaquero field test. The entire design was conceived in order to monitor the pore water pressure build up and the surface movements.
- Results obtained from penetration tests and in-situ vane shear tests carried out in Lagunillas field test, show that the casing driving and ramming technique improves the engineering properties of both granular and high plasticity soils.
- The construction sequence appears to have an important effect on the pore water pressure generation. The highest excess pore pressure was registered in stratum III, during the casing driving operation.
- No increment of pore pressure due to stone column construction was registered 10-12 meters away from the piezometers.
- Vertical displacements, as expected, showed heave during pile construction. When construction stopped, ground level started to settle down to values even lower than the initial ones. This indicates soil compaction after the appropiate time period.
- Horizontal displacements showed the pattern followed by the berm, when subjected to strong compaction of the soil. This means, the berm deformed landwards.

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