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CPT/DMT QC OF GROUND MODIFICATION AT A POWER PLANT

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ABSTRACT

The use of 908 static cone penetration test (CPT) and 33 Marchetti dilatometer test (DMT) soundings provided the quality control (QC) needed to improve a power plant site by the use of dynamic compaction and compaction grouting. The engineers used an equivalant CPT-based relative density acceptance criterion, which they modified during the work to include a DMT-based modulus criterion. They achieved the objective of reducing differential settlements to permit the successful use of shallow foundations.

1. Introduction

Extensive insitu testing provided a key element in the successful ground modification effort described in this case history. The Jacksonville Electric Authority (JEA) and the Florida Power and Light (FP&L) jointly own and have under construction two 600 MW coal fired units at their St. Johns River Power Park site 15 km (10 mi) NE of downtown Jacksonville, Florida, and 12 km (8 mi) inland from the Florida coastline. The heavy loadings in the power block area of this plant required the use of either pile foundations or some form of ground improvement in combination with shallow foundations. The design engineers, EBASCO, elected to use ground modification because of a potential cost savings of approximately $6,000,000. JEA-FP&L then contracted with the Hayward Baker Company (now GKN Hayward Baker, Inc.) to perform a combination of dynamic compaction and compaction grouting at the 21 acre site. The production work over the entire site followed an intensive investigation in a one acre test section. Table 1 presents a chronological summary of the major events concerning this ground modification work.

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1 Principal, Schmertmann & Crapps, Inc., Gainesville, FL
2 Consulting Engineer, Odenton, MD
3 Geotechnical Engineer, GKN-Baker, Odenton, MD
4 Principal Geotechnical Engineer, EBASCO Services Inc., Norcross, GA

The Engineers set the standards for acceptable soil after ground improvement. The Contractor had the responsibility for establishing and implementing a quality control (QC) program acceptable to the Engineers. The firm of Schmertmann & Crapps, Inc. acted as consultants to the Contractor to assist with the design and implementation of their QC operations. The QC work consisted of a mixture of electric Dutch cone penetration test (CPT) and Marchetti flat dilatometer (DMT) soundings, performed from a vehicle especially designed for the efficient performance of such soundings. This vehicle and its operators performed the enormous amount of QC testing that played a major role in the timely and successful documentation and completion of the ground improvement at this site.

TABLE 1 - CHRONOLOGY OF GROUND MODIFICATION AND QC EVENTS

1. Site Densification Contract	Dec 22, 1982
2. Start Site Work	Dec 23,
3. Start of Pretreatment Testing	Mar 15, 1983
4. Start of Dewtatering	Mar 18,
1 Acre Test Area	
5. Start of Dynamic Compaction	April 6,
6. Start of During & After Treatment Testing	April 6,
7. Start of Compaction Grouting.	April 21,
8. Completion of Dynamic Compaction in Test Area . .	May 26,
9. Completion of Compaction Grouting in Test Area. .	June 14,
10. Completion of QC Testing in Test Area	June 22,
Remaining 20 Acres	
12. Start of Production QC Testing, Before Treatment.	April 6,
13. Start of Production Ground Modification Work. . .	April 20,
14. Completion of Ground Modification Work.	Feb 3, 1984
15. Completion of QC Testing.	Feb 3, 1984

The reader can find several references dealing with aspects of this ground modification work other than the insitu testing QC emphasized in this paper. Kessler (1985) and Kessler and Kuretski (1985) emphasized the engineering decisions and the contractural arrangements involved in designing and completing the subject ground modification. Also, an advertisement in Engineering News Record (1984) presented a digest of the Contractor's work at this JEA-FP&L site.

2. The Site Soils

The plant area consisted of a low-lying, naturally filled-in, former marine estuary. The Florida coastline has probably advanced and retreated many times over the site. Figure 1 presents the average soil conditions as determined from about 50 preliminary standard penetration tests (SPT) borings.

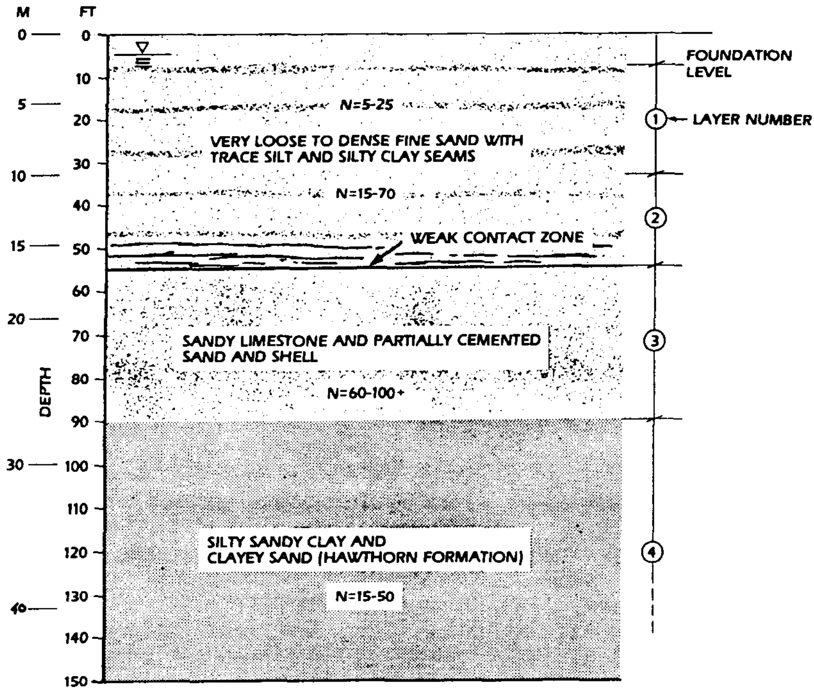


FIGURE 1 - GENERALIZED SUBSURFACE PROFILE

As expected in a former marine estuarine environment, the soil conditions over this 21 acre site are quite variable, but have a definite overall layer sequence. Starting from the surface, Layer 1 consists of uncemented, recent Pleistocene, relatively clean quartz sand, loose in many areas, and extending to a depth of about 10 m. (33 ft). This layer has local seams or pockets of dark colored, slightly organic, finer grained soil, varying from very silty sands to mixtures of silt and clay.

Layer 2 underlies Layer 1. It is more variable and finely interlayered but still an uncemented quartz sand, and it contains more silt and clay than Layer 1. Particularly important is its contact zone with the underlying Layer 3, which occurs at a typical depth of 17 m (55 ft). This contact zone had many very weak pockets which probably consisted of voids or loose material filling former voids. Layer 2 is believed to be of early Pleistocene age and the loose pockets probably represent the result of the solution of shells in Layers 2 and 3. The drainage at the site is primarily vertical into Layer 3.

Layer 3 consists of a very competent, approximately 11 m (36 ft) thickness of cemented and partially cemented sands and shells forming a young limestone of upper Miocene age. Layer 4 below consists of a thick, highly overconsolidated and strong, primarily cohesive deposit of Miocene age, known as the Hawthorn Formation.

The Engineers wanted to modify and improve the loose areas in the Layer 1 sand, and the loose/weak zones in Layer 2, especially within the contact zone with Layer 3 at a depth of about 17 m (55 ft). Except for the contact zone between Layers 2 and 3, Layers 3 and 4 provided excellent support for the power block loads and required no treatment. The pile alternate would have involved driving into Layer 3.

Figure 2 shows a plan of half of the 21 acre site with some of the major structural loadings, the grid coordinate system and the location of the QC test area described subsequently.

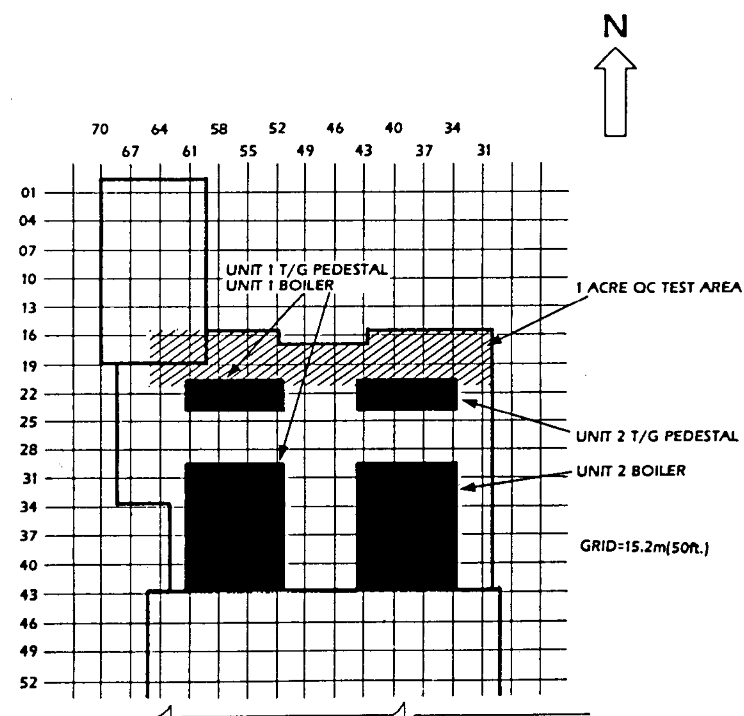


FIGURE 2 - PLAN OF THE NORTH HALF OF THE POWER BLOCK GROUND IMPROVEMENT AREA, INCLUDING THE TEST AREA AND TYPICAL STRUCTURES WITH SETTLEMENT MEASUREMENTS (Fig. 9)

3. Gradient The Ground Modification Scheme

After suitable demonstration by the Contractor, the Engineers decided to improve Layer 1 by means of dropped-weight dynamic compaction (DC). However, they did not expect improvements to extend significantly into Layer 2 because of its depth and because the layer contains many silty and clayey zones. They expected little or no improvement in the very weak pockets at the contact with Layer 3. The Engineers therefore elected to improve Layer 2, and its contact with Layer 3, by use of the compaction grouting (CG) pressure injection method suggested and test-demonstrated by the Contractor before the bidding.

The Contractor used 300-320 KN weights (33-36 tons) for the dynamic compaction, dropped 30 m (105 ft) by an especially modified crane, from 2 to 7 times per print location, using a square print pattern with primary and secondary (where necessary) print locations. The primary grid had a 10 m (33 ft) spacing between prints. The secondary prints, located in the center of each primary square, had a spacing of 7.1 m (23.3 ft). The photograph in Figure 3a shows the dynamic compaction in progress.

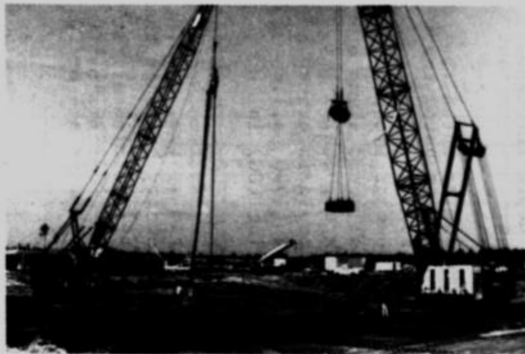
As an aid to the DC work, and also to permit the easy removal of surface or near surface fine grain soil pockets and seams, the Contractor lowered the initial ground water table from approximately the 1 m (3 ft) to the 3 m (10 ft) depth throughout the duration of the modification work.

The Contractor used a highly automated system to batch the CG grout and deliver it to the vertical grout pipes. The grout consisted of an automatically proportioned mixture of imported and site sands, fly ash, cement and water, with a delivered slump of approximately 75 m (3 in). A hydraulically powered, dual cylinder, variable speed compaction grout pump could inject up to 45 m^3 (60 yd^3) per hour at pressures up to 7 MPa (1000 psi). Figure 3b shows a photo of the compaction grouting (CG) in progress at the site.

The CG also involved a primary-secondary grid system, with primary holes spaced at 7.1 m (23.3 ft) and secondary holes at the centers of the primary grid with a 5.1 m (16.7 ft) spacing. In almost all cases the QC insitu sounding tests were located at the approximate 1/4 to 1/2 distances between adjacent DC prints or CG holes.

4. Test Methods for Quality Control

The Engineers set the requirements for the results of soil modification: Initially in terms of relative density, D_r, and material displacement and subsequently modified them to include



(a) DYNAMIC COMPACTION WORK IN PROGRESS

(b) CG EQUIPMENT IN OPERATION

(c) CPT-DMT TRUCK WORKING AT THE SITE

(d) VIEW INSIDE TRUCK SHOWING THE DATA ACQUISITION EQUIPMENT

FIGURE 3 - PHOTOS OF THE GROUND IMPROVEMENT AND QUALITY CONTROL WORK IN PROGRESS

the vertical, one-dimensional, compression modulus, M. Table 2 summarizes these requirements.

The Engineers used the CPT as the job method for checking the adequacy of the ground modification work. They selected the curves and equations presented in Schmertmann (1978) to correlate between D_r and the electric Dutch cone test (CPT) bearing capacity, q_c. The D_r = 65% and D_r = 85% lines shown in Figs 4 and 6 came from the equations in Schmertmann (1978). They apply only to clean, uniform, normally consolidated, unaged sands. Thus, in any real sand insitu they represent only an "equivalent D_r" in terms of q_c and the aforementioned sands. The later use of the DMT-determined modulus M incorporated the correlation for vertical M presented in Marchetti (1980).

The Contractor had responsibility for the quality control (QC) testing and documentation and realized immediately the need for a large volume of timely q_c data during both the test area and production phases of the work. Figure 3 presents photos of various work in progress at the site. The use of the aforementioned sounding vehicle (also see Fig. 3c) and its onboard equipment produced a real time strip chart record of an ongoing CPT sounding with simultaneous digital recording, at 50 mm depth intervals, of q_c, local sleeve friction f_s, and inclinometer readings to warn of excessive tilting of the tip. Immediately after each sounding, these digital data were processed to produce figures of the type shown in Fig. 4. This rapid reduction and plotting of data proved very valuable in expediting QC work, and provided a standardized form for the CPT testing records. This single vehicle, with a crew of two, typically performed about 10 CPT soundings to 17 m (56 feet) depth per 8 hour shift, with the results immediately available to the Engineers in final graphical format. The digital tape recording from each sounding was also immediately available for computer processing in the field to check each for conformance to the q_c-D_r-M criteria listed in Table 2. The field crew knew the next day whether a location had passed or required more work.

The Contractor chose to use the Marchetti flat dilatometer test (DMT) as a supplement to the required CPT work. See Marchetti (1980) for a description of the DMT. The DMT has excellent compatibility with the CPT truck equipment and provided supplementary data to help interpret, if necessary, the CPT results. This proved fortuitous. The DMT became of great value, as described subsequently, because it permitted the Engineers to use a rational, alternate acceptance criterion under circumstances when it became very difficult to meet the relative density criterion using CPT results and/or the material volume displacement criterion.

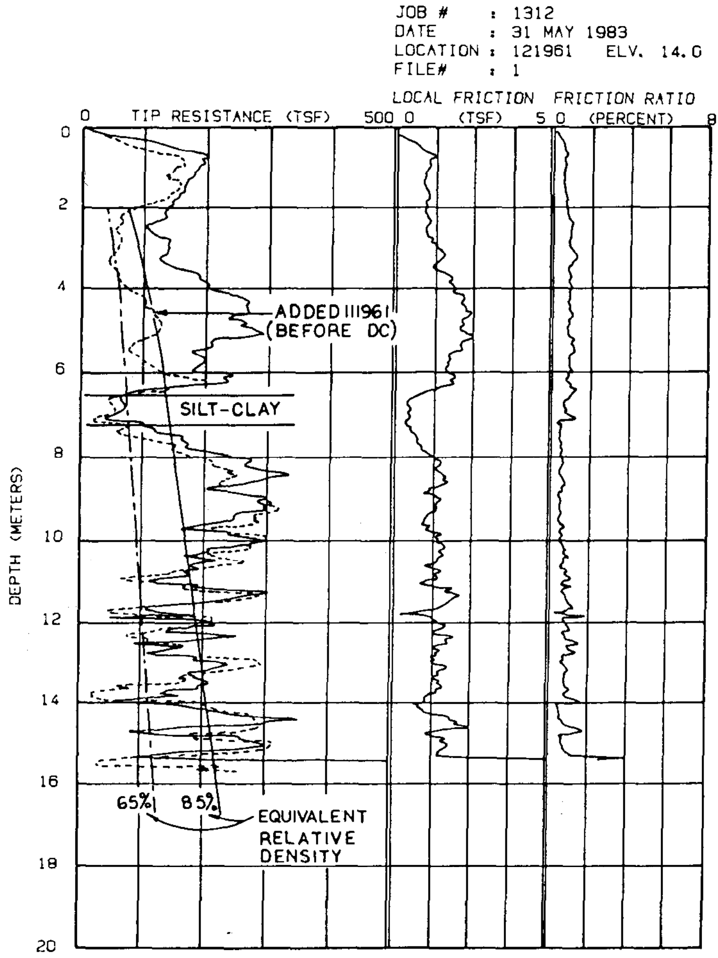


FIGURE 4 - EXAMPLE OF CPT LOG PRODUCED DIRECTLY IN THE FIELD, ANNOTATED

The Contractor performed the CPT and DMT testing during all phases of the work -- before any ground modification, during the modification work, and after its completion. The testing effort was particularly concentrated in the test area, where 99 CPT soundings, 9 CUPT (piezocone) soundings and 23 DMT soundings were performed. The Engineers also added 4 SPT borings in the test area. During the subsequent production work the Contractor performed another 800 CPT and 10 DMT soundings, but no more CUPT soundings. Sounding depths averaged approximately 17 m (56 ft).

TABLE 2 - SUMMARY OF COMPACTION ACCEPTANCE REQUIREMENTS

Initial:

1. Layers 1 and 2 are to be densified.

2. The correlation in Schmertmann (1978) shall be used to determine an equivalent relative density, D_r, from electric CPT soundings made in conformance with ASTM D-3441. Clearly defined silty-clay layers are excluded from the averages in the D_r criteria that follow.

3. The average D_r at each test location shall be at least 85% over the 2-7 m depth interval.

4. The average D_r over any 3 m depth interval below 7 m shall be at least 75%.

5. No D_r average over any 1 m depth interval below 2 m shall be less than 65%.

6. The material volume displacement ratio of the compaction grout shall be at least 15 to 25% depending upon, and averaged over, the depth interval of the grout zone and the prorata plan area of that grout injection column.

Modifications After the Test Area:

7. Below the 10 m depth, and with both primary and secondary CG points, an average M equal or greater than 100 MPa (1020 tsf) shall be acceptable in lieu of the relative density requirement. With CG only at primary points, the minimum average M shall be 120 MPa (1230 tsf).

8. The 1-D modulus M shall be determined from the DMT, and by site specific q_c-M correlations when using the CPT. (Ref. Fig. 8)

9. A time improvement factor may be applied to q_c if needed to pass. (Ref. Table 3)

10. The minimum volume injection ratio criteria was replaced by minimum and maximum grout injection pressure criteria, a maximum volume per injection point, and a maximum allowable ground heave of 30 mm (0.1 ft) mid-between grout injection points.

5. Some Results From the QC Testing

5.1 Typical results from ground improvement methods: Figure 4 shows a typical CPT output page obtained by the computing and plotting equipment immediately after each sounding. In this case the figure also includes, superposed, an additional q_c log for the before-improvement condition to illustrate the improvement obtained as a result of DC work in this part of the test area.

Figure 5 shows a typical DMT log, in this case near the location of the subsequent Figure 6 CPTs. Part (a) gives the DMT results in tabular form, and part (b) shows a few selected parameters (OCR, ϕ and M) in graphical form. In this part of the test area the modification work included CG followed by DC. The Engineers typically obtained these DMT results in the field office the same afternoon, or the day after each DMT sounding.

HAYWARD BAKER CO. TEST NO. 32-1934
FILE NAME: JEA GROUND IMPROVEMENT, POST COMPACTION
FILE NUMBER: 82-514

RECORD OF DILATOMETER TEST NO. 32-1934
USING DATA REDUCTION PROCEDURES IN MARCHETTI (ASCE,J-GED,MARCH 80)
KO IN SANDS DETERMINED USING SCHMERTMANN METHOD (1983)
PHI ANGLE CALCULATION BASED ON DURGUNOGLU AND MITCHELL (ASCE,RALEIGH CONF,JUNE 75)
PHI ANGLE NORMALIZED TO 2.72 BARS USING BALIGH'S EXPRESSION (ASCE,J-GED,NOV 76)
MODIFIED MAYNE AND KULHAWY FORMULA USED FOR OCR IN SANDS (ASCE,J-GED,JUNE 82)

LOCATION: 5 FT. SOUTH OF 11-1934, BLADE FACING SOUTH, PUSH USING GEOPROBE CPT TRUCK
PERFORMED - DATE: 20 MAY 83
BY: GEOPROBE

CALIBRATION INFORMATION:
DELTA A = .12 BARS DELTA B = .37 BARS GAGE 0 = .01 BARS GWT DEPTH= 3.05 M
ROD DIA.= 3.70 CM FR.RED.DIA.= 4.80 CM ROD WT.= 6.50 KG/M DELTA/PHI= .50 BLADE T=13.70 MM

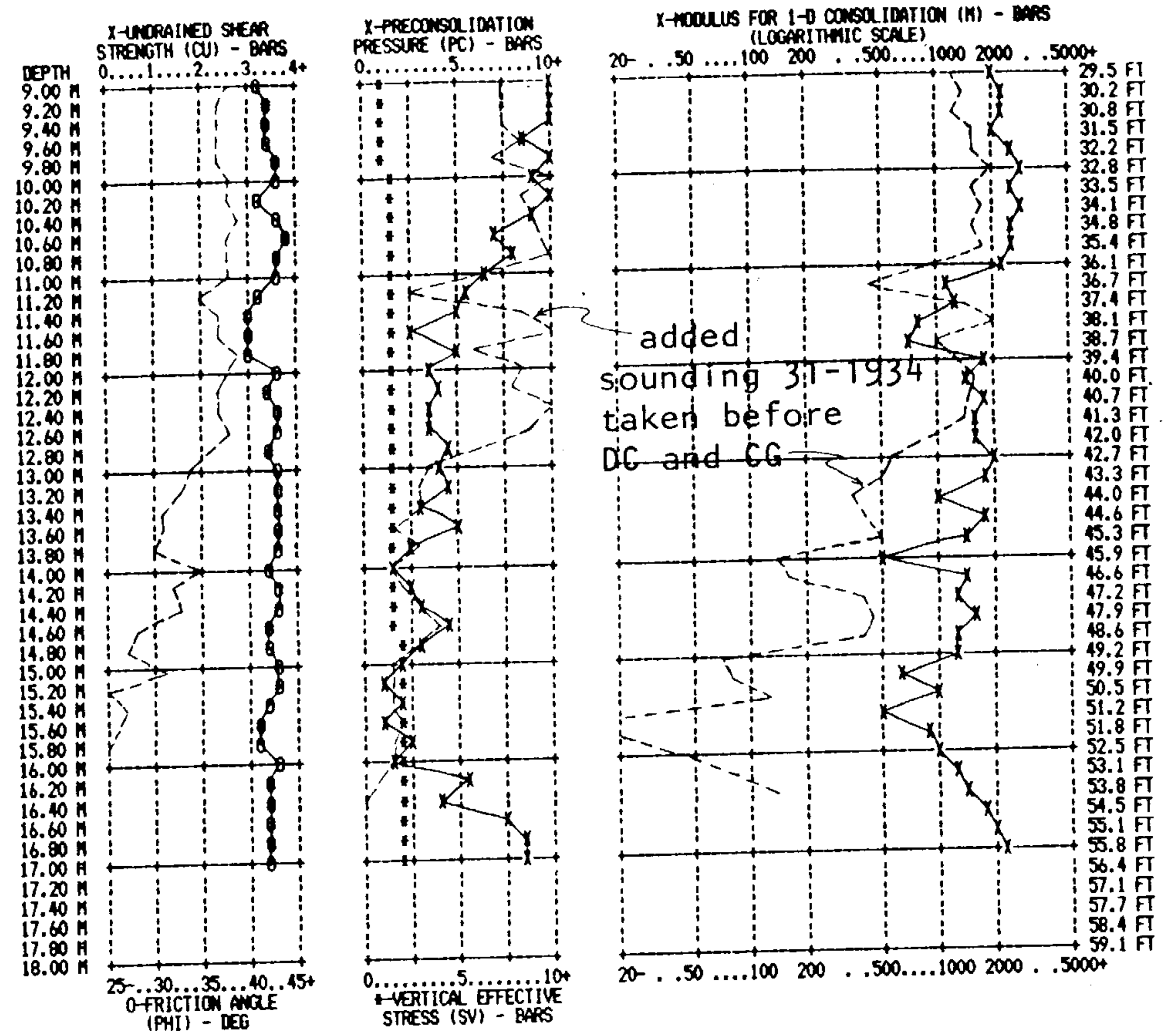
1 BAR = 1.019 KG/CM2 = 1.044 TSF = 14.51 PSI ANALYSIS USES H20 UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	KD	UO (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	KO	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
*****	******	*****	*****	*****	*****	*****	******	******	******	*****	*****	*****	*****	*****	******	***********
9.00	6519.	12.40	36.30	853.	2.29	9.34	.584	2.150	1.151	11.31	9.83	1.18		41.5	2079.4	SILTY SAND
9.20	7028.	12.80	39.50	955.	2.50	9.37	.604	2.150	1.174	11.35	9.67	1.16		41.9	2331.4	SILTY SAND
9.40	6901.	13.00	38.60	915.	2.35	9.39	.623	2.150	1.196	11.79	9.86	1.18		41.6	2234.7	SILTY SAND
9.60	6646.	11.50	36.30	886.	2.62	8.00	.643	2.150	1.219	8.71	7.15	1.00		41.9	2037.9	SILTY SAND
	NEW DA =	.14		NEW DB =	.51											
9.80	8554.	13.00	43.40	1084.	2.84	8.85	.662	2.150	1.241	9.92	7.99	1.04		43.2	2593.0	SILTY SAND
10.00	8681.	13.00	48.20	1259.	3.38	8.48	.682	2.150	1.264	9.19	7.27	.99		43.3	2967.1	SAND
10.20	7664.	15.40	42.30	956.	2.04	10.51	.702	2.150	1.286	15.95	12.40	1.32		41.5	2435.8	SILTY SAND
10.40	8681.	12.90	45.90	1179.	3.18	8.17	.721	2.150	1.309	8.89	6.79	.96		43.3	2739.4	SILTY SAND
10.60	8554.	12.00	45.30	1190.	3.51	7.33	.741	2.150	1.332	7.14	5.36	.85		43.5	2652.7	SAND
10.80	8300.	12.40	43.90	1124.	3.17	7.55	.761	2.150	1.354	8.02	5.92	.90		43.1	2536.0	SILTY SAND
11.00	7537.	11.10	40.60	1051.	3.36	6.54	.780	2.150	1.377	6.28	4.56	.79		42.9	2240.5	SAND
11.20	5120.	9.00	25.40	574.	2.19	5.40	.800	2.000	1.396	5.32	3.81	.74		40.6	1109.1	SILTY SAND
11.40	4865.	8.80	27.50	658.	2.63	5.09	.819	2.000	1.416	4.96	3.50	.72		40.4	1247.7	SILTY SAND
11.60	3975.	6.00	22.00	559.	3.56	3.15	.839	2.000	1.436	2.31	1.61	.49		40.1	837.2	SAND
11.80	4865.	8.50	19.80	388.	1.54	4.98	.859	1.950	1.454	4.94	3.40	.71		40.3	709.3	SANDY SILT
12.00	7537.	9.40	36.30	956.	3.76	4.97	.878	2.150	1.477	3.72	2.52	.58		43.4	1810.2	SAND
12.20	5883.	8.90	29.90	741.	3.00	4.75	.898	2.000	1.496	4.21	2.81	.63		41.7	1374.8	SILTY SAND
12.40	6455.	8.70	35.80	964.	4.21	4.35	.918	2.000	1.516	3.30	2.17	.55		42.5	1711.9	SAND
12.60	6646.	9.00	32.80	843.	3.46	4.58	.937	2.000	1.536	3.71	2.41	.58		42.5	1536.8	SAND
12.80	6773.	9.80	33.70	847.	3.13	5.01	.957	2.150	1.558	4.57	2.94	.64		42.3	1609.6	SILTY SAND
13.00	7791.	10.00	38.20	1004.	3.72	4.92	.976	2.150	1.581	4.02	2.54	.58		43.3	1890.9	SAND
13.20	7918.	10.40	35.30	884.	3.06	5.19	.996	2.150	1.603	4.62	2.88	.62		43.1	1705.5	SILTY SAND
13.40	7282.	8.60	25.40	588.	2.46	4.26	1.016	2.000	1.623	3.20	1.97	.52		43.1	1016.5	SILTY SAND
13.60	7537.	10.90	37.80	956.	3.17	5.28	1.035	2.150	1.646	5.21	3.17	.66		42.6	1859.9	SILTY SAND
13.80	6901.	8.20	32.50	862.	4.08	3.66	1.055	2.000	1.665	2.51	1.50	.45		42.9	1401.7	SAND
14.00	5883.	6.20	16.90	366.	2.22	2.82	1.075	2.000	1.685	1.73	1.03	.38		42.3	489.4	SILTY SAND
14.20	6519.	7.90	33.00	891.	4.49	3.35	1.094	2.000	1.704	2.30	1.35	.43		42.6	1381.3	SAND
14.40	6773.	8.60	31.20	800.	3.54	3.78	1.114	2.000	1.724	2.96	1.72	.49		42.5	1323.7	SAND
14.60	6901.	10.00	33.60	836.	3.07	4.49	1.133	2.150	1.747	4.35	2.49	.59		42.1	1509.5	SILTY SAND
14.80	6519.	8.40	31.00	800.	3.67	3.56	1.153	2.000	1.766	2.82	1.60	.47		42.3	1281.0	SAND
15.00	6773.	8.00	32.20	858.	4.28	3.24	1.173	2.000	1.786	2.25	1.26	.42		42.7	1304.3	SAND
15.20	6519.	5.90	21.00	526.	3.69	2.28	1.192	2.000	1.806	1.08	.60	.28		43.1	640.0	SAND
15.40	6137.	7.00	26.60	690.	4.00	2.72	1.212	2.000	1.825	1.83	1.00	.37		42.2	946.0	SAND
15.60	4611.	4.80	20.00	530.	5.14	1.61	1.232	1.900	1.843	1.02	.56	.29		41.0	485.0	SAND
15.80	5120.	7.20	25.80	654.	3.64	2.78	1.251	2.000	1.862	2.40	1.29	.44		40.7	908.2	SAND
16.00	7155.	7.20	29.20	778.	4.49	2.65	1.271	2.000	1.882	1.48	.79	.32		43.3	1048.0	SAND
16.20	7155.	11.20	31.80	727.	2.32	4.75	1.290	2.150	1.905	5.50	2.89	.64		41.7	1322.7	SILTY SAND
16.40	7028.	10.00	33.60	836.	3.14	3.98	1.310	2.150	1.927	3.95	2.05	.54		42.0	1421.5	SILTY SAND
16.60	7727.	13.30	39.80	942.	2.51	5.54	1.330	2.150	1.950	7.51	3.85	.74		41.6	1854.2	SILTY SAND
16.80	8045.	14.20	41.30	964.	2.38	5.91	1.349	2.150	1.972	8.55	4.33	.79		41.6	1948.7	SILTY SAND
17.00	8554.	14.70	44.50	1062.	2.55	6.02	1.369	2.150	1.995	8.74	4.38	.79		41.9	2170.7	SILTY SAND

END OF SOUNDING

FIGURE 5a - EXAMPLE OF LISTING OF RESULTS FROM A DMT SOUNDING

5.2 Screening effects of silt-clay layers: The continuous CPT logs and the near-continuous DMT sounding logs sometimes gave a clear picture of how certain soil layers affected or did not affect the DC work. For example, the comparative q_c logs in Figure 4 show that a silt-clay layer at about the 7 m (23 ft) depth greatly reduced the effectiveness of the DC effort below the layer. Figure 6 shows that a similar silt-clay layer at only 2.5 m (8 ft) depth did not prevent significant improvement below, although it may have interfered enough to prevent meeting criterion No. 3. in Table 2.



END OF SOUNDING

FIGURE 5b - GRAPHICAL LOG FOR SELECTED OUTPUT IN FIG. 5a, INCLUDES BEFORE-AFTER COMPARISONS

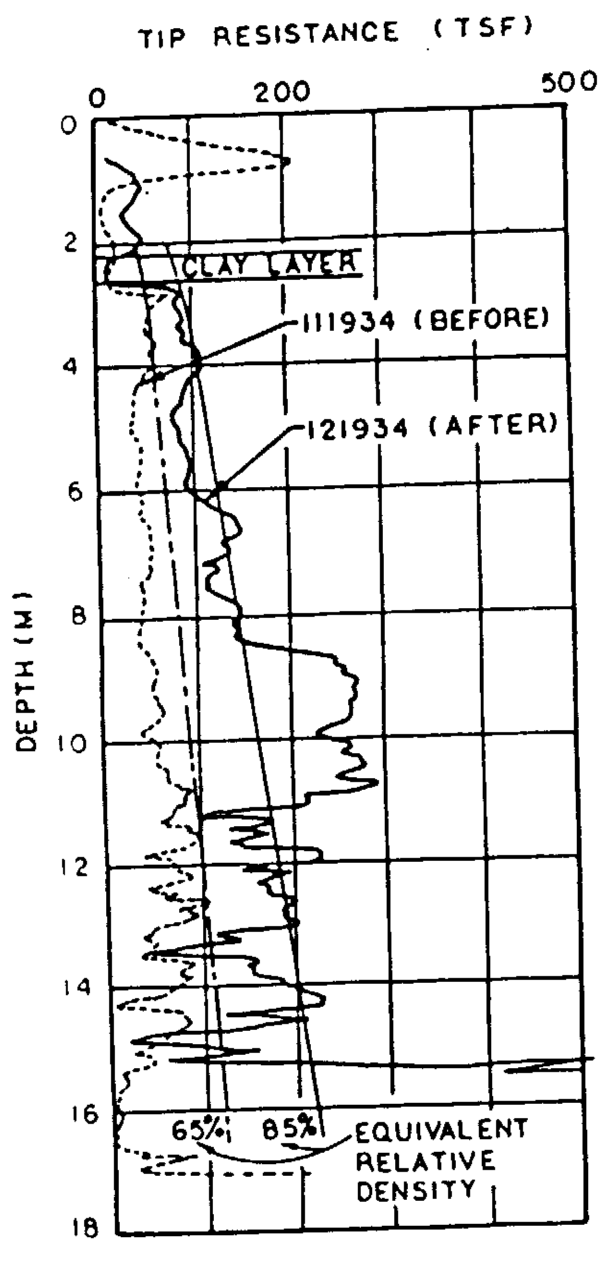


FIGURE 6

EXAMPLE OF COMPARATIVE BEFORE AND AFTER CPT LOGS WITH A NEAR-SURFACE CLAY LAYER

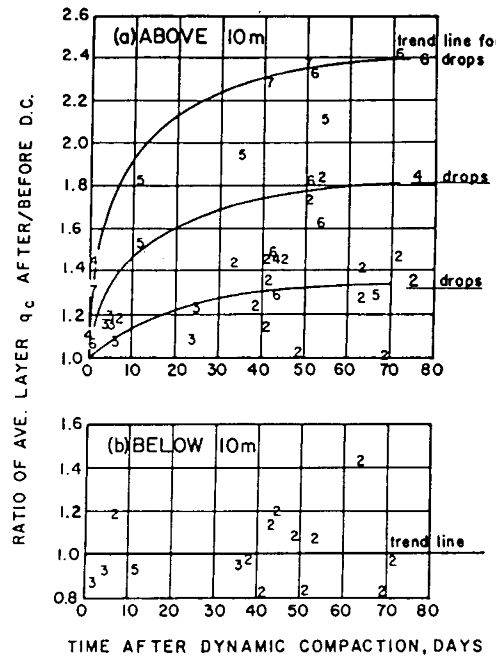


FIGURE 7 - EFFECTS OF TIME, NO. OF DC DROPS, AND DEPTH OF SOIL ON THE RELATIVE IMPROVEMENT IN LAYER AVE. q_c FROM CPTs (Nos. denote drops/data pt.)

5.3 Time-improvement effects: The large quantity of CPT sounding data also permitted the observation, documentation and use of another effect that has been observed by others, such as Solymar and Mitchell (1984), namely the improvement in q_c with time after ground improvement treatment such as dynamic compaction. Figure 7 summarizes this improvement as observed from the test section QC testing, with 7(a) showing an improvement above 10 m, and 7(b) showing the lack of improvement below 10 m. After obtaining these data the Engineers in special circumstances allowed a time improvement factor to be applied to q_c profiles obtained above the 10 m depth and within 60 days after the dynamic compaction work. Table 3 lists the improvement factor ratios that were allowed, based on Figure 7(a), but only when a sounding would otherwise have failed.

TABLE 3 - TIME-IMPROVEMENT FACTORS FOR q_c

Time between DC and CPT (days)	Factor by which to multiply q_c
5	1.35
10	1.20
15	1.15
20	1.12
30	1.06
40	1.03
50	1.01
60	1.00

5.4 A possible K_o barrier effect: The DMT soundings in the test area appear to indicate, at least in some parts of the site, that an upper limit might exist with respect to ground improvement achieveable by means of dynamic compaction. Table 4 presents a list of the 3 test sections in the test area where the writers had comparable before and after compaction K measurements averaged over the approximate 1 to 8 m depth interval. The results suggest that test section 2 showed the most improvement in M and q_c values because there occurred the greatest improvement in K (0.66 to 1.17). The least improvement occurred in test section 1, where the average DMT-determined K was initially high and improved very little (1.30 to 1.34). It appears that if an insitu condition of K = 1.4 or greater already existed in this part of the test area, then it would be difficult to achieve improvements in q_c with the magnitude of dynamic compaction effort used.

TABLE 4 - AVERAGE RESULTS FROM BEFORE AND AFTER DYNAMIC COMPACTION IN THE TEST AREA, USING 33 TON WEIGHT DROPPING 105′ (all tests in approx. center between DC prints 24′ apart)

In Test Section*	No. drops	depth interval	No. Tests	from DMTs K_o before	K_o after	M(bars = 100Kpa) before	M after	from electric CPTS q_c (bars) before	q_c after**
1	2	3-20′	26	1.30	1.34 (+3%)	1050	1680 (+60%)	83	n.a.
2	6	5-24′	30	0.66	1.17 (+77%)	680	2290 (+237%)	83	165 (+99%)
3	6	6-27′	32	0.98	1.19 (+21%)	1230	1590 (+54%)	121	150 (+24%)

* Tests 30 m (100 ft) apart between Sections 1 and 2.
Tests an additional 50 m (160 ft) apart between Sections 2 and 3.

** q_c values increased with time after the DC, as did M values. The q_c values given are for the time of the DMTs.

6. DMT Modulus and Settlement

One of the major changes made in the QC criteria involved a partial conversion from a q_c-to-D_r acceptance criterion to a q_c-to-M criterion after study of the test area results. It proved very difficult in some areas of the site to achieve the minimum q_c requirements after both dynamic compaction and compaction grouting work at both their primary and secondary points. The deficiency was most troublesome within the 10 to 17 m depth interval, particularly the lower portions around the contact zone with Layer 3. This probably occurred for a variety of reasons, such as the screening effects of various overlying soil layers, the grout pressure volume and pressure absorption effects of the cavity condition at the contact zone, unknown time effects associated with the compaction grouting in the variable Layer 2, etc. The situation was difficult with respect to how best to achieve QC assurance.

The various engineers involved discussed the great body of CPT and DMT sounding information available from the test area work and decided to apply a criterion based more directly on the design objectives of the ground improvement. The primary objective was to limit differential settlements between adjacent structures in the power block to a maximum of 6 mm (1/4"). Thus, the DMT M data relates more closely to the objective than the q_c-D_r criteria. It was noted that M values increased relatively much more than q_c values after the ground modification work. Table 4 includes data on this point, with an average (percent increase in M)/(percent increase in q_c) ratio of about 2.3 shown in these data. This led to a more systematic comparison of M and q_c from DMT and CPT soundings in the test area.

Figure 8 shows the correlation established in the test area between the DMT M and q_c, based on comparative points before and after ground modification treatment. At that time there appeared to be a small difference between Layer 1 (above 10 m) and Layer 2 (below 10 m). Figure 8 shows only the best-fit curves going through the origin, each based on about 15 comparative points. Each curve had a correlation coefficent $r^2 = 0.94$. Subsequently, in the production area, the third author obtained the q_c-M comparisons shown by the approximately 150 points shown in Figure 8. These subsequent 150 points increase the scatter but they also confirm the reasonableness of the curves obtained from the first 30 points in the test area.

The Engineers modified the acceptance criteria by allowing the use of the curves shown in Figure 8. They reasoned as follows: For a maximum expected surface blanket load increase of 350 kPA (3.5 tsf), a Layer 2 thickness of 8 m (26 ft), and an average M = 100 MPa (1000 tsf) for Layer 2, its maximum contribution to total settlement would = (stress increase x thickness)/modulus = (350 x 8000)/100,000 = 28 mm.

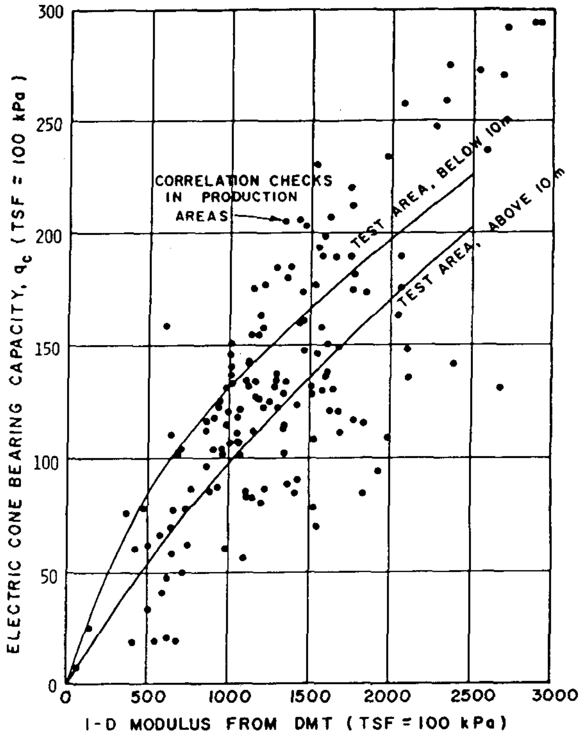
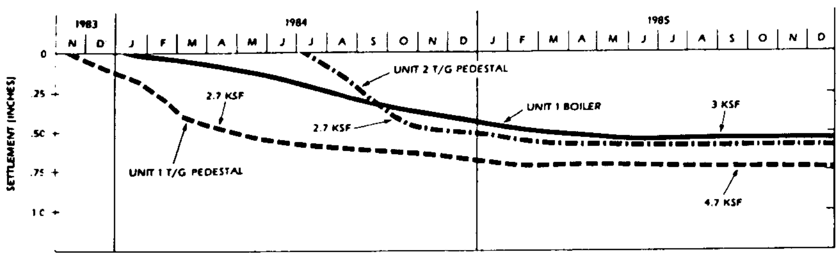


FIGURE 8 - SITE-SPECIFIC CORRELATIONS BETWEEN M FROM DMTs AND q_c FROM CPTs

Considering the arching action of the overlying DC-densified Layer 1, plus the arching action to the compaction grout columns in Layer 2, the Engineers believed that no more than 10%, or 3 mm, of the maximum settlement would become differential settlement. They considered a maximum 3 mm differential settlement contribution from layer 2 as acceptable vs. the maximum specified 6 mm from all the layers. They applied the 100 MPa minimum average M criterion when using both primary and secondary compaction grout points. Because of reduced arching possibilities when using only primary points, the Engineers increased the criterion to an average M below 10 m (33 ft) of 120 MPa (1200 tsf). All the settlement calculations neglected any contribution from the much less compressible Layers 3 and 4. Much of the plant dead loads are now in place, and surveyors have measured the actual settlement performance. Figure 9 presents average settlement vs. time curves and calculated vs. measured settlements to date for a representative group of three structures in the power block.

The settlements resulted from calculations using the same simple method that produced the aforementioned 28 mm, except using the average actual loading, the average thickness of Layer 1 and 2 soil under the structure, and the average M from the after-treatment soundings made at the location of the structure. The M values used came from the post-treatment CPT data and the q_c-M correlation curves in Figure 8. The reader can see from Figure 9 that good agreement exists between the computed and the actual settlements. More importantly from the point of view of performance, the Fig. 9 and other similar data indicate that the objective of 6 mm or less differential settlement between adjacent structures will be achieved.

SETTLEMENT VS. TIME



CALCULATED VS. MEASURED SETTLEMENT

STRUCTURE	UNIT 1 T/G PEDESTAL		UNIT 2 T/G PEDESTAL	UNIT 1 BOILER
APPROXIMATE LOAD/DATE	2.7 ksf 3/84	4.7 ksf 9/85	2.7 ksf 10/84	3 ksf 9/85
* CALCULATED SETTLEMENT	0.47 In.	0.82 In.	0.49 In.	0.42 In.
MEASURED SETTLEMENT	0.44 In.	0.73 In.	0.42 In.	0.46 In.

*BASED ON DILATOMETER MODULUS

FIGURE 9

SETTLEMENT DATA FROM THREE OF THE MAJOR STRUCTURES ON THE SITE (see Fig. 2)

7. Findings and Conclusions

7.1 The combination of CPT and DMT soundings provided excellent quality control for the improvement of a 17 m (56 ft) thick sand layer under a 21 acre site for a power plant.

7.2 A special CPT and DMT sounding truck, supplied and operated by the Contractor, permitted the timely and efficient QC testing that guided and thoroughly documented the work.

7.3 As a result of work in a test area, it proved possible and acceptable to adjust the initial relative density densification acceptance criteria to include one that used the 1-D compression modulus M.

7.4 The objective of using more economical shallow foundations, and limiting differential settlements between structures to 6 mm or less, appears to have been achieved.

7.5 The direct use of the DMT modulus M in simple settlement calculations, via a q_c-M correlation, produced good agreement with measured settlements.

8. Acknowledgements

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