Case History: Shallow Foundation Settlement Prediction Using the Marchetti Dilatometer

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ABSTRACT: A four-story, steel-framed office building was planned for construction in Jacksonville, Florida. It was hoped that the building could be supported on a shallow foundation system. Test borings revealed the presence of 9 to 14 feet of loose to firm clean sands overlying a 6- to 20-foot thick layer of very loose (N-value ranging from 0 to 5) very silty fine sand. Due to foundation loads ranging from approximately 400 to 1200 kips, combined with up to 2-1/2 feet of permanent structural fill, total settlements of up to 2 inches and differential settlements of up to 1 inch were estimated using conventional standard penetration resistance data and correlations with elastic modulus values. These predicted settlements were considered to be intolerable; therefore, several soil improvement concepts were evaluated to reduce settlement potential, including preloading of the building area, and the use of the vibro-replacement (stone column) technique. These methods, however, were considered to be either too costly or time consuming. Dilatometer testing was subsequently performed to refine the soil compression modulus values and settlement estimates. Total settlements of up to 1-1/4 inch and differential settlements of 3/4-inch or less were predicted using the dilatometer data which were considered to be generally acceptable to the structural engineer. The building was constructed using shallow foundations, and a settlement monitoring program was conducted during a portion of construction. The measured settlements were slightly less than those predicted using the dilatometer data. Use of the dilatometer at this site provided soil compressibility data which enabled the structure to be constructed successfully on a conventional shallow foundation system without utilizing costly and time consuming soil improvement techniques.

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Introduction

This case history involves a four-story office building which was constructed in 1991 in the Southpoint area of Jacksonville, Florida, which is located near the intersection of I-95 and J. Turner Butler Boulevard (refer to Figure 1 for a map of the general vicinity). Although the initial subsurface exploration program indicated the presence of a compressible soil layer, subsequent in-situ testing using the flat plate dilatometer, which provided a more accurate prediction of soil compressibility, allowed the building to be supported on a conventional shallow foundation system with no improvement of the compressible layer.

Background Information

The structure consists of a steel frame building with a central masonry core for elevators and stairs, and an exterior glass curtain wall. The overall plan dimensions of the building are 155 by 215 feet. Columns are generally spaced 30 feet on centers.

The following table presents the structural loading conditions for the subject building, as well as typical plan dimensions of the shallow foundations as designed by the structural engineer and the actual applied bearing pressures

Location	Maximum Foundation Load, DL+LL (kips)	Shallow Foundation Size (Feet)	Applied Bearing Pressure (ksf)
Typical Interior Columns	420	10½ by 10½	3.81
Stairwell Foundations	1175	22 by 29	1.84
Elevator Foundations	930	14 by 26	2.55

DL: Dead Load
LL: Live Load

Our evaluation of bearing capacity indicated that footings could be designed using an allowable bearing pressure of 4000 psf, provided a separation distance at least equal to one footing width existed between

LEGEND

- State route
- Marker
- City
- Small town
- Interstate route
- U.S. route
- Open water

Scale 1:175,000 (at center)

2 Miles

5 KM

Mag 12.00
Wed Sep 08 15:39:09 1993

Figure 1: Site Location Map

the bottom of the footing and the upper surface of the very loose silty sand layer. The actual applied bearing pressures of the stairwell and elevator foundations were somewhat lower due to wind loading considerations and minimum foundation size requirements.

For the column loads, the dead load was about 70 percent of the total (dead plus live) load. In designing the building foundation, the structural engineer established tolerable post-construction total and differential settlements of one inch and 3/4-inch, respectively. The tolerable differential settlement was determined by the structural engineer to be 0.002 times the column spacing

Prior to construction, the site was relatively flat and level, and consisted of maintained grass. In order to bring the site to the desired grade, up to 2½ feet of permanent structural fill was required. Some 3- to 4-foot high landscaped earth berms that were located on the property required removal as part of the site grading process. A similar three-story office building is located just east of the site of the proposed construction. This structure is supported on a shallow foundation system.

Field Exploration Program

In order to explore the subsurface conditions in the area of the planned building prior to construction, a total of twelve standard penetration test (SPT) borings were drilled on the property to depths ranging from 30 to 50 feet each. These borings included several that were drilled a few years earlier for another project which was subsequently abandoned. Also, the configuration and location of the subject office building were changed several times before the final configuration was determined. Of the twelve borings performed on the property, seven were located in the general vicinity of the final building configuration (refer to Figure 2).

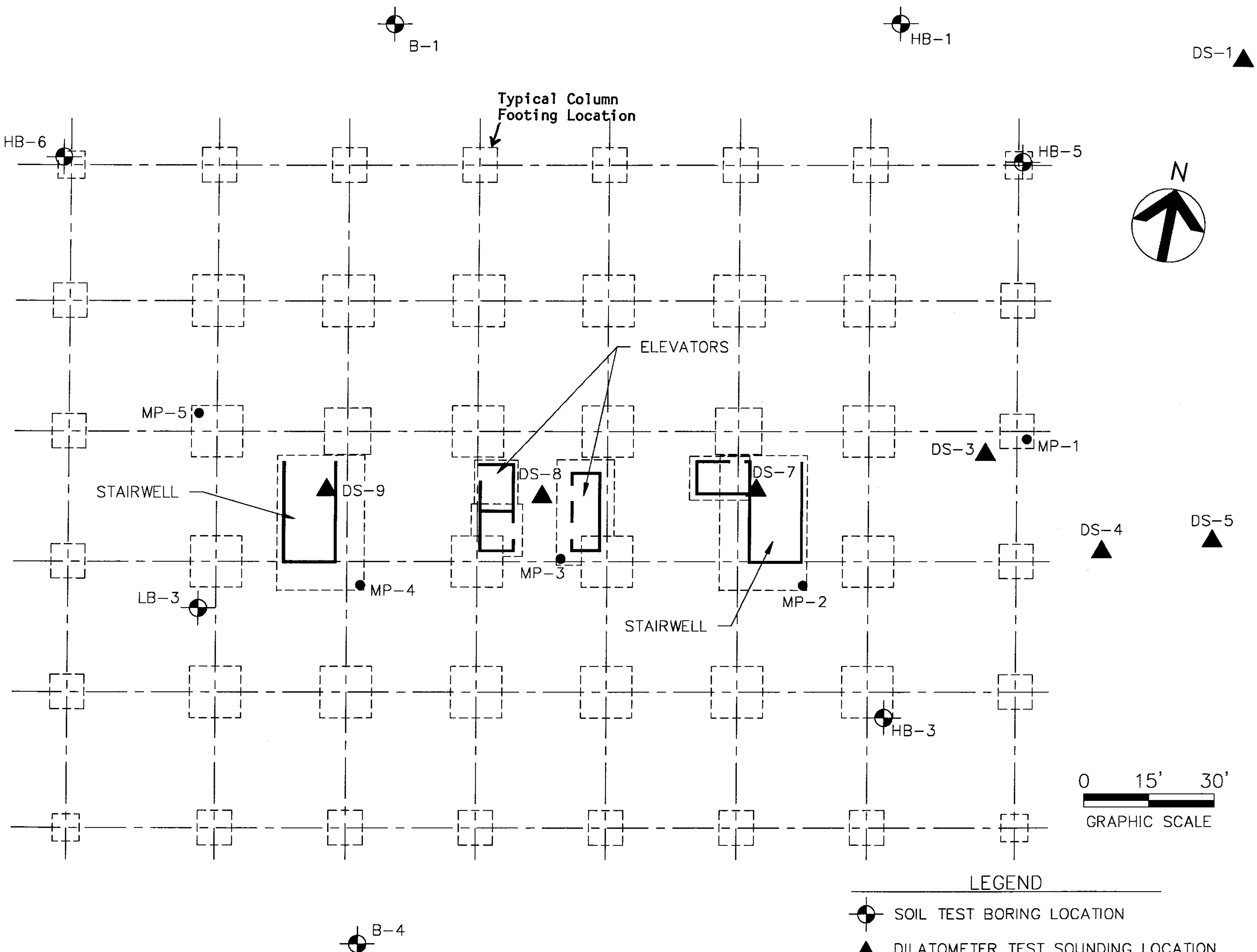


FIGURE 2: FIELD EXPLORATION PLAN

Subsurface Conditions

The SPT borings generally identified a three-layered subsurface profile, as shown on Figure 3 and in the following table:

Layer No.	Typical Depth Range (Feet)	Actual Layer Thickness Range, Feet (Average in Parentheses)	Generalized Soil Description (Unified Soil Classification Symbol)	Standard Penetration Resistance (N, Blows/Foot)
1	0 to 12	8 to 17 (11)	LOOSE to FIRM fine SANDS (SP)	3 to 20
2	12 to 30	5 to 20 (13)	VERY LOOSE slightly clayey silty to very silty fine SANDS (SM)	0 to 5 (typically WOH*)
3	30 to 50	4 to 31 (11½)	LOOSE to VERY FIRM fine SANDS (SP)**	6 to 21

* Weight of hammer and drilling rods.
** All borings were terminated in this material

Laboratory Testing Program

Laboratory classification testing of representative soil samples in Layer 2 obtained from the SPT borings indicated the following properties:

Test	Range	Average
Water Content (W_n)	31% to 40%	35%
Liquid Limit (LL)	29 to 34	32
Plasticity Index (PI)	1 to 6	4
Fines Content	13% to 39%	24%

Unified Soil Classification System Symbol: SM

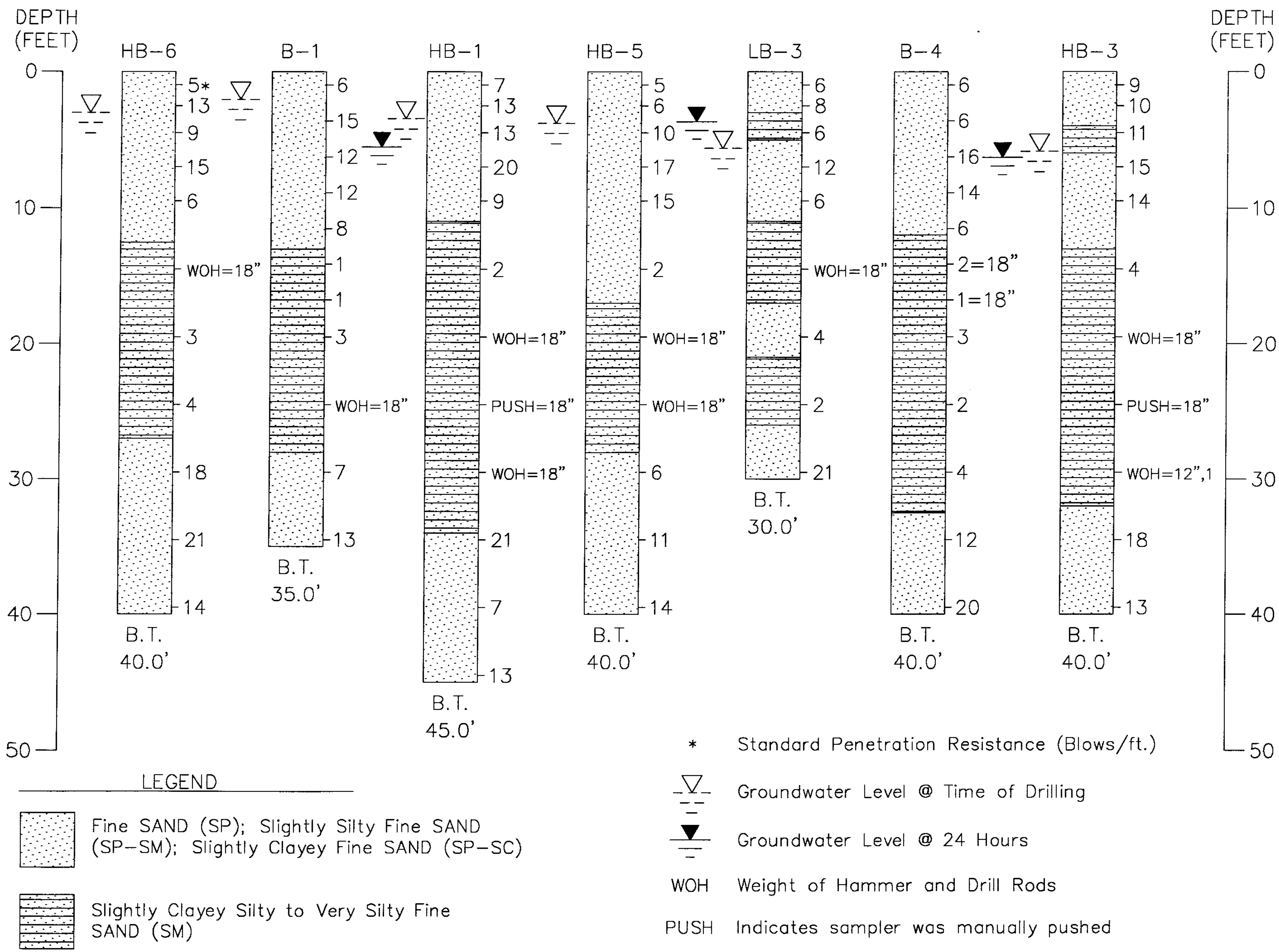


FIGURE 3. GENERALIZED SUBSURFACE PROFILE

It is noted that laboratory consolidation testing of the soils from Layer 2 was not conducted because undisturbed samples were not able to be obtained due to the predominantly sandy nature of these soils, and also because there was not sufficient time in the project schedule to permit such testing (this was a design-build project). As discussed in the next section, in-situ testing was determined to be a better method to estimate the strength and compressibility characteristics of these soils.

Preliminary Evaluation

Based on the results of the SPT borings, the primary geotechnical concern related to the proposed construction was the settlement potential of the very loose clayey/silty sands comprising Layer 2 due to foundation and structural fill loads. This very loose clayey/silty sand layer is typical of the Southpoint area of Jacksonville. There are numerous three- to six-story office buildings and hotels located in this area, with one hotel having nine stories. To our knowledge, all of the buildings are supported on conventional shallow foundation systems, with the nine-story building being supported on a mat foundation. Several of these sites, however, have required preloading with a temporary surcharge fill mound to precompress the very loose clayey/silty sand layer with a temporary surcharge fill mound in conjunction with settlement monitoring programs. The temporary surcharge fill mounds were typically 5 to 7 feet in height (above any required permanent fill) to account for the structural loading, and were required to remain in place for one to three months, after which they were removed and construction of the buildings was permitted to take place on conventional shallow foundation systems.

Based on the results of the SPT borings performed for this project, an analysis was conducted to estimate potential foundation settlements that could occur. The majority of the potential settlements would be due to compression of the very loose silty sands in Layer 2. The soils in Layers 1 and 3 were not believed to contribute significantly to the total settlement due to the generally firm relative density condition of these soils. Despite its very low strength, the laboratory test results indicated that the Layer 2 soils would compress relatively quickly due to the relatively low fines content and plasticity index. We therefore initially estimated settlements using elastic theory with a Westergaard stress distribution Modulus of elasticity values of the soils were estimated using published empirical correlations with SPT N-values. The modulus of elasticity value for Layer 2 utilized in our preliminary settlement evaluation was 100 ksf, and for Layer 1, a value of 850 ksf was utilized. Settlements for each layer were calculated using elastic theory, as follows

$$\Delta h_i = \frac{h_i \Delta \sigma}{E_i} \tag{1}$$

where Δh_i = settlement of the ith layer, h_i = thickness of the ith layer, $\Delta\sigma$ = increase in vertical effective stress in the ith layer due to the foundation loading, and E_i = modulus of elasticity of the ith layer. The total settlement was then computed by summing the estimated settlements of each individual layer. The results of our preliminary evaluation indicated estimates of total settlement to be in excess of 2 inches, which was deemed intolerable by the structural engineer.

Two soil improvement techniques were then evaluated to reduce shallow foundation settlement potential: (1) preloading with a temporary surcharge fill mound, and (2) installing stone columns into Layer 3 at footing locations only. These alternatives were presented to the structural engineer for consideration. There were, however, corresponding cost and time factors associated with each of these ground improvement alternatives. With the preloading alternative, the major consideration was the time and expense required to construct, monitor, and remove the temporary surcharge fill mound. With the stone column alternative, the additional cost of $30,000 to $40,000 was determined to be too costly by the structural engineer and the owner. Although the cost of the surcharge alternative was not estimated at the time, we have estimated a cost on the order of $90,000 to $100,000 to haul the required fill material to the site, monitor settlements, and then to remove the fill at the completion of the preloading period.

We then suggested to the structural engineer that the need for deep soil improvement could be avoided if the estimated soil parameters used in our settlement evaluation could be refined. We suggested the use of the Marchetti flat-plate dilatometer, an in-situ testing device owned by LAW that we could mobilize quickly to the site. It has been our experience that the dilatometer device allows more realistic estimates of settlement by providing more accurate values of soil compression modulus (modulus of elasticity). Furthermore, while SPT sampling is typically performed on 5-foot depth intervals, the dilatometer test (DMT) soundings are typically performed on 1-foot depth intervals, providing less need for interpolation and more accurate soil layer boundary stratification (when utilized in conjunction with SPT borings).

Dilatometer Testing Program

For the supplemental in-situ testing program, we performed a total of nine dilatometer test (DMT) soundings on the property to depths of 35 to 40 feet each. The dilatometer blade was pushed with a drill rig. Five of the DMT soundings were located in the general vicinity of the final building configuration, with three of these soundings being located in the area of the central masonry core (refer to Figure 2 for DMT sounding locations). The DMT soundings were performed on 12-inch depth intervals. A schematic of the dilatometer device is shown in Figure 4. A typical DMT sounding record is presented in Figure 5. As shown in Figure 6, the plot of compression modulus (one-dimensional vertical compression modulus, M) versus depth for a typical sounding readily identified the relatively weak, compressible soils of Layer 2.

The M-values in Layer 2 generally ranged from about 40 to 50 ksf at the east end of the building with an average value of about 45 ksf. In the central masonry core area, M-values generally ranged from about 80 to 100 ksf, with an average value of about 85 ksf. As mentioned previously, the compression modulus value for Layer 2 utilized in our initial settlement evaluation based on published correlations with SPT N-values was 100 ksf, and in Layer 1, a value of 850 ksf was utilized. By comparison, the M-values as determined by the DMT soundings in the upper sands (Layer 1) ranged from about 400 to over 2,900 ksf, with an average of about 1,575 ksf. The ratio of the modulus values for Layer 1 to Layer 2 was 8.5 for the values correlated with the SPT N-values, whereas this ratio was on the order of 20 to 35 for the modulus values determined from the DMT soundings.

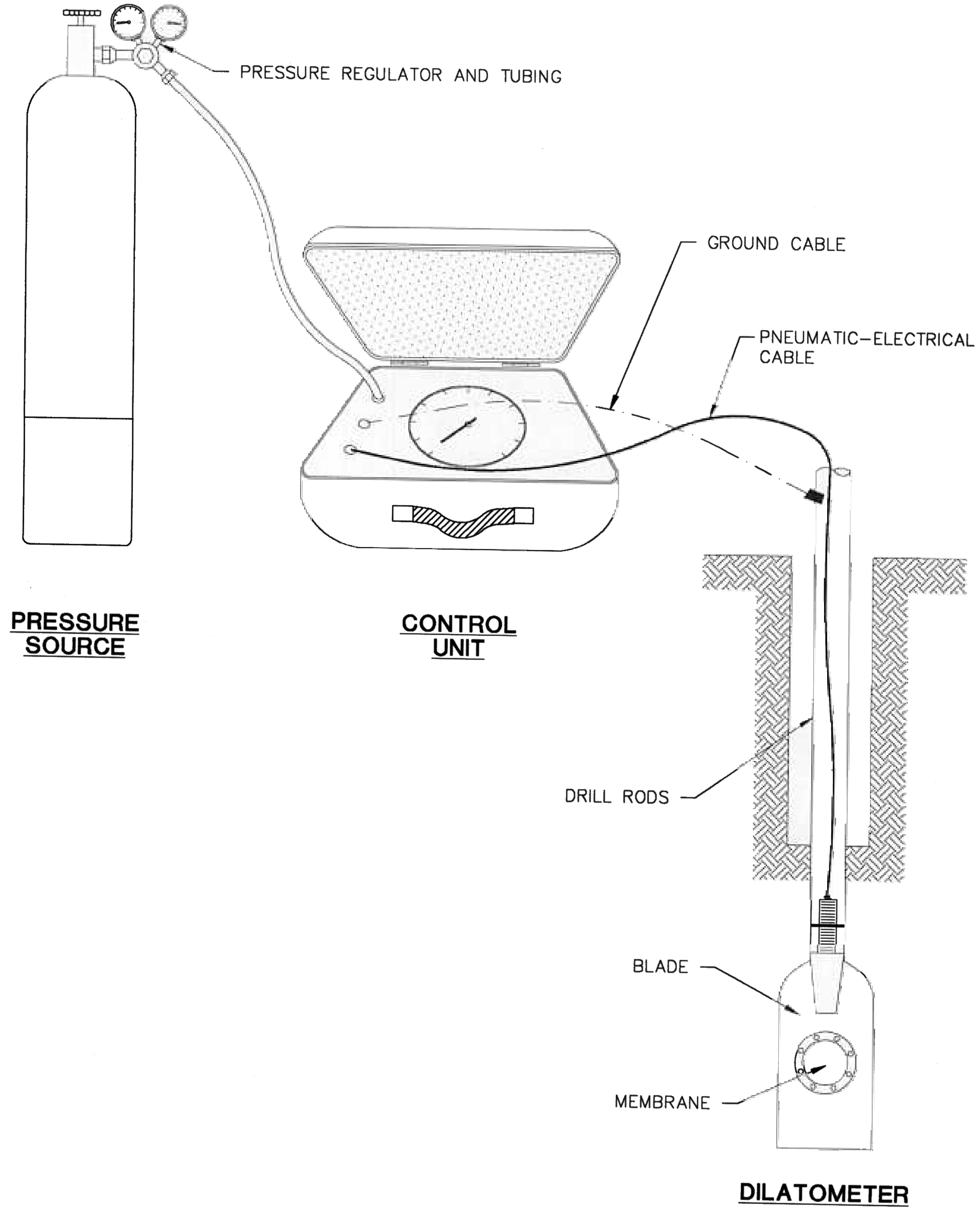


FIGURE 4: SCHEMATIC OF THE DILATOMETER EQUIPMENT

Figure 5: Typical DMT Sounding Record

LAW ENGINEERING INC — TEST NO. DS-3

FILE NAME: OFFICE BUILDING SITE

FILE NUMBER: J-6200A

RECORD OF DILATOMETER TEST NO. DS-3

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: SOUTHWEST SIDE OF SITE

PERFORMED - DATE: OCTOBER 8, 1990

BY: T. SELFRIDGE, J. ALEXANDER, R. DRAWDY

CALIBRATION INFORMATION:

DELTA A = .19 BARS DELTA B = .13 BARS GAGE 0 = .03 BARS GWT DEPTH= 1.92 M

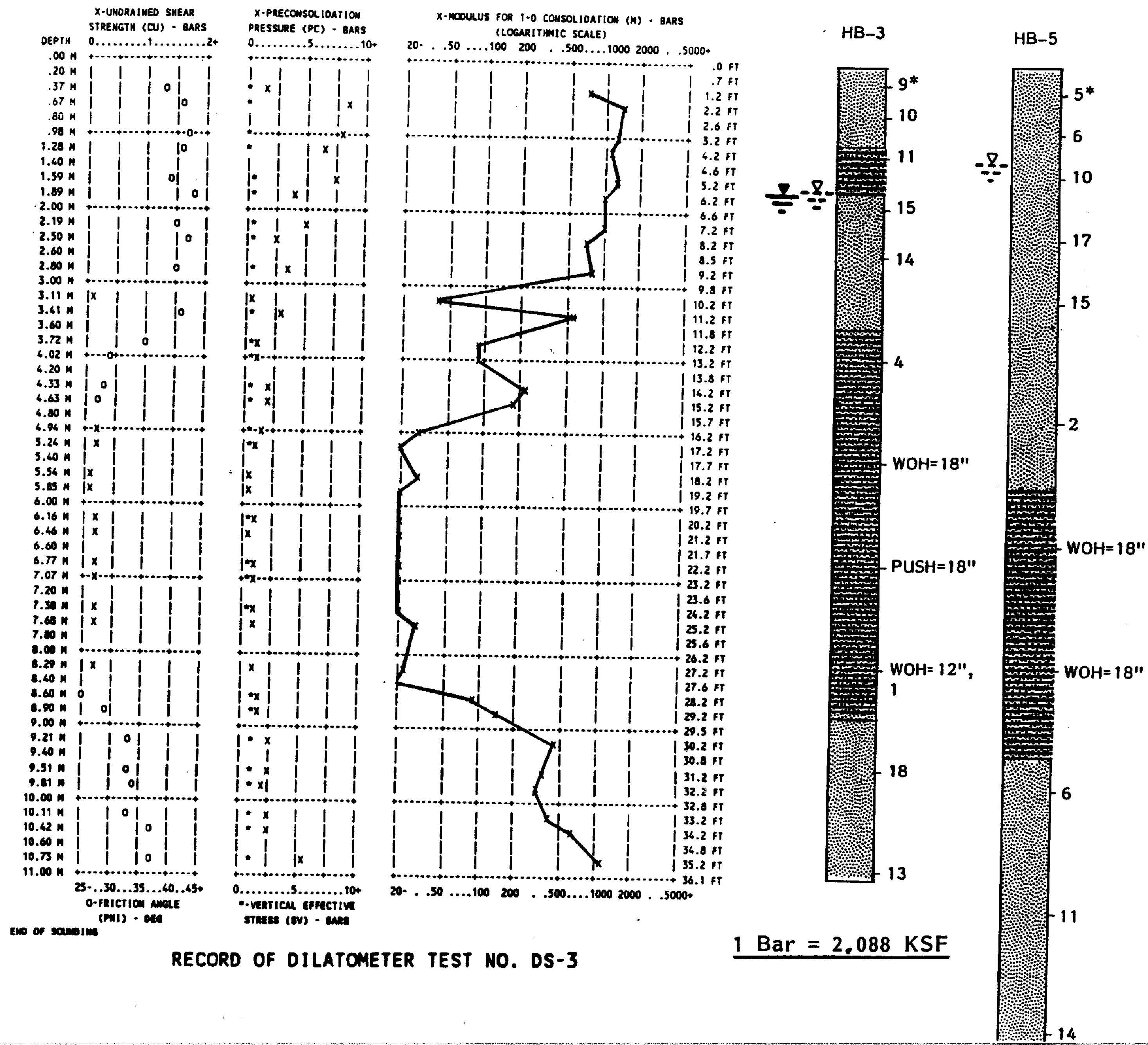
ROD DIA.= 4.40 CM FR.RED.DIA.= 5.40 CM ROD WT.= 6.30 KG/M DELTA/PHI= .50 BLADE T=15.00 MM

1 BAR = 1.019 KG/CM2 = .044 TSF = 14.51 PSI ANALYSIS USES H2O UNIT WEIGHT = 1.000 T/M3

Z (M)	THRUST (KG)	A (BAR)	B (BAR)	ED (BAR)	ID	KD	U0 (BAR)	GAMMA (T/M3)	SV (BAR)	PC (BAR)	OCR	KO	CU (BAR)	PHI (DEG)	M (BAR)	SOIL TYPE
.37	508.	1.21	8.32	247.	6.92	14.93	.000	1.800	.069	1.62	23.47	1.81		38.0	712.4	SAND
.67	2032.	3.74	15.30	410.	3.54	27.04	.000	1.900	.123	8.65	70.09	3.18		40.9	1409.5	SAND
.98	2830.	4.31	15.20	385.	2.82	21.56	.000	2.000	.183	8.09	44.23	2.51		41.7	1243.0	SILTY SAND
1.28	2903.	4.41	15.40	389.	2.78	16.70	.000	2.000	.242	6.65	27.51	1.96		41.4	1160.7	SILTY SAND
1.59	2250.	4.94	16.80	420.	2.68	14.95	.000	2.000	.303	7.62	25.20	1.88		38.5	1211.3	SILTY SAND
1.89	4137.	4.44	15.00	373.	2.63	11.31	.000	2.000	.361	4.16	11.51	1.24		43.3	976.3	SILTY SAND
2.19	2685.	4.63	15.30	377.	2.56	10.78	.026	2.000	.394	5.11	12.98	1.35		39.8	969.7	SILTY SAND
2.50	3266.	3.64	12.80	322.	2.81	7.81	.057	1.900	.423	2.53	5.98	.90		42.1	735.1	SILTY SAND
2.80	2830.	4.21	14.30	356.	2.70	8.45	.086	1.900	.449	3.58	7.96	1.06		40.4	836.7	SILTY SAND
3.11	1887.	1.20	2.54	37.	.90	2.53	.117	1.600	.472	.68	1.44	.68	.139		41.5	CLAYEY SILT
3.41	2903.	3.96	11.00	245.	1.94	7.36	.146	1.900	.494	3.02	6.12	.93		40.5	541.8	SILTY SAND
3.72	1089.	1.72	3.94	69.	1.24	3.10	.177	1.700	.518	1.04	2.01	.58		35.2	93.4	SANDY SILT
4.02	508.	1.45	3.99	81.	1.80	2.39	.206	1.800	.540	1.05	1.95	.64		28.8	92.7	SILTY SAND
4.33	581.	2.33	6.58	143.	2.01	3.64	.237	1.800	.565	1.89	3.35	.80		28.2	223.0	SILTY SAND
4.63	508.	2.23	5.94	124.	1.82	3.32	.266	1.800	.588	1.85	3.14	.80		26.8	180.1	SILTY SAND
4.94	290.	2.11	3.00	21.	.31	3.19	.296	1.600	.610	1.26	2.07	.83	.240		27.6	CLAY
5.24	145.	1.99	2.76	16.	.26	2.87	.326	1.600	.627	1.10	1.76	.76	.217		20.0	CLAY
5.54	73.	1.41	2.59	31.	.77	1.82	.355	1.600	.645	.56	.86	.49	.126		26.6	CLAYEY SILT
5.85	145.	1.56	2.55	24.	.54	1.96	.386	1.600	.663	.64	.97	.53	.142		20.7	SILTY CLAY
6.16	73.	1.79	2.60	18.	.34	2.21	.416	1.600	.682	.80	1.17	.60	.170		17.0	CLAY
6.46	218.	1.69	2.51	18.	.38	1.97	.446	1.600	.699	.68	.98	.54	.151		15.5	SILTY CLAY
6.77	145.	2.00	2.45	5.	.08	2.34	.476	1.500	.716	.92	1.28	.63	.192		4.8	MUD
7.07	145.	2.00	2.45	5.	.08	2.26	.505	1.500	.731	.88	1.21	.61	.187		4.6	MUD
7.38	145.	2.10	2.85	16.	.27	2.28	.536	1.600	.747	.92	1.23	.62	.193		15.4	CLAY
7.68	145.	2.41	3.46	27.	.39	2.57	.565	1.600	.765	1.13	1.48	.69	.231		29.5	SILTY CLAY
7.99	73.	.25	1.25	24.	-3.66		.596	P01 =	.38	P0 =	.41	P1 =	1.09			QUESTIONABLE
8.29	145.	2.07	3.09	26.	.47	1.96	.625	1.600	.801	.78	.97	.53	.172		21.7	SILTY CLAY
8.60	290.	2.04	5.32	108.	2.23	1.70	.656	1.800	.822	1.57	1.91	.74		21.4	93.9	SILTY SAND
8.90	726.	2.53	6.42	130.	2.05	2.16	.685	1.800	.846	1.52	1.79	.62		29.1	139.7	SILTY SAND
9.21	1379.	4.30	12.20	276.	2.36	3.86	.715	1.900	.872	2.73	3.13	.74		32.9	451.3	SILTY SAND
9.51	1379.	4.12	10.70	228.	2.04	3.59	.745	1.900	.898	2.54	2.83	.71		33.0	352.5	SILTY SAND
9.81	1451.	3.49	10.00	226.	2.53	2.78	.774	1.900	.925	1.79	1.94	.59		34.0	303.9	SILTY SAND
10.11	1524.	4.40	11.50	247.	2.08	3.59	.804	1.900	.951	2.66	2.79	.70		33.5	382.8	SILTY SAND
10.42	2395.	4.73	15.30	373.	3.04	3.62	.834	1.900	.979	2.34	2.39	.62		37.1	604.1	SILTY SAND
10.73	3048.	7.60	23.50	568.	2.67	6.07	.865	2.000	1.007	5.50	5.46	.92		37.1	1167.1	SILTY SAND

END OF SOUNDING

Figure 6: Comparison of DMT Sounding With SPT Boring Stratification



Revised Settlement Evaluation and Recommendations

Using the compression modulus (M) values obtained from the DMT soundings, a revised settlement evaluation was conducted. This revised settlement evaluation took into account the ratio of the modulus values of Layers 1 and 2 in estimating the stresses distributed through the soil profile from the foundation loads We utilized Palmer and Barber's Method as described in Elastic Solutions for Soil and Rock Mechanics by H.G. Poulos and E.H. Davis (1974) to estimate the stresses imposed on the soil layers for this "hard over soft" system. This method assumes that the upper layer of thickness h_1, modulus M_1, and Poisson's ratio v_1, may be replaced by an equivalent thickness h_e of lower layer material (modulus M_2, Poisson's ratio v_2) as follows:

$$h_e = h_1 \left[\frac{M_1 \; (1-v_2^2)}{M_2 \; (1-v_1^2)} \right]^{1/3} \qquad (2)$$

A Westergaard stress distribution was then utilized with this revised (or "warped") system to calculate the stress increase at various depths for each sublayer throughout the subsurface profile due to the foundation loads. The result of this "warped" system is that less stress reaches the underlying softer soil layer than with the actual layer thickness, which more closely models the actual field conditions. Furthermore, the Westergaard stress distribution more closely represents the elastic conditions of a stratified soil mass than does the widely used Boussinesq stress distribution. Once the stress distribution was determined, settlements were estimated for each layer using elastic theory along with the original, actual layer or sublayer thicknesses, as follows

$$\Delta h_i = \frac{h_i \Delta \sigma}{M_i} \qquad (3)$$

where M_i = the compression modulus of the ith layer. The total settlement was determined by summing the settlements computed for each individual layer. The results of our revised settlement evaluation are presented in the table presented below:

Location	Maximum Estimated Settlement (Inch)	
	Total	Differential
Typical Column Footing	1-1/4	1/2 to 3/4
Stairwell Foundations	1	1/2
Elevator Foundations	1-1/4	1/2 to 3/4

Based on our previous experience in the Southpoint area, we estimated that this settlement would take about one to two months to occur after the load had been applied.

For the purpose of this paper, we calculated settlements due to a typical interior column load using the standard Boussinesq and Westergaard stress distributions, and also using these stress distributions with Palmer and Barber's Method of layer "warping" to account for the hard over soft effects as described above. Compression modulus values of 1575 ksf and 45 ksf were utilized for Layers 1 and 2, respectively. The results of these settlement estimates, performed for comparison purposes only, are presented in the following table:

Column Load (kips)	Footing Size (feet)	Total Estimated Settlement (inches)			
		Standard		Hard Over Soft	
		Westergaard	Boussinesq	Westergaard	Boussinesq
420	10.5 x 10.5	2	2-3/4	1-1/4	1-3/4

As is evident from the above table, the hard over soft analysis results in lower estimated settlements than does the standard analysis that assumes a homogeneous system throughout. Also, the Westergaard stress distribution results in lower settlements for both the standard and hard over soft analyses than does the Boussinesq stress distribution. It is noted that the stress increase in Layer 2 using the hard over soft analysis was approximately 30 percent lower than that using the standard analysis.

The revised settlement estimates were acceptable to the structural engineer. The preloading and stone column alternatives were therefore deemed to be unnecessary. The structural engineer, however, still wanted to limit the total post-construction settlements to a value of one inch or less We therefore recommended a construction sequence such that the central masonry core would be partially constructed prior to floor construction and steel erection, thereby reducing differential settlements between the central core and the closest column footings In order to confirm the settlement estimates and to determine when erection of the surrounding steel frame could begin, we also recommended that a settlement monitoring program be implemented during construction

Settlement Monitoring Program

The recommended construction-phase settlement monitoring program was performed by technicians from our office using a rod and engineer's level over a three-month period. A total of five footing locations were monitored on a weekly basis. These locations included two column footings, two stairwell footings, and one elevator footing, as shown on Figure 2. The monitoring program was initially performed through construction of the central masonry core Based on these initial measurements, we were able to recommend when the steel frame erection could begin. The monitoring program was continued through steel erection, floor slab construction, and glass curtain wall installation The settlement monitoring program was terminated when it became necessary to cover up the monitoring points due to the progression of construction.

The structural engineer indicated that the majority of the structural dead load was in place at the completion of the monitoring program. The live load, which had not yet been in place, was estimated by the structural engineer to be about 25 percent of the total load in the central masonry core area, and about 30 percent of the total load at the column locations

At the completion of the monitoring program, the total measured settlements ranged from about 1/3 to 2/3 inch in the central masonry core area. At the column footing locations, the total measured settlement was about 1/4 inch. Over the last month of the settlement monitoring program, the elevations of the monitoring points had remained relatively constant, indicating that foundation settlements had essentially stopped. A possible reason the measured settlements were less than the predicted settlements include the fact that less than 70 percent of the total load was in place at the end of the monitoring program.

Conclusions

In summary, the following conclusions were drawn from this case history:

1. The actual measured settlements were slightly less than the estimated settlements using dilatometer data and the Westergaard stress distribution with the hard over soft layer analysis. In general, reasonably good agreement was observed between the predicted and measured settlements.

2. Using dilatometer testing, the structure was able to be constructed successfully on a shallow foundation system without utilizing more expensive and time-consuming soil improvement techniques.

3. Use of the dilatometer, and acceptance of this technique by our client, allowed the original desired schedule to be maintained. Also, a savings of roughly $30,000 to $40,000 in foundation costs was realized by eliminating the need for stone columns, and the need for preloading was eliminated, saving approximately $90,000 to $100,000.

4. The standard penetration test cannot measure the strength and compressibility of soils as accurately as the flat-plate dilatometer, which is a more sensitive device. Also, dilatometer testing is typically performed on closer depth intervals than the standard penetration test. For this project, the SPT data underpredicted the modulus of the relatively firm upper sands of Layer 1.

5. The process of driving the spoon in very loose saturated sandy soils can create a temporary quick condition, which may result in low strength predictions.

6. The ratio of the compression modulus values of Layer 1 versus Layer 2 ranged from about 20 to 35, which greatly reduced the loading applied to the lower layer using the hard over soft concept. Use of the hard over soft concept resulted in more accurate settlement predictions, even though the compression modulus values in Layer 2 determined from the DMT soundings were lower than the modulus values estimated initially from SPT N-value correlations. This is believed to be a result of the increased modulus values measured by the DMT soundings for Layer 1, which were approximately 2 to 3 times the modulus values estimated using published SPT N-value correlations.

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