Tower Foundations Bearing Above Weak Soils

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SYNOPSIS A 13-level reinforced concrete structure was constructed on Marco Island in southwest Florida. The tower is located 200 feet from the Gulf of Mexico and has plan area dimensions of 115 by 170 feet.

The field testing revealed the site was mantled with a 17-foot thick layer of firm sand. The sand stratum was underlain by a compressible 9-foot thick layer of silty sand which had an average SPT N-value of less than 2.

Various methods of engineering analyses estimated total tower settlements to range from 1 to 8 inches. Actual measured settlement following the application of dead load was about 1 3/4 inches. An engineering inspection following construction revealed diagonal shear wall cracks.

INTRODUCTION

Usual high-rise building foundations in the vicinity of this project site involve 50 to 70-foot long driven piles. The project owners requested that other foundation options be studied, emphasizing the need for cost and time reductions. A field and laboratory test program was structured to quantify engineering properties of the compressible stratum with hopes that predicted settlements could be tolerated.

The project site is located on Marco Island on the west coast of Florida. The building was positioned 200 feet off the Gulf of Mexico and had site elevations of +8 feet MSL. Building construction involved a 6-foot deep excavation.

The 13-story tower covers a plan area of about 170 by 115 feet. A 2-story parking structure was constructed adjacent to the tower. Column loads in the tower ranged from 300 to 800 kips each, which resulted in a net applied building footprint bearing pressure of 1.3 ksf. Shear walls would generate a maximum moment of about 50,000 foot-kips.

AREA GEOLOGY

Peninsular Florida is characterized by sedimentary deposits. Southwest Florida is located in the Coastal Lowlands geomorphic province and more specifically in the Mangrove and Coastal Glades physiographic provinces. The geologic profile is composed of undifferentiated sands, silts, and silty clays to a depth of 50 to 75 feet. The Tamiami Formation is present below this depth and is composed of sands, sandstones and limestones.

FIELD AND LABORATORY DATA

The field exploration consisted of 5 soil test borings. The standard penetration test and undisturbed sampling were used as exploration tools.

Identification and strength tests were performed on soil samples transported to the laboratory. Laboratory tests consisted of: percent passing No. 200 sieve, Atterberg Limits, moisture content, and consolidation. The generalized subsurface conditions are presented in Figure 1. Field and laboratory test data are summarized in Table I.

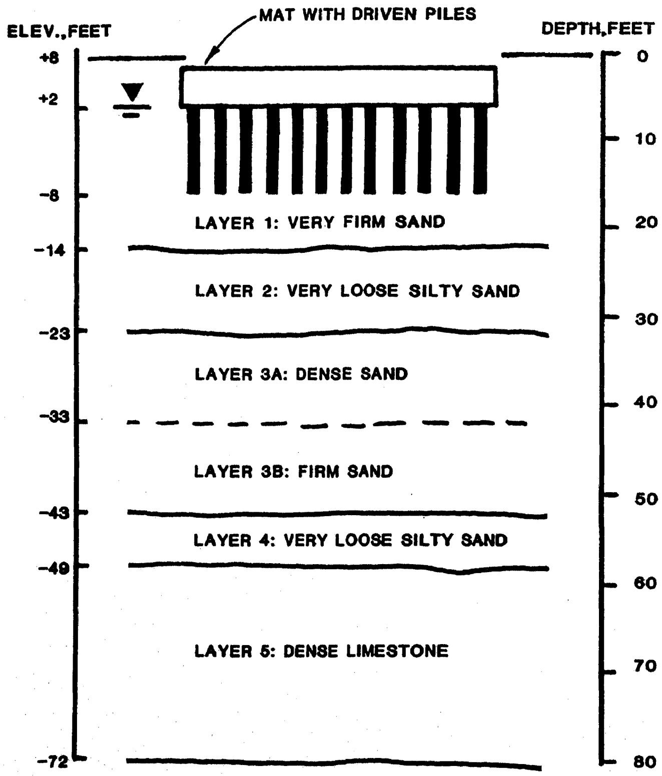

Figure 1 - Generalized Subsurface Profile

TABLE I. Field and Laboratory Data

	Layer Number					
	1	2	3A	3B	4	5
Description USCS	SP/SW	SM	SP/SW	SP/SW	SM	Lime-stone
Range SPT N-Values	10-52	1-2	6-79	6-22	1-7	15-100
Average SPT N-Values	27	2	44	15	3	35
Laboratory LL, %		21			27	
In-Situ W %		31		24		
% Passing No. 200 Sieve		23				
Initial Void Ratio		0.8			0.6	
Compression Index		0.04			0.06	
Consolidation Coefficient, ft^2/day		0.6			1.4	

ENGINEERING PROPERTIES

The field and laboratory data was used to es mate soil modulus values. Average standard penetration N-values were first converted to from the cone penetration test. The ratio of Q_c/N was estimated to be 4 (Schmertmann, 197

Modulus values were estimated from several sources. Schmertmann, 1970, suggests soil modulus values equivalent to twice Q_c based screw plate compressibility tests. Leary an Langan, 1982, suggest soil modulus equivalen to 2.5 Q_c based on preload data. Webb, 1969 performed in-situ screw plate compressibilit tests on fine to medium sands below the wate table. Based on these tests, Webb suggests soil modulus value equivalent to: E(tsf)=2.5 (Qq_c + 30 tsf). Schultze and Melzer, 1965, estimated modulus values for non-cohesive so using large-scale laboratory equipment. The results of this study present soil modulus values as a function of standard penetration N-values varying with the effect of overburd pressure. Laboratory consolidation test dat was also used to estimate soil modulus.

A comparison of the various soil modulus val is presented in Table II.

Table II. Soil Modulus Estimates

	Layer Number					
	1	2	3A	3B	4	5
Average SPT N-Value Blows/Foot	27	2	44	15	3	35
Approximate CPT, q_c ksf	215	15	350	120	25	280
E, ksf (Schmertmann, 1970)	430	30	700	240	50	560
E, ksf (Leary and Langan, 1982)	540	40	880	300	60	700
E, ksf (Webb, 1969)	690	190	1030	450	210	850
E, ksf (Schültze and Melzer, 1965)	1200	500	900	550	100	400
E, ksf (Consolida-tion Test Data)	--	110	--	--	110	--

SETTLEMENT ESTIMATES

The four methods of estimating modulus prese ted above are coupled with three elastic settlement solutions. One method utilized w that suggested by Kaderabek and Reynolds, 19 and is represented below.

$$S_i = \frac{p\, H_i\, I\, (12)}{E_i} \quad (1)$$

where: S_i = settlement within layer H_i, inches
p = footprint surface bearing pressure, ksf
H_i = layer thickness, feet
I = influence factor for the middle of H_i, dimensionless, NAVFAC DM7 1971
E_i = soil modulus, ksf

The second method of analysis is that suggested by Poulos and Davis, 1974, with the settlement equation being that shown below.

$$S = \frac{1.5\, p\, a(12)}{E_e} \quad (2)$$

where: S = settlement resulting from p, over the depth represented by E_e, inches
p = footprint bearing pressure, ksf
a = radius of the bearing area when the building plan geometry is normalized to a circular shape, feet
E_e = equivalent soil modulus, various soil layers and moduli are normalized into a single equivalent layer and modulus, ksf

The equivalent soil modulus is obtained from a repetitive process of two-layer equivalencies until a single equivalent layer is obtained. The method for determining equivalent modulus is shown below.

$$E_e = \left[\frac{h_1\sqrt[3]{E_1} + h_2\sqrt[3]{E_2}}{h_1 + h_2}\right]^3 \quad (3)$$

where: E_e = equivalent soil modulus of two soil layers h_1 and h_2, ksf
h_1 = layer thickness shallower strata, feet
h_2 = layer thickness deeper strata, feet
E_1 = soil modulus shallower strata, ksf
E_2 = soil modulus deeper strata, ksf

A third method of estimating settlement is that procedure described by Kay and Cavagnaro, 1983. This method is identical to the method suggested by Kaderabek and Reynolds, but utilizes a different influence factor.

Table III presents total surface settlements as a result of an applied surface pressure of 1.3 ksf for various elastic solutions and modulus values.

Table III. Surface Settlement Estimates (Inches) From a Surface Pressure of 1.3 ksf, Beneath the Center of the Loaded Area

	Kaderabek and Reynolds, 1979	Poulos and Davis, 1974	Kay and Cavagnaro, 1983
Modulus from Schmertmann, 1970	8.3	5.0	6.8
Modulus from Leary and Langan, 1982	6.5	4.5	5.3
Modulus from Webb, 1969	2.3	3.1	1.9
Modulus from Schultze and Melzer, 1965	2.5	3.2	2.1

NOTE: The recorded settlement under full dead load was 1.8 inch.

FOUNDATION DESIGN

The tower portion of the project was designed to be supported on a mat varying in thickness from 30 to 54 inches. The modulus of subgrade reaction for the soils immediately beneath the mat was estimated to be 50 lbs./inch^3. The two-level garage structure utilized isolated shallow foundations.

Concern over soil scour as a result of hurricane storms dictated that short piles be used under the mat portion of the project. Twelve-inch square driven concrete piles which had an allowable capacity of 70 tons were used. A pile length of 10 feet was selected for bearing in the firm, near-surface sands. These piles were driven with a diesel pile hammer having an energy of 18,000 ft-lbs. A successful pile load test was performed to twice the working capacity. Driven piles were positioned beneath column areas in clusters of two, three, four, and five piles. Beneath shear wall areas pile clusters were increased to 10, 14, and 18 piles. Figure 2 illustrates the foundation layout in plan view.

The mat foundation, even with short piles, proved to be cost effective in both time and dollar savings.

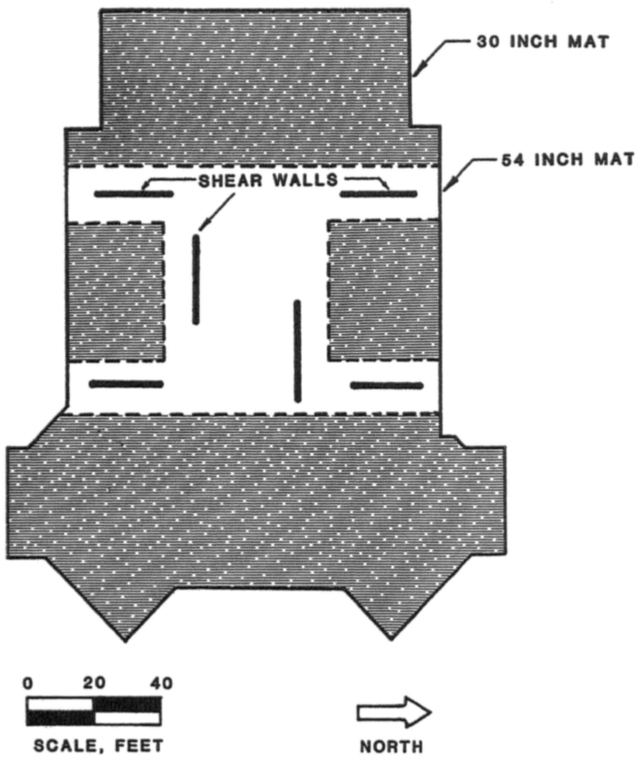

Figure 2 - Foundation Plan

FOUNDATION PERFORMANCE

A detailed settlement monitoring program was not performed for this project. However, top-of-mat elevations at the time of mat placement and following application of full dead load indicate total settlement on the order of 1 3/4 inches. A photograph of the completed concrete frame is shown in Figure 3.

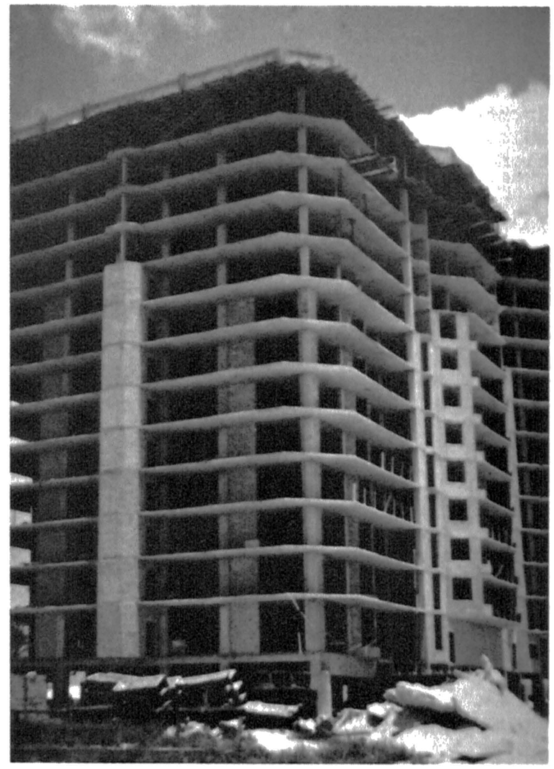

Figure 3 - Photograph of the Completed 13-Sto Concrete Tower

Site observations following application of ful dead load indicated diagonal cracks within all north-south shear wall areas. One to four no intersecting cracks were observed in each of the four shear walls. These cracks went through the entire 12-inch thick wall section. The cracks ranged from two to 10 feet in leng and slanted about 45° from vertical toward the center of the building. Crack widths ranged from .015 to .053 inch. Drawing perpendicula lines through these diagonal cracks pointed toward the source of settlement being the center of the structure. This is as expected since the load is concentrated in the center the building, resulting in a gradual dish-sha settlement profile. Crack length or width di not increase, and no modifications were made the structural engineer.

CONCLUSIONS

1. A mat foundation with 10-foot long driven piles (scour protection) was a viable alternate to 70-foot long driven piles and resulted in total settlements of about 1 3/4 inches.

2. Elastic solutions and modulus values are suggested in Table III for estimating settlements. It appears that all three methods of elastic solutions coupled with modulus values from Webb, 1969 or Schultze and Melzer, 1965 produce reasonable and conservative settlement numbers.

3. The modulus values suggested by Schmertmann, 1970 and Leary and Langan, 1982 overestimate settlement by a factor of 2 to 4. It appears these modulus correlations underestimate modulus values when the standard penetration value is below 10 blows per foot.

4. Building construction resulted in differential settlement between the center and edge of the mat. This mat curvature resulted in diagonal shear wall cracks. These cracks averaged about 0.03 inch in thickness. No structural changes were made as a result of these cracks.

5. The structure has been in service for three years. Structural and geotechnical building performance has been observed; no modifications in the original design were made.

ACKNOWLEDGMENTS

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