Ken Hankins, John Nixon, W.C. Garbade, A.O. Hilgers, T.J. Walthall, R.H. Magers, M.H. Hardy, M. Picornell, D.R. Snethen, R.D. Lockhart, Pablo De Arkos, and Rick Norwood.

REFERENCES

1. D.E. Jones and W.G. Holtz. Expansive Soils--The Hidden Disaster. Civil Engineering Magazine, Aug. 1973, pp. 49-51.
2. J.P. Krohn and J.E. Slosson. Assessment of Expansive Soils. Proc., 4th International Conference on Expansive Soils, Denver, Colo., ASCE, New York, Vol. 1, June 1980.
3. D.R. Snethen et al. A Review of Engineering Experiences with Expansive Soils in Highway Subgrades. Report FHWA-RD-75-48. FHWA, U.S. Department of Transportation, 1978.
4. E.B. McDonald. Experimental Moisture Barrier and Waterproof Surface. Report HR 0200(3645). South Dakota Department of Transportation, Pierre, Oct. 1973.
5. M.L. Steinberg. Ponding on Expansive Clay Cut: Evaluations and Zones of Activity. Transportation Research Record 641, TRB, National Research Council, Washington, D.C., 1978, pp. 61-66.
6. M.L. Steinberg. Deep Vertical Fabric Moisture Barriers in Swelling Soils. Transportation Research Record 790, TRB, National Research Council, Washington, D.C., 1981, pp. 87-94.
7. M. Picornell, R.L. Lytton, and M.L. Steinberg. Assessment of Effectiveness of a Vertical Moisture Barrier. 5th International Conference on Expansive Soils, Adelaide, Australia, 1984.
8. B.J. Dempsey and O.L. Robnett. Influence of Precipitation Joints and Sealing or Pavement Marriage. Transportation Research Record 705, TRB, National Research Council, Washington, D.C., 1979, pp. 13-23.
9. M.L. Steinberg. Horizontal Placement of a Geotextile on a Subgrade to Control a Swelling Soil. Research Report 187-9. Texas State Department of Highways and Public Transportation, Austin, Feb. 1983.

Publication of the paper sponsored by Committee on Environmental Factors Except Frost.

Long-Term Behavior of a Drilled Shaft in Expansive Soil

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ABSTRACT

A vertical load test was performed in November 1982 on an instrumented 30-in. diameter drilled shaft 36 ft long with a 4-ft underream. Shaft LAFB-2 was constructed at Lackland Air Force Base, Texas, during July 1966, in stiff, expansive clay soil that contained a perched water table in a clayey gravel stratum. Soil adjacent to LAFB-2 had heaved 7.7 in. at the ground surface by September 1981, while the shaft had heaved 3.4 in. Because soil within 3 ft of the shaft base had heaved only 1.8 in., LAFB-2 had stretched or fractured along the shaft length. Results of the vertical load test indicated a discontinuity in the load-displacement curve that separated the observed skin friction resistance of 250 tons from the end bearing resistance of 130 tons. Uplift thrust of the adjacent swelling soil mobilized skin friction and tensile forces in the shaft equivalent to the shear strength of the adjacent soil times the total shaft surface area. Long-term end bearing capacity was reduced to about 75 percent of short-term capacity because of long-term wetting of soil beneath the shaft base.

In July 1966 seven test shafts were constructed at Lackland Air Force Base to study the performance of drilled shafts in expansive soil. The test site was instrumented with porous stone piezometers, free-standing benchmarks, and two deep reference benchmarks to provide accurate pore water pressure and elevation profiles. Pressure heads recorded in the piezometers indicated a perched water table 8 ft below ground surface extending down to about 50 ft. A deep water table was observed 80 ft below ground surface.

A vertical load test was performed on shaft LAFB-2 in November 1982 to investigate the long-term performance of a drilled shaft in expansive soil. This 2.5-ft diameter by 36-ft long shaft, including a 4-ft diameter bell, is located in the southeast corner of a 100 x 100-ft covered area constructed in 1974 to observe trends in long-term heave in expansive soil beneath lightly loaded areas (1). LAFB-2 was instrumented with 21 strain gauges secured on steel reinforcement and distributed in groups of three at periodic intervals along the full shaft length. Steel reinforcement is 3 number 14S bars tied with number 4 bars at 12-in. centers for 6.75 in.^{2} or 1 percent of the shaft cross section. LAFB-2 was also instrumented with eight Carlson earth pressure cells that were mounted in specially prepared holes cut into undisturbed soil on opposite sides of the shaft perimeter at depths of 2.4, 6, 15, and 32 ft below ground surface. Compressive strength tests performed on 3-in. diameter concrete core samples taken from the adjacent shaft (LAFB-1) in 1982 at 3.8 and 11.5 ft deep indicated a Young's modulus of 216,00 tsf and compressive strength of about 400 tsf (2,3).

DESCRIPTION OF SOILS

The overburden material consists of about 8 ft of expansive black to gray CH clay and 4 to 5 ft of GC clayey gravel with caliche. The primary material encountered below the gravel is fissured expansive CH clay shale of the Upper Midway group of the Tertiary system. The soils are uniform within the test area. Laboratory tests to classify the soil and to evaluate strength and consolidation parameters were performed on 6-in. diameter undisturbed samples obtained between 1966 and 1982 by Shelby tube or piston samplers. Relatively undisturbed samples could not be obtained from the clayey gravel. Details of the results of these soil tests and site layout are described elsewhere (2,3).

HEAVE PROFILE

The soil heave profile measured from the freestanding benchmarks, PSTBM, shown in Figure 1 relative to July 1966 indicate the least measured vertical swell at the test site. Heave recorded at the ground surface by the PSTBM was about 3 in. in 1981. Soil heave of about 1 in. occurred less than 5 ft below ground surface, 0.9 in. between 15 and 25 ft in the fissured clay shale of the Upper Midway group, 0.6 in. from 25 to 40 ft, and 0.4 in. below 40 ft. Soil adjacent to LAFB-2 had heaved 7.7 in. by 1981, considerably more than the freestanding benchmarks. Soil near the base of shaft LAFB-2 at 35 ft had also heaved about twice that of the 35-ft benchmark. Shaft LAFB-2 had heaved 3.4 in. by 1982, much more than the 1.8 in. of soil heave observed near the shaft base; therefore, the shaft appeared to stretch 1.6 in. Because LAFB-2 is underreamed, it may have fractured at one or more locations along the shaft length.

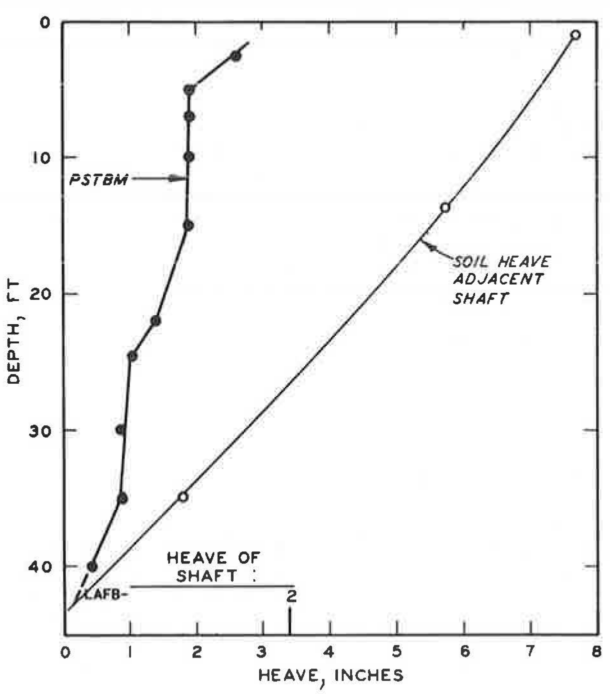


FIGURE 1 Heave profile of LAFB-2 in September 1981.

THEORETICAL MODELS

Vertical axial loads are resisted by skin friction along the shaft-soil interface and by end bearing capacity of the soil beneath the shaft base. The ultimate capacity Q_u of the shaft is commonly given by

$$Q_u = Q_{su} + Q_{bu} \tag{1}$$

$$Q_{su} = \pi D_s \int_0^L f_{\bar{s}}\, dL \tag{2}$$

$$Q_{bu} = q_{bu} A_b \tag{3}$$

where

Q_{su} = ultimate mobilized skin resistance, tons;
Q_{bu} = ultimate end bearing resistance, tons;
D_{su} = shaft diameter, ft;
$f_{\bar{s}}$ = full mobilized skin friction, tsf;
dL = increment of shaft length L, ft;
q_{bu} = ultimate base resistance pressure, tsf; and
A_b = base area, ft^2.

The ultimate capacity Q_u is normally reduced to an allowable bearing capacity Q_a for design by selection of suitable factors of safety.

End Bearing Capacity

Shaft foundations normally fail by punching shear in which the shear surface is not well defined and failure is progressive with continuing downward movement or punching of the soil. The end bearing capacity, q_{bu}, for deep foundations may be given by

$$q_{bu} = cN_c + \sigma_v' N_q \tag{4}$$

where c is the cohesion, tsf; σ_v' is the effective vertical overburden pressure, tsf; and N_c, N_q is the bearing capacity factors. Factor N_c equals 9 and N_q equals 1 for total stress (undrained) analysis. σ_v is equally ignored to compensate for the shaft weight. The cohesion c equals the undrained shear strength C_u.

The bearing capacity factor N_q for effective stress (drained) analysis has been evaluated by a variety of useful procedures in which cohesion is normally ignored. Two procedures for local and general shear failure are

Local (4): $$N_q = (1 + \tan\phi')e^{\tan\phi'} \tan^2 (45 + \phi'/2) \tag{5a}$$

General (5): $$N_q = [e^{(270-\phi')\pi \tan\phi'/180}] \div [2\cos^2 (45 + \phi'/2)] \tag{5b}$$

where ϕ' is the effective friction angle. Settlements required to achieve ultimate end bearing vary widely, but practical estimates for drilled shafts are up to 25 percent of the shaft diameter.

Skin Friction

The development of skin friction f_s in deep foundations depends on the relative displacement between soil and shaft. The full mobilized skin friction $\bar{f_s}$ occurs at relative displacements much less than those usually required to achieve full end bearing and often less than 0.5 in. of total shaft head displacement. Cylindrical shear models are considered most appropriate for analysis of skin friction resistance as used in Equation 2. The magnitude of ultimate skin friction for practical design applications may be computed by the same cylindrical shear model for applied structural loads, pullout loads, downdrag forces from consolidating soil, and uplift thrust from swelling soil. The maximum uplift thrust developed in swelling soil is considered in this paper.

Methodology for evaluating $\bar{f_s}$ by undrained analysis is

$$\bar{f_s} = \alpha C_u \tag{6}$$

where α is a factor relating adhesion along the soil-shaft interface to the undrained cohesion C_u. The factor α appears to vary with the type of soil and is on the order of 0.5 for the stiff clays of this test site (6); however, α for modeling uplift thrust from expansive soil may be larger than 0.5 and could approach 1.0 because the soil expands tightly against the shaft perimeter over the full length of the swelling soil.

Methodology for evaluating the full mobilized skin friction by drained analysis is given by

$$\bar{f_s} = \beta \sigma_v' \tag{7}$$

where β is the lateral earth pressure and friction angle factor. Among the models available for evaluating β, the model (7)

$$\beta = K_o \tan \delta' \tag{8}$$

is commonly used where K_o is the coefficient of lateral earth pressure at rest. In practice the most appropriate effective friction angle δ' appears to be several degrees ($\approx$ 5 degrees) less than the measured angle ϕ' or about $0.8\phi'$.

The uplift thrust from swelling of expansive soil is expected to increase the actual coefficient of lateral earth pressure toward the passive coefficient K_p, which could be modeled by the Rankine passive state ignoring cohesion and $\delta' = \phi'$. The uplift thrust Q_{us} may also be modeled by (8,9)

$$Q_{us} = 1.3\pi D_s \int_0^L \sigma_s \tan \phi'_{residual} \, dL \tag{9}$$

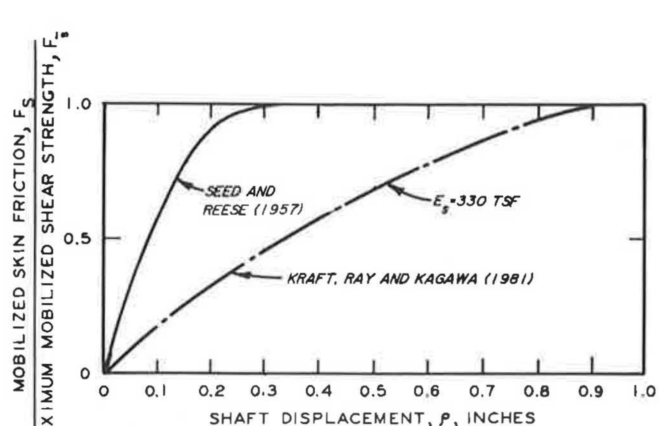
where σ_s is the swell pressure, tsf, and ϕ' residual is the residual effective friction angle, degrees. Possible explanation for the factor 1.3 in Equation 9 are that measured swell pressures may be less than actual swell pressures, and the friction angle may be between the peak and residual friction angle.

Load-Displacement Behavior

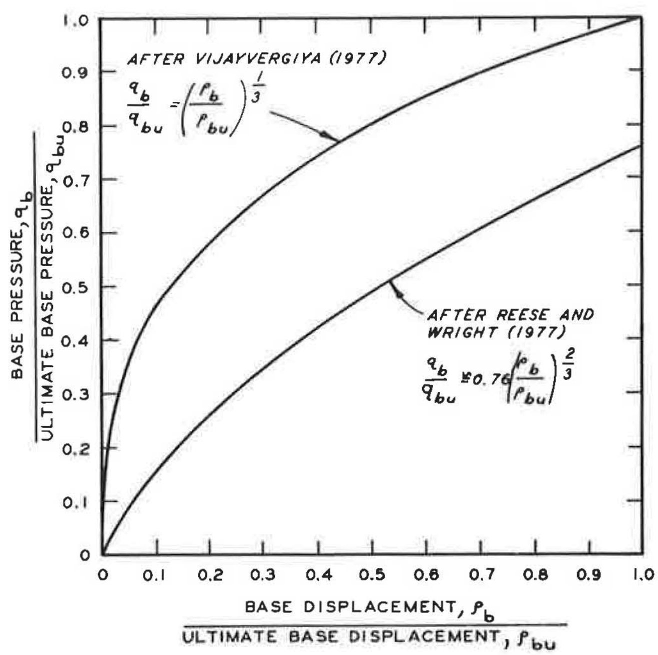
The load-displacement behavior may be estimated by a variety of techniques. Results of several elastic and load transfer models applied to LAFB-2 are described elsewhere (3). The finite element method using computer program AXIPLN (10) and load transfer functions programmed into a computer code AXILTR (2) are applied in this paper to evaluate load-displacement behavior of LAFB-2. Load transfer functions programmed into AXILTR are discussed next.

Shaft Load Transfer Functions

The load transfer function or t-z curve defined by Seed and Reese (11) uses a shape function that depends on the type of soil. Experimental data (11) indicated the load transfer (or normalized t-z) function for normally consolidated soil shown in Figure 2a. Kraft, Ray, and Kagawa (12) also developed a load transfer hyperbolic t-z function after Randolph and Wroth (13) that leads to the normalized



a. SHAFT TRANSFER (NORMALIZED t-z) FUNCTIONS



b. BASE TRANSFER (NORMALIZED q-z) FUNCTIONS

FIGURE 2 Normalized load transfer functions.

load transfer function (Figure 2a) for the following parameters consistent with shaft LAFB-2:

Soil Young's modulus,
E_s = 330 tsf
Soil Poisson's ratio,
ν_s = 0.3
Soil shear modulus,
G = 127 tsf
Soil shear strength,
$\bar{f_s}$ = 1.5 tsf

Shaft diameter,
D_s = 2.5 ft
Shaft length,
L = 36 ft
Curve fitting constant, R_f = 0.96

This load transfer function is less stiff than the empirical function of Seed and Reese (11) for the preceding parameters.

Base Load Transfer Functions

Transfer functions have also been developed for load transfer to the soil beneath the shaft base. Figure 2b shows two (normalized q-z) base functions after Reese and Wright (14) and Vijayvergiya (15). The ultimate base settlement ρ_{bu} has been related to the strain in laboratory tests by (14)

$$\rho_{bu} = 2D_b\varepsilon_{50} \qquad (10)$$

where D_b is the base diameter in feet and ε_{50} is the strain at 1/2 maximum deviator stress of undrained Q triaxial test. Confining pressure during the Q test was not specified. In the interest of simulating in situ conditions, the confining pressure should be similar to the in situ soil overburden pressure. Vijayvergiya (15) assumed ρ_{bu} is about 4 to 6 percent of the base diameter, D_b. The ultimate base pressure, q_{bu} (Figure 2b) is taken as $9C_u$ where C_u is the undrained shear strength. Figure 2b shows that the data after Vijayvergiya (15) indicate substantially stiffer soil than the mean data after Reese and Wright (14).

RESULTS AND ANALYSIS OF VERTICAL LOAD TEST

The vertical load test was performed in November 1982 using a 1,250-ton capacity loading frame supported by two 3-ft diameter shafts 36 ft long with 7-ft, 6-in. diameter bells. Each anchor shaft contained ten 1 3/8-in. diameter, 150,000 psi high-strength bars equally spaced around a 28-in. diameter ring. Vertical loads were applied with a calibrated and electronically operated loading jack of 1,200 ton capacity. Displacements were measured by two dial gauges, sensitive to 0.001 in., positioned on each side of the test shaft and mounted on a wood frame independent of the shaft. Backup ruler and wire gauges were also in position. The load test was conducted in accordance with ASTM Standard Test Method D1143 (16).

Results of Load Test

The shaft experienced an intermediate plunging failure at 250 tons causing rapid displacement to 1.5 in. (Figure 3). This intermediate failure was attributed to applied loads exceeding the maximum skin resistance and the presence of a void or soft soil, or both, beneath the base. Because LAFB-2 had heaved

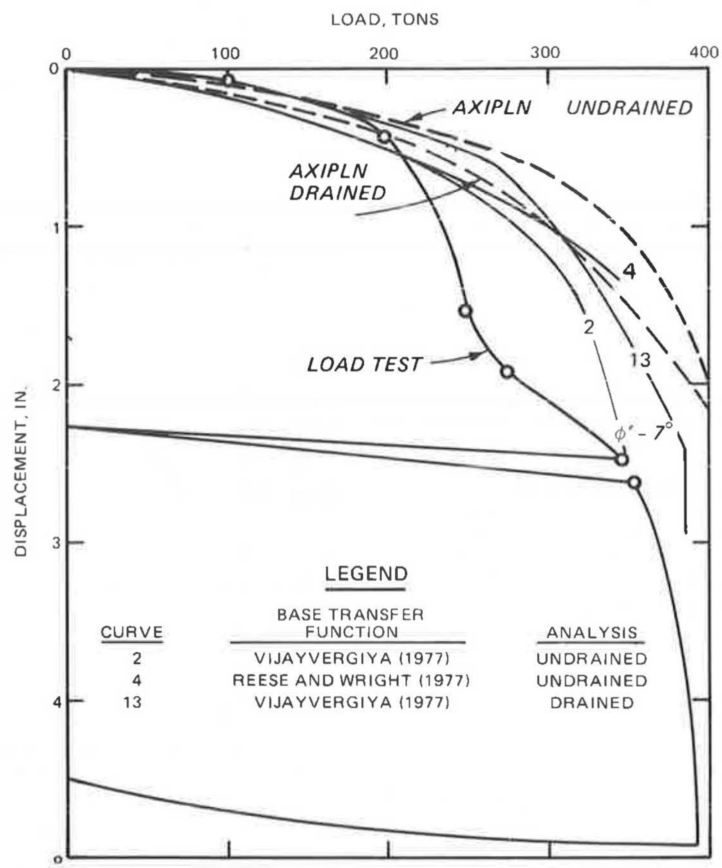


FIGURE 3 Comparison of observed with calculated load-displacement behavior by programs AXILTR and AXIPLN.

1.6 in. more than the soil had heaved near the vicinity of the base, a gap had occurred in the shaft and could have been caused by tensile fracture near the base. The strain gauge data (2) indicated negligible elastic modulus in the shaft near the shaft base and the strain gauge data also had shown that the shaft had compressed about 1 in. during the load test, mostly below 20 ft. Therefore, a fracture in the shaft appears to have existed near the base, and 0.6 in. of possible space may have existed beneath the base. The shaft held an additional 130 tons after the intermediate plunging failure, which is attributed to the end bearing capacity. The failure load of 380 to 400 tons was maintained for 8 hr with a creep rate of 0.001 in./min. Loads exceeding 400 tons significantly increased the creep rate and could not be maintained.

An analysis of the skin friction distribution with depth (2,3) indicated that the α factor at maximum skin resistance relative to the undrained shear strength is about 0.85 and 1.0 from 0 to 16 ft and 16 to 34 ft, respectively (Figure 4). These α factors are larger than those expected in nonswelling soil, indicating intimate contact at the soil-shaft interface. The skin friction distribution is bounded by the Seed and Reese (11) and Kraft, Ray, and Kagawa (12) load transfer models. The large magnitude of skin friction observed over the full length of the shaft indicates that any fracture in the shaft must have occurred near the base. Strain gauge readings (2,17) show that large tensile strains had developed in the lower portion of the shaft.

The maximum end bearing load, Q_{bu}, was 130 tons at a displacement of 4.9 in. A plot of the normalized base transfer q-z functions in Figure 5 shows that the load test results are between the functions after Vijayvergiya (15) and Reese and Wright (14)

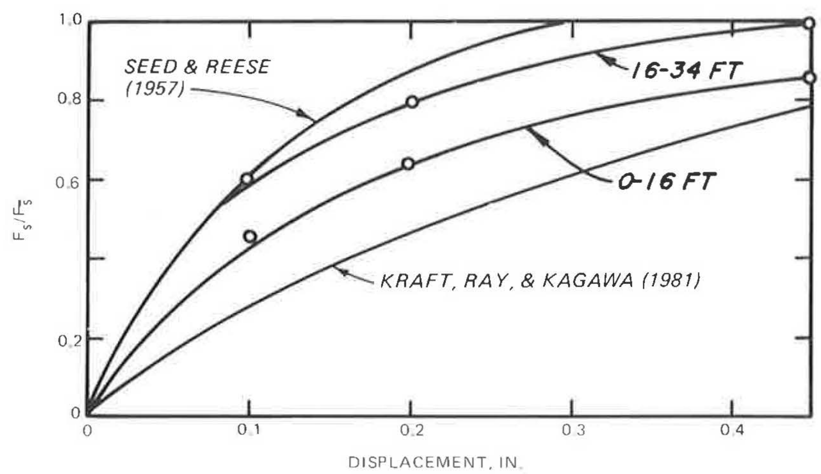


FIGURE 4 Normalized t-z skin friction curves of shaft LAFB-2.

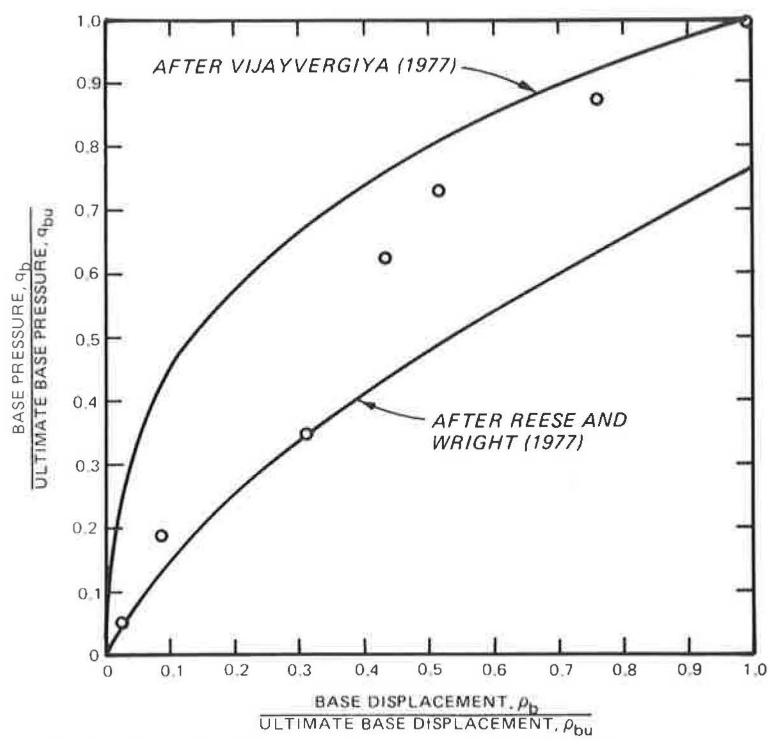


FIGURE 5 Normalized base transfer q-z curve for shaft LAFB-2.

shown in Figure 2b. The ultimate base displacement, ρ_{bu}, estimated by Equation 10 is 0.6 in., which is much less than observed even excluding the 1.5 in. of rapid drop at 250 tons. The value of ρ_{bu} after the criterion of 4 to 6 percent of D_b (15) is 2 to 3 in., which reasonably simulates the observed ρ_{bu} if 1 to 2 in. of displacement are subtracted from the observed 4.9 in. to compensate for crack closure in the shaft.

The skin friction model used to calculate curves 2, 4, and 13 in Figure 3 was after the load-transfer skin friction model of Kraft, Ray, and Kagawa (12). The general shear Equation 5b was used to compute ultimate base capacity, Q_{bu}, for the drained analysis of curve 13, and N_c = 9 was used to compute Q_{bu} for the undrained analysis. The shaft modulus was 108,000 tsf, simulating a long-term concrete modulus. The input parameters used in programs AXIPLN and AXILTR are given in Tables 1 and 2. The calculated curves tend to underestimate displacements for loads exceeding mobilization of skin friction. Skin friction parameter α = 0.45 for undrained analysis of curves 2 and 4 is too low. The finite element drained (C_u = 0) and undrained (ϕ' = 0) results from AXIPLN; both overestimate end bearing capacity similar to results of AXILTR.

Ultimate Capacity

End bearing capacity, Q_{bu}, for undrained analysis using N_c = 9 is 170 tons, which exceeds the actual end bearing capacity by 40 tons. A bearing capacity factor N_c = 7 reasonably consistent with a recommended value of 7.4 (18) properly simulates end bearing, Q_{bu}, for drained analysis using ϕ' = 27 degrees, and the general shear model for N_q grossly overestimates end bearing at 278 tons. N_q approximated by local shear failure from Equation 5a reasonably simulates actual end bearing resistance.

Skin resistance was underestimated for α = 0.45 and should have been near 1.0 for undrained analysis consistent with results in Figure 4. Skin resistance for drained analysis is properly calculated using the measured effective friction angle ϕ' from results of $\bar{R}$ tests with pore pressure measurements and a coefficient of lateral earth pressure K approaching 3.0. The measured earth pressure distribution on LAFB-2 (2,3) indicates lateral pressures about three times the vertical pressure. The Rankine friction model or the swell pressure model given by Equation 9 using ϕ' residual = 9 degrees and 1966 swell pressures both lead to skin frictions comparable to those measured during the 1982 load tests.

CONCLUSIONS

The α factor for skin friction resistance by total stress (undrained) analysis in expansive soil is larger than may occur in nonswelling soils. It can approach 1.0 over the entire length of the shaft subject to lateral thrust from swelling soil.

End bearing capacity for LAFB-2 was less than expected and approximately given by N_c = 7 for undrained analysis and N_q given by local shear failure for drained analysis. The reduced end bearing capacity is attributed to shaft heave, which lifted the base off the soil contributing to a possible void and soil heave with softening beneath the base and shaft fracture near the base.

Load-displacement behavior may be modeled by a variety of methods of which load transfer functions appear practical. Skin friction was bounded by the Seed and Reese (11) and Kraft, Ray, and Kagawa (12) models and end bearing by the Vijayvergiya (15) and Reese and Wright (14) models.

Drilled shaft foundations for supporting permanent structures in expansive soil should be loaded near the allowable bearing capacity assuming a minimum factor of safety consistent with good engineering practice. The amount of reinforcement steel required to resist uplift thrust caused by swelling soil should be based on the maximum shear strength of the adjacent soil. Reinforcement should be full length extending into any existing underream and well secured in the underream. Elements of deep foundations

TABLE 1 Soil Parameters for Analysis of Vertical Load-Displacement Behavior, AXILTR

Depth, ft	Specific Gravity	Poisson's Ratio	Void Ratio	Water Content (%)	Shear Strength Parameters C_u, tsf	ϕ', deg.	K_o	Soil Modulus, tsf
0-10	2.72	0.3	0.90	29.0	0.8	25	0.7	150
10-20	2.75	0.3	0.90	29.0	0.8	27	0.7	150
20-34	2.77	0.3	0.88	30.5	1.5	34	1.5	400
34-40	2.77	0.3	0.88	30.5	1.5	35	1.5	700

Note: Concrete modulus = 108,000 tsf; ϵ_{50} = 0.006; R_f = 0.97; soil shear modulus = 58 tsf; a = 0.6.

TABLE 2 Soil Parameters for Analysis of Vertical Load-Displacement Bheavior, AXIPLN

Depth, ft	Poisson's Ratio	Unit Weight tons/ft^3	Shear Strength Parameters C_u tsf	ϕ' deg	K_o	Hyperbolic Parameters R_f	K_i	K_{ui}	n
0-10	0.3	0.059	0.8	18	0.7	0.97	190	380	0.64
10-20	0.3	0.059	0.8	20	0.7	0.97	190	380	0.64
20-80	0.2	0.060	1.5	28	1.5	0.90	380	760	0.28
Concrete	0.2	0.075	1.0						
Interface element	0.495		0.8	20		0.97			0.64

Note: Concrete modulus = 108,000 tsf, K_j = 6440.

should not penetrate through perched water tables into deeper desiccated soil if practical.

ACKNOWLEDGMENT

This load test and analysis were supported by a grant to the Waterways Experiment Station from the Office, Chief of Engineers, and conducted in cooperation with the U.S. Army Engineer District, Fort Worth, Texas.

REFERENCES

1. L.D. Johnson. Field Test Sections on Expansive Soil. Proc., 4th International Conference on Expansive Soils, Denver, Colo., ASCE, New York, 1981.
2. L.D. Johnson. Methodology for Design and Construction of Drilled Shafts in Cohesive Soils. Technical Report GL-84-5. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., 1984.
3. L.D. Johnson and W.R. Stroman. Vertical Behavior of Two 16-Year Old Drilled Shafts in Expansive Soil. Analysis and Design of Pile Foundations. J.R. Meyer, ed., ASCE, San Francisco, Calif., 1984, pp. 154-173.
4. A.S. Vesic. Design of Pile Foundations. NCHRP Synthesis of Highway Practice 42, TRB, National Research Council, Washington, D.C., 1977.
5. J.E. Bowles. Foundation Analysis and Design. McGraw-Hill, New York, 1968, pp. 607-609.
6. L.C. Reese, F.T. Touma, and M.W. O'Neill. Behavior of Drilled Piers Under Axial Loading. Journal of the Geotechnical Engineering Division, ASCE, Vol. 102, No. GT5, 1976, pp. 493-510.
7. R.J. Chandler. The Shaft Friction of Piles in Cohesive Soils in Terms of Effective Stress. Civil Engineering and Public Works Review, Vol. 63, 1968, pp. 48-51.
8. W.P. Jobes and W.R. Stroman. Structures on Expansive Soil. Technical Report M-81. Construction Engineering Research Laboratory, Champaign, Ill., 1974.
9. M.W. O'Neill and N. Poormoayed. Methodology for Foundations on Expansive Clays. Journal of the Geotechnical Engineering Division, ASCE, Vol. 106, No. GT12, 1980, pp. 1345-1357.
10. J.L. Withiam and F.H. Kulhawy. Analytical Modeling of the Uplift Behavior of Drilled Shaft Foundations. Report B-49(3). School of Civil and Environmental Engineering, Cornell University, Ithaca, N.Y., Niagara Mohawk Power Corporation, Syracuse, N.Y. 1978.
11. H.B. Seed and L.C. Reese. The Action of Soft Clay Along Friction Piles. Transactions ASCE, Vol. 122, pp. 731-753.
12. L.M. Kraft, R.P. Ray, and T. Kagawa. Theoretical t-z Curves. Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, No. GT11, 1981, pp. 1543-1561.
13. M.F. Randolph and C.P. Wroth. Analysis of Deformation of Vertically Loaded Piles. Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT12, 1978, pp. 1465-1488.
14. L.C. Reese and S.J. Wright. Drilled Shaft Design and Construction Guidelines Manual, Construction of Drilled Shafts and Design for Axial Loading. Vol. I, FHWA, U.S. Department of Transportation, 1977.
15. V.N. Vijayvergiya. Load-Movement Characteristics of Piles. Proc., Port '77 Symposium, Long Beach, Calif., ASCE, New York, 1977, pp. 269-284.
16. Piles Under Axial Compressive Load. ASTM D 1143, Part 19, ASTM, Philadelphia, Pa., 1981.
17. Investigations for Building Foundations in Expansive Clays. Vols. I and II, U.S. Army Engineer District, Fort Worth, Texas, 1968.
18. Procedures for Foundation Design of Buildings and Other Structures (Except for Hydraulic Structures). Technical Manual 5-818-1. Department of the Army, Washington, D.C., 1983.

Publication of this paper sponsored by Committee on Environmental Factors Except Frost.