Comparison of Measured and Computed Drilled Shaft Capacities Based on Utah Load Tests

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Fourteen load tests were performed at a number of bridge sites in Utah in an effort to reduce foundation costs. Soil profiles generally consisted of cohesionless materials. Failure loads were determined from load-settlement data using four methods, including Davisson's method, double tangent method, 1 in. criterion, and the Hirany and Kulhawy's method. Hirany and Kulhawy's method typically failure loads 30 percent higher on the average than Davisson's method. Load capacities were computed using equations proposed by Reese and O'Neill/FHWA, Meyerhof, and Reese et al. Good agreement between measured (Davisson's method) and computed capacities was generally found for the FHWA and Meyerhof methods, but the Reese et al. method significantly underpredicted capacity. The design methods were generally more than 25 percent conservative in comparison with the Hirany and Kulhawy failure load interpretation. In three cases involving dense gravels and weathered shale the design equations resulted in computed capacities that were only 20 to 30 percent of the measured failure load.

In an effort to reduce construction costs, the Utah Department of Transportation has performed a number of load tests on drilled shafts. Since the measured capacities were generally higher than estimates based on design equations, it was possible to increase design loads for each shaft. The costs of load testing proved to be insignificant when compared with savings that were realized by reducing the size and number of drilled shafts needed. It is estimated that the testing program resulted in cost savings of more than $890,000 in the first 2 years. Unfortunately, load testing cannot be performed for each project, and reliance must be placed on load prediction equations. Reasonable capacity prediction equations are necessary to provide safe yet economical shaft foundations for highway structures.

OBJECTIVES AND SCOPE OF INVESTIGATION

Since the inception of the Utah load testing, Reese and O'Neill (1) prepared a drilled shaft handbook with new design recommendations for nationwide use. This work was prepared for the Federal Highway Administration in cooperation with the Association of Drilled Shaft Contractors. In addition, a new procedure for interpreting axial load test data was proposed by Hirany and Kulhawy (2).

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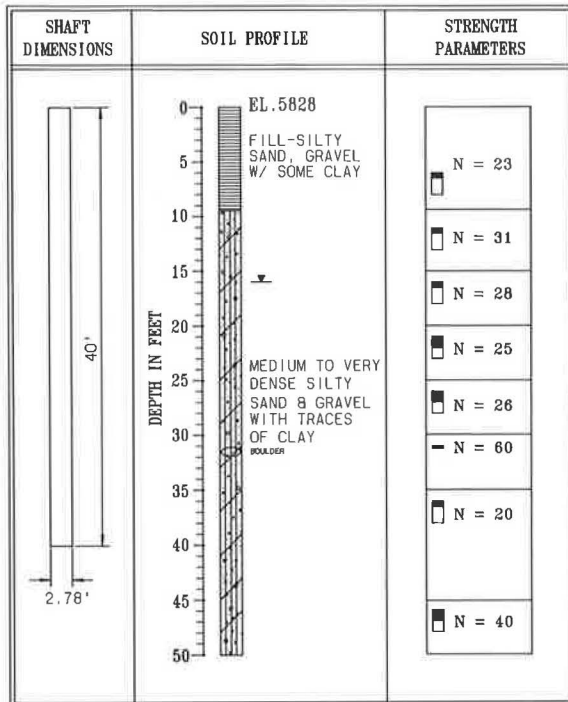
It was determined that a review and summary of the existing load test information would be valuable for (a) establishing a computer data base on the performance of drilled shafts in Utah that could be updated with future tests, (b) evaluating the variation in load capacity on the basis of various load test interpretation procedures, (c) determining the suitability of various design equations for Utah soil conditions, and (d) making recommendations for future load testing procedures.

Since the load test data set was not used in the development of any of the design methods, it provides an independent check on the various design methods proposed for use. Load test data can easily be compared with calculated values based on design methods. By comparing load test data with these methods, one can better determine which method is more applicable for a given circumstance.

In this study, four methods were used for determining failure load from the load versus settlement test data. The procedures are Davisson's Method (3), the double tangent method (4,5), Hirany and Kulhawy's Method (2), and Terzaghi's simple procedure (6), which defined failure at a settlement of 1 in. The results from each interpretation procedure were compared with three prediction methods. The load prediction equations included those proposed by Reese and O'Neill (1), Meyerhof (7), and Reese et al. (8).

DESCRIPTION OF DATA BASE CHARACTERISTICS

A total of 14 load tests were available for evaluation, and additional tests are presently under way. In each case the function of the foundation was to support a bridge or overpass.

Geotechnical Conditions

The subsoils at the test sites generally consisted of granular materials deposited by stream flow along with some man-made fill. Clay layers were encountered in only two of the tests. Subsoils encountered at these sites ranged from very loose to very dense sand and gravel with some silt and cobbles. SPT N values ranged from 10 to 80 but were typically between 20 and 40 blows/ft. A fairly typical boring log in the alluvial deposits is shown in Figure 1. Most of the shaft tips were founded in soil; however, in three cases, the shaft tips were

FIGURE 1 Typical boring log in granular alluvial soils at Test Site 6, F-493-North.

holes were excavated with rotary drilling machinery. Excavations in granular soils below the water table were carried out using casing, which was subsequently removed. In most cases, however, the dry method of construction was used. The top of each test shaft was finished smooth and level to within $\pm \frac{1}{32}$ in. The unconfined compressive strength, f'_c, was typically between 4,000 and 5,000 psi. Table 1 gives the location and dimensions of the test shafts. Where concrete volumes were known, an equivalent diameter was determined on the basis of the volume of concrete placed, and this diameter was used in subsequent calculations. Whereas this procedure is an approximation, it accounts for overages in excavation diameter assuming that the shaft wall is relatively uniform with depth.

Load Testing Procedure

Hydraulic load cells were used to apply the desired loads, and the load frame was capable of applying loads of just over 1,100 tons. Each test shaft was placed between two larger reaction shafts that anchored the load frame. The average of two extensometer readings was used to monitor settlement at the top of the shaft.

Test-Shafts F-493-South and F-501-1 were loaded in 25-ton increments at 3-min intervals. All three of the C-630 shafts were loaded in 50-ton increments at 2.5-min. intervals, and the remainder were loaded in 50-ton increments at 3-min. intervals. Settlement readings were taken immediately before increasing the load. Test shafts with the word "skin" on the end of the location are specially cast shafts with compressible material at the tip to prevent end bearing, thus measuring skin resistance only.

Instrumentation

To evaluate the total load being carried at different depths along the shaft, vibrating wire strain gauges were installed at regular depth intervals in several of the test shafts. The gauges were attached to the rebar cage in sets of 3 at each depth and protected. Figure 2 shows the load in the pile as a function

terminated in shale bedrock. Borehole logs showing soil type and groundwater elevation were available for each structure along with SPT N values, gradations, and limited shear test results. Unconfined compressive strength data were obtained for the shafts founded in shale. At Test Site 10, the unconfined compressive strength of the shale was between 1 and 2 tsf, whereas at the site for Tests 11 and 12 the compressive strength varied from 300 to 3,000 psi.

Drilled Shaft Types and Construction

Each drilled shaft was cast-in-place portland cement concrete with reinforcement provided by rebar cages. In each case the

TABLE 1 SUMMARY OF DRILLED SHAFT GEOMETRIES INCLUDED IN THE DATA BASE

Ref.	Structure	Effective Depth (Ft.)	Diameter (Ft.)	Equivalent Dia. (Ft.)
1	F-435-East	23.3	2	2.09
2	F-435-West	20.6	2	2.08
3	F-438	40.8	3	3.08
4	F-489	40	2	
5	F-489-Skin	40	2	
6	F-493-North	40	2.5	2.78
7	F-493-South	40	2.5	3.14
8	F-495-North	40	2.5	2.62
9	F-495-South	40	2.5	3.82
10	F-501-1	33	2	
11	F-501-2	33	2	
12	C-630Skin	14	2	
13	C-769-1	38.5	3	
14	C-769-2	28	3	
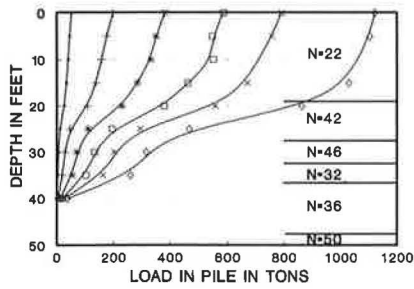
FIGURE 2 Load in pile versus depth below ground for Load Test 5, F-489, SKIN.

of depth for Load Test 5 along with the SPT N values at each depth. This shaft was designed to have skin friction, and this is borne out by the strain gauge data. Whereas a few of the strain gauge readings provided useful information, the majority were unreliable and exhibited increasing inconsistency with increasing load. Readings made little to no sense when the applied load approached 400 tons, which was well before failure. The cause of the problem is undetermined.
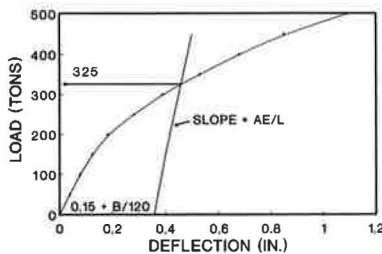
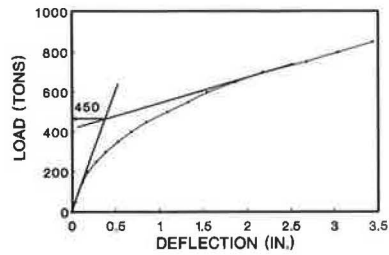## LOAD TEST INTERPRETATION

Description of Methods Used

To evaluate the predictions of ultimate capacity given by the various design methods, the ultimate capacity must first be determined from the field load test. Four methods for interpreting the ultimate load were used: Davisson's double tangent, Hirany and Kulhawy, and Terzaghi's 1 in. of settlement criteria.

Davisson's Method

The Davisson method (*3*) is a graphical method that defines the ultimate capacity as that load corresponding to the settlement that equals the elastic compression of the shaft plus 0.15 in. plus a factor equal to the diameter of the shaft in inches divided by 120. In computing the elastic compression, the modulus of elasticity, E_c, of the concrete was evaluated using the equation $E_c = 57,000(f'_c)^{0.5}$ (psi), and the shaft area was based on the effective diameter. Figure 3 shows an example of this method using the data from Load Test 1.

FIGURE 3 Failure load interpretation using Davisson (1972) method.

FIGURE 4 Failure load interpretation using double tangent method.

Double Tangent Method

The double tangent method (*4*) is a graphical method that defines the ultimate capacity as the load corresponding to the intersection of lines drawn tangent to the initial and failure portions of the load-settlement curve. This method involves subjectivity in determining the failure slope. An example of the method for Load Test 1 is shown in Figure 4.

One in. of Settlement

A typical parameter for design of shallow foundations is to limit settlement to 1 in. Terzaghi (*6*) also suggested that this value be used in determining the ultimate capacity of drilled shaft foundations. In evaluating the load test data, defining ultimate capacity as the load corresponding to 1 in. of settlement was also considered.

Hirany and Kulhawy's Method

After a detailed review of 41 methods proposed in the literature for determining failure loads from load tests, Hirany and Kulhawy (*6*) proposed that the ultimate capacity be defined at the beginning of the final straight line portion of the load-displacement curve. This point was found to correspond to a displacement of 4 percent of the shaft diameter, and this value has been used as the failure criterion. Failure displacements would therefore be defined at displacements of 1 and 2 in. for shaft diameters of 2 and 4 ft. Figure 5 shows an example of this method, again using the data from Test 1. Hirany and Kulhawy indicate that if the failure load is divided by a factor of safety of 2, the average displacement is about
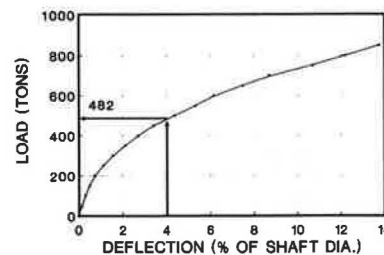
FIGURE 5 Failure load interpretation using Hirany and Kulhawy (1989) method (4 percent of shaft diameter).

TABLE 2 SUMMARY OF MEASURED AND COMPUTED CAPACITIES
USING VARIOUS METHODS FOR EACH DRILLED SHAFT IN THE
DATA BASE

SUMMARY SHEET								
Test Locations		Load Test Results				Prediction Methods		
		(Tons)				(Tons)		
Ref	Site	Hirany and Kulhawy	Davisson	Double Tangent	1*	FHWA	Reese	Meyerhof
1	F-435-E	490	325	450	480	244	162	262
2	F-435-W	355	220	200	355	226	150	339
3	F-438	1100	905	730	1000	716	382	811
4	F-489	1020	995	570	1100	302	238	226
5	F-489SKIN	>1100	>1100	610	>1100	234	134	76
6	F-493-N	580	365	390	500	395	295	330
7	F-493-S	650	445	480	570	495	336	464
8	F-495-N	505	325	325	455	265	158	328
9	F-495-S	915	565	535	710	533	376	561
10	F-501	>1100	>1100	625	>1100	291	205	372
11	C-630SKIN	1010	645	355	1100	662	N.A.	N.A.
12	C-630	1015	600	355	1100	837	N.A.	792
13	C-769-1	770	550	295	680	443	333	289
14	C-769-2	520	338	280	455	262	262	232

0.4 percent of the shaft diameter or about 10 percent of the failure displacement. This method is simple and non-subjective, yet it takes into account the shape of the load-displacement curve. It does not, however, account for elastic compression of the shaft. This factor is not critical for the Utah data base, since all shafts are less than 40 ft long.

Comparison of Methods

A summary of the ultimate capacity as determined by each of the four methods is given in Table 2. In addition, a comparison of the average ultimate capacity for each method is shown graphically in Figure 6. On the average, the double tangent method yielded the lowest failure load and was about 70 percent of the average Davisson failure load. The 1-in. criterion, on the other hand, yielded a failure load 26 percent higher on average than the Davisson failure load. The capacity values obtained with the 1-in. settlement criterion and the Hirany and Kulhawy method were typically within about 10 percent of one another. The agreement of the last two methods results because the Hirany and Kulhawy failure criterion
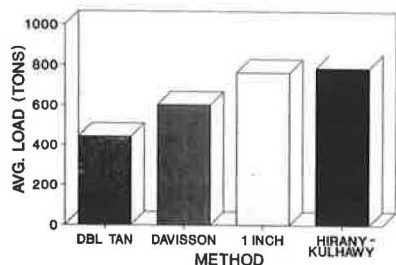

FIGURE 6 Comparison of average load determined by various failure load interpretation methods.

(4 percent of the shaft diameter) is about equal to 1 in. for the shaft diameters under consideration. The primary reason for the higher loads with the latter two methods is that the methods define failure at a somewhat higher deformation. In two cases, failure as defined by all but the double tangent method was not achieved, although the total applied load exceeded just over 1,100 tons. In these two cases the double tangent method estimated the failure load at around 600 tons, which is a conservative estimate considering that deformations were very low at this load.

If the failure load interpreted with the Hirany and Kulhawy 4 percent criterion is divided by a factor of safety of 2, the allowable load typically corresponded to a deformation of 1 percent of the shaft diameter. This is about 2.5 times greater than the 0.4 percent average reported for the data base used by Hirany and Kulhawy (2).

DESIGN METHODS

Three design methods were used to predict ultimate capacities: Reese and O'Neill/FHWA, Meyerhof, and Reese. Each method for computing ultimate capacity, Q_u, uses the following basic equation:

$$Q_u = Q_s + Q_b \tag{1}$$

where Q_s equals shaft resistance and Q_b equals base or tip resistance.

Reese and O'Neill/FHWA Method

The Reese and O'Neill/FHWA method (1) is a semiempirical method developed by Reese and O'Neill on the basis of a

large data base of drilled shaft load tests. The method establishes design procedures for shafts in sands, clays, and rock. Side resistance in clay is calculated by multiplying the product of the shear strength and surface area by the factor α, whereas side resistance in sand is calculated by multiplying the product of the surface area and the average effective stress by a factor β. The factor α is 0.55 for all points along the shaft except from ground surface to a depth of 5 ft, and the bottom one diameter of the drilled shaft or one stem diameter above the top of the bell, where α is zero.

The factor β is obtained from the equation

$$\beta = 1.5 - 0.135\sqrt{z} \qquad 1.2 \geq \beta \geq 0.25 \tag{2}$$

where z is the depth below the ground surface in feet. Limiting values of load transfer in side resistance have been set at 2.75 tsf for clays and 2.0 tsf for sands. These limits are not theoretical but are the largest values that have been measured. It is interesting to note that the β factor is independent of sand density or shear strength.

The ultimate unit end bearing pressure, q_b, for drilled shafts in saturated clay is calculated using the equation

$$q_b = N_c \cdot c_{ub} \qquad q_b \leq 40 \text{ tsf} \tag{3}$$

In Equation 3,

$$N_c = 6.0 \cdot \left[1 + 0.2 \cdot \left(\frac{\text{length}}{\text{base diam.}} \right) \right] \qquad N_c \leq 9 \tag{4}$$

and c_{ub} is the average undrained shear strength of the clay computed one to two diameters below the base.

The net ultimate unit end bearing pressure, q_b, for drilled shafts in sand (tsf) is 0.6 times the N_{SPT} (uncorrected), with a limit of 45 tsf. When base diameters exceed 50 in., q_b is reduced by a factor of q_b divided by the base diameter.

Meyerhof's Method

The Meyerhof method (7) is an empirical procedure based on load test data and allows computation of capacity in sands and clays. The ultimate unit skin friction f_s of drilled shafts in sands is computed using the equation

$$f_s = \frac{N}{100} \leq 0.5 \text{ tsf} \tag{5}$$

where N is the standard penetration blow count along the shaft. This is half of the skin friction specified for driven piles in sands. Skin friction in clays is taken as zero when the base is resting on soil significantly stiffer than the soil around the stem. For shafts in soils with no soil of exceptional stiffness below the base, the average cohesion is reduced by a factor of 0.15 to 0.6 and applied to the area of the shaft 5 ft beneath the ground surface to 5 ft above the base or top of bell.

The ultimate tip bearing pressure, q_u, in tsf is calculated with the following equation:

$$q_u = \frac{0.133 \cdot \overline{N} \cdot D}{B} \leq q_l \tag{6}$$

where

$\overline{N} = C_N * N$,
N = standard penetration resistance (blow/ft) near shaft tip,
$C_N = 0.77 \log_{10} 20/p$ (for $p \geq 0.25$ tsf),
p = effective overburden stress at shaft tip (tsf),
D = depth drilled into granular bearing stratum (ft),
B = width or diameter of shaft (ft), and
q_l = limiting point resistance (tsf), equal to $1.33N$ for sand and N for nonplastic silt.

According to Meyerhof, the ultimate tip resistance for driven piles in sands is three times the value allowed for drilled shafts in similar materials. Tip resistance values for clays are taken as 9 times the undrained strength near the base as with the other methods.

Reese et al. Method

The Reese et al. method (8) is a forerunner of the FHWA method discussed previously. This method is felt to be somewhat conservative relative to the newer FHWA method. In clay, the skin friction is obtained by multiplying the undrained strength by an α factor, which varies from 0 to 0.5 depending on the method of construction. For skin friction in sand, Reese multiplies the average effective stress by a lateral pressure factor K and by the tangent of the friction angle of the sand. K is 0.7 from the ground surface to a depth of 25 ft, 0.6 from 25 to 40 ft, and 0.5 for depths exceeding 40 ft.

End bearing in clay is computed as it is with the FHWA method. In sand the following equation is used:

$$Q_p = \frac{q_p}{\alpha_p} A_p \tag{7}$$

where

q_p = 0 for loose sand, 16 tsf for medium dense sand, and 40 tsf for dense sand; and
α_p = base reduction factor to limit base settlement to 1 in.
$= 0.6B$ (B in feet).

Capacity Computation Results

A simple spreadsheet program was developed to facilitate computations for each shaft and reduce the potential for errors. Figure 7 shows a typical spreadsheet printout containing soil profile information, capacity estimates for each method, and various factors that were used in the calculations. Table 2 presents a summary of results for each of the methods used. A review of this table indicates that Reese's method generally yields the lowest estimate of the ultimate capacity and is typically 60 percent of the maximum value predicted by the three methods. Meyerhof's method gave the maximum computed capacity in 64 percent of the cases, whereas the FHWA procedure yielded the highest capacity in 36 percent of the cases. On the average, the difference between the capacity computed by the FHWA and the Meyerhof procedures was about 20 percent.

TEST F-493 - NORTH #6
LENGTH 40
DIAM. 2.78

GAMMA	SOIL	DEPTH	AREA	Po'	FHWA		REESE			MEYERHO	
					B	Qs	K	PHI	Qs	N	Qs
120	SAND	0	9.5	83.0	0.285	1.2	28.4	0.7	34	11.2	23	19.08
120	SAND	9.5	16	56.8	0.765	1.018	44.2	0.7	36	22.1	29	16.46
57.6	SAND	16	20	34.9	1.018	0.927	33	0.7	36	18.1	29	10.13
57.6	SAND	20	35	131.0	1.291	0.792	134	0.6	35	71	25	32.75
52.6	SAND	35	40	43.7	1.573	0.673	46.2	0.6	33	26.8	20	8.734
57.6	SAND	40	0	0.0	1.063	0	0	0.7	36	0	30	0
			0	0.0	0.487	0	0	0.7		0		0
			0	0.0	0.487	0	0	0.7		0		0
			0	0.0	0.487	0	0	0.7		0		0
			0	0.0	0.487	0	0	0.7		0		0
			0	0.0	0.487	0	0	0.7		0		0

TOTAL SIDE	286	149	87
			N br 25
TOTAL TIP	109	146	243
Q ult.	395	295	330

* NOTE: All Q values are in Tons.

FIGURE 7 Typical spreadsheet printout containing required input data and computed capacity for each prediction method.

FIGURE 8 Comparison of measured capacity using Davisson method (1972) and computed capacity using FHWA method (1987).

Ease of Application

Because it is simple and straightforward for the design engineer and yields reasonable results, the FHWA method is favored over the other methods presented in this study. The β factor needed for unit skin resistance in sand is simple to apply, and the α factor for clays is a constant or zero. For unit tip resistance, the uncorrected N value is multiplied by a constant with a limiting value and in clays is almost always nine times the undrained cohesive strength of the clay. With a knowledge of the blow counts for sands and the undrained strength for a soil profile, one can quickly calculate the estimated ultimate load capacity of a drilled shaft.

COMPARISON OF COMPUTED AND MEASURED CAPACITY

Comparisons between the computed and measured load capacities are a function of both the load test interpretation procedure and the prediction equation. Comparisons are presented for the three computation procedures and both the Davisson method and the Hirany and Kulhawy method (similar values to the 1-in. criterion).

Plots comparing the measured ultimate capacity using Davisson's method (x-axis) and the calculated ultimate capacity (y-axis) for the FHWA, Meyerhof, and Reese et al. procedures are presented in Figures 8, 9, and 10, respectively. Boundaries representing 25 percent conservative and 25 percent unconservative estimates of the load capacity are shown to provide perspective regarding the degree of error in the computed values.

A review of Figures 8 through 10 indicates that the FHWA method probably provides the best agreement with measured values but the Meyerhof method also provides relatively good agreement. The Reese et al. procedure typically underestimated the measured capacity by more than 25 percent and, therefore, represents a more conservative estimate of capacity. For three cases the measured capacity was four to five
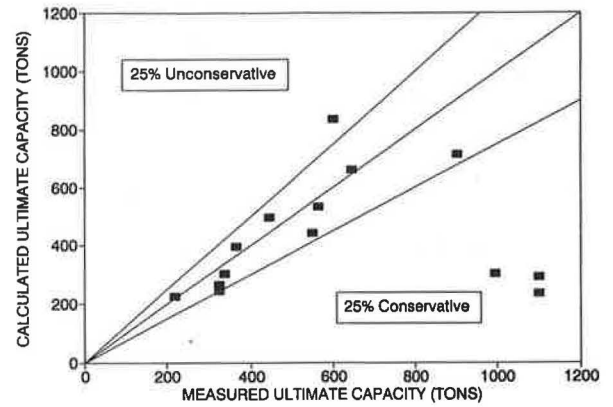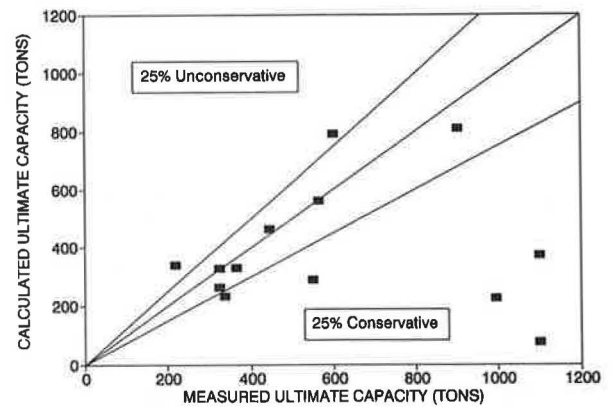

FIGURE 9 Comparison of measured capacity using Davisson method (1972) and computed capacity using Meyerhof method (1976).
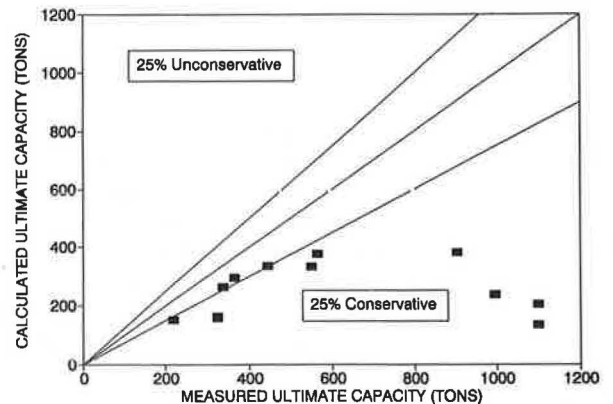

FIGURE 10 Comparison of measured capacity using Davisson method (1972) and computed capacity using Reese et al. method (1976).

times higher than the computed capacity regardless of the design equation used. In two of these cases, the soils were relatively dense sandy gravels (Load Tests 4 and 5) and the higher resistance can generally be attributed to side friction. The third shaft penetrated into weathered shale, and the higher resistance can be attributed to increased base resistance.

Comparisons between computed capacity and measured load capacity based on Hirany and Kulhawy's technique are presented in Figures 11 through 13. As indicated previously, the Hirany and Kulhawy (4 percent shaft diameter) interpretation is the least conservative of the various interpretation methods used. Because the FHWA design method is based on a failure load at a settlement of 5 percent of the shaft diameter, the computed capacities would be expected to be somewhat unconservative. The results in Figure 11, however, show the opposite. Almost all the load tests were more than 25 percent conservative. The Meyerhof method was similar to the results for the FHWA method, whereas the Reese method was more conservative. The results in Figures 11 through 13 indicate that increases in computed capacities of 25 percent or more are justifiable in cases where the higher deformation level associated with the Hirany and Kulhawy failure criteria can be tolerated.

CONCLUSIONS

Three methods for predicting the ultimate load capacity for drilled shafts (FHWA, Reese's, and Meyerhof's) were evaluated with load test data. Four load test interpretation methods were used in the comparison: Davisson's, double tangent, Hirany and Kulhawy's, and 1 in. of settlement. On the basis of the results, the following conclusions are possible.

1. The load test interpretation suggested by Davisson generally yields ultimate capacities about 37 percent higher than those from the double tangent method. The Hirany and Kulhawy interpretation procedure produces ultimate capacities typically 25 percent higher than the Davisson method because failure is defined at a higher deformation level.
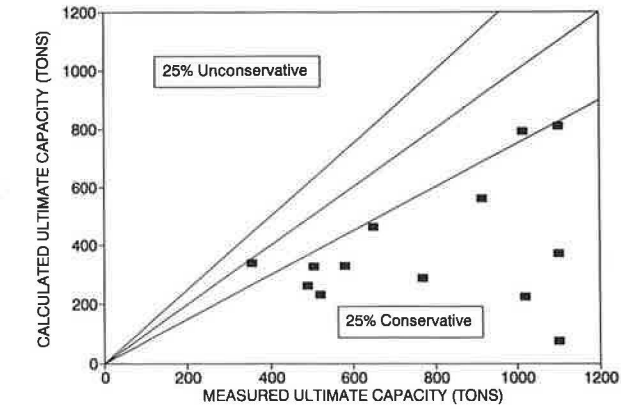
FIGURE 12 Comparison of measured capacity using Hirany and Kulhawy method (1989) and computed capacity using Meyerhof method (1976).

2. Whereas the Davisson method is a common interpretation procedure, the higher capacities provided by the Hirany and Kulhawy method may be acceptable if somewhat greater shaft deformations are tolerable (about 1 in. for 2-ft-diameter shafts at failure, 0.25 in. with FS = 2).

3. Of the three prediction methods used, the FHWA method yielded the most favorable results for both ease of application and correlation with measured load capacity (on the basis of Davisson's method). In no case did the FHWA method overestimate measured capacity by more than 25 percent, and there were few cases where it was more the 25 percent conservative.

4. The Meyerhof equation also predicted load capacities in good agreement with measured capacities, but Reese's method typically underpredicted measured values by more than 25 percent.

5. In comparison with the Hirany and Kulhawy interpretation, almost all the computed capacities were more than 25 percent conservative, suggesting that modifications of the design equations may be appropriate for gravelly soils.

6. Additional load tests should be performed in gravelly soils to allow appropriate modifications of existing design equations.
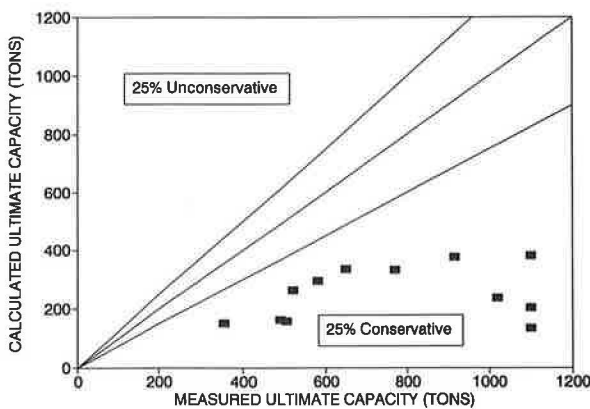
FIGURE 11 Comparison of measured capacity using Hirany and Kulhawy (1989) method and computed capacity using FHWA method (1987).
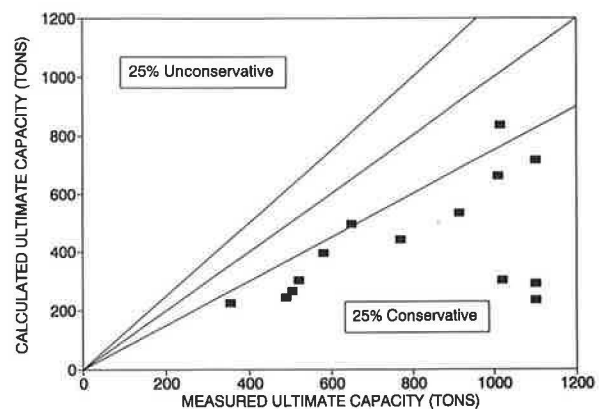
FIGURE 13 Comparison of measured capacity using Hirany and Kulhawy method (1989) and computed capacity using Reese et al. method (1976).

ACKNOWLEDGMENTS

REFERENCES

1. L. C. Reese and M. W. O'Neill. *Drilled Shafts: Construction Procedures and Design Methods*. Federal Highway Administration, U.S. Department of Transportation, McLean, Va., 1987.
2. A. Hirany and F. H. Kulhawy. Interpretation of Load Tests on Drilled Shafts, Part 1: Axial Compression. *Proc., Foundation Engineering Congress*, Vol. 2, ASCE, Evanston, Ill., 1989, pp. 1132–1149.
3. M. T. Davisson. High Capacity Piles. *Proc., Innovations in Foundation Construction*, ASCE, Illinois Section, 1972.
4. C. I. Mansur and J. M. Kaufman. Pile Tests, Low-Sill Structure, Old River, Louisiana. *Journal of the Soil Mechanics and Foundation Division*, ASCE, Vol. 82, No. SM5, Oct. 1956, pp. 1079-1–1079-33.
5. H. D. Butler and H. E. Hoy. *User's Manual for the Texas Quick-Load Method for Foundation Load Testing*. Report IP 77-8. Federal Highway Administration, U.S. Department of Transportation, 1977.
6. K. Terzaghi and R. B. Peck. *Soil Mechanics in Engineering Practice* (corrected printing of 2nd ed.) John Wiley and Sons, New York, 1968.
7. G. G. Meyerhof. Bearing Capacity and Settlement of Pile Foundations. *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 102, No. GT3, March 1976, pp. 195–228.
8. 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. 5, May 1976, pp. 493–510.