

Effects of Stratigraphic and Construction Details on the Load Transfer Behavior of Drilled Shafts

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Drilled shafts are often designed by representing soil layers as ideal geomaterials, such as clay, sand, or rock, and using simple correlation factors to convert measured strength values into values of unit shaft and base resistance. The effects of apparently minor inclusions in layers of otherwise uniform soil and soft rock on load transfer, particularly in shaft resistance, are addressed. Also considered are the effects of the use of polymer drilling slurry and artificial roughening of the borehole on load transfer. Data from the load testing of six full-sized drilled shafts at three sites indicated that thin sandstone layers could increase load transfer by one-third in dense sand and that thin bentonite layers could decrease load transfer by two-thirds in clay-shale. No adverse effects could be detected in shaft load transfer by the use of polymer drilling slurry, and the rifling of a borehole wall in clay-shale increased the shaft load transfer by about 40 percent over that in an unrifled shaft.

Current design procedures for drilled shafts are based primarily on experience that has been accumulated in the testing of full-scale shafts and in the correlation of test results with soil or rock properties obtained in a straightforward manner. In recent years several data bases of loading tests and corresponding soil and rock properties have been established and have been used effectively to develop design procedures and parameters (1–3). However, these procedures do not directly address the issue of apparently minor variations in the subsurface conditions that are often neglected in establishment of design loads. For example, Tomlinson (4) cited variations in average unit shaft (side) resistance of from 0.4 to in excess of 5.0 tsf in similar chalk formations in the United Kingdom due to locally present flints and fissures that affect both the shaft load transfer and the strength indicated in the laboratory. To provide more information on this phenomenon and information on the effects of certain construction details (namely, rifling of the borehole and use of polymer drilling slurry) on load transfer, the results of six loading tests on full-sized drilled shafts at three test sites in three geological settings are presented in this paper.

The three test sites are all characterized by the presence of fairly thick layers of geomaterial that can be characterized in

a straightforward manner with SPT or triaxial compression tests. However, interbedded with some of the layers are thin layers and seams, usually no thicker than 4 in. and making up less than 10 percent of the vertical profile of the major layer, of either harder or softer material (or both). The extent to which this interbedding affects load transfer is examined.

SITE A—MONTGOMERY FORMATION

The Montgomery Formation is a Pleistocene-aged deltaic terrace of the Texas Gulf Coast region. It consists mainly of layers of submerged silty fine sands with occasional layers of clay and sandy clay. Within the fine sand layers are occasionally found seams of cemented sand, locally called weak sandstone. These seams are normally too thin to sample and test in the laboratory but appear to have compressive strengths in the range of 60 to 150 ksf. They can normally be penetrated during drilled shaft construction with an auger but occasionally require the use of a core barrel.

During the design phase for a major interchange between U.S. Highway 290 and the Sam Houston Tollway, located approximately 17 mi northwest of downtown Houston, Texas, two test shafts were constructed and tested to assist in finalizing the design parameters (5). The site conditions and test shaft profiles are shown in Figure 1. The drilled shaft denoted Shaft 1 was purposely situated in an area of the interchange in which sandstone seams were known to exist, whereas the shaft denoted Shaft 2 was constructed in an area where the seams were known to be absent. The two test shafts were separated by about 1,500 ft, so that the soil layering was somewhat different at each test shaft location, as shown in Figure 1. *N* values in Figure 1 and the figures that follow represent uncorrected values, and *c_u* values are undrained shear strength values obtained from *UU* triaxial compression tests.

Each shaft at this site was constructed under bentonitic slurry controlled as recommended by Reese and O'Neill (3). Slurry samples from the bottom of the borehole just before concreting indicated the following:

- Unit weight: 64.3 and 69.3 pcf for Shafts 1 and 2, respectively;
- Sand content (by volume): Shaft 1, < 1 percent; Shaft 2, 11 percent; and

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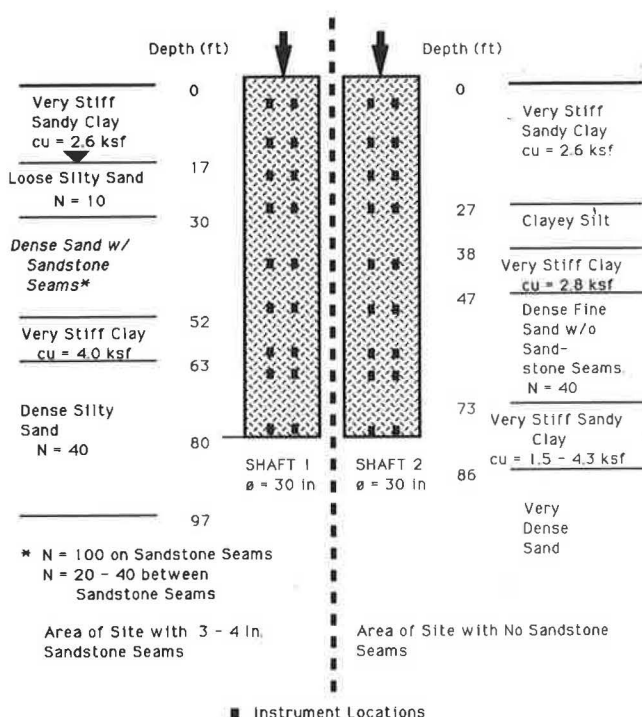


FIGURE 1 Soil and shaft profiles: test shafts at US-290/Sam Houston Tollway test site.

- Marsh funnel viscosity (sec/quart): Shaft 1, 37; Shaft 2, 49.

The slurry was displaced directly with tremie-placed, high-slump concrete, which is the standard procedure in the area. For these shafts, as well as the other four shafts described herein, concrete slump was in the range of 6 to 7 in., and the time between opening the borehole and completion of concreting was 5 to 7 hr.

Load-settlement relations for both shafts on the first cycle of loading are shown in Figure 2, which also contains the load-settlement curves for all other tests described in this paper. In all tests reported in this paper, loading was according to the quick test method, in which load increments of approxi-

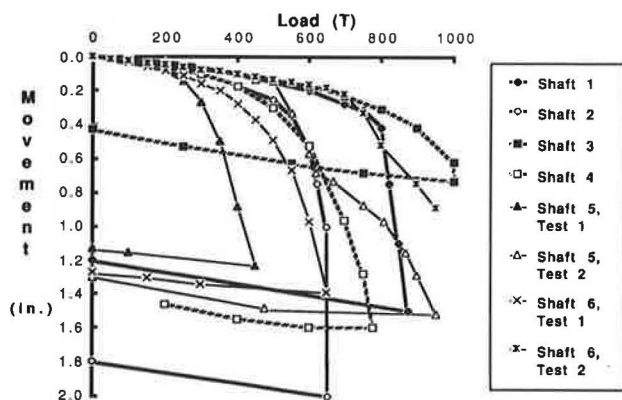


FIGURE 2 Load versus settlement: for test shafts.

mately 7 percent of the anticipated failure load are applied every 5 min. A second cycle of loading was applied to each shaft immediately after the completion of the first cycle to study potential loss of capacity due to large interface strains. In both shafts the second cycle of loading produced load-settlement relations almost identical to those on the first cycle, except that a slight increase of base resistance occurred in Shaft 1.

Shafts 1 and 2 (and Shafts 3, 4, and 5, described later) were instrumented with Mustran cells (6) to measure distribution of load along the shafts, and a load cell and displacement transducers at the shaft heads. Mustran cell locations are shown schematically in Figure 1. For the highest load applied to Shafts 1 and 2 for the first loading cycle, measured load distribution relations are shown in Figure 3. It is clear that load transfer was identical between shafts except for the depth range of roughly 30 to 52 ft. In that range, unit shaft load transfer was slightly more than twice as high in Shaft 1 as in Shaft 2. The major layer corresponding to the dense sand with interbedded sandstone seams in Shaft 1 (30 to 52 ft) was found at 47 to 73 ft in depth in Shaft 2. The ratio of shaft load transfer in these two corresponding layers was still about 2. Since differences in load transfer were minor elsewhere, it is argued that the differences in slurry composition in the two shafts had very little effect on shaft load transfer.

Base resistance in these two shafts will be described in a separate section.

SITE B—LIME HILLS FORMATION

The Lime Hills Formation is a component of the Wilcox Group, a heterogeneous system of formations deposited in Eocene times. It consists mainly of calcareous clay, silty clay, and seams and lenses of fine sand, some of which are cemented. Lignite seams also appear near the top of the formation. During the design phase of the I-20/I-49 interchange, 4 mi west of downtown Shreveport, Louisiana, two test shafts, here denoted Shaft 3 and Shaft 4, were constructed and loaded tested (7). The soil profiles at the location of each test shaft are indicated on Figure 4. Shaft 3 was placed in an area within the footprint of the interchange where thin seams of both sandstone and lignite were found, and Shaft 4 was placed in

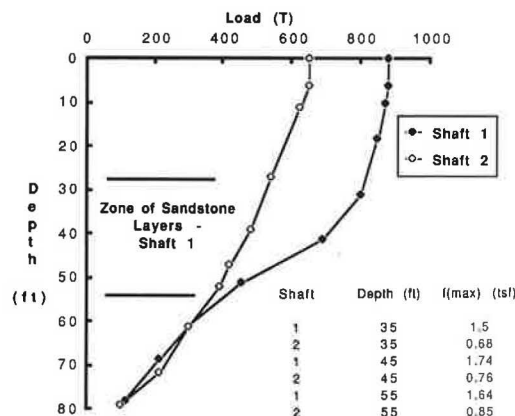


FIGURE 3 Load versus depth: Shafts 1 and 2.

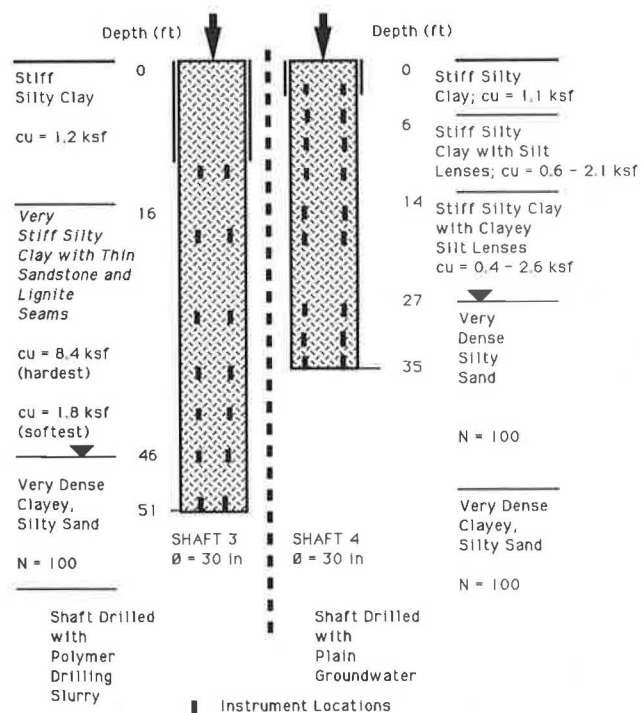


FIGURE 4 Soil and shaft profiles: Test Shafts 3 and 4 at I-20/I-49 Interchange test site.

an area where no such inclusions existed. The lignite was somewhat softer and more brittle than the clay, whereas the sandstone was somewhat stronger than the clay in which it was included. No samples could be obtained for definitive strength tests. Each shaft was designed to penetrate to a very dense sand layer found consistently, but at varying elevations, over the site. The elevation of the soil surface at Shaft 3 was 9 ft below that at Shaft 4, so that the surface of the very dense sand was about 20 ft lower at Shaft 3 than at Shaft 4. As with Shafts 1 and 2, the test locations were about 1,500 ft apart.

The piezometric surface appeared to be at the top of the very dense sand at both test locations, and the overlying strata were dry. Shaft 4 was drilled first, without the use of casing (other than surface casing) or drilling slurry. When the very dense sand was penetrated, some groundwater flowed into the borehole to about the top of the sand layer, but the hole remained stable, which indicates the presence of at least some cohesion in the sand. The shaft was concreted using tremie methods. To avoid a collapse of Shaft 3, the entire hole was drilled under polymer drilling slurry, in this case a PHPA emulsion. During drilling, the expected thin sandstone and lignite seams, not encountered in Shaft 3, were found within the clay between depths of 16 and 46 ft. After completion of the borehole the polymer slurry was allowed to remain unagitated for 30 min to allow sand to settle out, following which the base was cleaned and the shaft concreted using tremie methods. Bottomhole samples of the slurry taken after 30 min of settling indicated a unit weight of 62.5 pcf, a sand content of < 1 percent, and a Marsh funnel viscosity of 30 sec/quart. Shaft 3 was cased in the top 11 ft, and bond was broken between the casing and soil to account for later excavation at

that location. Both shafts were instrumented with Mustran cells at the locations indicated.

Load-settlement curves for the two test shafts are shown in Figure 2. Clearly, Shaft 3 carried the greater load with the lesser settlement. The reason for the higher capacity appears to be increased unit load transfer in the stiff clay layer with sandstone and lignite seams, greater contact area along the sides of the shaft, and higher unit base resistance (described later). Measured load distribution relations for Shafts 3 and 4 at the maximum load applied are given in Figure 5. No significant effect of the sandstone and lignite seams can be presumed in Shaft 3, however, since the clay matrix was about 50 percent stronger at Shaft 3 than at Shaft 4, which is the approximate difference in unit load transfer. It appears that the stiffer sandstone and softer lignite in effect each canceled the strengthening or weakening effect of the other.

SITE C—EAGLE FORD FORMATION

The Eagle Ford Formation is a Cretaceous-aged marine deposit consisting mainly of laminated clay shales. Embedded within the Eagle Ford are occasional seams of calcite and bentonite. The bentonite seams, which are seldom more than 3 in. thick, are much softer than the shale. During installation of drilled shafts, this formation can normally be penetrated, without the use of casing or slurry, using soil augers, owing to the horizontal bedding and frequency of the laminations (10 to 20 to the inch).

Two test shafts were installed at the site of the GTE World-wide Operations headquarters building in Irving, Texas, 15 mi northwest of downtown Dallas (8). Two objectives were established for the testing: (a) determine the effect of the presence of thin bentonite seams on the shaft resistance in the clay-shale and (b) determine whether simple rifling of the borehole could increase shaft resistance substantially. One other important detail is noted: about 20 ft of overburden soils had been removed at the location of the test shafts, exposing unweathered clay-shale, only about 1 week before the shafts were installed and 3 weeks before the shafts were load tested.

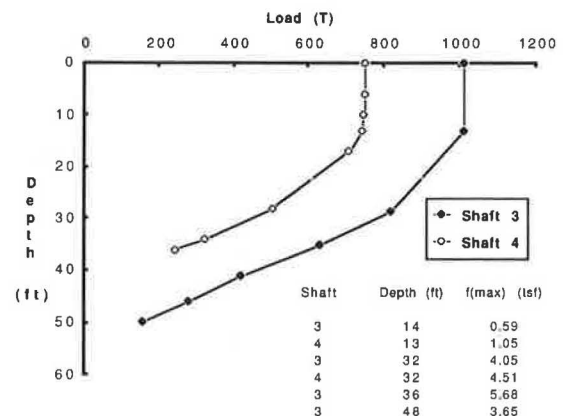


FIGURE 5 Load versus depth: Shafts 3 and 4.

Figure 6 shows the geomaterial and shaft profiles for the two test shafts. Bentonite seams appeared within the clay-shale formation in the depth range of 36 to 41.5 ft. The general groundwater surface was well below the base elevation of the test shafts, although a few seeps from perched water were encountered at various elevations during drilling. The geomaterial profiles can be considered identical at both shaft locations, because the shafts were only about 20 ft apart.

Shaft 5 was drilled with normal drilling procedures (without casing or drilling fluid), except that the top 26 ft was cased off and bond broken between the shaft and soil. This permitted the test section to include the zone with bentonite seams. Shaft 5 was instrumented with Mustran cells at the locations indicated. Shaft 6, on the other hand, was drilled using normal procedures and then rifled using a simple 1-in. long side cutter affixed to the drilling auger. Rifling was done on a pitch of about 3 ft, and two separate passes were made so that two separate rifled grooves, estimated to be about 0.75 in. deep, were developed on the borehole wall. Shaft 6 was not instrumented. Both Shafts 5 and 6, however, were cast with voids beneath their bases so that it would be possible to produce failure in side resistance at loads below the capacity of the available testing system (1,000 tons). Each void was vented to the atmosphere to prevent air pressure buildup. With the void it was possible to determine the average unit shaft load transfer in the contact zone for Shaft 6.

Load-settlement relations for two cycles of loading are shown in Figure 2. A reference of zero settlement is used for the beginning of each test, although the accumulated deflection of the shafts at the beginning of the second cycle of loading was about 5 in., as the shafts were pushed slowly down to a

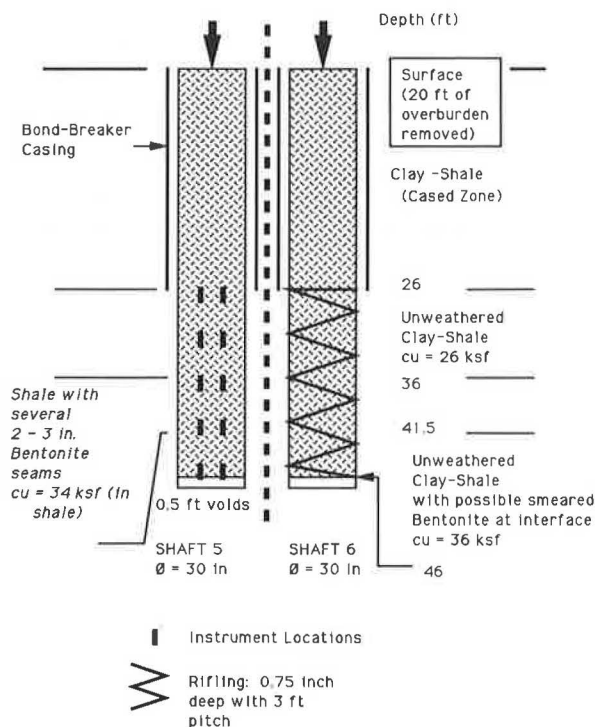


FIGURE 6 Soil and shaft profiles: Test Shafts at GTE test site.

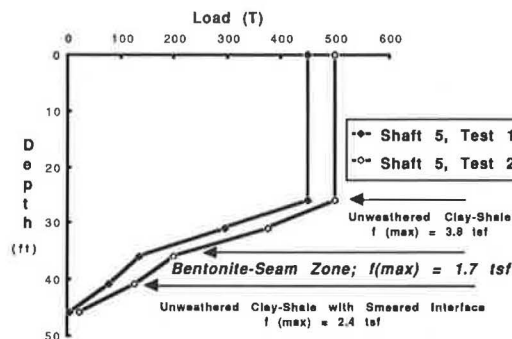


FIGURE 7 Load versus depth: Shaft 5, load and reload.

point at which the shaft base was just above the bottom of the borehole prior to unloading and beginning the second cycle of loading. Cyclic loading was used to observe whether degradation of side shear occurred with large deformations during relatively rapid loading. Both shafts were considerably stiffer during reloading than during initial loading despite the fact that no base resistance existed during either sequence. This behavior, which is not typical of drilled shafts in the Eagle Ford shale, appears to be because the shale had been recently unloaded by excavation of overburden, resulting in the opening of the horizontal laminations, which were apparently again closed by the initial loadings. It is also evident that Shaft 6 carried considerably more load during initial loading than did Shaft 5 at a given settlement, once the settlement exceeded about 0.1 in. This phenomenon is apparently due to the effects of borehole rifling.

The load distribution at failure for both loading cycles in Shaft 5 (denoted Test 1 and Test 2) are shown in Figure 7. Reduced load transfer is seen in the zone in which the bentonite seams are embedded and also below that zone, compared with the load transfer in the zone of unweathered shale above. The only explanation for the reduced load transfer in the zone below the bentonite zone is that bentonite clay cuttings were carried down with the auger and smeared on the sides of the borehole for some distance below the deepest bentonite seam. It is also seen in Figure 7 that little difference in shaft load transfer occurred between the two cycles of loading despite the fact that the accumulated shear displacement was more than 5 in. for Test 2.

COMPARATIVE SIDE AND BASE UNIT LOAD TRANSFER RELATIONS

Representative relations between unit shaft load transfer (shear stress), f , and local shaft movement, w , from among the first cycle loadings on the instrumented shafts at Sites A and C, are shown in Figure 8. The f - w curves for comparable layers for Shafts 1 and 2, with and without sandstone seams, clearly indicate that the sandstone seams had a major reinforcing effect on the dense sand at the site, although more deflection was required to mobilize the full resistance when the sandstone seams were present. On the other hand, in Shaft 5, the bentonite seams clearly reduced the ultimate load transfer, but their presence also significantly reduced the local settle-

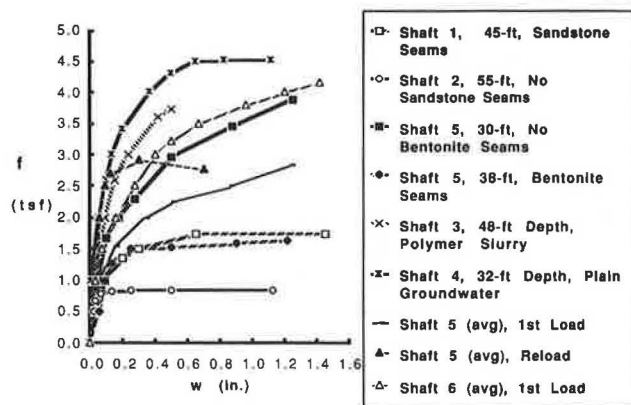


FIGURE 8 Comparative unit side load transfer relations.

ment required to mobilize full shaft resistance. In the unweathered clay-shale, full unit shaft resistance had not yet been mobilized at a settlement of 1 in.

Comparative f - w relations are also shown in Figure 8 for the lower very dense sand layer at Site B. The relations are almost identical, indicating no discernible difference in shaft behavior between drilling with natural groundwater and drilling with polymer slurry.

For Site C the average shaft unit load transfer relations over the entire depth of contact are compared for the two shafts in Figure 8, since no depthwise determination of f - w curves was possible in Shaft 6. The initial average f - w relations are of similar shape for both shafts, except that f values are about 40 percent larger at corresponding values of w for Shaft 6, the rifled shaft, for $w > 0.1$ in. Comparative f - w relations are shown for Shaft 5 during initial loading and reloading, indicating the result of the effect speculated earlier that initial loading had closed laminations that had opened during site excavation. No loss of ultimate shaft resistance can be observed between the two loadings.

Relations of net unit base load transfer (q) to base settlement (w) measured for Shafts 1 through 4 are given in Figure 9. Shaft 1 (founded in dense sand) developed an ultimate unit base resistance of 19 tsf at a settlement of 3 in., or 10

percent of the shaft diameter. According to Reese and O'Neill (3), a value of 24 tsf would be expected on the basis of the SPT N value. Their correlative expression for base resistance is $q(ult) (tsf) = 0.6N \leq 45$ tsf. The difference is minor and the cause of the difference is unknown. It may have been associated with the relatively small size of the test shaft, the need to use bentonitic slurry, and the difficulty of cleaning the bases of relatively small shafts constructed under slurry. The base of Shaft 2 at Site A was in a sandy clay material. The ultimate base resistance was 22 tsf, developed at a movement of about 1.25 in., or 4 percent of the shaft diameter. According to Reese and O'Neill (3), a value of 16 tsf would have been expected on the basis of an average value of c_u of 3.5 ksf in the base layer. The correlative expression for base resistance is $q(ult) = 9 c_u$. The small overprediction of capacity may have been because the sandy clay drained slightly during loading.

Shafts 3 and 4 were both founded in a very dense sand. Drilling reports indicated that the sand was siltier at Shaft 3, although the SPT blow counts at both locations were identical. The interpreted ultimate base resistance for Shaft 4 is approximately 53 tsf, compared with 45 tsf predicted by Reese and O'Neill (3). However, for Shaft 3 the ultimate base resistance is only 22 tsf, about half of the predicted value, and it occurred at a settlement of 0.5 in., or 1.7 percent of the shaft diameter, which is characteristic of undrained or partially drained behavior, perhaps due to the presence of silt at that location.

Shafts 5 and 6 were not designed to evaluate base resistance. However, Shaft 5 was pushed during Test 2 until approximately 57 tsf of base resistance was developed, with no indication that bearing capacity failure was impending.

EXPERIMENTALLY DERIVED CORRELATIVE FACTORS

A number of procedures exist whereby predictions of ultimate unit shaft and base resistance can be made. Reese and O'Neill (3), whose work is representative, cite several expressions for shaft and base resistance. Those for base resistance have already been indicated.

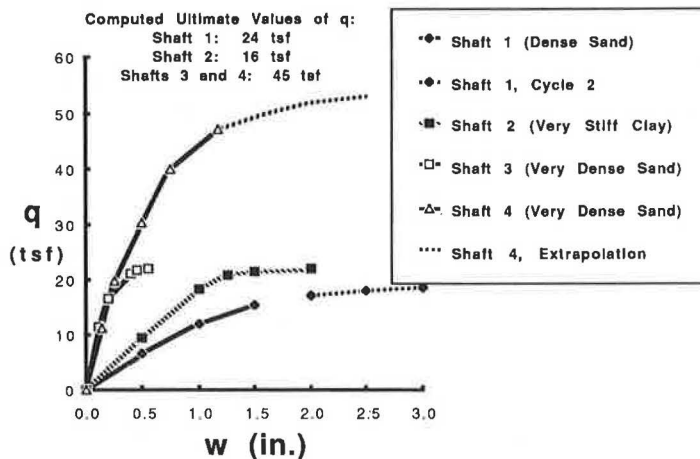


FIGURE 9 Unit base load transfer relations.

For shaft resistance, four expressions are used for soil and soft rock:

$$f_{\max} = \alpha c_u \text{ (in clay)} \quad (1)$$

where $\alpha = 0.55$ except in the top 5 ft and bottom diameter of the shaft, where $\alpha = 0$;

$$f_{\max} = [1.5 - 0.135 (z)^{0.5}] \sigma'_v = \beta \sigma'_v \text{ (in sand)} \quad (2)$$

where z = depth in feet and σ'_v = vertical effective stress;

$$f_{\max} = \kappa q_u \text{ (in very soft shale)} \quad (3)$$

where q_u is the unconfined compressive strength of shale cores and κ is a correlation factor, traditionally taken as 0.15; and

$$f_{\max} = \mu [q_u (\text{psi})]^{0.5} \text{ (in harder shale)} \quad (4)$$

where μ is typically recommended to be 2.5.

Equation 4 was adapted from Horvath and Kenney (9) and is predicated on massive rock with no weak seams and no special roughening of the borehole wall. Generally, in shale, f_{\max} is evaluated from both Equations 3 and 4, and the smaller value is used.

Values of correlative parameters α , β , κ , and μ backcalculated from the loading tests described in this paper are given in Table 1. Values for these parameters recommended by

TABLE 1 SUMMARY OF CORRELATIVE FACTORS FOR UNIT SHAFT RESISTANCE

Site	Shaft	Layer	Avg. $f(\text{max})$ (tsf)	Correlative Factors			
				α	β	κ	μ
290/Tollway	1	Dense sand with sandstone seams (depth = 30 - 52 ft)	1.6	-	0.88 [0.64]	-	-
	2	Dense sand without sandstone seams (depth = 47 - 73 ft)	1.0	-	0.42 [0.45]	-	-
	1	Very stiff sandy clay (depth = 0 - 17 ft)	0.60	0.46 [0.55]	-	-	-
	2	Very stiff sandy clay (depth = 0 - 27 ft)	0.62	0.48 [0.55]	-	-	-
120/149	3	Very dense sand (depth = 46 - 51 ft) (polymer slurry)	3.7	-	1.3 [0.56]	-	-
	4	Very dense sand (depth = 27 - 35 ft) (plain water)	4.7	-	2.4 [0.74]	-	-
	3	Very stiff silty clay with sandstone / lignite seams (depth = 16 - 46 ft) (polymer slurry)	2.5	[insufficient soil test data to evaluate correlative factors]			
	4	Very stiff silty clay with no sandstone / lignite seams (depth = 6 - 27 ft) (plain water)	1.5	[insufficient soil test data to evaluate correlative factors]			
GTE	5	Unweathered shale (depth = 26 - 36 ft)	4.2	-	-	0.16 [0.15]	3.1 [2.5]
	5	Shale / bentonite seams (depth = 36 - 41 ft)	1.7	-	-	0.049 [0.15]	1.1 [2.5]
	5	Shale / smeared face (depth = 41 - 46 ft)	2.0	-	-	0.056 [0.15]	1.3 [2.5]
	5	Average (26 - 46 ft) (unrilled)	2.9	-	-	0.10 [0.15]	2.0 [2.5]
	6	Average (26 - 46 ft) (rilled)	4.2	-	-	0.15 [0.15]	2.8 [2.5]

[] Indicates value computed from data base in Reese and O'Neill (3)

Range of correlative factors from data base in (3):

α - [] value $\pm 25\%$

β - [] value $+50\%/-25\%$

κ - [] value $\pm 25\%$

μ - [] value $+100\%/-25\%$

Reese and O'Neill (3), which were evaluated from analysis of a data base of 41 large test shafts, are shown in brackets immediately below the values derived from the tests. At Site A, the sandstone seams increased β to a value about one-third greater than is normally recommended, whereas the value is very close to that recommended where sandstone seams were absent. At Site B β was 2.5 to 3.5 times the value recommended in the lower dense sand, probably because the sand possessed some cohesion or cementation. No clear adverse effect of the use of the polymer slurry is seen. At Site C, Factor κ was about as recommended and Factor μ was slightly greater than is recommended in the unweathered shale with no rifling. Where bentonite seams were encountered and in the zone immediately below, these factors were considerably below the recommended values. Rifling increased the average shaft load transfer by 40 percent over the unrifled shaft and restored Factors κ and μ to their recommended values (or slightly above) in spite of the presence of the bentonite seams.

CONCLUSIONS

The following conclusions can be drawn from analysis of the full-scale loading tests described in this paper. It is not suggested that they can be generalized to other sites, but they serve to point out the magnitude of the effects of relatively minor anomalies in the subsurface profile and in the method used to construct the shafts on the load transfer behavior in drilled shafts.

1. Inclusions of sandstone seams no thicker than 3 to 4 in. and making up less than 10 percent of the thickness of the layer increased the β factor for a dense sand layer by about one-third over that in a comparable layer without such inclusions.

2. Thin, hard sandstone and soft, brittle lignite seams included within the same layer of stiff clay appeared to have mutually canceling effects.

3. The use of polymer slurry during construction appeared to produce load transfer and settlement behavior comparable with construction under plain groundwater.

4. Inclusions of bentonite seams no thicker than 3 in. and composing less than 10 percent of the thickness of the layer decreased the κ and μ factors by about two-thirds compared with similar factors in that part of a clay-shale formation not containing the bentonite seams. Load transfer in the zone below the bentonite layers also appeared to be adversely affected.

5. Unloading of horizontally laminated clay-shale by removal of about 1.25 tsf of overburden pressure before the

construction of drilled shafts caused drilled shaft settlements at full mobilization of side load transfer to be much larger than is normally expected.

6. Rifling of the borehole wall by using a simple side cutter on the drilling auger increased unit side resistance by about 40 percent in clay-shale.

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