USE OF ULTIMATE LOAD THEORIES FOR DESIGN OF DRILLED SHAFT SOUND WALL FOUNDATIONS

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(Abstract)

A study was performed to investigate the factors that affect the accuracy of the procedures used by the Virginia Department of Transportation for design of drilled shaft sound wall foundations. Field load tests were performed on eight inch and nine inch diameter drilled shafts, and the results were compared to theoretical solutions for ultimate lateral load capacity. Standard Penetration Tests were run in the field and laboratory strength tests were performed on the soils from the test sites. It was found that published correlations between blow count and friction angle for sands and gravels can be used to estimate friction angles for the partly saturated silty and clayey soils encountered at the test sites. A spreadsheet program was developed to automate the process of determining design lengths for drilled shaft sound wall foundations. The spreadsheet was used to investigate the effects of different analysis procedures and parameter values on the design lengths of drilled shaft sound wall foundation.

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CHAPTER 1 - INTRODUCTION

The study described in the following chapters was performed under sponsorship of the Virginia Transportation Research Council and the Virginia Department of Transportation (VDOT) through a research contract with Virginia Polytechnic Institute and State University, and support from the Federal Highway Administration through an Eisenhower Fellowship.

The objective of the study was to investigate the factors that affect the accuracy of the procedures being used for design of drilled shaft foundations for sound barriers (sound walls) in Virginia. These foundations are usually designed by contractors and checked by the personnel of the Bridge Division at VDOT. In most cases designs are based on Broms's (1964b) theory for ultimate load capacity of eccentrically loaded drilled shafts in cohesionless soils, using values of friction estimated on the basis of Standard Penetration Test (SPT) blow count values.

The study focused on four broad questions regarding this process:

1. Are published correlations between SPT blow counts and friction angles, which have been developed for sands and gravels applicable for the silty and clayey soils found in many locations in Virginia?

2. What is the consequence of neglecting the cohesion intercepts that are characteristic of the partly saturated silty and clayey soils in Virginia, and treating these soils as if they were cohesionless for the purposes of analysis and design using Broms's (1964b) theory?

3. Does Broms's (1964b) theory, in combination with the factors of safety that are used for design (usually F = 1.88 or F = 2.5), result in safe and economical dimensions for drilled shaft sound wall foundations in Virginia?

4. Can the process of designing drilled shaft foundations for sound walls be automated effectively through computer analysis, without removing the opportunity to apply essential engineering judgment to input values and results?

To address these questions, a program of research has been completed that included these activities:

- Theories for the ultimate load capacity of eccentrically loaded drilled shafts were reviewed. These theories, by Broms (1964a and 1964b) and Brinch-Hansen (1961) are summarized in Chapter 2. Broms's theory for cohesionless soils (Broms, 1964b) and Brinch-Hansen's theory for soils with both cohesion and friction (Brinch-Hansen, 1961) were programmed in an EXCEL spreadsheet called LCAP, which provides an efficient tool for automating the calculations and displaying the results.

- Field load tests were performed on eccentrically loaded drilled shafts. In all, 20 tests were performed at five sites in Virginia (Price's Fork, Salem, Suffolk, Fairfax County Parkway, and Roberts Road). The procedures used and the results of these tests are described in Chapter 3.

- Laboratory tests were performed on the soils from the load test sites. The friction angles measured for these soils, and the SPT blow counts measured at the test sites, were compared to published correlations between friction angles and blow count. This work is summarized in Chapter 4.

- The theories described in Chapter 2, together with the properties measured in lab tests, were used to estimate the ultimate lateral load capacities of the drilled shafts tested in the field. These theoretical capacities are compared to the measured failure loads in Chapter 5.

- Finally, a study was made, using the spreadsheet LCAP, to compare the Broms (1964b) and the Brinch-Hansen (1961) theories. LCAP analyses were also used to assess the effects of ignoring soil resistance for some distance below the ground surface, to evaluate two methods of accounting for ground slope adjacent to the foundations, to evaluate the effect of varying the value of friction angle used in the calculations, and to determine the change in the dimensions of the foundation that results from changing the value of the factor of safety. The results of the parametric study are summarized in Chapter 6.

The appendices contain details of Brinch-Hansen's (1961) theory (Appendix A), a summary of wind pressures and wind loads from the AASHTO guidelines (Appendix B), and a summary of the calculation procedures used in LCAP (Appendix C).

CHAPTER 2 - THEORIES FOR ULTIMATE LATERAL CAPACITIES OF ECCENTRICALLY LOADED DRILLED SHAFTS

2.1 Introduction

Ultimate load theories are often used to determine the sizes of drilled shaft sound wall foundations. Among the most frequently used theories are those developed by Broms, and by Brinch-Hansen. The assumptions involved in these theories, and procedures for their use are reviewed in this chapter.

The theories reviewed are:

- Broms's (1964a) theory for cohesive soils (soils with $c > 0$ and $\phi = 0$)
- Broms's (1964b) theory for cohesionless soils (soils with $c = 0$ and $\phi > 0$)
- Brinch-Hansen's (1961) theory for soils having both cohesion and friction (soils with $c > 0$ and $\phi > 0$)

2.2 Mechanisms of Deformation and Soil Resistance

In ultimate load theories, the drilled shaft foundation is assumed to behave as a rigid body, and it is assumed that bending deformations of the shaft are negligibly small in comparison with movements due to deformation of the soil around the shaft. The lateral load capacity computed using these ultimate load theories is that associated with failure of the soil. The moment and shear capacity of the shaft that are required to prevent structural failure are calculated separately.

As an eccentric load is applied to a shaft, the shaft rotates and displaces in the direction of the applied load (to the right as shown in Figure 2.1). As the shaft moves, it rotates around a center of rotation located somewhere above the bottom of the shaft. Above the center of rotation, passive pressures develop on the front of the shaft (the right side as shown in Figure 2.1), and active pressures develop on the back. Below the center of rotation, passive pressures develop on the back of the shaft, and active pressures develop on the front. The location of the center of rotation depends on the eccentricity of the applied load and the properties of the soil.

Figure 2.1: Mechanism of Deformation of an Eccentrically Loaded Drilled Shafts (PDF, 100K, fig21.pdf).

Eventually, when the full shear strength of the soil around the shaft has been mobilized, no further change in earth pressure is possible, and the shaft rotates and deflects freely, with no further increase in load. In this state the shaft has reached the "ultimate load condition." The purpose of ultimate load theories is to calculate the magnitude of the applied load required to reach this ultimate load condition. This load is called the "lateral load capacity" of the foundation. It depends on the dimensions of the foundation, the properties of the soil, and the eccentricity of the applied load.

One of the most important aspects of the interaction between a drilled shaft and the surrounding soil is that the difference between active and passive earth pressures that resists movement of the shaft is larger than would be calculated using conventional earth pressure theories, such as the Rankine theory. Conventional earth pressure theories consider only two-dimensional (2D) conditions. These 2D conditions correspond to a long wall moving in the soil. In the case of a circular drilled shaft, larger passive pressures are possible due to three-dimensional (3D) effects: a zone within the soil that is wider than the face of the shaft is involved in resisting movement of the shaft. The ratio between the 3D and the 2D soil resistance varies with the friction angle of the soil and the depth below the ground surface, but is usually on the order of two or three.

2.3 Broms's (1964a) Theory for Cohesive Soils with $\phi = 0$

This theory is applicable to saturated cohesive soils loaded rapidly, under undrained conditions. The distribution of the passive soil reaction used in the theory is shown in Figure 2.2. Important assumptions made in the theory are:

- As a result of 3D earth pressure effects, the difference between the passive and active earth pressure is 9c, or 2.25 times as large as would be calculated using the Rankine earth pressure theory for $\phi = 0$. This approximation was found to be in reasonable agreement with load test results in $\phi = 0$ soils.

- The soil near the top of the shaft, within 1.5 diameters below the ground surface, provides no resistance to movement of the shaft. This assumption is considered reasonable in view of the fact that the soil near the ground surface may be weaker than the soil at greater depth due to disturbance during construction of the shaft and frost action.

Figure 2.2: Broms (1964a) - Assumed Soil Reaction for Cohesive (ϕ = 0) Material (pdf, 100K, fig22.pdf)

Using the requirements of horizontal equilibrium and moment equilibrium, together with the distribution of soil resistance shown in Figure 2.2, the following equations can be derived:

$$P = f \bullet 9cD \quad \text{(horizontal equilibrium)} \tag{2.1}$$

where:

P = ultimate lateral load, or lateral load capacity of the foundation (force),

f = depth to point of zero shear (units of length),

c = cohesion (units of stress), and

D = diameter of shaft (units of length).

$$P(e + 1.5D + 0.5f) = 2.25D(L - 1.5D - f)^2 \bullet c \quad \text{(moment equilibrium)} \tag{2.2}$$

where:

e = eccentricity of applied load (units of length), and

L = shaft length (units of length).

To compute the magnitude of the lateral load capacity (P), the following procedure can be used:

1. Use Eq. (2.1) to express f in terms of P.
2. Substitute this expression in Eq. 2.2, and solve for P using an iterative process.

To compute the length required to develop a given value of load capacity (P), the following procedure can be used:

1. Assume a shaft length (L).

2. Follow the procedure above to compute the value of P corresponding to this shaft length.

3. If the computed value of P is smaller than the given value, increase the value of L and repeat steps (1) and (2). If the computed value of P is larger than the given value, reduce the value of L and repeat steps (1) and (2).

2.4 Broms's (1964b) Theory for Cohesionless Soils

This theory is applicable to soils such as sands or gravels, with c = 0. It can also be used for soils like partly saturated silts or clays that have some cohesion, if the contribution of cohesion to shaft capacity is neglected.

The distribution of the passive soil reaction used in the theory is shown in Figure 2.3. Important assumptions made in the theory are:

- As a result of 3D earth pressure effects, the difference between the passive and active earth pressure is $3K_p$, where K_p is the Rankine passive earth pressure coefficient ($K_p = \tan^2(45 + \phi/2)$. This approximation has been found to be in reasonable agreement with the results of load test in cohesionless soils.

- The point of rotation is at the bottom of the shaft. This approximation greatly simplifies the computations, and has only a small effect on the results.

Figure 2.3: Broms(1964b) - Assumed Soil Reaction for Cohesionless (c=0) Material (pdf, 100K, fig23.pdf)

Using the requirements of moment equilibrium, together with the distribution of soil resistance shown in Figure 2.3, the following expression for P can be derived:

$$P = \frac{\gamma D L^3 K_P}{2(e + L)} \qquad (2.3)$$

where:

γ = unit weight of soil (units of weight per unit volume),

K_p = Rankine passive earth pressure coefficient = $\tan^2(45 + \phi/2)$ (dimensionless), and

ϕ = angle of internal friction (degrees).

The magnitude of the ultimate lateral load can be computed directly using Eq. (2.3). To compute the length required to develop a given load capacity (P), the following procedure can be used:

1. Assume a shaft length (L).
2. Use Eq. (2.3) to compute the value of P corresponding to this shaft length.
3. If the computed value of P is smaller than the given value, increase the value of L and repeat, or, if the computed value of P is larger than the given value, reduce the value of L and repeat.

2.5 Brinch-Hansen's (1961) Theory for Soils having both Cohesion and Friction

This theory is applicable to soils such as partly saturated silts or clays that have both cohesion and friction, i. e. $c > 0$ and $\phi > 0$. It can also be used as an alternative to the Broms theories for frictionless ($\phi = 0$) soils or for cohesionless ($c = 0$) soils by setting $c = 0$ or $\phi = 0$ in the equations involved in the theory. This theory has the drawback that it is much more complex than the Broms theories, but it has the advantage that it can be used for soils that have both cohesion and friction.

The difference between passive and active earth pressures was expressed by Brinch-Hansen as:

$$\sigma_h = \gamma D z K_q + c D K_c \qquad (2.4)$$

where:

σ_h = the difference between passive and active earth pressure (units of stress),

z = depth below ground surface (units of length),

K_q = coefficient for the frictional component of net soil resistance under 3D conditions (dimensionless), and

K_c = coefficient for the cohesive component of net soil resistance under 3D conditions (dimensionless).

The distribution of the net soil reaction used in the theory is shown in Figure 2.4. The net soil resistance increases nonlinearly with depth in this relatively complex theory. The values of K_q and K_c vary with the ϕ and z/D, as shown in Figures 2.5 and 2.6. The equations needed to compute values of K_c and K_q are given in Appendix A.

Figure 2.4: Brinch-Hansen (1961) - Assumed Soil Reaction (pdf, 100K, fig24.pdf)

Figure 2.5: Variation of K_c with Depth (pdf, 50K, fig25.pdf)

Figure 2.6: Variation of K_q with Depth (pdf, 50K, fig26.pdf)

In deriving this theory, it was assumed that the active and passive earth pressures at very shallow depths correspond to those of conventional earth pressure theory. At larger depths a three-dimensional failure mechanism was used to evaluate the earth pressures. At great depths it was assumed that the maximum pressures will be governed by the bearing capacity of a deep vertical strip loaded horizontally.

2.6 Use of Ultimate Load Theories to Compute Bending Moments

Although these ultimate load theories are suitable for computing the ultimate load capacities of eccentrically loaded drilled shafts, they are inherently conservative when used to compute bending moments in shafts under working load conditions.

The reason for this conservative bias in working load moments is illustrated in Figure 2.7. Three distributions of earth pressures are shown in the figure:

- The ultimate earth pressure distribution
- One-half of the ultimate earth pressure
- The actual distribution for one-half of the ultimate load applied to the shaft

Figure 2.7: Earth Pressure Distribution at Ultimate Load and One-half of Ultimate Load (pdf, 100K, fig27.pdf)

When ultimate load theories are used to calculate bending moments, it is assumed that the earth pressures are distributed like the ultimate earth pressure. This is not true, however. With half of the ultimate load applied, the earth pressure varies with depth something like the solid line in Figure 2.7. Where the movements of the shaft are largest (at the top and the bottom of the shaft), a larger fraction of the ultimate pressure is mobilized. Where the movements of the shaft are smaller, a smaller fraction of the ultimate pressure is mobilized.

The magnitudes of the bending moments in the shaft are affected by the earth pressure distribution. The moment arm between the applied load and the resultant force due to the earth pressure is smaller for the actual earth pressure distribution than for one-half of the ultimate earth pressure. As a result, the maximum bending moment calculated for the actual distribution is smaller than that calculated using one-half of the ultimate distribution. Thus bending moments for working load conditions calculated using ultimate load theories are conservatively large. More accurate bending moments for working load conditions can be calculated using pile-soil interaction computer programs such as LPILE1 (Reese and Wang, 1989)

2.7 Summary

The three ultimate load theories summarized in this chapter can be used to calculate lateral load capacities of drilled shaft foundations. Broms's theories are simpler than Brinch-Hansen's. While it is possible to solve all of them using hand calculation, the complexity of Brinch-Hansen's theory makes its use by hand impractical. Any of the three theories can be solved using spreadsheets or other types of computer programs.

Broms's (1964b) theory and Brinch-Hansen's (1961) theory have been programmed in the spread sheet LCAP discussed in subsequent chapters, and capacities calculated using these theories have been compared to capacities measured in field tests on drilled shafts. LCAP provides a simple and practical tool for design of drilled shaft sound wall foundations.

CHAPTER 3 - FIELD LOAD TESTS

3.1 Introduction

Field load tests on 20 drilled shafts were performed at five sites in Virginia to assess the lateral load behavior of drilled shafts in Virginia soils. The soils in many locations within Virginia are partly saturated, and their strengths are characterized by both cohesion and friction. The most widely used theories for estimating drilled shaft capacity assume that the strength of the soil is characterized entirely by cohesion or entirely by friction. It was therefore considered important to investigate the capacities of drilled shafts in Virginia soils experimentally, to provide a sound basis for design of drilled shaft sound wall foundations in the Commonwealth. The methods used in these investigations, and the results of the load tests, are described in the following sections.

3.2 Test Shaft Construction

Four shafts were constructed at each site, as shown in Figure 3.1. Their diameters were either eight inches or nine inches, and their lengths were all 48 inches. At Price's Fork (the first site), the holes were drilled using the Virginia Tech drill rig, and were reamed to their final diameters by hand. The holes for the model shafts at the other four sites were drilled by VDOT drill crews, and were reamed by hand to their final dimensions.

Figure 3.1: Arrangement of Test Shafts (pdf, 50K, fig31.pdf)

Eight Standard Penetration Tests (SPTs) were performed at each site. At Price's Fork, the SPTs were performed by VDOT after the drilled shafts had been constructed. At the other sites, the SPTs were performed by the VDOT drill crews in the holes being drilled for the tests shafts.

The concrete for the shafts was mixed by hand at the sites, using commercial pre-mixed bagged cement and aggregate to which only water had to be added. The concrete conformed to ASTM C-387 specifications. The strength of the concrete for all of the sites was checked by performing compression tests, following the ASTM C-39 test procedure. The concrete at the first site (Price's Fork) was allowed to cure for 28 days, and the compressive strength was determined to be 3,500 psi at the end of testing. High-early strength concrete was used at the other four sites to permit earlier load testing; this concrete achieved compressive strengths in excess of 3,500 psi at the time of testing.

The shafts were reinforced with two No. 6 deformed bars and four 0.75 inch threaded rods, configured as shown in Figure 3.2. The deformed bars were secured to a wire mesh cylinder 6.5 inches in diameter, which was positioned in the hole before the concrete was poured. The four threaded rods, which served as both tensile reinforcement and anchor bolts, were positioned using a template after the concrete was poured. The threaded rods and the deformed bars extended the full depth of the shafts.

Figure 3.2: Cross-Section of Concrete Shaft (pdf, 50K, fig32.pdf)

The test shafts, like full-scale sound wall drilled shaft foundations, have length-to-diameter ratios of five to six, and can be considered rigid with regard to their interaction with the surrounding ground. The criterion suggested by Bierschwale et al. (1981), as noted in Mayne et al. (1992), states that a shaft can be considered rigid if the length-to-diameter ratio is six or less. Another criterion for rigidity, suggested by Poulos and Davis (1980), employs a stiffness ratio K_R, which is defined as follows:

$$K_R = \frac{E_p I_p}{E_s L^4} \qquad (3.1)$$

where:

K_R = a dimensionless stiffness ratio,

E_p = modulus of elasticity of shaft concrete (3,000,000 psi),

I_p = gross (uncracked) moment of inertia of the shaft (201 in^4 for D = 8 inches),

E_s = Young's modulus of soil (2,000 psi for the stiffest soil, at the Salem site), and

L = shaft length (48 inches).

A shaft is considered to be rigid if $K_R > 0.01$. For the conditions tested in this study the value of K_R was 0.06 or more, which is considerably greater than the minimum value for a rigid shaft. Thus, by both criteria (those of Bierschwale et al. and Poulos and Davis) the shafts can be considered to be rigid. As a result, the bending deformations that occur when they are loaded are not appreciable compared to the deflections and rotations resulting from the deformation of the surrounding soil.

3.3 Loading Equipment

The loading equipment used for the tests was developed with these objectives in mind:

- Easy setup and disassembly,
- Multiple tests at each site, and
- Efficient and practical load reaction system.

To achieve these objectives, the system shown in Figures 3.3, 3.4, and 3.5 was developed.

Figure 3.3: Test Apparatus (pdf, 2000K, fig33.pdf)

Figure 3.4: Section View of Test Setup (pdf, 100K, fig34.pdf)

Figure 3.5: Plan View of Test Setup (pdf, 100K, fig35.pdf)

The shafts themselves were used to provide load reaction, as shown in Figure 3.4. Reaction posts were attached to two diagonally opposed shafts, like shafts 2 and 3 in Figure 3.5. These posts supported a horizontal reaction beam which was used to apply loads to a post attached to another shaft, like shaft 4 in Figure 3.5. The jacking system was arranged as shown in Figure 3.5, so that the load was applied in tension, through tie rods.

When the first test had been completed, the reaction beam was moved to the opposite side of the reaction posts, and the opposite shaft (like shaft 1 in Figure 3.5) was loaded in the same way. Subsequently, the reaction posts were moved to the two shafts that had already been loaded, and the two shafts that had originally been used to support the reaction posts and beam (shafts 2 and 3 in Figure 3.5) were then each loaded in turn. Despite the fact that they had already served as reaction shafts, the load-deflection responses of the third and fourth shafts were not significantly affected. Since the reaction loads were shared between them, they had been subjected to loads that were only one-half as large as the loads applied to the test shafts. Also, the direction of the previous reaction loading was perpendicular to the direction in which they were subsequently loaded to failure.

As shown in Figure 3.4, the reaction beam was placed at a height of 4 feet above the ground surface. This height (eccentricity) was chosen so that the ratio of eccentricity to shaft length was roughly consistent with typical drilled shaft sound wall foundations.

The applied loads were measured using a 5,000 lb. load cell calibrated for tension loading. Using the load cell in tension reduced extraneous loads on the load cell due to possible eccentricity. The jack used to apply the loads was a 25-ton hydraulic jack, operated with a hand pump. The jack had a stroke of 14 inches. For tests where greater deflection was desired, the jack was repositioned during the test.

The lateral deflections of the test shafts were measured using two position transducers located as shown in Figures 3.4 and 3.5. Because the load posts were essentially rigid, the readings from these two transducers could be extrapolated to determine the deflection at the top of the shaft and the difference in readings could be used to determine the angular rotation of the shaft.

The readings of the load cell and the position transducers were measured by a computer data acquisition system. This allowed for measurements to be taken at five-second to ten-second time intervals, providing great redundancy in the data. Backup readings were recorded manually.

3.4 Loading Procedure

As mentioned previously, the loads of greatest concern for sound walls are those due to wind. These loads are exerted over short periods of time. To model these short-term load applications, the tests were performed rapidly, reaching failure in 25 minutes or less.

The first load increment was approximately 500 lbs, and subsequent load increments were approximately 200 lb. Each load was maintained for a period of one minute, at which time another load increment was applied. During the one minute period between load increments, the jack was pumped at a slow rate to maintain the load as nearly constant as possible. As can be seen in Figure 3.6, this test procedure resulted in a series of nearly horizontal load plateaus. Examination of the data in Figure 3.6 shows where each load was maintained, and the movement that occurred during the one minute time period. As expected, the amount of deflection over the one minute time period increased as the level of loading increased.

Figure 3.6: Typical Load Deflection Curve (pdf, 50K, fig36.pdf)

The loading sequence was continued until the load could not be increased. This occurred in a dramatic manner in some cases, where brittle failure occurred. In some tests, less dramatic ductile failure occurred. In these ductile failure cases, the load could be maintained but not increased. Once failure was reached, loading was discontinued, to avoid additional deflection. Thus, especially for the cases where brittle failure occurred, it is unclear what shape the load deflection curve would have exhibited if further deflections had been imposed. It was clear, based on observations made during testing, that failure occurred due to reaching the ultimate capacity of the soil around the shafts. The only exception was at Price's Fork, where there was some crushing of the concrete beneath the base plate on the first shaft constructed, due to the poor quality of the concrete at the top of the shaft.

3.5 Test Results

A variety of soil conditions were encountered at the five sites. The properties of the soils are discussed in Chapter 4. The load deflection results from the five sites are shown in Figure 3.7 through 3.11. The deflection values shown on the horizontal axes are those at the tops of the shafts (at ground level). The average ultimate loads for the five sites are listed in Table 3.1. The Unified Soil classifications, the average degree of saturation(S), and the Standard Penetration Test blow count (N) are shown in the figures. The values of $(N_1)_{60}$ are values of N corrected for overburden pressure and hammer energy, as explained in Chapter 4.

Figure 3.7: Prices Fork Load Deflection Curves (pdf, 50K, fig37.pdf)

Figure 3.8: Salem Load Deflection Curves (pdf, 50K, fig38.pdf)

Figure 3.9: Suffolk Load Deflection Curves (pdf, 50K, fig39.pdf)

Figure 3.10: Fairfax County Parkway Load Deflection Curves (pdf, 50K, fig310.pdf)

Figure 3.11: Roberts Road Load Deflection Curves (pdf, 50K, fig311.pdf)

The results of Test No. 1 at Price's Fork, in which poor quality concrete affected the behavior, were not used in determining the average ultimate load for Price's Fork that is shown in Table 3.1.

The shafts at the Salem site all exhibited brittle soil failure, with the loads dropping off rapidly after the maximum load was reached. As noted previously, no attempt was made to maintain the load after failure when brittle failure occurred, to avoid unnecessarily large deflections.

Test shaft No. 4 at Suffolk (Figure 3.9) was subjected to two unload-reload cycles. The first unloading was at approximately 45% of the ultimate load and the second cycle was at about 62% of the ultimate load. These cycles did not appear to affect the ultimate load.

Test shaft No. 4 at Fairfax County Parkway (Figure 3.10) was also subjected to two unload-reload cycles. As at Suffolk, these load cycles did not appear to affect the failure load for the test.

The average deflection measured at the Roberts Road site was the smallest for any site. The deflections at failure in Tests No. 1 and No. 3 were only about 1.0 inch. The load deflection curves showed dramatic drop-off, characteristic of brittle failure.

Two unload-load cycles were performed during test No. 4 at Roberts Road (Figure 3.11), at approximately 40% and 64% of the ultimate load. The ultimate load achieved during this test was slightly lower than the ultimate for the other three tests, but the difference was not great.

3.6 Summary

As would be expected because of the variations in soil conditions at the five sites, the maximum loads and the magnitudes of deflection varied somewhat from one site to another. The values of the average ultimate load for each site are shown in Table 3.1, and average load-deflection curves for the five sites are shown in Figure 3.12. The load deflection curves shown in Figure 3.12 are average curves for the four tests performed at each site.

Figure 3.12: Summary of Field Load Tests (pdf, 50K, fig312.pdf)

The values of ultimate load at the Roberts Road site were the highest and the deflections were the smallest of any of the five sites. As discussed in Chapter 4, the soil at Roberts Road was non-plastic, and had the highest corrected blow count of any site, $(N_1)_{60} = 38$. The smallest values of ultimate load, and the largest values of deflection, were measured at the Fairfax County Parkway site. The soil at this site contained highly plastic clay and silt (CL to CH), and had the second lowest corrected blow count of any site, $(N_1)_{60} = 13$.

The load-deflection curves for the Salem site and the Roberts Road site exhibited the most dramatic drop-off of load after peak load was reached. The Salem soil contained silt and clay of low plasticity, and had a corrected blow count $(N_1)_{60} = 22$.

The load and unload cycles performed at the Suffolk, Fairfax County Parkway, and Roberts Road Site showed that two cycles had little effect on the measured values of ultimate load. However, each load cycle did induce added deformations in the range of 0.1 inches at about 40% of the ultimate load. Since only two cycles of loading were performed, the behavior under additional cycles cannot be generalized from this data, and the effects of cyclic load variations needs further study.

Table 3.1: Average Ultimate Loads from Field Tests

Site	$(N_1)_{60}$	Average Ultimate Load (lbs)
Prices Fork	15	4000
Salem	22	4600
Suffolk	8	4500
Fairfax County Parkway	13	3700
Roberts Road	38	4650

CHAPTER 4 - SOIL PROPERTIES

4.1 Introduction

Field and laboratory tests were performed to assess the properties of the soils at the sites where the load tests were performed. Standard Penetration Tests (SPT) were performed in the field. Index tests and triaxial compression tests were performed in the laboratory.

4.2 Standard Penetration Tests

The SPT tests were performed according to ASTM D-1586. A total of eight tests were performed at each site, two tests in each of four boreholes. For all sites except Prices Fork, the SPT's were performed in the boreholes in which test shafts were later constructed, as shown Figures 4.1 through 4.5. Within each borehole, SPT values were determined at two depths.

Figure 4.1: Site Plan, Prices Fork (pdf, 50K, fig41.pdf)

Figure 4.2: Site Plan, Salem (pdf, 50K, fig42.pdf)

Figure 4.3: Site Plan, Suffolk (pdf, 50K, fig43.pdf)

Figure 4.4: Site Plan, Fairfax County Parkway (pdf, 50K, fig44.pdf)

Figure 4.5: Site Plan, Roberts Road (pdf, 50K, fig45.pdf)

The type of hammer and the release mechanism varied from site to site. These variations were taken into account by correcting the SPT N-values for hammer energy. The measured N-values were also corrected for the effect of overburden pressure. The corrected N-values are denoted as values of $(N_1)_{60}$, which corresponds to a standardized overburden pressure of one ton per square ft., and a standardized hammer energy equal to 60 percent of the theoretical value. Additional corrections were made to account for the test procedures used with respect to borehole diameter, sampler liners, and rod length. The following equation was used to account for these effects:

$$(N_1)_{60} = N_{Field} C_N C_E C_R C_B C_S \tag{3.1}$$

where:

$(N_1)_{60}$ = N-value corrected to 60% of the theoretical energy and 1.0 t/ft^2 overburden pressure,

N_{Field} = number of blows per foot measured in the field,

C_N = overburden pressure correction factor,

$C_N = 0.77 \log\left(\frac{20}{\sigma_{vo}'}\right)$ (Peck, Hansen, and Thornburn, 1974),

σ_{vo}' = effective overburden pressure ($\geq$0.25 t/ft^2)

C_E = hammer energy correction factor,

$C_E = \frac{(E.R.)}{60}$ (Skempton 1986),

C_R = correction factor for rod length (=0.75 for rod length, 4 meters),

C_B = borehole diameter correction factor (=1.15 for 8 in diameter hole),

C_S = sampler correction factor (=1.2 since no liners were used),

σ_{vo}' = effective vertical overburden pressure ($\geq$ 0.25t/ft^2), and

E.R. = energy ratio (%) for specific SPT hammer.

Hammer energies were not measured during the tests. To account for hammer energy effects the following energy ratios, recommended by Kovacs (1994), were used for the hammers and release mechanisms used in tests:

- E.R. = 60 for safety hammer and cathead release mechanism, and
- E.R. = 90 for automatic trip safety hammer.

The blow count values from the SPTs are summarized in Table 4.1. Also shown in Table 4.1 are the hammer types and release mechanisms, the unit weights of the soils, and the resulting correction factors for hammer energy and overburden pressure used for each site in determining $(N_1)_{60}$ values. The average values of N_{field}, $(N_1)_{60}$, and standard deviations of these quantities for each site are shown in Table 4.2.

4.3 Summary of SPT Results

The values of $N_{field} = 17$ and $(N_1)_{60} = 38$ for Roberts Road were the highest of the five sites. Both Suffolk and Fairfax County Parkway had $N_{field} = 6$, the lowest for any of the sites. Due to differences in the release mechanisms of the SPT equipment used at these sites, the values of $(N_1)_{60}$ were 8 for Suffolk and 13 for Fairfax County Parkway.

The values of N_{field} and $(N_1)_{60}$ measures at Roberts Road varied more widely than at the other sites. The values of N_{field} ranged from 10 to 22 and the values of $(N_1)_{60}$ ranged from 23 to 50. The smallest variation in blow count was found at Suffolk, where the values of N_{field} varied from 5 to 6, and the $(N_1)_{60}$ varied from 8 to 9.

Examining the field blow count values in comparison to the corrected blow count values, it is evident that the corrections for overburden pressure and hammer energy are significant. The overburden corrections are greater than unity and are quite large due to the shallow depths at which the tests were performed. Since in most cases drilled shaft sound wall foundations will be constructed at shallow depths (although not as shallow as the test shafts) the overburden pressure effect can be expected to be quite significant.

4.4 Classification Tests

The split spoon samples from the Standard Penetration Tests were used for Atterberg Limit and grain size analysis tests. The ASTM D-4318 and ASTM D-422 test procedures respectively were followed in performing the tests. Using the results of these tests, the soils were classified according to the Unified Soil Classification System described in ASTM D-2487. The results of the classification tests are summarized in Table 4.3, and the results of the Atterberg Limits tests are shown in Figure 4.6. Individual characteristics of the soils at each site are described below.

Figure 4.6: Summary of Atterberg limits (pdf, 50K, fig46.pdf)

Table 4.1: SPT Blow Count Values and Corrections

Prices Fork						**Salem**						**Suffolk**					
Automatic Trip Safety Hammer						Automatic Trip Safety Hammer						Cathead Release Safety Hammer					
Unit Weight = 110 pcf.						Unit Weight = 127 pcf.						Unit Weight = 122 pcf.					
Boring No.	Depth (ft.)	N_{Field}	C_N	C_E	$(N_1)_{60}$	Boring No.	Depth (ft.)	N_{Field}	C_N	C_E	$(N_1)_{60}$	Boring No.	Depth (ft.)	N_{Field}	C_N	C_E	$(N_1)_{60}$
B-1	2’-3.5’	6	1.47	1.5	14	B-1	.5’-2’	12	1.47	1.5	27	B-1	.5’-2’	6	1.47	1	9
B-1	4’-5.5’	4	1.43	1.5	9	B-1	2.5’-4’	9	1.47	1.5	21	B-1	2.5’-4’	5	1.47	1	8
B-2	2’-3.5’	7	1.47	1.5	16	B-2	.5’-2’	11	1.47	1.5	25	B-2	.5’-2’	6	1.47	1	9
B-2	4’-5.5’	8	1.43	1.5	18	B-2	2.5’-4’	7	1.47	1.5	16	B-2	2.5’-4’	5	1.47	1	8
B-3	2’-3.5’	8	1.47	1.5	18	B-3	.5’-2’	14	1.47	1.5	32	B-3	.5’-2’	5	1.47	1	8
B-3	4’-5.5’	7	1.43	1.5	16	B-3	2.5’-4’	8	1.47	1.5	18	B-3	2.5’-4’	6	1.47	1	9
B-4	2’-3.5’	7	1.47	1.5	15	B-4	2’-3.5’	6	1.47	1.5	14	B-4	2’-3.5’	5	1.47	1	8
B-4	4’-5.5’	5	1.43	1.5	11	B-4	4’-5.5’	11	1.39	1.5	25	B-4	4’-5.5’	6	1.40	1	9

Note: $C_B = 1.15$, $C_R = 0.75$, $C_S = 1.2$

Table 4.1: (Continued) SPT Blow Count Values and Corrections

Fairfax County Parkway						**Roberts Road**					
Automatic Trip Safety Hammer						Automatic Trip Safety Hammer					
Unit Weight = 114 pcf.						Unit Weight = 110 pcf.					
Boring No.	Depth (ft.)	N_{Field}	C_N	C_E	$(N_1)_{60}$	Boring No.	Depth (ft.)	N_{Field}	C_N	C_E	$(N_1)_{60}$
B-1	.5’-2’	5	1.47	1.5	11	B-1	.5’-2’	10	1.47	1.5	23
B-1	2.5’-4’	4	1.47	1.5	9	B-1	2.5’-4’	15	1.47	1.5	34
B-2	.5’-2’	4	1.47	1.5	9	B-2	.5’-2’	14	1.47	1.5	32
B-2	2.5’-4’	9	1.47	1.5	21	B-2	2.5’-4’	22	1.47	1.5	50
B-3	.5’-2’	4	1.47	1.5	9	B-3	.5’-2’	15	1.47	1.5	34
B-3	2.5’-4’	3	1.47	1.5	7	B-3	2.5’-4’	20	1.47	1.5	46
B-4	2’-3.5’	8	1.47	1.5	18	B-4	2’-3.5’	16	1.47	1.5	37
B-4	4’-5.5’	9	1.42	1.5	21	B-4	4’-5.5’	22	1.43	1.5	50

Note: $C_B = 1.15$, $C_R = 0.75$, $C_S = 1.2$

Table 4.2: Summary of SPT Results

Site	Average N_{field}	Standard Deviation	Average $(N_1)_{60}$	Standard Deviation
Prices Fork	7	1.4	15	3.2
Salem	10	2.7	22	6.2
Suffolk	6	0.5	8	0.8
Fairfax County Parkway	6	2.5	13	5.7
Roberts Road	17	4.2	38	9.7

The material encountered at Prices Fork was consistent in all the holes, with all the material classifying as highly plastic elastic silt (MH). All of the Prices Fork samples plot in the same general area on the Plasticity Chart in Figure 4.6. The natural moisture content was below the Plastic Limit for most of the samples.

The Salem soils showed more variation than those from Prices Fork. Most of the samples classified as sandy lean clay (CL). The natural moisture content of the soil was below the Plastic Limit for all the samples.

The classification of the Suffolk soils ranged from sandy silt (ML) to sandy lean clay (CL). The natural moisture content was near the Plastic Limit, ranging from 3 percent below to 4 percent above the PL.

The soils at Fairfax County Parkway showed the widest variability of any of the sites. Soil classifications ranged from sandy silt (ML) to highly plastic clay (CH). SPT values also varied widely: values of N_{field} varied from 3 to 9, and values of $(N_1)_{60}$ varied from 7 to 20. The natural moisture contents ranged from 9 percent below the PL to 8 percent above.

All of the Roberts Road samples classified as non-plastic sandy silt (ML). Natural moisture contents varied from 14 to 23 percent. Values of Plastic Limit could not be determined for these non-plastic soils.

4.5 Summary of Classification Test Results

It is evident from the data in Table 4.3, a variety of soils were encountered at the sites included in this study. This was one of the objectives of performing field load tests at different locations in the Commonwealth. The soils at Prices Fork and Roberts Road were silts (MH and ML). The other three sites showed more variability, but for the most part the soils classified as lean clay (CL). The greatest variability was evident in the Fairfax County Parkway soils with classifications ranging from ML to CH. All of the soils tested in this study were fine-grained.

4.6 Triaxial Tests

Triaxial tests were performed to assess the shear strengths of the soils from each site. The tests were performed on undisturbed samples obtained following ASTM D-1587, using 3 inch diameter thin-walled sample tubes. Because the most severe loading

Table 4.3: Classification Test Results

Site	Depth Interval	Boring Number	Visual Description	USCS Symbol	Field N-Value	$(N_1)_{60}$	Natural Moisture Content (%)	Plastic Limit	Liquid Limit	% Fines
Prices Fork	2'-3.5'	B-1	reddish orange elastic silt with sand	MH	6	13	48	61	84	72.2
	2'-3.5'	B-2	reddish orange elastic silt with sand	MH	7	15	45	66	77	72.7
	2'-3.5'	B-3	reddish orange elastic silt with sand	MH	8	18	41	52	89	80.1
	2'-3.5'	B-4	reddish orange elastic silt with sand	MH	7	15	45	58	93	82.7
	4'-5.5'	B-1	reddish orange elastic silt with sand	MH	4	9	55	47	84	70.2
	4'-5.5'	B-2	reddish orange elastic silt with sand	MH	8	18	44	70	89	80.1
	4'-5.5'	B-3	reddish orange elastic silt with sand	MH	7	15	45	39	73	73.5
	4'-5.5'	B-4	reddish orange elastic silt	MH	5	11	53	59	82	91.4
Salem	.5'-2'	B-1	gray, sandy lean clay	CL	12	26	12	18	38	69.0
	.5'-2'	B-2	brown, sandy silt	ML	11	24	12	23	31	62.1
	.5'-2'	B-3	brown, sandy lean clay	CL	14	31	14	20	44	54.0
	2'-3.5'	B-4	gray lean clay with sand	CL	6	13	15	18	47	71.2
	2.5'-4'	B-1	brown, sandy lean clay	CL	9	20	15	18	40	64.5
	2.5'-4'	B-2	gray, sandy lean clay	CL	7	15	14	18	36	58.8
	2.5'-4'	B-3	gray, sandy lean clay	CL	8	18	15	16	28	66.4
	4'-5.5'	B-4	brown silt with sand	ML	11	24	21	31	46	79.6
Suffolk	.5'-2'	B-1	gray, sandy silty clay	CL-ML	6	9	16	17	23	62.1
	.5'-2'	B-2	gray, sandy silty clay	CL-ML	6	9	15	18	22	56.5
	.5'-2'	B-3	gray, sandy silt	ML	5	7	18	18	22	54.5
	2'-3.5'	B-4	gray, sandy lean clay	CL	5	7	23	19	38	68.0
	2.5'-4'	B-1	brown, sandy lean clay	CL	5	7	18	18	35	57.0
	2.5'-4'	B-2	brown, sandy lean clay	CL	5	7	23	19	40	64.6

Table 4.3: (Continued) Classification Test Results

Site	Depth Interval	Boring Number	Visual Description	USCS Symbol	Field N-Value	$(N_1)_{60}$	Natural Moisture Content (%)	Plastic Limit	Liquid Limit	% Fines
Suffolk	2.5'-4'	B-3	brown, sandy lean clay	CL	6	9	20	18	29	65.0
	4'-5.5'	B-4	brown, sandy lean clay	CL	6	9	24	20	38	68.8
Fairfax County Parkway	.5'-2'	B-1	light brown, sandy lean clay	CL	5	11	21	23	41	67.1
	.5'-2'	B-2	light brown, lean clay	CL	4	9	35	27	49	86.9
	.5'-2'	B-3	brown, lean clay with sand	CL	4	9	23	24	45	83.8
	2'-3.5'	B-4	brown, sandy silt	ML	8	18	24	31	43	68.8
	2.5'-4'	B-1	brown, fat clay	CH	4	9	35	31	68	93.6
	2.5'-4'	B-2	brown, sandy lean clay	CL	9	20	17	26	36	51.8
	2.5'-4'	B-3	brown, fat clay	CH	3	7	33	32	76	88.0
	4'-5.5'	B-4	brown, sandy silt	ML	9	20	23	29	37	54.6
Roberts Road	.5'-2'	B-1	light brown, nonplastic sandy silt, some organics present	ML	10	22	23	N/A	N/A	67.3
	.5'-2'	B-2	light brown, nonplastic sandy silt, trace of organics	ML	14	31	18	N/A	N/A	57.6
	.5'-2'	B-3	light brown, nonplastic sandy silt, some organics present	ML	15	33	17	N/A	N/A	53.7
	2'-3.5'	B-4	light brown, nonplastic sandy silt, some organics present	ML	16	35	15	N/A	N/A	51.7
	2.5'-4'	B-1	light brown, nonplastic sandy silt, trace of organics	ML	15	33	18	N/A	N/A	58.5
	2.5'-4'	B-2	light brown, nonplastic sandy silt, trace of organics	ML	22	49	14	N/A	N/A	55.1
	2.5'-4'	B-3	light brown, nonplastic sandy silt, trace of organics	ML	20	44	14	N/A	N/A	59.7
	4'-5.5'	B-4	light brown, nonplastic sandy silt	ML	22	49	16	N/A	N/A	56.8

conditions for sound wall foundations are short term wind loads, unconsolidated-undrained triaxial tests were performed on the samples following ASTM D-2850. A loading rate of 1% axial strain per minute was used in performing the tests. This led to testing times of approximately 20 minutes, because the tests were continued to 20% axial strain.

The confining pressures used during the tests were in the same range as the estimated overburden pressures at the sample depths.

Peak deviator stress was used as the failure criterion if the peak was reached at 10% axial strain or less. If the peak did not occur below 10% axial strain, the deviator stress at 10% axial strain was used as the failure criterion.

Deviator stress versus axial strain curves for the tests are shown in Figures 4.7a, 4.8a, 4.9a, 4.10a, and 4.11a. The strength envelopes derived from the test results are shown in Figures 4.7b, 4.8b, 4.9b, 4.10b, and 4.11b. The depth, dry density, degree of saturation, and natural moisture content for each specimen are summarized in Table 4.4, and the strength properties and material properties are given in Table 4.5.

Figure 4.7a: Prices Fork, Deviator Stress vs. Axial Strain Curves (pdf, 50K, fig47a.pdf)

Figure 4.7b: Prices Fork, Strength Envelope (pdf, 50K, fig47b.pdf)

Figure 4.8a: Salem, Deviator Stress vs. Axial Strain Curves (pdf, 50K, fig48a.pdf)

Figure 4.8b: Salem, Strength Envelope (pdf, 50K, fig48b.pdf)

Figure 4.9a: Suffolk, Deviator Stress vs. Axial Strain Curves (pdf, 50K, fig49a.pdf)

Figure 4.9b: Suffolk, Strength Envelope (pdf, 50K, fig49b.pdf)

Figure 4.10a: Fairfax County Parkway, Deviator Stress vs. Axial Strain Curves (pdf, 50K, fig410a.pdf)

Figure 4.10b: Fairfax County Parkway, Strength Envelope (pdf, 50K, fig410b.pdf)

Figure 4.11a: Roberts Road, Deviator Stress vs. Axial Strain Curves (pdf, 50K, fig411a.pdf)

Figure 4.11b: Roberts Road, Strength Envelope (pdf, 50K, fig411b.pdf)

The soil at Prices Fork was very brittle, and it was not possible to trim 1.4 inch test specimens. The soil was extruded from the thin-walled samplers, cut to the appropriate length, and tested without trimming the diameter. Four tests were performed with somewhat varied results. The failure envelope was selected conservatively. Because the soil is partially saturated, it has both a cohesion intercept and a friction angle in UU tests, as can be seen in Figure 4.7b. The material from Prices Fork had the largest cohesion intercept of any site, c = 400 psf.

The same method of specimen preparation was used for the Salem soil as for the Prices Fork brittle soil. The Salem soils contained particles which approached one-half inches in diameter. Since the diameter of triaxial test specimens should be at least six times the largest particle size, a specimen diameter of 1.4 inches would not have been adequate. The test results show considerable variability. In drawing the failure envelope shown in Figure 4.8b, the results from the tests at 2, 3, and 5 psi. confining pressure were weighed heavily.

Table 4.4: Specimen Data from Triaxial Tests

Site	Depth Interval	Dry Density (pcf.)	Degree of Saturation (%)	Natural Moisture Content (%)
Prices Fork	2’-4’	74.9	99.8	46
	2’-4’	83.3	100.0	39
	3’-5’	77.2	95.6	42
	3’-5’	73.7	96.2	46
Salem	2’-4’	110.0	96.3	19
	2’-4’	101.5	99.9	24
	3’-5’	97.8	83.2	22
	3’-5’	105.1	97.2	22
	1’-3’	111.4	92.4	18
Suffolk	3’-5’	99.8	82.3	21
	3’-5’	101.0	85.0	21
	3’-5’	101.8	91.2	22
	3’-5’	100.4	84.2	21
Fairfax County Parkway	2.0’-2.5’	80.9	95.1	38
	2.5’-3.5’	80.4	90.7	37
	3.0’-3.5’	81.0	92.2	37
	3.5’-4’	88.5	100.0	34
Roberts Road	1’-3’	101.4	61.9	15
	2’-4’	86.0	43.7	16
	2’-4’	94.7	55.3	16
	2’-4’	98.2	56.1	15

Table 4.5: Triaxial Test Results and Average N-values

Site	Average Dry Density (pcf.)	Average Degree of Saturation (%)	Natural Moisture Content (%)	Moist Density (pcf.)	Friction Angle (Degrees)	Cohesion (psf.)	Average Field N-Value	Average $(N_1)_{60}$
Prices Fork	77	98	43	110	31	400	7	14
Salem	105	94	21	127	34	350	10	21
Suffolk	100	86	22	122	31	350	6	8
Fairfax County Parkway	83	94	37	114	30	200	6	13
Roberts Road	95	54	16	110	40	150	17	37

Tests on samples from Suffolk were performed on 1.4 inch diameter specimens. Upon extrusion from the thin-walled sampler, the samples were trimmed to the appropriate diameter and height. The soil was less brittle then the Prices Fork and Salem soils, and the results are more consistent.

Material from Fairfax County Parkway was also tested using 1.4 inch diameter specimens. This material had the lowest friction angle value (30^o) and a small cohesion intercept (200 psf).

The tests on the Roberts Road soil were performed on 1.4 inch diameter specimens. It can be seen that the deviator stress versus axial strain curves in Figure 4.11a have peculiar shapes. It was noticed during the tests that there were very thin bands within the specimens which ran in the same direction as the eventual failure plane. The bands appeared to be thin zones of weaker soil. The strains within these zones may have been higher than the strains in the surrounding material, which may be related to the small values of average strain at failure and the erratic stress-strain behavior of these specimens. This material exhibited the largest friction angle value (40^o) and the smallest cohesion intercept (150 psf) of any of the soils tested.

The values of friction angle for the soils tested ranged from 30^o to 40^o and the values of cohesion intercept ranged from 150 psf to 400 psf, as shown in Table 4.5.

4.7 Relationship Between ϕ Values from Triaxial Tests and SPT Blow Counts

Blow count values from Standard Penetration Tests are often used to estimate values of friction angle (ϕ). As is evident from the data in Table 4.1, the energy of the hammer and the overburden associated with the test can affect the blow count value. It is desirable to account for these effects when SPT test results are used to estimate values of ϕ, because the corrected results provide a better measure of the strength of the soil.

The values of ϕ determined in this study with N_{field}, N_1, and $(N_1)_{60}$ are shown in Figures 4.12, 4.13, and 4.14. Also shown in Figures 4.12, 4.13, and 4.14 are empirical correlations for N_{field}, N_1, and $(N_1)_{60}$ respectively. Although the empirical correlations were developed for effective stress friction angles for sands and gravels, they nevertheless give reasonable values of ϕ for the partly saturated Virginia silts and clays.

Figure 4.12: Variation of Friction Angle with N_{field} (pdf, 50K, fig412.pdf)

Figure 4.13: Variation of Friction Angle with N_1 (pdf, 50K, fig413.pdf)

Figure 4.14: Variation of Friction Angle with $(N_1)_{60}$ (pdf, 50K, fig414.pdf)

It should be noted that the Virginia soils tested had cohesion intercepts ranging from 150 psf to 400 psf. This is an additional component of strength not reflected in correlations between blow count and friction angle developed for sands and gravels. The effect of cohesion on the load capacities of drilled shaft sound wall foundations is examined later in this report.

It is possible that the degree of saturation, therefore the cohesion and the friction angle values, would vary seasonally with rainfall and or other weather related factors such as ground freezing and snow melt. To be fully reliable the soil strength parameters, the method of analysis, and the factor of safety should account for the most severe set of conditions likely to be experienced during the design life of the sound wall.

CHAPTER 5 - COMPARISON OF MEASURED AND CALCULATED ULTIMATE LATERAL LOAD CAPACITIES

5.1 Introduction

The field load tests described in Chapter 3, together with the triaxial compression tests described in Chapter 4, provide a basis for evaluating the accuracy of Broms's theory and Brinch-Hansen's theory for estimating the lateral load capacities of drilled shafts.

Drilled shaft capacities were calculated using the values of unit weight, cohesion intercept, and angle of internal friction discussed in Chapter 4, which are summarized in Table 5.1.

Capacities were calculated using five different methods:

1) Broms's (1964b) theory for cohesionless soils, using the measured friction angles and setting the cohesion values equal to zero.

2) Brinch-Hansen's (1961) theory for soils with both cohesion and friction,

3) Brinch-Hansen's (1961) theory with cohesion set equal to zero,

4) Brinch-Hansen's (1961) theory neglecting soil resistance in the upper 1.5 times shaft diameter, as recommended by Broms (1964a), and

5) Brinch-Hansen's (1961) theory, multiplying calculated values of capacity by 0.85.

The capacities calculated using these methods are compared to the measured loads in the following sections.

5.2 Broms's (1964b) Theory

As shown in Table 5.2, the measured shaft capacities exceeded those calculated using the Broms's (1964b) theory by a considerable margin. The values of measured capacity/calculated capacity varied from 3.1 to 4.4, with an average of 3.8.

These large differences between theory and measurement are due to the fact that the Broms's (1964b) theory is formulated for soils with no cohesion. The values of calculated shaft capacity shown in Table 5.2 were computed using the values of ϕ shown in Table

5.1. Since cohesion is not included in this theory, the values of cohesion (c) shown in Table 5.1 were in effect set equal to zero.

5.3 Brinch-Hansen's (1961) Theory

As shown in Table 5.3, the measured shaft capacities agreed well on average with those computes using Brinch-Hansen's Theory. The values of measured capacity/calculated capacity varied from 0.8 to 1.4, with an average of 1.0.

5.4 Brinch-Hansen's (1961) Theory with Cohesion Set Equal to Zero

To provide a direct measure of the contribution of cohesion to shaft capacity, capacities were also computed using Brinch-Hansen's (1961) theory with the values of cohesion set equal to zero. The results are shown in Table 5.4. It can be seen that the calculated values of shaft capacity are even smaller than those shown in Table 5.2, which were calculated using the Broms's (1964b) theory. The values of measured capacity/calculated capacity vary from 3.4 to 7.3, with an average of 5.9.

It is clear that with cohesion (c) set equal to zero, the Brinch-Hansen theory is more conservative than the Broms theory. As shown in Table 5.5, this difference decreases with increasing values of ϕ.

5.5 Brinch-Hansen's (1961) Theory Neglecting Soil Resistance in the Top 1.5 Times Shaft Diameter

Broms (1964a) recommended that, for cohesive soils, the soil resistance in the top 1.5 shaft diameters should be neglected as an allowance for possible disturbance and loosening of the material during construction. Shaft capacities calculated in this way using Brinch-Hansen's Theory are summarized in Table 5.6.

It can be seen that neglecting the soil resistance near the top of the shaft results in conservative estimates of shaft capacity. The calculated values of measured capacity/calculated capacity in Table 5.6 vary from 1.2 to 1.9, with an average of 1.4. Although the soil near the tops of the shafts may have been somewhat disturbed during construction, considerable passive resistance was nevertheless developed in this area by the time the ultimate load condition was reached.

5.6 Brinch-Hansen's (1961) Theory, with Calculated Capacities Multiplied by 0.85

The best agreement with measured capacities is achieved with the Brinch-Hansen (1961) Theory, including cohesion and including soil resistance near the top of the shaft.

Table 5.1 Summary of Soil Properties

Site	Soil Description	γ_m (lb/ft^3)	ϕ (deg)	c (psf)
Price's Fork	Elastic Silt (MH)	110	31	400
Salem	Sandy Lean Clay (CL)	127	34	350
Suffolk	Sandy Silt (ML) to Sandy Lean Clay (CL)	122	31	350
Fairfax County Parkway	Sandy Silt (MH) to Highly Plastic Clay (CH)	114	30	200
Roberts Road	Non-plastic Sandy Silt (ML)	110	40	150

Table 5.2: Comparison of Measured and Calculated Ultimate Lateral Capacities for Broms's (1964b) Theory

Site	Measured Capacity (lbs)	Calculated Capacity (lbs)	Measured/Calculated
Price's Fork	3900	920	4.24
Salem	4600	1180	3.90
Suffolk	4500	1020	4.41
Fairfax County Parkway	3700	1030	3.59
Roberts Road	4650	1520	3.06
Average			3.84

Table 5.3: Comparison of Measured and Calculated Ultimate Lateral Capacities for Brinch-Hansen's (1961) Theory

Site	Measured Capacity (lbs)	Calculated Capacity (lbs)	Measured/Calculated
Price's Fork	3900	4780	0.82
Salem	4600	5430	0.85
Suffolk	4500	4320	1.04
Fairfax County Parkway	3700	2745	1.35
Roberts Road	4650	4660	1.00
Average			1.01

Table 5.4: Comparison of Measured and Calculated Ultimate Lateral Capacities for Brinch-Hansen's (1961) Theory with Cohesion Set Equal to Zero

Site	Measured Capacity (lbs)	Calculated Capacity (lbs)	Measured/Calculated
Price's Fork	3900	560	6.96
Salem	4600	820	5.61
Suffolk	4500	620	7.26
Fairfax County Parkway	3700	580	6.38
Roberts Road	4650	1350	3.44
Average			5.93

Table 5.5: Comparison of Brinch-Hansen's (1961) Theory and Broms's (1964b) Theory for Differing Friction Angles (Cohesion = 0)

ϕ (deg)	Brinch-Hansen Theory Calculated Capacity (lbs)	Broms Theory Calculated Capacity (lbs)	Ratio (Broms/Brinch-Hansen)
30	560	960	1.71
32	660	1040	1.58
34	780	1130	1.45
36	930	1230	1.32
38	1100	1340	1.22
40	1320	1470	1.11

* Note: shaft diameter = 8 in, shaft length = 4 ft, eccentricity = 4 ft,

unit weight = 120 pcf , and cohesion = 0.

Table 5.6: Comparison of Measured and Calculated Ultimate Lateral Capacities for Brinch-Hansen's (1961) Theory Neglecting Soil Resistance in the Top 1.5 Times Shaft Diameter

Site	Measured Capacity (lbs)	Calculated Capacity (lbs)	Measured/Calculated
Price's Fork	3900	3370	1.16
Salem	4600	3910	1.18
Suffolk	4500	3070	1.47
Fairfax County Parkway	3700	1930	1.92
Roberts Road	4650	3520	1.32
Average			1.41

As shown in Table 5.3, the average value of measured capacity/calculated capacity for this case is 1.0.

Although capacities calculated using the Brinch-Hansen (1961) theory are close to the measured values on average, the measured values were smaller than the calculated values for Prices' Fork and Salem. As shown in Table 5.7, the reliability of Brinch-Hansen's (1961) theory can be improved if the theoretical values are reduced by 15% (multiplied by 0.85). When this is done, the values of measured capacity/calculated capacity vary from 1.0 to 1.6, with an average of 1.2.

5.7 Summary

Based on these comparisons between measured and calculated capacities, the following conclusions appear to be justified:

1. Broms's (1964b) theory, which is formulated for cohesionless soils, underestimates the capacities measured at all five sites where tests were performed.

2. The contribution of cohesion to the capacities of the test shafts is quite large. Note however, that the percentage contribution of cohesion to shaft capacity decreases as shaft size increases, as discussed in Chapter 6.

3. Brinch-Hansen's (1961) theory, which includes both cohesion and friction angle, provides an accurate measure of estimating shaft capacities, on average. The theory slightly overestimates shaft capacities for the Price's Fork and Salem sites.

4. The reliability of Brinch-Hansen's (1961) theory can be improved if the theoretical capacities are reduced by 15% (i.e. if the theoretical capacities are multiplied by 0.85).

On the basis of these comparisons, it is recommended that reliable and reasonably accurate estimates of shaft capacity can be made by:

- Using Brinch-Hansen's (1961) theory.

- Including cohesion as well as friction.

- Including soil resistance near the top of the shaft.

- Reducing the calculated values by 15% to improve reliability.

Table 5.7: Comparison of Measured and Calculated Ultimate Lateral Capacities for Brinch-Hansen's (1961) Theory with Calculated Capacity Multiplied by 0.85

Site	Measured Capacity (lbs)	Calculated Capacity (lbs)	Measured/Calculated
Price's Fork	3900	4060	0.96
Salem	4600	4610	1.00
Suffolk	4500	3680	1.22
Fairfax County Parkway	3700	2300	1.61
Roberts Road	4650	3960	1.17
Average			1.19

CHAPTER 6 - EFFECTS OF ANALYSIS PROCEDURES AND PARAMETER VALUES

6.1 Introduction

A parametric study was performed to assess the effect of various factors on the required shaft length. The study was performed using the program LCAP, described in Appendix C. Wall heights of 15 feet and 30 feet were used in the study, together with a wind speed of 80 mph, and exposure category D. Shown in Table 6.1 are the specific values used in the study. The effects of the analysis procedures and parameter values are summarized in the following paragraphs.

6.2 Effect of Theory Used in the Analysis

Analyses performed using Brinch-Hansen's (1961) theory for c = 0 usually result in shaft lengths that are longer than those found using Broms's (1964b) theory. The difference is greatest for larger shaft diameters, and may be as much as 50%.

6.3 Effect of Cohesion

When cohesion is included in Brinch-Hansen's (1961) theory, the resulting shaft lengths may be longer or shorter than those found using Broms's (1964b) theory with c = 0. Shaft lengths calculated using Brinch-Hansen's (1961) theory with c = 150 psf range from 20% shorter to 20% longer than those calculated using Broms's (1964b) theory with c = 0. Shaft lengths calculated using c = 400 psf in Brinch-Hansen's (1961) theory are shorter than those calculated using Broms's (1964b) theory with c = 0. The difference is as much 40%, and the difference tends to be greater for small shaft diameters.

6.4 Effect of Sloping Ground

Method 1 for including the effect of ground slope in analyses (the method cited in the AASHTO Guide Specifications for the Structural Design of Sound Barriers, 1992, and discussed in Appendix C) is based on fundamental soil mechanics principles. Method 2 (suggested by Boghrat, 1990 and discussed in Appendix C) results in shaft lengths which are as much as 20% longer than those found using Method 1. Method 2 thus appears to be excessively conservative.

For 2:1 ground slope, shaft lengths calculated using Broms's (1964b) theory are 50% longer than those found for level ground conditions. For Brinch-Hansen's (1961) theory, shaft lengths for 2:1 ground slope are about 30% longer than for level ground conditions. (These effects were evaluated using Method 1.)

6.5 Effect of Ignoring Soil Resistance at the Top of the Shaft

For cohesionless soils, neglecting the soil resistance in the top 2 feet results in shaft lengths that are 2% to 8% longer than those found when this resistance is included. The effect is greatest for large diameter shafts.

The effect of ignoring soil resistance at the top of the shaft is greater for soils that have cohesion. For c = 400 psf, ignoring soil resistance in the upper 2 feet results in 15% to 20% longer shafts.

6.6 Effect of Soil Friction Angle, ϕ

Reducing ϕ by 2 degrees results in shaft lengths that are 3% to 4% longer as computed by Broms's (1964b) theory, and 6% to 7% longer as computed by Brinch-Hansen's (1961) theory.

6.7 Effect of Changing the Factor of Safety

The factor of safety is applied by multiplying the total horizontal load and the total moment by the factor of safety before the required shaft length is calculated. Increasing the factor of safety from 1.88 to 2.50 results in a 10% to 15% increase in the required shaft length.

Table 6.1: Summary of Variables Used in the Parametric Study

Variable	Value of Variable
Height of Sound wall	15 ft and 30 ft
Friction Angle	28, 30, 32, 34, and 36 degrees
Cohesion	0, 150, and 400 psf
Factor of Safety	1.88 and 2.50
Ground Slope	0 and 2H:1V
Depth of Soil Neglected	0 and 2 ft

CHAPTER 7 - CONCLUSION

The study described in the previous chapters was performed to investigate the factors that affect the accuracy of the procedures being used by VDOT and its contractors for design of drilled shaft sound wall foundations in Virginia. The following conclusions can be drawn from the study:

1. Published correlations between SPT blow counts and friction angles, shown in Figure 4.12, 4.13, and 4.14, appear to be reasonable for estimating friction angles for the silty and clayey soils found in many locations in Virginia. The accuracy of the correlations appears to be best if the blow counts are corrected for overburden pressure and hammer energy, that is, if ϕ is estimated using $(N_1)_{60}$ values.

2. The undrained strengths of the partly saturated silts and clays from five test sites in Virginia are characterized by cohesion values varying from 150 psf to 400 psf, and friction angles varying from 30 degrees to 40 degrees. Both cohesion and friction can be included in ultimate load analyses if Brinch-Hansen's (1961) theory is used. Although this theory is too cumbersome for easy use by hand, it can be applied readily through the EXCEL spreadsheet LCAP, which was developed during the study.

   Although the contribution of cohesion was found to be a significant factor in the failure loads measured in 20 load tests performed on eight inch diameter and nine inch diameter drilled shafts, calculations using Brinch-Hansen's (1961) theory show that the contribution of cohesion to the ultimate load capacity would be less important for shafts of larger diameter.

   The two most logical alternatives for design of drilled shaft sound wall foundations in Virginia appear to be (a) Using Brinch-Hansen's (1961) theory and including the cohesive component of strength as well as friction, or (b) Using Broms's (1964b) theory and ignoring cohesion (setting c = 0). Both of these theories are included in the spreadsheet LCAP.

   Using Brinch-Hansen's (1961) theory requires that the value of c as well as ϕ be measured or estimated. Measuring c would involve performing triaxial tests on the foundation soil, which would add to the time and cost required for drilled shaft foundation design. Estimating values of c would require a more extensive database for strengths of Virginia soils than is currently available.

Parametric studies show that if c = 150 psf, the required shaft lengths calculated using Broms's (1964b) theory without c are 20% longer to 20% shorter than those found using Brinch-Hansen's (1961) theory with c. If c = 400 psf the required shaft lengths calculated using Broms's (1964b) theory without c are as much as 40% greater than those found using Brinch-Hansen's (1961) theory with c.

3. Using Broms's (1964b) theory, in combination with factors of safety of 1.88 and 2.50, results in safe designs of drilled shaft sound wall foundations in Virginia soils, provided the value of ϕ used in the calculations is representative of the foundation soil. It seems logical to use F = 1.88 when the characteristics of the foundation soil are reliably known, and to use F = 2.50 when there is greater uncertainty about the foundation soil conditions.

4. The spreadsheet program LCAP, described in Appendix C, provides an effective computational tool for the design of drilled shaft sound wall foundations. It calculates loads on sound wall foundations from input wind speeds, exposure categories, and sound wall dimensions; performs analyses using both Broms's (1964b) theory and Brinch-Hansen's (1961) theory; and includes calculations for sloping and level ground conditions. It provides shaft lengths, concrete volumes, and bending moments for shaft diameters ranging from 18 inches to 54 inches.

   LCAP provides an engineer designing drilled shaft sound wall foundations , or checking designs, an effective tool for performing computations efficiently, for rapidly assessing the effects of possible changes in conditions, and, through these processes, for applying engineering judgment effectively.

5. The principal shortcomings of the field load test program performed during this study relate to the small size of the drilled shafts that were tested, and the fact that only a few cyclic load tests were performed. It would be desirable to extend this study by testing larger-size drilled shafts, and by subjecting them to extensive cyclic loading. Such studies would further reduce uncertainties in design of drilled shaft sound wall foundations in Virginia.

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APPENDIX A - EQUATIONS FOR K_q AND K_c FACTORS FOR THE BRINCH-HANSEN'S (1961) THEORY

A.1 Introduction

Included in this Appendix are the equations used to calculate the earth pressure coefficients for Brinch-Hansen's (1961) theory. The assumptions involved with these factors are discussed in Chapter 2.

A.2 Equations for K_q and K_c

$$K_q = \frac{K_q^o + K_q^\infty \alpha_q \frac{z}{D}}{1 + \alpha_q \frac{z}{D}} \tag{A.1}$$

where:

K_q = passive earth pressure coefficient due to weight of soil at intermediate depth,

K_q^o = passive earth pressure coefficient due to weight of soil at ground surface,

$$K_q^o = (e^{(\frac{1}{2}\pi+\phi)\tan\phi} \cos\phi \tan(45^o + \frac{1}{2}\phi)) -$$
$$(e^{-(\frac{1}{2}\pi-\phi)\tan\phi} \cos\phi \tan(45^o - \frac{1}{2}\phi))$$

K_q^∞ = passive earth pressure coefficient due to weight of soil at great depth,

$K_q^\infty = N_c d_c^\infty K_o \tan\phi$,

$$\alpha_q = \frac{K_q^o K_o \sin\phi}{(K_q^\infty - K_q^o)\sin(45^o + \frac{1}{2}\phi)}$$

z = depth below ground (units of length),

D = shaft diameter (units of length),

K_o = at-rest earth pressure,

$K_o = 1 - \sin\phi$, and

ϕ = friction angle of foundation soil.

$$K_c = \frac{K_c^o + K_c^\infty \alpha_c \frac{z}{D}}{1 + \alpha_c \frac{z}{D}} \tag{A.2}$$

where:

K_c = passive earth pressure coefficient due to cohesion at intermediate depth,

K_c^o = passive earth pressure coefficient due to cohesion at ground surface,

$K_c^o = [e^{(\frac{1}{2}\pi+\phi)\tan\phi} \cos\phi \tan(45^o + \frac{1}{2}\phi) - 1]\cot\phi$,

K_c^∞ = passive earth pressure coefficient due to cohesion at great depth,

$K_c^\infty = N_c d_c^\infty$

$\alpha_c = \frac{K_c^o}{K_c^\infty - K_c^o} 2\sin(45^o + \frac{1}{2}\phi)$,

ϕ = friction angle of foundation soil,

N_c = bearing capacity factor,

$N_c = [e^{\pi\tan\phi} \tan^2(45^o + \frac{1}{2}\phi) - 1]\cot\phi$,

d_c^∞ = depth coefficient at great depth, and

$d_c^\infty = 1.58 + 4.09\tan^4\phi$.

APPENDIX B - WIND PRESSURES AND WIND LOADS FOR DESIGN OF SOUND BARRIERS

B.1 Introduction

Wind load conditions are of critical importance in the design of sound walls. This appendix describes the procedure used in calculating wind pressures and wind loads in LCAP. These procedure are based on the "Guide Specifications for Structural Design of Sound Barriers," 1989 and 1992 Revisions.

B.2 Wind Pressures

Sound walls are designed for wind pressures resulting from 50-year recurrence interval wind speeds. These vary from 70 mph in the southwest corner of Virginia to 95 mph in the southeast corner. The wind pressure is calculated using the expression:

$$P = 0.00256(1.3V)^2 C_d C_c \tag{B.1}$$

where:

P = wind pressure (psf),

V = wind speed (mph) based on a 50 year mean recurrence interval,

$(1.3V)$ = 30% gust speed,

C_d = drag coefficient (1.2 for sound barriers), and

C_c = combined height, exposure, and location coefficient.

The exposure categories are:

Exposure B1 - urban and suburban areas with numerous closely spaced obstructions, having the size of single-family dwellings or larger, that prevail in the upwind direction from the sound barrier for distance of at least 1500 feet.

Exposure B2- Urban and suburban areas with more open terrain not meeting the requirements of Exposure B1.

Exposure C - Open terrain with scattered obstructions. This category includes flat, open country and grasslands. This exposure is used for sound barriers located on bridge structures, retaining walls, or traffic barriers.

Exposure D - Flat unobstructed coastal areas directly exposed to wind flowing over large bodies of water, extending inland from the shoreline a distance of one half mile.

Table B.1 gives values of C_c for four different exposure categories and for differing wall heights.

B.3 Wind Loads

The loads on sound barrier walls are calculated by multiplying the wind pressure by the tributary area:

$$W_t = P_1A_1 + P_2A_2 + P_3A_3 \tag{B.2}$$

where:

W_t = total wind load (lbs),

P_1, P_2, P_3 = wind pressures for height zone 1, 2, and 3, and

A_1, A_2, A_3 = areas for height zones 1, 2, and 3.

The area for each height zone is calculated by multiplying the vertical distance from the bottom of the height zone to the top of the height zone by the distance between drilled shafts:

$$A = \Delta Y \bullet L \tag{B.3}$$

where:

ΔY = vertical distance from bottom to top of height zone as shown in Figure B.1 (ft), and

L = horizontal distance (spacing) between foundations (ft). If the spacing varies, the value of L for a particular shaft is the average of the values for adjacent shafts.

Figure B.1 shows the distribution of pressure on a sound barrier, and the procedure used to convert the pressure to a total horizontal load and total moment on the drilled shaft.

Figure B.2 shows an example calculation for wind loads on a sound barrier.

Figure B.1: Moments and Loads on Sound Wall Foundations due to Wind (pdf, 100K, figb1.pdf)

Figure B.2: Example Calculation of Wind Load (pdf, 50K, figb2.pdf)

Table B.1: C_c Values for Sound Barriers

Distance from average level of adjoining ground surface to centroid of loaded area in each height zone, H, ft.	C_c for Exposure Category B1	C_c for Exposure Category B2	C_c for Exposure Category C	C_c for Exposure Category D
$0 < H \leq 14$	0.37	0.59	0.80	1.2
$14 < H \leq 29$	0.59	0.75	1.00	1.37
$H \geq 29$	0.59	0.85	1.10	1.49

APPENDIX C - DESIGN OF DRILLED SHAFT SOUND WALL FOUNDATIONS USING THE SPREADSHEET LCAP

C.1 Introduction

A spreadsheet named LCAP was developed for use in the design of drilled shaft sound wall foundations. LCAP was programmed in Microsoft Excel to perform analyses using both the Broms (1964b) theory and the Brinch-Hansen (1961) theory. A reduction factor of 15% is applied to values of P_{ult} for the Brinch-Hansen (1961) theory, based on field load tests on eccentrically loaded drilled shafts performed in this study.

C.2 Input for LCAP

Copies of printed input and output from LCAP are shown in Figures C.1 and C.2. This information appears on screen as the spreadsheet is being used, and can be printed by pressing control-s (s for sloping ground) for the section of the spreadsheet shown in Figure C.1 or control-l (l for level ground) for the section of the spreadsheet shown in Figure C.2.

The input values that define the problem are typed in the tables shown in the upper left areas of Figures C.1 and C.2. All of the other cells in the spreadsheet, except the user's name and the project name, are locked, and cannot be altered except by changing the input data values.

As shown in the upper left area in of Figure C.1, the values input to define sloping ground conditions for analysis are:

- wind speed (mph),
- drag coefficient (dimensionless) = 1.2 for sound walls, (Appendix B),
- exposure category, (Appendix B),
- wall height (ft),
- shaft spacing (ft),
- additional horizontal load (lbs),
- eccentricity of additional load (ft),
- cohesion of foundation soil (psf),
- friction angle of foundation soil (degrees),

- unit weight of foundation soil (pcf),
- slope of ground surface on downhill side of the sound wall (degrees),
- factor of safety, and
- depth over which soil resistance at top of shaft is neglected (ft).

For level ground conditions, the ground slope can be set equal to zero, or the level ground section of the sheet, shown in Figure C.2, can be used. Using the level ground section of the sheet for zero ground slope conditions has the advantage that the results section of this part of the sheet are more compact, being tailored for the level ground condition. The same results are calculated, whichever procedure is followed.

Using the input variables listed above, values of design load, moment, and eccentricity are computed as displayed in the upper right portion of Figures C.1 and C.2

C.3 Components of LCAP

The spreadsheet LCAP consists of a series of worksheets which perform specific operations. The specific function of each worksheet is given below:

- Main Module - consists of tables for input of design variables for display of computed results.
- BrinchHansen1 - performs calculations for shaft diameters of 18 inches and 24 inches for Brinch-Hansen's (1961) theory using Method 1 to account for sloping ground effects.
- BrinchHansen2 - performs calculations for shaft diameters of 30 inches and 36 inches for Brinch-Hansen's (1961) theory using Method 1 to account for sloping ground effects.
- BrinchHansen3 - performs calculations for shaft diameters of 42 inches and 48 inches for Brinch-Hansen's (1961) theory using Method 1 to account for sloping ground effects.
- BrinchHansen4 - performs calculations for a shaft diameter of 54 inches for Brinch-Hansen's (1961) theory using Method 1 to account for sloping ground effects.

Drilled Shaft Sound Wall Foundation Design (for Sloping Ground)

Input Variables	
Wind Speed (mph)	80
Drag Coefficient	1.2
Exposure Coefficient	D
Wall Height (ft.)	15
Shaft Spacing (ft.)	15
Added Load (lb.)	0
Eccentricity of Added Load (ft.)	0
Cohesion, c, (psf.)	400
Friction Angle (degrees)	36
Unit Weight of Soil	125
Slope (degrees)	0
Factor of Safety	1.88
Depth to Neglect (ft.)	0

Exposure B1 - urban areas with numerous structures

Exposure B2 - urban areas with fewer structures

Exposure C - open terrain with scattered obstructions

Exposure D - coastal areas

Derived Variables	
Wind Load (lb.)	8971
Wind Moment (lb-ft.)	67284
Added Load (lb.)	0
Added Moment (lb-ft.)	0
Total Load (lb.)	8971
Total Moment (lb-ft.)	67284
Resultant Eccentricity (ft.)	7.5

	Broms (c=0), Method 1			Broms (c=0), Method 2			Brinch-Hansen, Method 1			Brinch-Hansen, Method 2		
Diameter	Length	Vol.	Max. Moment	Length	Vol.	Max. Moment	Length	Vol.	Max. Moment	Length	Vol.	Max. Moment
in.	ft.	yd^3	lb-ft.	ft.	yd^3	lb-ft.	ft.	yd^3	lb-ft.	ft.	yd^3	lb-ft.
18	9.2	0.6	171,072	9.2	0.6	171,072	6.2	0.4	167,693	6.2	0.4	167,693
24	8.2	1.0	165,100	8.2	1.0	165,100	5.8	0.7	165,182	5.8	0.7	165,182
30	7.5	1.4	161,024	7.5	1.4	161,024	5.5	1.0	163,123	5.5	1.0	163,123
36	7.0	1.8	158,016	7.0	1.8	158,016	5.3	1.4	161,470	5.3	1.4	161,470
42	6.6	2.3	155,677	6.6	2.3	155,677	5.0	1.8	160,265	5.0	1.8	160,265
48	6.2	2.9	153,792	6.2	2.9	153,792	4.9	2.3	159,247	4.9	2.3	159,247
54	5.9	3.5	152,231	5.9	3.5	152,231	4.7	2.8	158,331	4.7	2.8	158,331

Method 1: Value of K_p modified as an allowance for ground slope Method 2: Soil at top neglected as an allowance for ground slope.
The drilled shaft lengths and maximum moment values correspond to wind loads multiplied by the factor of safety.
The ultimate moment capacity of the drilled shaft should be greater than or equal to the maximum moment.

Calculated by:	Project:	Date and Time:	6/1/97 4:38 PM

Figure C.1: Input and Output from LCAP for Sloping Ground Conditions

Drilled Shaft Sound Wall Foundation Design (for Level Ground)

Input Variables	
Wind Speed (mph)	0
Drag Coefficient	1.2
Exposure Coefficient	D
Wall Height (ft.)	15
Shaft Spacing (ft.)	15
Added Load (lb.)	20484
Eccentricity of Added Load (ft.)	15
Cohesion, c, (psf.)	400
Friction Angle (degrees)	36
Unit Weight of Soil	125
Factor of Safety	1.88
Depth to Neglect (ft.)	0

Exposure B1 - urban areas with numerous structures

Exposure B2 - urban areas with fewer structures

Exposure C - open terrain with scattered obstructions

Exposure D - coastal areas

Derived Variables	
Wind Load (lb.)	0
Wind Moment (lb-ft.)	0
Added Load (lb.)	20484
Added Moment (lb-ft.)	307260
Total Load (lb.)	20484
Total Moment (lb-ft.)	307260
Resultant Eccentricity (ft.)	15.0

	Broms (c=0)			Brinch-Hansen		
Diameter	Length	Vol.	Max. Moment	Length	Vol.	Max. Moment
in.	ft.	yd^3	lb-ft.	ft.	yd^3	lb-ft.
18	14.7	1.0	731,451	10.7	0.7	754,892
24	13.1	1.5	710,846	10.0	1.2	746,566
30	12.0	2.2	696,784	9.5	1.7	739,769
36	11.2	2.9	686,404	9.1	2.4	733,956
42	10.5	3.8	678,336	8.8	3.1	729,151
48	10.0	4.7	671,833	8.5	4.0	725,589
54	9.6	5.6	666,447	8.2	4.8	722,384

The drilled shaft lengths and maximum moment values correspond to the wind loads multiplied by the factor of safety.

The ultimate moment capacity of the drilled shaft should be greater than or equal to the maximum moment.

Calculated by:	Project:	Date and Time:	6/1/97 4:38 PM

Figure C.2: Input and Output from LCAP for Level Ground Conditions

- Broms Cohesionless - performs calculations for Broms’s (1964b) theory using Method 1 to account for sloping ground effects.

- Broms Cohesionless-Slope - performs calculations for Broms’s (1964b) theory using Method 2 to account for sloping ground effects.

- BrinchSloped - performs calculations for shaft diameters of 18 inches and 24 inches for Brinch-Hansen’s (1961) theory using Method 2 to account for sloping ground effects.

- BrinchSloped2 - performs calculations for shaft diameters of 30 inches and 36 inches for Brinch-Hansen’s (1961) theory using Method 2 to account for sloping ground effects.

- BrinchSloped3 - performs calculations for shaft diameters of 42 inches and 48 inches for Brinch-Hansen’s (1961) theory using Method 2 to account for sloping ground effects.

- BrinchSloped4 - performs calculations for a shaft diameter of 54 inches for Brinch-Hansen’s (1961) theory using Method 2 to account for sloping ground effects.

- BrinchHansen1L - performs calculations for shaft diameters of 18 inches and 24 inches for Brinch-Hansen’s (1961) theory for level ground conditions.

- BrinchHansen2L - performs calculations for shaft diameters of 30 inches and 36 inches for Brinch-Hansen’s (1961) theory for level ground conditions.

- BrinchHansen3L - performs calculations for shaft diameters of 42 inches and 48 inches for Brinch-Hansen’s (1961) theory for level ground conditions.

- BrinchHansen4L - performs calculations for a shaft diameter of 54 inches for Brinch-Hansen’s (1961) theory for level ground conditions.

- BromsL - performs calculations for Broms’s (1964b) theory for level ground conditions.

- A series of 8 macro sheets control the iterative process involved in calculating the required shaft lengths for the input design parameters.

C.4 Calculations Performed by LCAP

Required shaft lengths are calculated for shaft diameters ranging from 18 inches to 54 inches in 6 inch increments. This process is performed for both the Broms (1964b) theory and the Brinch-Hansen (1961) theory (with a 15% reduction factor).

The calculations for the Broms (1964b) theory are performed using an iterative process to solve the following equation for the required value of L:

$$\frac{L^3}{e + L} = \frac{2WF}{\gamma D K_p} \tag{C.1}$$

where:

L = shaft length (ft.),

e = eccentricity of resultant load (ft.),

W = wind load (lbs),

F = factor of safety,

γ = unit weight of foundation soil (pcf),

D = shaft diameter (ft.)

K_p = coefficient of passive earth pressure = $\tan^2(45 + \phi/2)$ (dimensionless),

ϕ = friction angle of foundation soil (degrees), and

F = factor safety (dimensionless).

The computations for Broms's (1964b) theory are easily performed, but the computations involved with the Brinch-Hansen (1961) theory are more complicated. The required shaft length for each shaft diameter is determined by performing the following operations:

1. A shaft length is assumed.

2. The length of the shaft is divided into 20 segments.

3. The lateral pressure on each segment is calculated by determining the values of K_c and K_q at the middle of each segment.

4. The lateral pressure is multiplied by the height of the segment to determine a resulting horizontal force at the middle of each segment.

5. A point of rotation is assumed and the forces above the point of rotation are considered to act on the front of the shaft, while the forces below the point of rotation are considered to act on the back of the shaft. The spreadsheet is set up so the point of rotation is first assumed to be at mid-depth and is shifted downward during subsequent trials.

6. The forces on the front of the shaft (+) and the back of the shaft (-) are summed to determine the net soil resistance corresponding to the assumed point of rotation.

7. The net soil resistance is considered to act at a distance above the ground surface equal to the design eccentricity, and a resulting moment at the ground surface is calculated.

8. The moment at the ground surface due to the forces of the 20 individual segments is computed.

9. The two moments (moment due to lateral resistance applied above the ground surface, and the moment due to the 20 segments) are compared.

10. If the moment due to the lateral resistance applied above the ground surface is smaller, the assumed point of rotation is moved down. If it is larger, the assumed point of rotation is moved up. This process is continued until the point of rotation has been located.

11. The ultimate lateral capacity is calculated by dividing the ground level moment by the design eccentricity.

12. The calculated lateral capacity is compared to the design load. If the calculated capacity is less than the design load, the shaft length is increased. If the calculated capacity is greater than the design load, the shaft length is decreased. This process is continued until the shaft length has been determined with an accuracy of 0.1 feet.

Once the shaft lengths have been computed for the given load conditions, the maximum bending moment is calculated. This is accomplished by calculating the bending moment at a number of depths below the ground surface. As depth below ground increases, the bending moment first increases and then decreases. The maximum bending moment is taken as the largest value calculated

C.5 Sloping Ground Conditions

LCAP performs calculations for two methods of accounting for sloping ground conditions.

Method 1 is outlined in the AASHTO - Guide Specifications for Structural Design of Sound Barriers (1989, revised in 1992). This method involves modifying the passive earth pressure coefficient on the front of the shaft to account for the sloping ground. The Coulomb method for determining passive earth pressure coefficients is an appropriate method to use in determining a coefficient accounting for the sloped conditions.

Method 2 is described by Boghrat (1990). This method accounts for the effect of ground slope by neglecting a portion of soil on the front side of the shaft. The portion of soil neglected is the hatched area in Figure C.3. The equation used to evaluate the distance A from Figure C.3 is as follows:

$$A = \frac{(D \tan\theta \tan\beta)}{(\tan\theta \tan\beta + 1)} \tag{C.2}$$

where:

A = depth of neglected soil (ft),

D = depth to bottom of shaft as shown in Figure C.3 (ft),

θ = slope angle (degrees), and

$\beta = 45+\phi/2$.

Figure C.3: Method 2 for Accounting for Sloping Ground Surface (pdf, 50K, figc3.pdf)

The soil in the hatched region in Figure C.3 is considered to provide no resistance to lateral movement. Using Method 2 the total embedment depth of the shaft is defined by:

$$L = A + D \tag{C.3}$$

where:

L = total embedment depth.

C.6 Output from LCAP

Once the desired values have been input into LCAP, new results are calculated by pressing control-b. The computations usually take 45 seconds to one minute, depending on the speed of calculation of the computer used

Once the calculations have been completed, the results appear in the lower portion of the spreadsheets, as shown in Figures C.1 and C.2. Shaft lengths are shown for shaft diameters from 18 inches to 54 inches, in 6 inch increments. The output also includes the volume of concrete and the maximum bending moment for each shaft size.

Results for sloping ground conditions can be printed by pressing control-s (s for sloping ground), and results for level ground conditions can printed by pressing control-l (l for level ground). Before results are printed, they are recalculated to insure that the results correspond to the input values. Figures C.1 and C.2 are example printouts from the sloping ground and the level ground sections of the sheet.

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