Ultimate side friction of drilled shafts in gravels

Le frottement latéral ultime des pieux forés en graviers

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ABSTRACT: To evaluate side friction, 28 axial tension (uplift) load tests were performed on drilled shafts in soil profiles ranging from uniform medium sand through well graded sandy gravel. Measured load capacities were compared with capacities computed using equations proposed by Reese and O'Neill (FHWA) and Meyerhof. Reasonable agreement between measured and computed capacities was generally found for sandy profiles. However, measured capacities were typically 2 to 4 times higher than predicted at sites where the gravel fraction was over 50%. Additional load test data for gravelly soils were collected and combined with the data for Utah load tests. Based on this data set, modifications to the two design equations were then developed to better predict ultimate side friction capacity while still maintaining a comfortable margin of safety.

RESUME: A fin d'évaluer le frottement latéral, 28 essais de chargement à traction (d'arrachement) ont été conduits sur des pieux forés dans des profils de sol qui varies entre de sable moyen à granulométrie uniforme et gravier sableux à granulométrie bien repartie. Les capacités de chargement mesurées ont été comparées aux capacités calculées des équations proposées par Reese et O'Neill (FHWA) et Meyerhof. Une concordance raisonable entre les capacités mesurées et celles calculées a été observée généralement pour les profils de sol sableux. Pourtant, les capacités mesurées ont été typiquement 2 à 4 fois plus grandes q'on l'avait prévu aux sites où la fraction de gravier a été plus de 50 %. Les données d'essais de chargement additionnelles pour des sols gravieux ont été rassemblées et combinées avec les données d'essais de chargement conduites en Utah. En partent de cette base de données, des modifications aux deux équations de dessein ont été ensuite développées afin de prédire mieux la capacité du frottement latéral ultime alors qu'un facteur de sécurité agréable est maintenu.

1. INTRODUCTION

A series of load tests conducted by the Utah Department of Transportation (UDOT) in the mid 1980's suggested that side friction for drilled shafts in gravelly soils was significantly higher than in sandy soils. In addition, the side friction was significantly underestimated by design equations commonly used in the U.S. (Price et al, 1992). The increased side friction was initially attributed to the roughness of the soil-shaft interface and the tendency for gravels to dilate during shearing. Since most design equations are based on test results in sands and clays where the soil-shaft interface is much smoother, they would not be expected to account for the increased capacity.

To more accurately evaluate side friction in gravelly soils and the equations used to predict capacity, a series of uplift load tests were performed on drilled shafts in soil profiles ranging from uniform sand to sandy gravel. In addition, an effort was made to assemble available load test data on drilled shaft capacity in gravelly soils from throughout the world. This data set was then used to suggest modifications to design equations to better fit the measured capacities.

2. DESIGN EQUATIONS FOR SIDE FRICTION IN COHESIONLESS SOILS

The ultimate side friction capacity, Q_s is given by the equation

$$Q_s = f_s \cdot A_s \tag{1}$$

where: f_s = ultimate unit side friction on shaft
A_s = area of shaft producing side friction

The ultimate side resistance of the drilled shafts in this study was estimated using methods proposed by Meyerhof (1976) and by Reese and O'Neill (1988) for the U.S. Federal Highway Administration (FHWA). The Meyerhof method (1976) is an empirical procedure based on the results of field load tests. The ultimate unit side friction, f_s, of a drilled shaft in sand is given by

$$f_s\,(\text{kPa}) = N, \quad \leq \; 50 \text{ kPa} \tag{2}$$

where N is the average standard penetration resistance, in blows per 300 mm. The side friction specified for drilled shafts in Equation 2 is only half of that recommended by Meyerhof for displacement piles.

The FHWA method (Reese and O'Neill, 1988) is a semi-empirical method based on a data base of 41 drilled shaft load tests. The ultimate unit side resistance in sand is given by

$$f_s = \beta\, \sigma_z' \le 200\ \text{kPa} \qquad (3)$$

where σ_z' = vertical effective stress in soil at depth z
β = $1.5 - 0.244\, z^{0.5}$, $1.2 \ge \beta \ge 0.25$
z = depth below ground surface in m.

The parameter, β, depends only on depth and is independent of soil density although σ_z' does account for density.

3. UTAH LOAD TESTS

3.1 Site conditions and drilled shaft geometry

A total of 28 axial tension (uplift) tests were performed at 8 sites in Northern Utah. All the sites are located in Holocene age alluvial soil deposits. One site consisted of medium-grained uniform clean sand, 3 sites consisted of gravelly sands, and 4 sites consisted of sandy gravels or silty gravels. Two exploratory borings were made within 5 m of the tests shafts at each site and SPT N values were obtained at 1 m intervals.

Test shafts were generally installed to depths of 1.5, 3, 4.5, and 6 m so that side friction could be evaluated as a function of depth without the need for expensive instrumentation. However, in three cases it was not possible to drill the holes to the full depth due to either large boulders, groundwater, or excessive caving in the uncased auger hole. The shaft diameters were typically 0.5 to 0.7 m, which was judged to be large enough to eliminate significant scale effects but small enough so that a massive load frame was not required. Additional information on soil properties and construction details are provided by Rollins et al (1994).

3.2 Load testing and interpretation

The load testing was performed in general accordance with the quick load method as specified by ASTM D3689-83. The uplift load was applied in increments of about 10% of the expected shaft capacity and maintained for three minutes. Uplift loads as large as 1780 kN could be applied to the drilled shafts through the load frame apparatus.

Load versus displacement curves obtained from the load testing were interpreted using three procedures for estimating the ultimate load capacity in uplift. These methods were the double tangent, the slope tangent and the 12.7 mm displacement criterion. The 12.7 mm displacement criterion typically gave the highest ultimate capacity and was 30% higher on average than the double tangent method which typically gave the lowest ultimate capacity. If the ultimate capacity defined by 12.7 mm displacement is divided by a factor of safety of 2, the average displacement is typically about 2.5 mm.

3.3 Comparison of Measured and Computed Capacities

On average, the FHWA method yielded predicted ultimate capacities which were 23% higher than the Meyerhof equation. The Meyerhof equation often predicted the largest ultimate capacity in soil profiles consisting of gravels with high SPT blow counts. The FHWA procedure, which is not highly sensitive to blow count, generally yielded the highest predicted capacities in the sand profiles.

Comparisons between the measured capacity using the 12.7 mm criterion and the predicted capacity using the FHWA and Meyerhof equations are shown in Fig.1 for all load tests. For brevity, comparisons with other capacity interpretation procedures are not presented in this summary, however, the general results are similar. The measured capacity tends to increase with the percentage of gravel. Reasonable agreement between measured and computed capacities was generally found for the FHWA method in sandy profiles. However, measured capacities were 100 to 300% higher than predicted at sites where the gravel fraction was over 50 percent.

The Meyerhof method was somewhat less conservative for the gravels at shallow depths because the SPT blow counts in these materials were relatively high. Nevertheless, measured capacities were also 100 to 300% higher than predicted by the Meyerhof equation. The higher capacities were once again associated with the gravelly soil profiles. The side resistance predicted by the

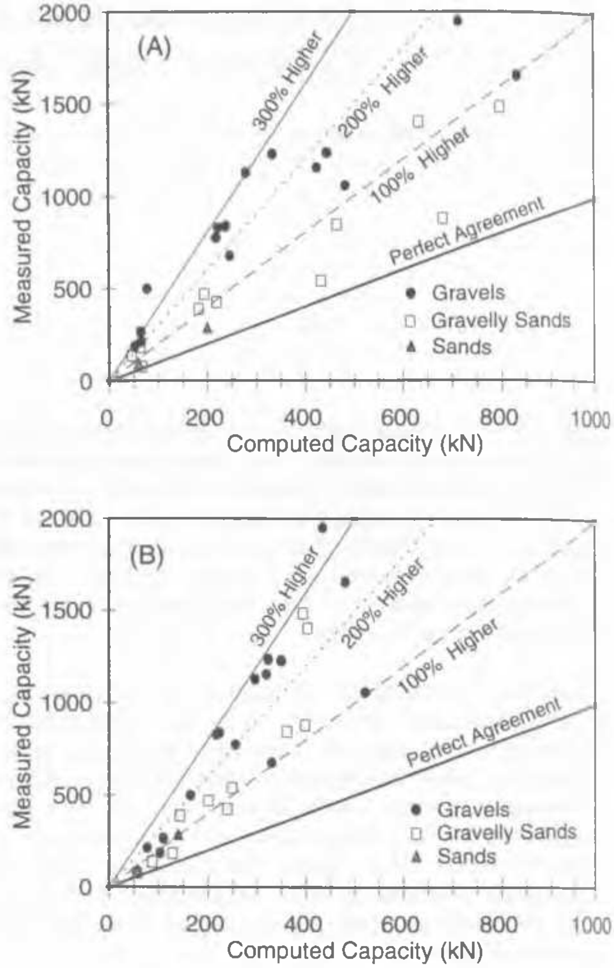

Figure 1. Measured ultimate capacity interpreted using the 12.7 mm criteria versus predicted capacity using (A) the FHWA equation and (B) the Meyerhof equation.

Meyerhof equation could generally be doubled for both gravelly sands and gravels without any problem. This would make the side friction equation in these soils the same as that originally proposed by Meyerhof (1976) for driven piles.

4. ADDITIONAL LOAD TEST DATA IN GRAVELLY SOILS

Drilled shaft load test results from other sites were added to the data base to better define the behavior of drilled shafts in gravelly soils. In addition, these data points were used to confirm that the higher load capacities measured in the Utah load tests were not unique to the geotechnical characteristics of Utah soils. Additional data were collected for sites in the states of California, Nevada, New Mexico, and Hawaii. In addition, data were collected from the countries of Czechoslovakia, Germany, Greece and Japan. The search for data yielded a total of 45 additional data points in gravelly soils which could be used in this study.

For analysis purposes, the data has been divided into three categories: gravels, gravelly sands, and sands based on the Unified Soil Classification System. The gravels classify as GW, GP, GM or GC soils and contain between40 and 80% gravel size particles. The gravelly sands classify as SW, SP, SM or SC soils and contain between 15 and 40% gravel size particles. The sands also classify as SW, SP, SM or SC soils, but they contain less than 15% gravel size particles. Data for sands were only obtained at sites where gravelly material was also present and was primarily used for comparison purposes.

5. RECOMMENDED MODIFICATIONS TO DESIGN EQUATIONS BASED ON COMPLETE DATA SET

The results of this study indicate that modifications to the two design methods presented previously are necessary to reflect the behavior of drilled shafts in gravelly soils. The following suggested modifications are based on the complete data set which includes load tests conducted during this study and load tests obtained from the literature. Each modification is directly tied to the 12.7 mm failure criteria used to determine the ultimate shaft capacity.

5.1 Modifications to FHWA Design Equation

As indicated previously, the FHWA equations multiplies the vertical effective stress by a β-factor to determine the unit side friction on the shaft (see Eq. 3). Back-calculated β-values from all the available load tests are plotted as a function of depth in Fig. 2. The FHWA design curve for sand is also shown in Fig. 2 for comparison. A review of the data indicates that the β-values generally tend to increase as the percentage of gravel increases. Although the FHWA β curve appears appropriate for sands, this curve significantly underestimates the measured β values for gravelly sands and gravels especially considering that a factor of safety of 2 to 3 is subsequently applied.

Also shown in Fig. 2 are recommended design curves defining β-values for gravelly sands (25 to 50% gravel size) and gravels (>50% gravel size), respectively. These

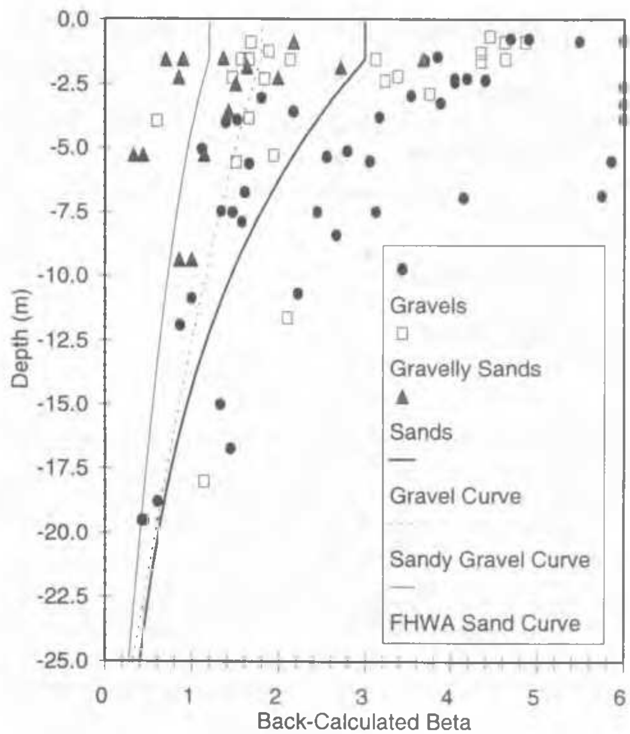

Figure 2 Back-calculated β values in comparison with design curves for drilled shafts in sands, gravelly sands and gravels.

curves fall slightly below the mean minus one standard deviation bounds for their respective data sets. If a factor of safety greater than 2.0 is used, the computed capacity is always below the lowest measured capacity, however, a factor of safety of 2.5 is recommended. The three design curves merge at a depth of 25 m.

The design curve recommended for use with gravelly sands was originally proposed by O'Neill (1994) for "gravelly soils" based on a subset of the data from this study. The equation for the gravelly sand (25 to 50% gravel size) β curve is

$$\beta = 2.0 - 0.15 \cdot z^{0.75}, \qquad 0.25 \leq \beta \leq 1.8 \qquad (4)$$

where z is the depth below the ground surface in meters.

The equation for the proposed design curve for gravels (>50% gravel size) is

$$\beta = 3.4 \cdot e^{(-0.085 \cdot z)}, \qquad 0.25 \leq \beta \leq 3.0 \qquad (5)$$

where e is the natural base (2.718) and z is the depth below the ground surface in meters. It should be noted that almost all the gravels in the data base had N values greater than 25. Therefore, use of equation 5 for low blow count gravels may not be appropriate.

5.2 Modifications to Meyerhof Equation for Gravelly Soils

A plot of the unit side friction versus the uncorrected SPT blow count, N, for all of the load test data is shown in Fig.3. The original correlation equation between unit side friction, f_s and N proposed by Meyerhof (1976) (See equation 2) is also shown with dashed lines in Fig. 3. There is a tendency for the unit side friction to increase as

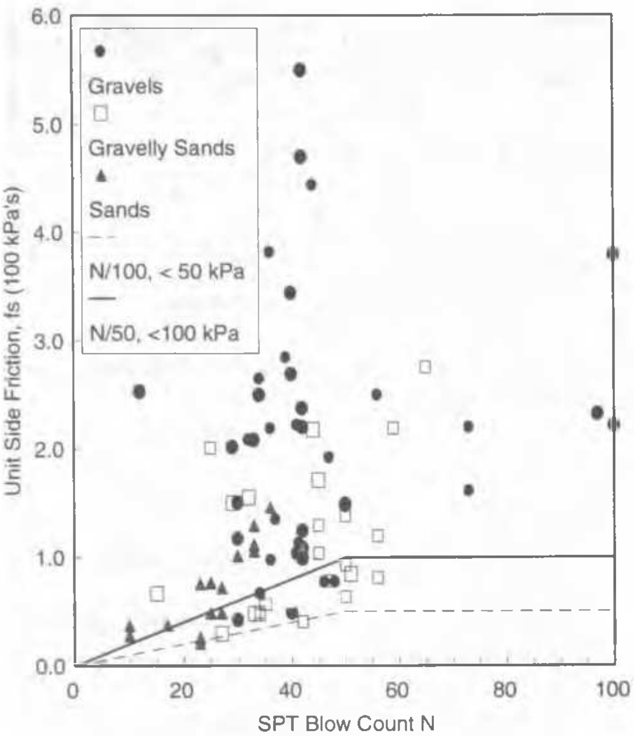

Figure 3 Measured unit side friction versus SPT blow count in comparison with design equations for sands, gravelly sands and gravels.

the blow count increases. In addition, the data show that the unit side friction generally tends to increase as the percentage of gravel increases, but so does the scatter. For example, for a given N value, the spread in unit side friction is about 50 kPa for the sands, 150 kPa for the gravelly sands, and 400 kPa for the gravels.

Essentially all the data points are equal to or greater than the side friction computed with equation (2). In fact, more than 80% of the gravel points and 50% of the gravelly sands are twice as great as predicted by equation (2). For design purposes, it appears reasonable to estimate the unit side friction for gravels, at least, using the equation

$$f_s\,(\text{kPa}) = 2 \cdot N, \quad \leq 100 \text{ kPa} \tag{6}$$

This equation doubles the predicted capacity in comparison with the Meyerhof (1976) equation, yet it is still relatively conservative for the gravels. If the ultimate capacity is divided by a safety factor of 2.5, the measured capacity would be safely above the allowable capacity for all the gravels data points as shown in Fig.3.

6. CONCLUSIONS

1. Ultimate side resistance appears to increase as the gravel content increases.
2. Measured side resistance in gravels is 100 to 300% higher than predicted by the FHWA or the Meyerhof equation for depths less than about 20 m.
3. The FHWA equation can be improved by using separate β curves for gravelly sands and for gravels. These curves produce a significant increase in capacity at shallow depths while still providing a margin of safety.
4. For design purposes, the side resistance predicted by the Meyerhof equation can be doubled for gravels gravels with a reasonable safety margin.

ACKNOWLEDGMENTS

This study was funded by the Utah Dept. of Trans. under research contract 93-8476 with supplemental support from ADSC, the International Association of Foundation Drilling. This support is gratefully acknowledged.

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