Drained uplift capacity of drilled shafts

Capacité sous-pression drainée du puits forés

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SYNOPSIS A general analysis/design model is presented for the drained uplift capacity of drilled shaft foundations. This model has evolved from extensive research to define the failure mechanisms and establish the controlling parameters. Detailed guidelines are given to evaluate these parameters, and the results of field load tests are used to illustrate the model reliability.

INTRODUCTION

Drilled shafts (also known as drilled piers, bored piles, etc.) have become the foundation of choice in many design situations, stimulating research to develop improved design methods. This paper presents a general analysis/design model for the drained uplift capacity of drilled shafts, which is based upon extensive research by our group during the past ten years and has included analytical studies, laboratory testing, large-scale model testing, and evaluation of full-scale field load tests. For brevity, the term shaft is used herein.

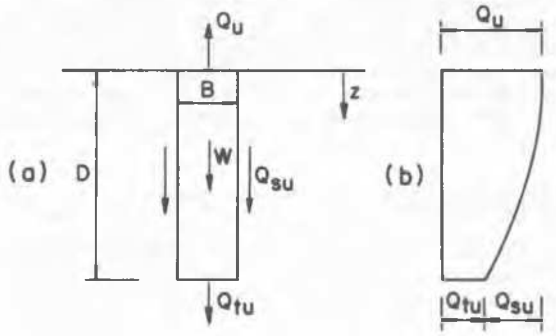

Fig.1 Shaft in Uplift

BASIC CONSIDERATIONS

In principle, the uplift capacity of shafts is given as in Fig. 1a by the vertical equilibrium equation below:

$$Q_u = W + Q_{tu} + Q_{su} = W + Q_{tu} + \int_{surface} \tau(z)dz \quad (1)$$

in which Q_u = uplift capacity, W = foundation weight, Q_{tu} = tip resistance, Q_{su} = side resistance, and τ = shearing resistance along a general shear surface. The use of this equation is limited only by our ability to predict the shear surface and the shearing resistance along it, and the tip resistance.

Studies to define the shear surface have been summarized recently by Kulhawy, et al. (1983), who showed that shafts fail principally along the soil-shaft interface leading to an overall cylindrical shear, as implied by Fig. 1a. The corresponding load transfer which normally occurs is shown in Fig. 1b (Stewart and Kulhawy, 1981b). Typically, the displacement to mobilize the full side resistance is about 5 to 10 mm.

The mechanism by which this type of failure occurs has been examined by Stewart and Kulhawy (1980, 1981a), as shown in Fig. 2. During initial uplift loading, Riedel shears develop in the soil along planes on which Mohr-Coulomb failure conditions are satisfied (Fig. 2a). Large displacements along the Riedel shears are not kinematically possible, so the soil is forced to develop displacement shears with further foundation movement (Fig. 2b), which finally result in a continuous displacement shear (Fig. 2c). This continuous shear is very close to the soil-shaft interface, and effectively defines a cylindrical shear surface.

However, in some cases the Riedel shears constitute a kinematically permissible failure mode before a continuous displacement shear develops. For this condition, a composite failure surface occurs, represented by a cone of soil near the ground surface with a cylindrical shear below. Fig. 3 illustrates this composite surface and gives suggested guidelines for determining the depth of the failure cone. In this figure, developed for both drained and undrained loading, the mean values over the depth, D, are used for the effective

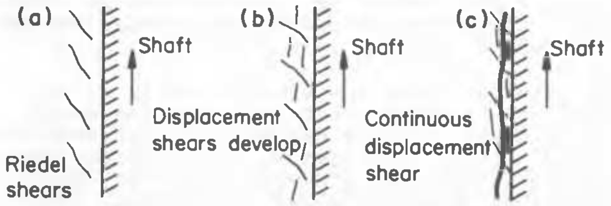

Fig.2 Development of Shear Surface

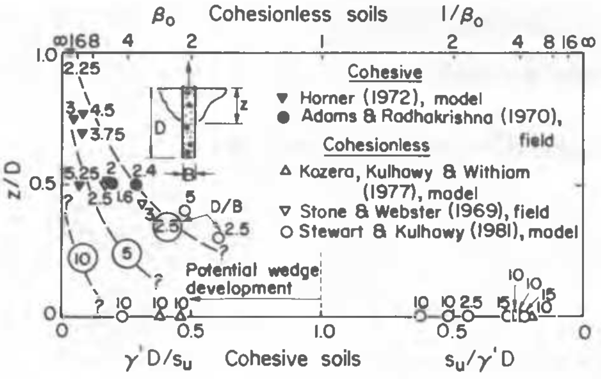

Fig.3 Composite Failure for Shafts in Uplift (Stewart and Kulhawy, 1981a)

unit weight, γ', the undrained strength, s_u, and β_o, defined as $K_o \tan\delta'$ in which K_o = in-situ horizontal soil stress coefficient and δ' = soil-shaft interface friction angle. Examination of this figure shows that cone development only occurs for small shaft D/B ratios when the shafts are installed in high strength soils with high in-situ stresses (large β_o).

SIDE RESISTANCE

For the dominant failure mode, the integral in Eq. 1 can be expressed as the shearing resistance on the surface of a vertical cylinder, or:

$$Q_{su} = \int_o^D P(z)\ \sigma_h'(z)\ \tan\ \delta'(z)\ dz \qquad (2a)$$

$$= \int_o^D P(z)\ \sigma_v'(z)\ K(z)\ \tan\ \delta'(z)\ dz \qquad (2b)$$

in which P = foundation perimeter, σ_h' = horizontal effective stress which acts as a normal stress on the soil-shaft interface, δ' = effective stress angle of friction for the soil-shaft interface, σ_v' = vertical effective stress, and K = operative coefficient of horizontal soil stress (σ_h'/σ_v'). These terms are often grouped to define $\beta = K \tan\delta'$ and the unit side resistance, $f = \beta\ \sigma_v'$. This grouping obscures behavioral factors and is not used herein.

The perimeter and vertical effective stress terms can be evaluated simply. The perimeter is computed from the foundation geometry and will be a constant for a straight shaft. The vertical effective stresses are computed from the soil unit weight and the water table location. For design purposes, the highest seasonal location normally is appropriate.

The interface friction angle commonly is related to the effective stress friction angle of the soil, ϕ', with the results expressed as a δ'/ϕ' ratio. Detailed study of soil-concrete interfaces (Kulhawy and Peterson, 1979) has shown that when the interface is rough, $\delta'/\phi' > 1$. Normal cast-in-place concrete with a slump greater than about 100 mm will yield a rough interface in granular soils. A ratio greater than 1 is of little design significance because it means that the failure surface will move out from the interface into the adjacent soil, giving a design ratio of 1.

The most difficult term to evaluate is K, because it is a function of the original in-situ K_o, and the stress changes caused in response to construction, loading, and time. Considering these factors, there may be either an increase or decrease from the original K_o. Values of K reported in the literature for a wide range of foundation types indicate low values approaching 0.1 and high values approaching 5. These bounds correspond roughly to the range from minimum active to maximum passive stress states.

The first step is to evaluate K_o. Most natural soil deposits are overconsolidated to some degree, at least near the surface. Therefore, assuming an at-rest $K_o = 1-\sin\phi'$ will almost always be overly conservative for design. What is necessary is to establish the K_o profile with depth which reflects the in-situ conditions more correctly. Measurements made by a pressuremeter or other evolving field devices can provide direct in-situ estimates (e.g., Baguelin, et al., 1978). A second approach is to construct the K_o profile based on the stress history of the soil (Mayne and Kulhawy, 1982). And thirdly, the K_o profile can be estimated from empirical correlations with field and laboratory test indices (Stas and Kulhawy, 1984).

Once the in-situ K_o profile has been estimated, the operative K can be determined. Studies by Kulhawy, et al. (1983) have shown that the K/K_o ratio for shafts varies between 2/3 and 1 when normal concretes are used. The lower range corresponds to slurry construction which leaves a thick cake on the shaft wall or causes softening in cohesive soils, while the upper range corresponds to dry construction and minimal sidewall disturbance. Casing construction below the water table is an intermediate case. Obviously, caving shaft walls or running sands will allow the ground stresses to relax much more and reach a minimum active state in the limit. These problem conditions require special study.

A further factor to consider is the use of expansive cement for the shaft concrete, which has the potential to increase K/K_o above 1. A preliminary study by Sheikh, et al. (1983) on shafts in stiff clay indicates as much as a 50% increase in capacity with expansive cements. Although comparable detailed data are not available for shafts in granular soils, there is every reason to suggest that K/K_o will be greater than 1 when expansive cements are used. This issue will have to be clarified in the future.

Incorporating all of these side resistance factors yields the following general equation:

$$Q_{su} = \frac{K}{K_o} \int_o^D P(z)\sigma_v'(z)K_o(z)\tan[\phi'(z)\cdot\frac{\delta'}{\phi'}]dz \quad (3)$$

with δ'/ϕ' = 1 and K/K_o = 2/3 to 1.

MODIFICATIONS FOR CONE BREAKOUT AND BELLS

When it is determined from Fig. 3 that a cone breakout may develop, leading to a composite shear surface, the side resistance will be reduced because the uplifted cone of soil is no longer exerting a shearing resistance to the shaft wall. Based on examination of the available data, Stas and Kulhawy (1984) suggested that a reduced β be used, defined as $\beta_r = (2 + \beta_o)/3$. The reduced side resistance then is:

$$Q_{su}(\text{reduced}) = Q_{su}(\text{from Eq. 3})\ \beta_r/\beta_o \qquad (4)$$

with β_o as defined previously. However, if D/B is larger than about 5, or if the determined z/D is less than about 0.25, the reduction should be disregarded because it is minor.

When a belled shaft is loaded in uplift, the shear surface changes from a cylindrical shape to a more complex one. No rigorous theory is available to evaluate this problem at the present time. However, observations made by the writer after reviewing the results of many load tests lead to the following tentative conclusions: (1) for deeper shafts with depth to shaft diameter ratios (D/B) greater than about 10, there is little apparent influence of the bell on side resistance, (2) for shorter shafts with D/B ratios less than about 5, a design assumption of an operative mean diameter appears to give computed side resistances in general agreement with the load tests, and (3) intermediate lengths can be treated by a linear interpolation between D/B of 5 to 10. This operative mean diameter is defined as B_{shaft} plus $(B_{bell} - B_{shaft})/3$ and is used herein.

TIP RESISTANCE AND WEIGHT

The tip resistance of shafts in uplift commonly is assumed to be zero, but this assumption may be overly conservative in some cases. Stewart and Kulhawy (1980, 1981b) discussed this problem and showed that tip resistance can be developed from both tension and suction. Suction is an undrained phenomenon and is not present during drained loading. However, tip tension can develop when the shaft concrete bonds with the soil at the tip. During uplift, the tensile strength of the soil would be mobilized over the area of the tip. However, common construction practices usually result in a thin "altered" zone of very low tensile strength at the tip. Also, the tensile strength of soil is low, commonly on the order of several percent of the compressive strength. These two points lead to the prudent conclusion of assuming zero tip tension. Conversely, where very careful cleanout is accomplished and the soil at the tip has significant tensile strength (e.g., basal till), tip tensions can develop which will add to the uplift capacity. The same is true for rock at the tip, in which case the tip tension will be controlled by the lower tensile strength of either the rock or the concrete.

The final term to evaluate in Eq. 1 is the shaft weight. This is computed simply from the shaft geometry, being certain to include the water table location to compute the effective shaft weight. For belled shafts, the operative mean diameter would be used.

FIELD LOAD TEST COMPARISONS

To evaluate the analysis/design approach outlined above, a comparison was made between the predicted uplift capacity, Q_{up}, and the measured uplift capacity, Q_{um}, as determined from 17 load tests available in the literature which were conducted on shafts installed in entirely granular soil deposits. The basic parameters are listed in Table I, with further details given by Stas and Kulhawy (1984). In all cases, δ'/ϕ' was taken as 1, and K/K_o was 1 except for the two 40 foot deep shafts with $K/K_o = 5/6$ because of casing construction under water. Customary U.S. units were used because these data were reported in that format.

Fig. 4 shows the results of the comparison. The agreement is very good with the solid line which represents a 1 to 1 or perfect prediction. A linear regression of these data resulted in the following:

$$Q_{up}(\text{tons}) = 1.7 + 0.91\ Q_{um}(\text{tons}) \qquad (5)$$

with a correlation coefficient of 0.961. The data were analyzed further by normalizing Q_{up} by Q_{um}, resulting in:

$$Q_{up}/Q_{um} = 1.04 - 0.0016\ Q_{um}(\text{tons}) \qquad (6)$$

This normalized fit gave a mean of 0.98 and a coefficient of variation of 28.8%. Overall, these results show a very good comparison with a small (and conservative) tendency for underestimation.

When it is considered further that as-built diameters are usually larger than the as-designed diameters, by as little as a few percent to as much as 15% in sands (Stewart and Kulhawy, 1981a), the correlations would be even better. Unfortunately, the load test data were not detailed enough to include the as-built diameters.

A final observation to be made relates to the mean value of K, back-calculated from the load tests and shown in the last column of Table I. The mean value of K is large and substantially higher than the 0.4 to 0.6 expected for normally consolidated (NC) soil. Only deep shafts approximate the NC case, while the data show very clearly that high horizontal stresses exist at shallow depths. These high stresses must be determined for economical design.

SUMMARY AND CONCLUSIONS

A general analysis/design model has been presented to compute the drained uplift capacity of straight-sided drilled shaft foundations,

TABLE I. Load Test Parameters

Shaft Depth (ft)	Shaft/Bell Diameter (ft)	Ground Water Depth (ft)	Total Unit Weight (pcf)	Effective Friction Angle (deg)	W (tons)	Q_{su} (tons)	Q_{up} (tons)	Q_{um} (tons)	Mean K from Q_{um}
8.0	3.00	b[a]	110[c]	31[d]	4.2	21.5[g]	25.7	24.0	1.96
10.0	3.00	b[a]	110[c]	32[d]	5.3	33.2	38.5	49.0	2.62
8.0	2.00	b[a]	120[c]	40[d]	1.9	34.2	36.1	45.0	4.25
10.0	3.00	7.5	110[c]	31[d]	4.8	34.9	39.7	40.4	2.33
9.0	2.00/3.00	b[a]	120[c]	36[d]	2.9	41.0	43.9	27.7	1.92
11.0	2.00/3.00	b[a]	120[c]	36[d]	3.5	53.1	56.6	58.0	2.82
4.0	2.00/3.00	b[a]	120[c]	36[d]	1.3	5.6[g]	6.9	14.3	5.09
7.0	2.50	2.5	111	31[e]	1.9	3.3	5.2	8.4	1.31
10.0	3.00	0.0	122	31[e]	3.1	26.3	29.4	43.1	4.71
4.5	1.10	1.0	120[c]	36[d]	0.2	3.0	3.2	2.5	2.26
8.0	1.20	3.0	120[c]	36[d]	0.5	9.6	10.1	7.0	1.56
12.0	1.20	1.0	120[c]	36[d]	0.6	13.5	14.1	14.3	2.08
21.0	3.50	2.0	120[c]	32[d]	9.4	80.2	89.6	93.7	1.62
21.0	3.50	2.0	120[c]	32[d]	9.4	80.2	89.6	100.0	1.74
6.5	2.00/3.50	b	110	30[d]	2.4	15.8	18.2	22.5	3.79
40.0	1.75	4.0	115	33[f]	4.5	45.5	50.0	38.4	0.45
40.0	1.25	4.0	115	33[f]	2.3	32.5	34.8	33.8	0.59

a - inferred from boring logs b - water level below shaft tip c - assumed
d - estimated from field SPT e - from direct shear test f - test type unknown
g - reduced for cone breakout

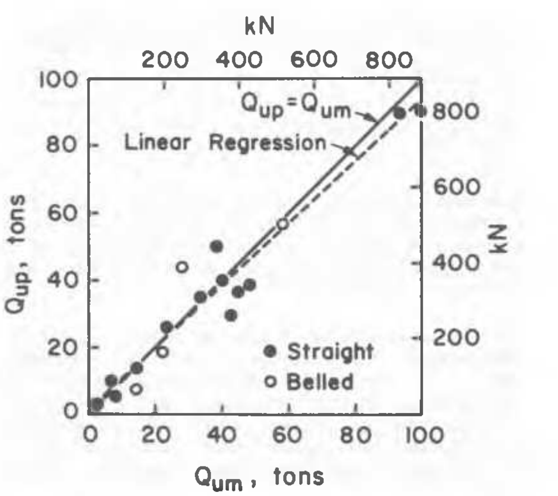

Fig.4 Capacity Comparison in Drained Uplift

and detailed guidelines have been given to evaluate the model parameters. Recommendations are also given to analyze cone breakout conditions if they are present, and to extend the model to belled shafts. Comparisons with available full-scale field test data are very good and support the model well.

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