Field measurements of laterally loaded drilled shafts

Mesures sur place de caissons charges latéralement

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SYNOPSIS: Horizontal wind forces and the resulting overturning moments represent a large portion of the loadings on the foundations of traffic signal control structures. Traditional design procedures (AASHTO, 1985) based on pseudo-static limiting equilibrium analysis often produce drilled shafts whose diameters and lengths increase very rapidly with increased lateral loads. Deflection analysis can now be incorporated into methods for improved design of laterally loaded drilled shafts. However, uncertainties remain relative to the characterization of the soil response to the applied loadings, including the effects of soil-structure interaction. This paper describes field performance measurements of full scale laterally loaded drilled shafts which support traffic signals. The measurements were made with the objective of developing improved design methods.

1. INTRODUCTION

Figure 1 illustrates the applied loadings on a span wire mounted traffic signal support structure. DL is the applied dead load and W_h, W_p, and W_v respectively are the wind loads on exposed horizontal supports, sign panels or traffic signals, and exposed vertical supports. The transverse components of these forces are not shown but must also be considered in design. The wind loads are usually calculated from design wind speeds using fluid flow theory to calculate the impact pressure of the air on the object (AASHTO, 1985). For the purposes of structural and foundation design, they are assumed to act statically and are the primary source of foundation overturning moment.

Drilled shafts at two sites in Taunton, MA USA were instrumented for this study. At location P4, a shaft 0.91 m in diameter and 9.07 m long was drilled through a 2.44 m layer of loose to medium fine sand underlain by a medium stiff inorganic silt and clay. The water table was encountered at a depth of 2.44 m. The top of the drilled shaft is level with the ground surface and is surrounded by a 0.15 m unreinforced concrete sidewalk. The sidewalk was cracked possibly due to deflections of the drilled shaft foundation. The Massachusetts Highway Department (MHD) designed the foundation for a maximum overturning moment of 235 kN-m, based on a 145 km/h wind loading and a limiting equilibrium procedure described by AASHTO(1985).

At location P17, a shaft 0.91 m in diameter and 3.43 m long was drilled into a medium dense fine to coarse sand layer. The water table was encountered at a depth of 2.44 m. The top of the drilled shaft extends 0.12 m above the ground surface, which is grassed and unpaved. This foundation was designed by MHD for a maximum overturning moment of 188 kN-m.

Both shafts are constructed of 28 MPa compression strength concrete, reinforced by 414 MPa minimum yield strength steel. Shaft P4 has 16-25 mm diameter bars and shaft P17, 10-25 mm diameter bars. The uncracked flexural rigidities (EI) of the shafts were calculated to be 979 MN-m^2 (P4) and 933 MN-m^2 (P17).

Initial analyses of soil limiting equilibrium capacity and foundation deflec-

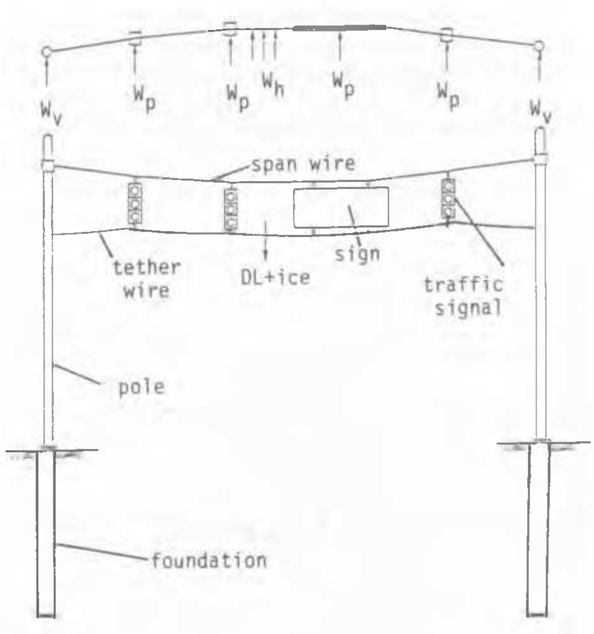

Figure 1. Loads on span wire mounted traffic signals (after AASHTO, 1985)

tions suggested that these drilled shafts are overdesigned. The soil limiting equilibrium analyses, using methods developed by Broms(1965), produced required shaft lengths of only 4.11 m and 2.44 m for shafts P4 and P17 respectively, significant reductions from the actual lengths.

Deflection analyses were performed using the finite difference computer program LPILE (Reese and Wang, 1989). The program requires the soil resistances to be expressed as a function of pile deflection. As the shaft deflects, the stresses in front of the shaft increase from initial values as stresses behind decrease. The soil reaction (p) per unit length which acts in opposition to the deflection (y) is characterized by p-y curves, Figure 2. The slope of a p-y curve is defined by the equivalent secant slope E, illustrated in Figure 2.

Figure 3 illustrates the shaft deflections and bending moments vs. depth computed by LPILE for shaft P4 due to the design overturning moment. The computed deflections are quite small and the deflections and moments both decrease very rapidly with depth. Below a depth of 4 m, the deflections are negligible, suggesting that the drilled shaft is considerably longer than necessary.

2. FIELD MEASUREMENTS

Field measurements were made at both sites during a winter storm in March of 1993. Maximum wind speeds of about 48 km/h were recorded at the levels of the span wires, about 8 m above street level. Three months later, the foundations were loaded statically by hanging dead weights at the mid-span of the signal span wires. The dead weight loads produced overturning moments of about 30 percent of the design live loads. The magnitude of these applied static loadings was in fact limited by the possibility of damage to the superstructure.

Field measurements included the lateral displacements and rotations of the top of the drilled shaft foundations, and wind speed and direction. Electrical resistance strain gages were attached to the poles, approximately 1 m above the base plates to avoid end boundary effects. The measured strains could then be used to backcalculate the loads applied to the structure.

Figure 4 illustrates the instrumentation set-up, including the orientations relative to the span wire of the coordinate X (parallel) and Y (perpendicular) axes. The field measurement system is described in detail by Smith (1993).

3. PRESENTATION OF DATA

X- and Y- displacements, strains and inclinations, wind speeds and azimuths, and temperatures were collected in 60 second duration data sets. Over 175 data sets were collected, approximately 150 during the storm loadings and another 25 during the static loading tests. Smith (1993) and DiFiore (1994) present the complete data.

All measured movements were very small, confirming the initial assessment that the foundations are overdesigned. During the storm wind loadings, the tops of the drilled shafts moved less than 0.025 mm. These movements are especially small possibly because of dynamic interactions between the wind and traffic signals. During the wind loadings, the traffic signals and cables oscillated in both the X- and Y- directions, at varying frequencies and amplitudes. The movements were not synchronous, and no overall system resonances were apparent. As a result, the overturning moments which were transmitted to the foundation are probably signifi-

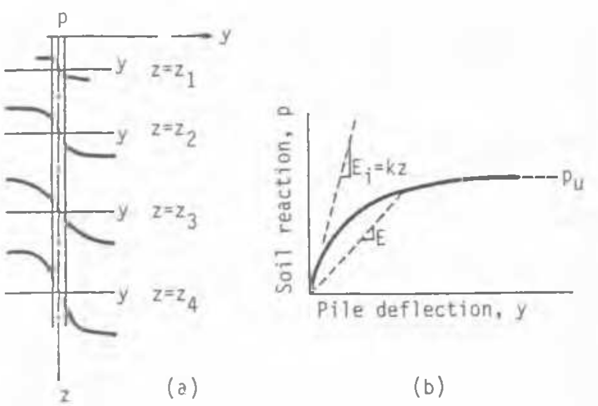

Figure 2. Typical p-y curves (after Reese and Wang, 1989).

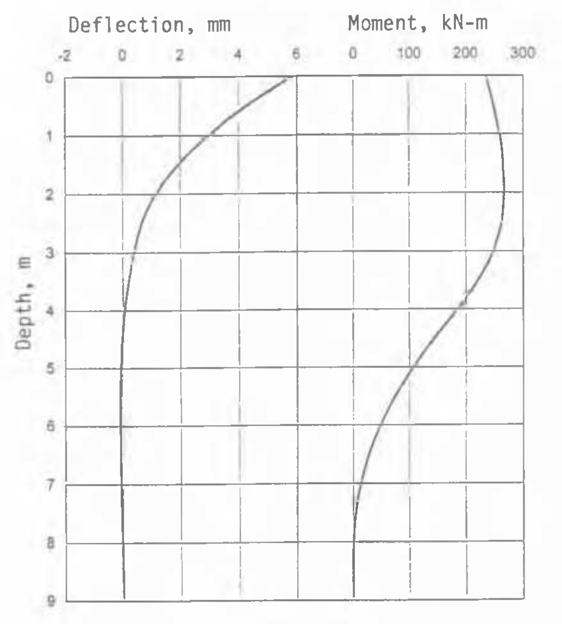

Figure 3. Computed deflections and bending moments vs depth due to design loading-P4.

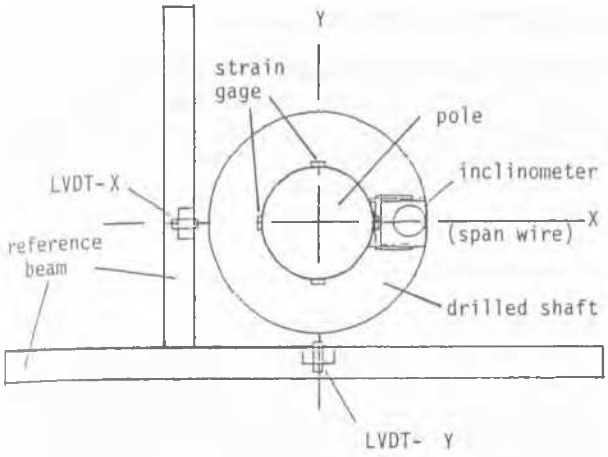

Figure 4. Plan view of field instrumentation arrangement (after Smith, 1993).

cantly smaller than the moments calculated by assuming that the wind applies a static force over the full exposed area of the traffic signals, span wire, and supports (Figure 1). The acquired strain gage data confirm this. For example, the overturning moment calculated from the strain measured during a wind gust of 34 km/h was 3.1 kN-m, only about one-third the overturning moment calculated by traditional pseudo-static analysis.

During the static controlled loading tests, the measured shaft head deflections at P4 were less than 0.05 mm, possibly due to the restraint offered by the 0.15 m thick concrete sidewalk.

Figure 5 illustrates a data set at P17 collected during the static tests. The data set show a loading sequence in which the foundation overturning moments were increased from 15.3 kN-m to 40.8 kN-m.

The recorded X- displacement and rotation are approximately 0.09 mm and 0.03 degrees. The recorded Y- displacement and rotation were much smaller since the lateral loads were applied in the X direction. When the pole at shaft P17 was loaded to an overturning moment of 52 kN-m and then unloaded, the top of the drilled shaft deflected and rebounded about 0.22 mm. This was the largest deflection measured during this study.

4. ANALYSIS AND DISCUSSION

Parametric studies were performed to compare the shaft head deflections measured during the controlled static loading tests with LPILE deflection analyses. Sets of recommended p-y type curves derived from field load test data (Reese and Wang, 1989) and coded as subroutines within LPILE were used for these computations. These p-y type curves must be scaled to account for site specific soil properties. The computed deflections depend on the factor,

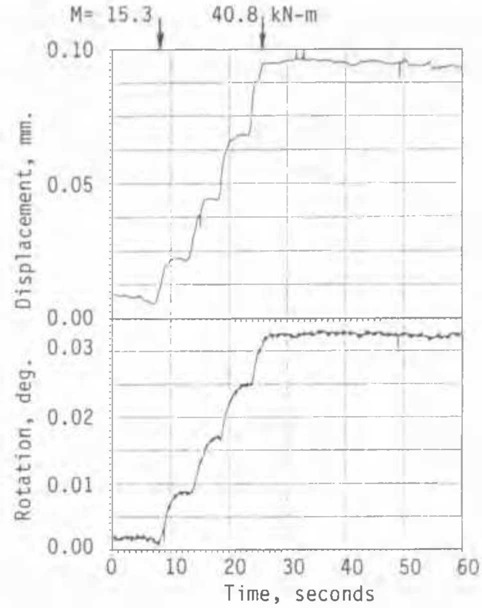

Figure 5. Measured X- displacement and rotation during static increases in moment from 15.3 kN-m to 40.8 kN-m - P17.

k, which defines the slope E_i, of the initial portion of the p-y curve (Figure 2). k corresponds to the constant of horizontal subgrade reaction defined by Terzaghi(1955), although the values recommended in this classic paper are extremely low (Habibagahi and Langer, 1984). Reese and Wang(1989) recommend somewhat higher k values based on interpretations of field load test data.

Table 1 summarizes the results of some LPILE computations for shaft P17. When the k values for sands recommended by Reese and Wang(1989) were used, the computed shaft head deflection was an order of magnitude larger than the measured deflection. However, when the k values were increased to approximately 20 times the Reese and Wang (1989) recommendations, the computed shaft head deflection equaled the measured value of 0.22 mm.

The very high k values backcalculated from the measured deflection at shaft P17 are probably a result of the very low stress levels experienced by the foundation soils during these field tests. Other possible contributing factors may include: neglecting base resistance at the tip of the shaft (Borden and Gabr, 1991); densification of the sand due to previous loadings; and the possibility of an enlarged shaft near the top due to overexcavation during construction (DiFiore, 1994).

These high k values are consistent with constants of horizontal subgrade reaction reported by others. Habibagahi and Langer (1984) describe the wide range of values reported in the literature, complicated by the lack of uniformity of the definitions used. Alizadeh and Davisson(1970) back-

Table 1. LPILE calculations - Shaft P17 (App. moment = 52 kN-m, meas. def.= 0.22mm)

	Constant of horizontal reaction, k kN/m^3	Calculated deflection, δ mm
a. Reese and Wang (1989) recommendations		
sand above water	24,000	
sand below water	16,000	2.1
b. Backcalculated k values		
sand above water	543,000	
sand below water	271,000	0.22

calculate from field measurements, constants of subgrade reaction as large as 81,000 kN/m^3 at ground line deflections of about 1.27 mm, still considerably larger than the deflections measured in this study. Their data also show that the constant of subgrade reaction increases rapidly with decreasing foundation deflection.

5. CONCLUSIONS

This paper presents data from field performance measurements of two full scale laterally loaded drilled shafts which support traffic signals. The lateral displacements and rotations of the top of the drilled shaft foundations and overturning moments from strain gage data were measured during a winter storm and later during controlled static tests. Conclusions of this study are:

a. The drilled shafts moved very small amounts for all loadings, supporting initial analyses which indicated that they are overdesigned.

b. The overturning moments measured during the storm wind loadings were much smaller than the loadings calculated by assuming that the wind applies a static force to exposed areas (AASHTO, 1985). Visual observations suggest dynamic interactions which reduce the moments transmitted to the foundation.

c. Deflection analyses by the computer program LPILE were then used to backcalculate equivalent constants of horizontal subgrade reaction (k). At shaft P17, the backcalculated k values were much larger than the values recommended by Reese and Wang (1989), possibly due to the very low stress levels experienced by the foundation soils. These high values are consistent with constants of subgrade reaction reported by others (Habibagahi and Langer, 1984).

6. ACKNOWLEDGEMENTS

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