Drilled Pier Design Based on Load-Settlement Curve

Calcul des Pieux d'après la Courbe Charge-Tassement

Z. BAŽANT Professor, Technical University and Geotechnical Consultant, Geoindustria, Prague, Czechoslovakia
J. MASOPUST Geotechnical Engineer, Geoindustria, Prague, Czechoslovakia

SYNOPSIS Dimensional analysis allowed to find the theoretical model of analysis of axial bearing capacity of drilled piers. Solution based on the limit state of deformation defines ultimate load-settlement curve as a combination of parabola and straight line. The parabola is defined by the settlement at ultimate yield load. The settlement is computed using cumulative modulus. Back-figured values of cumulative moduli are given in Tables. The capacity reduction factor 0.7 introduced in analysis of ultimate shaft resistance and of ultimate bearing capacity is suggested being adequate to confidence level 0.95.

INTRODUCTION

Three methods of the analysis of ultimate axial bearing capacity of drilled piers, based on limit state of deformation, can be used: the allowable capacity tables, the load test results and the construction of the load-settlement curve. Allowable capacity tables were prepared by J. Masopust, who analyzed statistically 226 load tests /Bažant 1979 Tables 16./a/ to /c//. These tables, as well as tables published in some Building Codes, are useful on small jobs and in preliminary estimates. Load tests which theoretically will be the best procedure are in many instances not feasible due to large loads required nor representative due to a great scatter of test results preventing the determination of ultimate value having the necessary confidence level. There remains the third method, the construction of the load-settlement curve used to define ultimate pier capacity as the load corresponding to permissible settlemet /Bažant 1979/. In this paper the final version of the third method is presented /Bažant 1980/.
The construction of load-settlement curve of drilled piers was attempted by many researchers. They proposed the curve in the form of two straight lines /Burland et al. 1966, Cambefort 1964, Christoulas 1976, Poulos 1972, Whitaker and Cooke 1966/, four straight lines /Franke 1977, Seefluth 1978/, hyperbolic curve /Alpan 1978/ and exponential curve /Schäffner and Walensky 1968/.
The attempt to make the method to work is facing considerable difficulties. The writers tried to overcome them by introducing /1/ the load-settlement curve composed of parabola and straight line, /2/ the modulus of deformation rising in direct proportion to depth and /3/ the introduction of appropriate capacity factor. The kind of theoretical analysis adequate to drilled pier problem was found by dimensional analysis.

DIMENSIONAL ANALYSIS

The solution of axial bearing capacity should fulfill the laws of mechanics of similitude. It is supposed that the axial bearing capacity of the drilled pier embedded in semi-infinite homogeneous isotropic elastic solid can be written as a function

$$f/Q,s,D,d,E_p,E_s/ = 0 \qquad /1/$$

where the independent variables of the problem are Q = head load, s = settlement of pier head, D = embedded length of pier, d = pier diameter, E_p = Young's modulus of pier, E_s = secant cumulative modulus of soil around pier. Shaft resistance Q_s and bearing capacity Q_b are not independent variables, because they can be determined knowing the proportion of base load /Poulos 1972/. Poisson's ratio of the soil was ignored, because its effect is relatively small. Dimensional analysis of Eq./1/ delivers following dimensionless parameters

$$\pi_1 = \frac{D}{d} \qquad \text{slenderness ratio} \qquad /2/$$

$$\pi_2 = \frac{E_p}{E_s} = K \qquad \text{pier stiffness ratio} \qquad /3/$$

$$\pi_3 = \frac{sdE_s}{Q} = I_s \qquad \text{settlement influence factor} \qquad /4/$$

$$\pi_4 = \frac{s}{d} \qquad \text{relative settlement} \qquad /5/$$

The analysis of axial bearing capacity of drilled piers by Poulos /1972/ respects the dimensionless parameters π_1, π_2, π_3 and delivers the values needed an ultimate load-settlement curve to be constructed. The parameter π_4 is useful to check the limits of validity of the elastic solution when comparing with the load test results.

ULTIMATE LOAD-SETTLEMENT CURVE

It is proposed, that the ultimate load-settlement curve consists of the parabola connecting origin o and point e, and the straight line ef /Fig.1/.

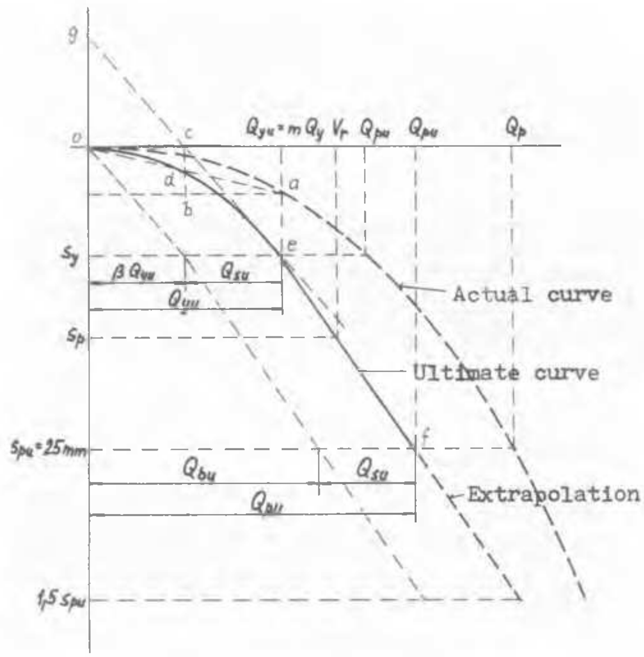

Fig.1 Ultimate load-settlement curve

Construction of the curve is simplified to the determination of two points, e and f. Point e is defined as the yield point at which the yield load Q_y is reached, i.e. shaft resistance in elastic state fully mobilized. Point f is the point in elastoplastic state, defined by a load pertaining to 25 mm settlement. Ultimate values are obtained by introducing capacity reduction factor m.
Ultimate yield load according to Poulos /1972/ is

$$Q_{yu} = \frac{Q_{su}}{1 - \beta} \qquad /6/$$

where Q_{su} = ultimate shaft resistance and β = proportion of base load. The corresponding settlement

$$s_y = I_s \frac{Q_{yu}}{mdE_s} \qquad /7/$$

Values of β and I_s were derived by Poulos/1972/. Shaft resistance Q_s for stiff clays was given by Reese et al./1976/ and for cohesionless soils by Touma and Reese /1974/. Ultimate value

$$Q_{su} = m\, Q_s \qquad /8/$$

Values of E_s and m are treated subsequently. Axial bearing capacity Q_p at 25 mm settlement is

$$Q_p = Q_b + Q_s \qquad /9/$$

At settlement larger than s_y the shaft resistance Q_s = const. The bearing capacity Q_b for stiff clays was derived Burland and Cooke /1974/ and in cohesionless soil by Touma and Reese /1974/. Ultimate value of axial bearing capacity is

$$Q_{bu} = m\, Q_b \qquad /10/$$

The axial bearing capacity Q_p at 25 mm settlement /as well as shaft resistance Q_s/ should be expressed with respect to dimensionless parameters π_1, π_2. If this is not the case, it happens that the line ef gives Q_p too great, exceeding the value defined by tangent to parabola in point e. This is not physically possible, because the modulus should decrease with rising load and therefore the tangent in the point e is the extreme position of the straight line ef. On the other side the smallest bearing capacity at 25 mm settlement $Q_p = Q_s$ which means that the yield point is reached the pier plunges into the soil as e.g. in soft clay. The shape of load-settlement curve at settlement higher than 25 mm is hard to predict. The assumption may be made that the prolongation of the straight line ef up to the settlement of about 30 to 40 mm is in stiff soils possible.

VERTICAL NONHOMOGENEITY OF SOIL

Modulus E_s is not a constant. It is nonlinear with load and increasing with depth. Nonlinearity is considered when the secant modulus at yield point e is introduced. Vertical nonhomogeneity is considered by introducing the cumulative modulus defined as an ideal modulus of linear homogeneous soil having the same stress-strain behavior as a real soil possessing vertical nonhomogeneity. Due to the definition of moduli it is possible to find the yield point e unequivocally. Approximately the cumulative modulus applying to 10 m long pile is

$$E_s = 3\, E_o \qquad /11/$$

where E_o is the secant triaxial modulus of deformation of the soil at the surface. The approximate value supposes the linear increas of modulus from the surface to the pier base where modulus $5E_s$ holds, from which the afforementioned value is obtained

$$E_s = 0.5\ /E_o + 5E_o/ = 3E_o \qquad /12/$$

If the pile length is smaller than 10 m, cumulative modulus is obtained by linear interpolation.
The insight into the real values of the cumulative moduli was obtained calculating backfigured moduli from equation

$$E_s = I_s \frac{Q_y}{s_y\, d} \qquad /13/$$

where I_s is introduced according to Poulos/1972/

The statistical analysis by Masopust /1980/ of 226 load tests provided the cumulative moduli dependent on diameter and length of drilled pier and on soil type which are given in Tables I, II and III.

The guess made in Eq./11/ lies within the limits of Tables which give E_{s10} = /2 to 4.5/$E_{s1.5}$ where E_{s10} is modulus for 10 m depth and $E_{s1.5}$ modulus for 1.5 m depth.

Theoretical solution of settlement influence factor I_s for linear increase of modulus with depth was presented by Poulos /1979/. Another solution was proposed by Randolph and Wroth /1978/.

Table I. Cumulative modulus E_s /MPa/ in rocks

D /meters/	d /meters/								
	0.6			1			1.5		
	A-3	A-4	A-5	A-3	A-4	A-5	A-3	A-4	A-5
1.5	50.3	28.2	20.0	72.3	35.0	24.7	85.5	33.5	22.3
3	64.5	43.1	30.8	105.5	57.3	41.0	138.3	58.8	41.2
5	-	58.2	41.3	-	75.3	54.8	-	87.9	63.7
10	-	87.5	61.6	-	114.5	83.2	-	133.0	97.0

A-3 Weathered igneous or metamorphic rocks, unweathered sedimentary rocks with layers under 5 cm

A-4 Completely disintegrated igneous or metamorphic rocks, slightly weathered sedimentary rocks having layers under 5 cm

A-5 Unweathered, indurated soft rocks, compressive strenght over 2 MPa

Table II. Cumulative modulus E_s /MPa/ in cohesionless soils

D /meters/	d /meters/								
	0.6			1			1.5		
	I_D								
	0.5	0.7	1	0.5	0.7	1	0.5	0.7	1
1.5	11.0	13.7	28.3	12.8	15.8	30.6	13.0	15.3	29.0
3	15.5	20.2	44.5	18.4	25.0	47.8	19.4	24.5	52.5
5	18.8	26.6	56.1	22.8	32.5	69.1	24.5	36.0	78.2
10	23.8	36.6	72.1	29.8	47.8	93.4	32.6	54.0	107.3

CAPACITY REDUCTION FACTORS

In the strenght design the capacity reduction factors m are introduced into the analysis of ultimate shaft resistances Q_{su} and ultimate bearing capacities Q_{bu}. It is assumed that capacity reduction factor m = 0.7 and all loadings are service loads with unity load factors. Analysis of load test results /Masopust 1978/ provided the mean values V_o/Q, /V_o = allowable pier capacity and Q = axial head load/ which vary between 0.63 and 0.83. If m = 0.7 is introduced into the strenght design, the overall confidence level is 0.96 which is higher than the prescribed value 0.95. Factor m = 0.7 does not fit in 9 cases out of 230. Therefore m = 0.7 is assumed as adequate. In detail, confidence level 0.95 is not fulfilled for all types of soil covered by Tables I to III, e.g. for loose to firm sands which, however, was not considered inadmissible, because drilled piers are used in these soils rather infrequently.

Pier installation methods have a pronounced effect on the bearing capacity. This is expressed by installation factor /shear strength reduction factor/ by which capacity reduction factor is multiplied. The values of installation factor for drilled piers embedded in stiff clays are approximately 1, which holds for drilled piers installed dry or in a thin slurry, 0.6 for piers installed in slurry and 0.4 for casing left in place.

CONCLUSIONS

The ultimate load-settlement curve is presented in a form suitable for determining the axial bearing capacity of drilled piers. In th analysis an attempt is made to present a simpl solution of the nonlinearity of load and settlement and the vertical nonhomogeneity of soil

Table III. Cumulative modulus E_s /MPa/ in cohesive soils

D /meters/	d /meters/					
	0.6		1		1.5	
	I_C					
	0.5	≥1	0.5	≥1	0,5	≥1
1.5	6.9	13.2	7.9	13.4	8.6	12.3
3	10.0	22.0	12.5	23.9	13.7	23.0
5	12.5	31.2	15.9	35.4	18.4	36.7
10	15.5	44.3	21.3	51.3	24.6	57.4

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