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DESIGN

BEARING CAPACITY OF DRIVEN PILES SUPPORTED ON SLIGHTLY COMPRESSIBLE SOILS

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Results of static tests of driven piles supported on neogenic limestones, dense sands, and clays are analyzed. It is noted that, depending on their length and the soil characteristics, driven piles combine within themselves the functional characteristics of both friction and end-bearing piles. Suggestions are presented for refinement of the design of foundations on driven end-bearing piles.

It is accepted that driven piles supported on rocky and slightly compressible soils be called end-bearing piles [1]. Use of these piles is reflected primarily in foreign literature. Thus, [2] contains recommendations for calculation of the bearing capacity of piles with consideration of the quality index of the rock (*RQD*) for the design of foundations, and also requirements for mandatory static tests. Tomilson [3] cites methods for determination of the bearing capacity of piles as the sum of the resistances along their lateral surface and the resistance under their lower ends. Moreover, the resistance along the lateral surface will depend on the displacements during pile tests. Methods for calculation of the bearing capacity of driven end-bearing piles have yet to be formalized, and are supported primarily on the experience acquired by the authors.

For a pile supported on bedrock, it is expedient, in conformity with [4], to neglect adherence with the soil along its lateral surface, and it is suggested that the load applied to the head be transmitted onto the bed beneath its lower surface. The calculation should be performed as for an elastic cylinder embedded in an elastic medium, however, even for a shallow pile in a rocky soil. It is precisely this approach that is basic in domestic regulatory literature. The requirement that the bearing capacity F_d of a driven end-bearing pile supported on rocky and slightly compressible soils should be determined from the formula $F_d = \gamma_c RA$ is stated in [1, 5], where γ_c is the working-conditions factor of the pile in the soil, R is the computed resistance of the soil beneath the lower end of the pile, which can be assumed equal to 20 MPa, and A is the support area of the pile on the soil with R. In determining the bearing capacity of an end-bearing pile, the *RQD* of the support layer is disregarded, i.e., these piles should be designed proceeding from the strength of the material in the shaft of the pile. For piles sunk in boreholes, Building Code [1] considers their depth of embedment in the layer of limestone, and contains the additional requirement that R be assumed to be no more than 20 MPa. For design, this approach assumes a broad spectrum of possible R values of rocky soils beneath the lower end of the piles. It is possible to conform to the requirement [6] that for weathered rocky soils, their ultimate uniaxial com-

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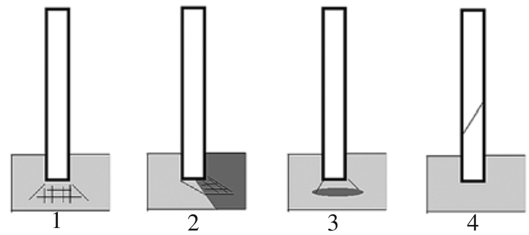

Fig. 1. Diagram showing failure of bed when supporting pile: 1) on uniform rocky bed; 2) on nonuniform rocky bed in plan; 3) bed with deep-seated weak layer; 4) diagram showing failure of pile material.

pressive strength should be assigned with consideration of results of plate tests or pile tests under a static load. It should, in the majority of cases, be considered that for plate tests of rocky soils, they are not brought to failure, and a value close to the standard ultimate compressive strength $R_{c,m,n}$ be determined. A reliability index γ_g = 1.2 or 1.25 is most probable for these tests.

A similar requirement for driven piles, piles driven in borehole, and end-bearing piles resting on weathered and softened rocky soils is contained in [1].

In essence requirements for calculation of driven piles supported on rocky, semi-rocky, and slightly compressible soils are conditional. Moreover, it is not understood in what cases the mechanical behavior corresponds to rocky soils, and in what cases to rock-free soils. From the standpoint of modeling their function, this is determined by a set of criteria for conversion to the plastic state, i.e., by the Coulomb-Mohr or Hoek-Brown criterion for slightly compressible soils with E close to 50 MPa is not ambiguous for the majority of researchers.

Positions for analysis of driven end-bearing piles are not incorporated in [1]; this is associated with a reduction in the volume of their usage. There are, by far, less experimental data on these piles than for cast-place piles; available information on the bearing capacity and results of field tests must therefore be analyzed.

Proof that the bearing capacity of driven piles on rocky and semi-rocky soils is achieved with small displacements is ambiguous. The settlement-load curve of these piles differs little from the curve for friction piles. Considering the differences in the function of end-bearing piles, however, use of dynamic tests for determination of the bearing capacity is not always competent. The required refusal calculated on the basis of dynamic formulas [1] cannot be achieved due to failure of the pile material during sinking. It is therefore necessary to determine the refusal for driven end-bearing piles with retention of their integrity.

It is obvious that embedment of driven piles in a layer of rocky soils is extremely complex. Moreover, the resistance R may vary owing to possible support of the pile on different rocks (Fig. 1), and the probability of failure of the pile material increases.

It has previously been difficult to determine which of the failure schemes shown in Fig. 1 will be realized. Moreover, refusal for each of the computed cases under consideration will differ. Failure in accordance with the fourth diagram is probable when the required refusal is incorrectly assigned. In the construction of a production and warehouse complex in the Noginsk District of the Moscow Oblast, however, end-bearing piles with a section of 30×30 cm and length of 6 m were driven to limestones that had broken down to a state of flour and fine crushed stone. During test driving, cases of failure of the body of the pile were observed prior to attainment of design refusal.

In designing piles supported on rocky and semi-rocky soils, approaches similar to analyses of end-bearing piles are possible [2]. Moreover, realization of friction forces is associated with possible settlement of the pile. The relation between the bearing capacity and length of the pile is indirectly evidenced. If the pile is short and is supported on a layer of limestones, the basic load is transferred onto its lower end. The longer the pile, the more critical the recovery time, since the lateral surface may be included in its functioning. The bearing capacity of end-bearing piles can be determined only with use of pile refusal, as determined by a refusal gage.

Examples of field tests of driven piles resting on slightly compressible soils in Volgodonsk are presented below.

Static testing of piles on experimental proving grounds were conducted in soils with their in-situ moisture content in conformity with [7]. The load was transferred onto the pile being driven by DG-200 hydraulic jacks, the reactive forces of which were taken up by an anchoring system. The driving load was brought to 1.8-2.02 MN, and no further loading was applied, since rupture of the reinforcement in the anchor piles was possible.

Test piles supported on neogenic limestones (proving ground No. 1). Soils prone to slump-type settlement, the stratum of which is 35 m thick in certain sections, are widespread throughout the area of Volgodonsk, and the industrial zone adjacent to it. A thick stratum of neogenic deposits, which is overburdened by fill soils from the surface to a depth of 4 m, was revealed at the site of a multistory tenement building.

The geologic-engineering section below the elevation of the bottom of the pit is composed of fill soils (2.1 m), heavily weathered limestone-crag (1.2 m), dense Sarmatsk clay (7.7 m), and Sarmatsk limestones weathered to a state of carbonate clay (2 m) with inclusions of limestone lumps. A layer of dense highly arenaceous clay (1 m), which is underlain by slightly moist sands of medium density (3 m) exists from a depth of 13 m beneath the sandstones. Sarmatsk clays (4 m) of hard and semi-hard consistency are exposed to a depth of 21 m. The water table is situated at a depth of 0.5 m from the bottom of the pit.

The foundations of the tenement building were designed from driven piles with a section 35×35 cm and length of up to 12 m in the ground, and rest on the Sarmatsk limestones. The "end-bearing pile" scheme was adopted based on interaction between the piles and soil. The decision to conduct test operations to ascertain the feasibility of driving piles to the design elevations and defining the bearing capacity of the piles more precisely was made in connection with the absence of geologic-engineering survey data.

Four test piles were driven in a broad section of the pit. The distance between piles (axes) was 2.15 m, and length in the soil 12.2 m (pile No. 1), 12.0 m (pile No. 2), and 10.6 m (piles No. 3 and 4). The lower ends of piles No. 1 and 2 were driven 1.3 m into the layer of Sarmatsk limestones, and piles No. 3 and 4 into the dense clays.

A scheme with groups of three anchor piles driven 11.6 m into the ground was used for the tests. The pile groups were positioned at the four corner points of a 5.2×5.7 m grid, and the test piles were positioned symmetrically within the central section of the grid.

The test and anchor piles with sections of 35×35 cm were sunk in leader holes 300 mm in diameter, which were bored to a depth of 3.3 m. The piles were driven by an SP-78 tubular diesel hammer with a 3.5-ton impact section. The total number of blows required to sink test piles 1-4 were, respectively, 645, 684, 366, and 308, and the refusal of the piles on termination of their driving was 0.19, 0.17, 0.62, and 0.77 cm, respectively.

The limiting resistances F_u of the piles, which were determined based on the refusal measured after their driving, were 2.52-2.37 MN (piles No. 1 and 2), and 0.83-0.74 MN (piles No. 3 and 4).

The settlements of piles No. 1-4 under the maximum impressing load of 2.02 MN ranged from 16 to 28 MN (Fig. 2).

The piles supported on the dense sands (proving ground No. 2) were tested in accordance with the procedure outlined in Republic Construction Regulation 50-87 [8] on soils prone to slump-type settlement.

The geologic section of the site from the surface to a depth of 11.2 m is composed of loess-like clayey loams classed as type-II in terms of proneness to slump-type settlement. Loess-like clayey loams not prone to slump-type settlement (5.6 m) lie below, and are underlain by a layer of alluvial clays 5 m thick. The support layer for the piles (saturated fine dense alluvial sands) resides at a depth of 21.8 m.

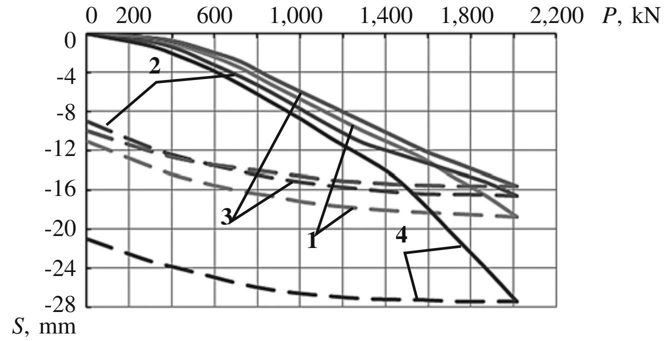

Fig. 2. Plots showing dependence of settlement S on load P for driven piles No. 1-4 supported on limestones (proving ground No. 1): 1-4) numbers of piles.

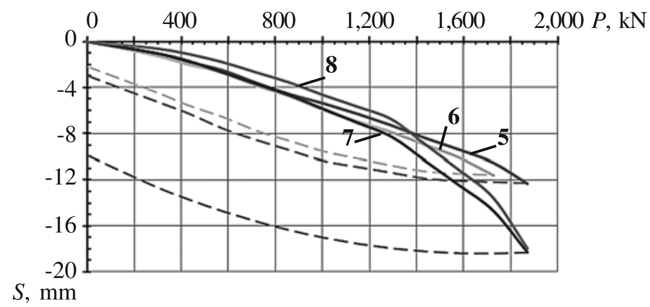

Fig. 3. Plots showing dependence of S on P for piles No. 5 and 6 and No. 7 and 8, supported, respectively, on dense sands (proving ground No. 2), and dense clays (proving ground No. 3): 5-8) numbers of piles.

The hydrogeologic conditions of the site are represented by two water-bearing horizons. The first is confined to the loess-like clayey loams not prone to slump-type settlement, and is situated at a depth of 12 m, while the second, which is under pressure, is exposed in the alluvial sands. Alluvial clays and Maikopsk clays serve as the upper and lower water-impervious layers confining the water-bearing horizon.

Two sectional piles with a cross section of 35×35 cm and length in the ground of 22.4 m were sunk in leader holes with a diameter of 500 mm and depth of 11.2 m. The piles were driven by an SP-78 diesel hammer with a 3.5-ton impact section to a refusal of 0.2 cm per single blow.

According to static-penetration data, the resistances of the alluvial sand under the cone of the probe q_{ci} ranged from 30.3 to 42.2 MPa; according to [9], this characterizes them as dense with a compression modulus $E > 41$ MPa.

In these soils, the settlements of piles No. 5 and 6 were 12 mm under a maximum impressing load of 1.73 and 1.87 MN, respectively (Fig. 3).

Piles supported on dense clays (proving ground No. 3) were tested in accordance with the procedure cited in Republic Construction Regulation 50-87 [8].

The geologic section of the proving ground from the surface to a depth of 8 m is comprised of loess-like clayey loams prone to slump-type settlement. Loess-like clayey loams not prone to slump-type settlement reside below (8.8 m), and are underlain by a layer of alluvial dense clays (7.4 m) with a roof at a depth of 16.8 m. A thick stratum of dense sands is revealed beneath the layer of clay.

The hydrogeologic conditions at proving ground No. 3 are similar to those at proving ground No. 2, but the water table in the former resides at a depth of 9 m.

TABLE 1

Pile No.	Pile-support layer	Effective length of pile, m	P_{max}, MN	S_{max}, mm	S after unloading, mm	F_u (MN) of pile when S = 10 mm	$R = F_u / (\gamma_c A)$, MPa
1	Limestone	12.2	2.02	19	11	1.30	10.6
2		12.0	2.02	17	9	1.20	9.8
3	Dense Sarmatsk clay	10.6	2.02	16	10	1.39	11.3
4		10.6	2.02	27	21	1.09	8.9
5	Dense sand	11.2	1.87	12	3	1.66	13.5
6		11.2	1.73	12	2	1.57	12.8
7	Dense alluvial clay	14.0	1.87	18	10	1.41	11.5
8		14.0	1.87	18	10	1.50	12.2

Piles No. 7 and 8 with a cross section of 35×35 cm and length in the ground of 22 m were sunk in leader holes with a diameter of 500 mm.

The lower ends of the piles were driven to a depth of 5.2 m in the dense alluvial clays.

According to static penetration of the alluvial clays, the resistance q_{ci} ranges from 2.7 to 7.6 MPa under the cone of the probe.

The impressing load during the tests of piles No. 7 and 8 were brought to 1.9 MN (see Fig. 3), during which their settlement amounted to 18 mm.

The settlements of buildings and structures raised on end-bearing piles cannot be calculated in conformity with [5], since the deformations of these beds do not exceed limiting standard values.

The plots presented above for the static tests of driven piles supported on the slightly compressible soils indicated, however, that the influence exerted on the piles by the load assigned from the condition R = 20 MPa under their lower ends, or a load equal to the computed resistance of the pile with respect to its material, will lead to settlements comparable to those of friction piles. Experience acquired from occupied structures on these piles indicates that their settlements may increase when they function as a component part of the foundation of the structure [1].For foundations supported on piles in the slightly compressible soils investigated, therefore, it is also necessary to perform the calculations with respect to deformations.

During loading, the settlement of piles No. 1-8 attained 12-27 mm (see Table 1), and ranged from 2-21 mm, respectively, during loading, and after unloading. Elastic compressive deformations of the shaft of amounted to 6-10 mm after unloading of the piles, and were found to be largest for piles No. 5 and 6, which are supported on the dense sands.

It is apparent from these plots that the mechanical behavior of piles No. 1-3 is similar, while pile No. 4 is supported on a weakened soil, and has a large settlement, i.e., high flexibility. As is apparent from Figs. 2 and 3, the piles function essentially elastically under load. Moreover, the plot showing the settlement of pile No. 3 is linear in nature over the entire load range, while that of pile No. 4 is in a bilinear relationship with the reduction in compression modulus under the lower end.

It should be considered that when loading groups of these piles, the forces within them are also distributed nonuniformly – a large load will be distributed onto the more rigid components, and the more flexible piles will be found less heavily loaded.

Values of the limiting resistance F_u of the piles, which are presented in Table 1, are determined from the condition that the limiting settlement of the structure S_u = 5 cm, but on the settlement-load curves, S = 1 cm is adopted for the calculation in accordance with [1]. Assignment of S_u = 5 cm is dictated by the actual settlements of entities erected on foundations formed from driven piles under the geologic-engineering conditions under consideration. The piles supported on the dense sands exhibit maximum resistance (1.6-1.7 MN). The piles supported on the limestones and dense clays (with the exception of pile No. 3) offer approximately the same resistance for an equal effective length (1.1-1.2 MN). The high resistance of pile No. 3 is apparently explained by the existence of limestone lumps in the section.

Equivalent computed resistances R of the soil under the lower end of the test piles, which were determined for an end-bearing pile with use of Fu and the formula $F_d = \gamma_c RA$ are presented in Table 1 for comparison. The R values range from 8.9 to 13.5 MPa, and on the whole, are substantially lower than the 20 MPa values indicated in [1]. Moreover, the relatively higher R values correspond to the dense sands, and lowest values to the dense Sarmatsk clays.

To ascertain the cause of the relatively lower settlements of the piles supported on the dense sands, we analyzed data derived from static penetration of the soils in the support layer. It was established that the q_{ci} obtained on the basis of penetration are higher than 20 MPa, while in the table presented in Building Code [9], the relation between q_c and E for sands is spread only over $q_{ci} \leq 20$ MPa. To obtain E with use of the table in [9], we constructed a hypothetical relationship for $q_{ci} > 20$ MPa. Attention was also focused on the relationship $E = 3q_c$, which is derived in [10]. The corresponding compression moduli of the sands ranged from 54 to 70 MPa, and $E > 90$ MPa, which is 50 MPa higher. Driven piles supported on these soils may, therefore be considered end-bearing piles, in principle, and their bearing capacity determined from formula $F_d = \gamma_c RA$, setting $R = 20$ MPa [1]. In [1], however, dense sands are not referred to as slightly compressible soils, and a pile supported on this soil is considered a friction pile, while the resistance R beneath the lower end of the pile should be assumed equal to 6.70 MPa, which is considerably lower than $R = 20$ MPa. The pile tests indicated that the actual values $6.70 < R < 20$ MPa.

The compression moduli E_{sl} at the level of the lower ends of piles No. 7 and 8, which correspond to static-penetration data on the alluvial clays ($q_{ci} = 2.7$-7.6 MPa), range from 19 to 53 MPa. Based on data derived from geologic-engineering surveys, $E = 19$ MPa. This implies that as component parts of the foundation, all piles should be considered friction piles, and the computed settlements of such a foundation alone will include a "safety factor." As component pasts of the foundation, some of the piles may function as end-bearing piles, and others as "friction" piles.

Thus, the compression modulus of the dense sands ($E = 54$-70 MPa), which were obtained from static-penetration data, are appreciably higher than the E of the dense alluvial clays (19-53 MPa). This may be one of the reasons for the relatively high settlement of the piles supported on the dense clays.

The settlements of large-panel buildings in Volgodonsk on foundations with piles supported on the dense sand and alluvial clays do not exceed 3 cm [11]. This fact, and the field-test data presented above for the piles supported on the slightly compressible soil confirm the author's suppositions concerning the functional characteristics of driven end-beating piles, which must be considered when refining design standards for pile foundations.

At the present time, it is expedient to confirm the bearing capacity assigned in the design of end-bearing piles by static tests.

Conclusions

The design of foundations on end-bearing piles offers a number of characteristic features that have yet to find sufficient reflection in regulatory literature. Results of field tests of these piles, and also data derived from geotechnical monitoring of structures erected on end-bearing piles must be systematized.

Proceeding from the standard condition $R = 20$ MPa, the bearing capacity of these piles implies that the strength of material in the shaft of the pile and the actual strength of the rocky soils under the lower end are determining factors in the calculation. This approach can be acknowledged permissible only for mandatory confirmation of the bearing capacity of the pile adopted in design from results of static or plate tests.

In determining the bearing capacity of a driven end-bearing pile supported on severely deteriorated limestones, it is necessary to consider the friction forces of the soils against its lateral surface, which increase with increasing length and flexibility of the pile.

The effect of the load assigned from the condition R = 20 MPa on an end-bearing pile under its lower end, or when equal to the computed resistance of the pile with respect to its material, may lead to settlements comparable to those of friction piles; analysis of deformations is therefore mandatory for the piles. Moreover, it is expedient to use correlation relationships between R and E, which can be established on the basis of characteristics obtained during geologic-engineering surveys.

If based on field tests of piles under design loads their settlement their settlements differ by more than 1.5 times, redistribution of forces between the piles must be considered in analyzing pile groups as a component parts of the foundation.

In the design of a foundation formed from end-baring piles, it is necessary, in addition to high elevations, to indicate the model of hammer used to drive the piles and the control refusal on completion of their driving, for which 0.002 m is recommended. The hammer should be selected in conformity with active norms, and consideration of the design parameters of the piles.

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