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Single Foundations of Framed Buildings and Skeleton Structures in a Compacted Base and Their Horizontal Load Resistance

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SINGLE FOUNDATIONS OF FRAMED BUILDINGS AND SKELETON STRUCTURES IN A COMPACTED BASE AND THEIR HORIZONTAL LOAD RESISTANCE

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ABSTRACT

In practice of construction in Russia in case of plastic soils bedding, the cast-in-place foundations are used, constructed in a compacted base in kind of tamping in the soil of the 2-4 m depth pyramidal holes with the subsequent ballast tamping into the hole bottom and placing concrete in the said hole.
Such foundations are advisable under the columns of framed buildings and structures. They are significantly more effective comparing to the pile foundations with a cast-in-place raft, as without a raft the volume of excavation works and formwork decreases significantly.
While design of such foundations for the framed buildings and structures, the horizontal load and bending moment calculation is of great importance. According to the above calculation, the horizontal displacements of a foundation and its reinforcing are evaluated. The aim of the given paper is to solve this problem.
The complex of the experimental – theoretical investigations has been carried out by a special program and included the horizontal load static tests of the foundations in tamped holes with the parallel CPT. By results of site experiments the rules of deformation of the stressed-deformed state are obtained and the criteria of achieving the limit state of a system “foundation-soil” are revealed. In particular, it was stated that the tamped in ballast creates a zone of an increased strength in a foundation base that prevents its horizontal displacement due to the horizontal load.
Based on experimental data, a design scheme was constructed and a method of such foundations horizontal load calculation was worked out including CPT.

INTRODUCTION

Lately, there is a trend in a foundation construction which combines the use of a cast-in-place concrete with an artificially improved base. The most long-termed foundations according to this trend are foundations in tamped holes (FTH). Such foundations construction includes the formation of the compacted soil along the side walls and under the bottom of a hole while its tamping. The holes are tamped by means of throwing down a 1.5 - 10 t tamper in the form of a future foundation from the height of 4 – 8 m. The excavating machines with the rigs including the guide rods and the tampers are used for tamping. The technological scheme of the above foundation construction is shown in figure 1.
The technology of the construction of the FTH with the enlarged base includes the following cycles: a- positioning of the tamper by the center of the future foundation and a hole tamping to the given depth; b- filling of the tamped hole with the ballast by separate portions with the tamper raised up; c- assembling of the reinforcing cages; d- placing concrete in a hole with the compaction of the freshly-placed concrete by the depth vibrators; e- placing of the column of the skeleton building or structure.

FTH use provides the high soil load capacity, decreases earthwork and formwork and has the high specific load capacity. The most effective is FTH as a separate foundation for the columns of the skeleton buildings and structures. In this case, the foundation is subjected to the essential lateral and moment loads which should be considered while design.
FTH has structural and technological design special features which should be taken into account when a design scheme working out. First, a foundation is a relatively short rod with the ratio l/d ≈ 2-4 (where l – a length in a soil, d – FTH cross-section). Second, a form of a foundation body is pyramidal (conical) by its length with the decreasing cross-section down by the depth. Third, at the level of a foundation base there is an enlargement consisting of the ballast tamped into the base. The said enlargement contacts with the concrete of the main foundation body. Fourth, a foundation behaves in a compacted soil, as the concrete is placed into the hole with the compacted soil around it.
To realize a design scheme and work out an engineering design method for practical use, the laws of the behavior of the system “laterally loaded foundation – base” should be experimentally evaluated considering the above design special features of the foundation.

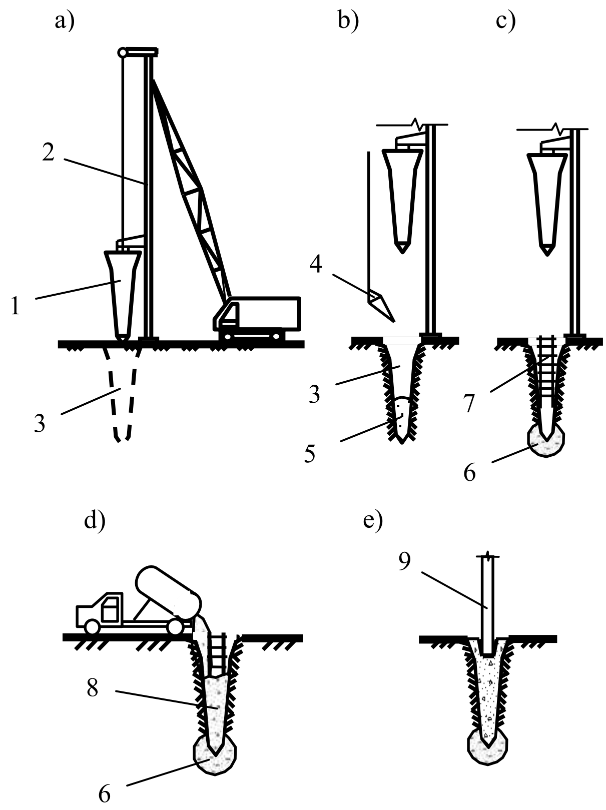

Fig. 1. The technological scheme of the foundation construction in the tamped hole

a – positioning of the tamper by center of the future foundation and tamping of the hole to a given depth ;
b – filling of ballast into the tamped hole by separate portions with the tamper raised up;
c – tamping of each ballast portion into the hole soil base with the same tamper;
d – positioning of the reinforcing cage into the hole and concrete placing with the compaction by a depth vibrator;
e – column assembly;
1- a tamper; 2- a guide rod; 3- a hole; 4- a bin with the ballast; 5- ballast; 6- ballast tamped into the soil; 7- a reinforcing cage; 8- concrete; 9- a column.

EXPERIMENTAL INVESTIGATIONS

The field experimental investigations were carried out on 5 test sites in cities Chelyabinsk and Ufa.

The geological structure of the test site in Chelyabinsk consists of deluvial-alluvial clay with the carbonate inclusions of semisolid consistency with the natural humidity W = 0.20…0.25, compactness at the natural humidity ρ = 1.91…1.95 g/cm^3, porosity factor e = 0.7…0.8, fluidity index I_L = 0.30…0.25, inner friction angle φ = 19^0, cohesion c = 0.021…0.022 MPa, modulus of general deformation E = 18.5…21 MPa. The sites of the experimental investigations in Ufa consist of talus deposits in kind of clays and loams with W = 0.26…0.32, ρ = 1.84…1.87 g/cm^3, e = 0.88…0.923, I_L = 0.12…0.35, φ = 19…20^0, c = 0.030…0.052 MPa, E = 10…18 MPa.

In places of test foundations construction, soil CPT was carried out with the unit C – 832 to the depth of 10 m.

When a hole tamping, a hexahedral tamper was used in Chelyabinsk (fig.2) and a rectangular tamper – in Ufa (fig.2). The hole was tamped to the full foundation length.

Fig. 2. General view of the unit and the tamper for the FTH construction on the site in the city Chelyabinsk

When investigations carrying out, FTH was constructed with the different volume of the tamped in ballast ranging from 0 to 4.4 m^3 (see table 1).

Results of FTH lateral load CPT are shown in kind of diagrams in figure 3 and in table 1.

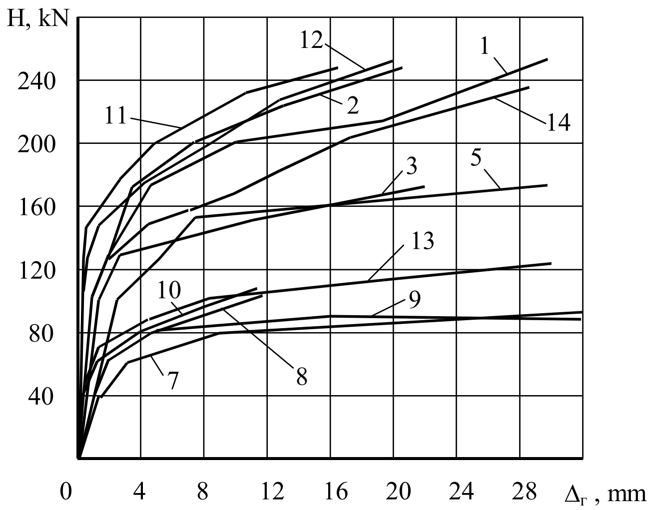

Fig. 3. Dependences “lateral load-displacement” (the number of the curve corresponds to the number of FTH in table 1).

Table 1. Results of FTH lateral load CPT

No.of site	No.of FTH	Geometrical dimensions of the tamper, m			The volume of tamped in ballast, m^3	Lateral load H_0 with u_0=10mm,kN	Depth of location of the point of zero displacement with H_0 action, m
		top	bottom	length			
				Chelyabinsk			
1	1	1,12 x 1,3	0,476 x 0,55	2,8	1,0	200	1,46
	2	- " -	- " -	- " -	1,0	213	1,73
	3	- " -	- " -	- " -	0	146	0,52
	5	- " -	- " -	- " -	1,5	153	1,74
2	7	- " -	- " -	- " -	0	81	2,30
	8	- " -	- " -	- " -	0,5	100	1,40
	9	- " -	- " -	- " -	1,0	84	2,20
	10	- " -	- " -	- " -	1,5	107	2,30
3	11	- " -	- " -	- " -	1,5	223	1,90
	12	- " -	- " -	- " -	1,0	210	1,44
				Ufa			
4	13	1,1 x 1,1	0,6 x 0,6	3,0	4,4	107	1,82
5	14	- " -	- " -	- " -	1,0	172	1,43

When CPT of the field foundations, the lateral load was applied by steps equal to 1/10 of the supposed load capacity of the foundation. Each next step was applied after the stabilization of the lateral displacement at the level of the soil surface after the previous load step. The change of the lateral displacement not more than 0.1 mm for the last 15 min was taken as the criterion of the stabilization.

The dependences "lateral load-displacement" (fig.3) are of clear nonlinear character. The linear part of the displacement is observed only with the 2-3 mm displacements at the level of soil surface. The stabilization in this case takes place in a significantly large range of displacements (to 24-28 mm), i.e. much more than it is Code admitted. At the same time, because of the large cross-sectional dimensions of the foundation and reinforcing, the bending of the foundation is practically impossible, i.e. the foundation behaves like a "rigid body", rotating in soil without bending. It follows that the nonlinearity of the diagram "load-displacement" is due to non –linear behavior of the soil base.

A DESIGN SCHEME WORKING OUT

Based on the analysis of the experimental data results, the design scheme can be worked out by two principles:

1) without considering the nonlinearity; then the design parameters of the soil should be chosen according to a condition of convergence of the design results with the experiments in the area of the admissible standard displacement (for instance, at 10 mm).
2) considering the nonlinearity of the soil base behavior using the iteration method when recalculation of the coefficient of sub grade reaction; however, this needs the special experiments with the measurement of the normal soil resistance along the lateral surface

As the first step, the design scheme is taken without considering the nonlinearity of the soil base.

The experimental results obtained show that to develop a design scheme of the FTH, some limit value of the base deformation should be taken as the criterion of the limit state, i.e. a problem of the evaluation of the FTH horizontal displacement should be solved. The strength of the FTH is ensured with its corresponding reinforcing.

By data of the lateral displacements at the level of the soil surface and by the angle of the test foundation rotation, the depth of location of the so-called point of zero displacements (p.z.d.) was evaluated (see table 1). Without the tamped in ballast p.z.d. is located on the average at the depth of 1.4 m, with the tamped in ballast in the volume of 0.5 m^3 – 1.4 m; 1.0 m^3 – 1.7 m; 1.5 m^3 – 2.0 m the depth of p.z.d. is 2.0 m, i.e. with the increase of the volume of the tamped in ballast p.z.d. approaches the foundation base. It is obvious, that the tamped in ballast creates a zone of increased strength in the area of FTH base, this prevents from the horizontal displacement of the FTH base when its rotation due to the lateral load.

To obtain data of the deformational zone at the level of the soil surface when FTH displacement due to the lateral load, the surface marks in kind of 8 mm diameter, 300 mm length reinforcing rods have been placed in front of one FTH (No.3). The marks displacements were measured with a measuring reel using reference points driven at about 4 m distance from the foundation. The diagrams of the lateral displacements of the marks show a zone of the soil base deformation to be observed not only in front of the foundation, but outside the foundation at the distance from 0.3 to 0.6 m as well (fig.4). Such type of deformations doesn't quite meet the Fuss-Vinkler hypothesis but to a greater degree it corresponds to the laws of the theory of an elastic halfspace which should be considered when choosing the model of the soil base and a design scheme for developing the design method.

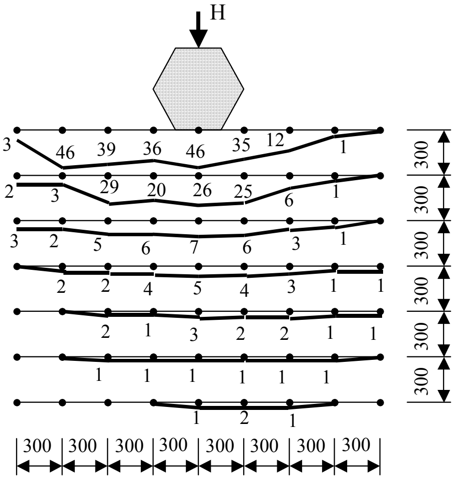

Fig. 4. A field of soil displacements (in mm) in front of FTH No.3 due to the lateral load H = 175 kN

A DESIGN METHOD

Considering the structural features as well as the experimental results for the developing of the design method, the following preconditions have been taken.

1. The base by its length is heterogeneous, multilayer and is divided into n-layers with the constant coefficient of subgrade reaction C_{zi} within each layer.
2. The dimension of the cross-sectional side of the FTH body d_z changes by the following linear dependence with depth.

$$d_z=d_o (1-\xi z) \ , \qquad (1)$$

where $\xi=(d_o-d_н)/ld_o$; (2)

d_0, d – the dimension of the section side according to the top and bottom of the foundation;

l – the length of the FTH in the soil;

z – the distance from the soil surface to a FTH section being considered.

3. Due to the lateral load and the bending moment, the foundation rotates in the soil as a rigid rod without bending, so the change of the FTH lateral displacement u_z by the depth is taken like for the rigid rod as follows:

$$u_z=u_o(1-z / l) \ , \qquad (3)$$

where u_0 –the lateral displacement of FTH at the level of the soil surface;

z, l – the same as in formula (1).

4. The soil pressure q_z per unit of length is proportional to its lateral displacement

$$q_z = d_z C_z u_z \ , \qquad (4)$$

where C_z – coefficient of subgrade reaction at the depth z;

d_z, u_z - are evaluated according to formulas (1) and (3).

5. The enlargement of the tamped in ballast prevents the lateral displacement of the FTH bottom end. That's why, the displacement of the FTH base due to the lateral load is taken equal to 0 in the design scheme.

The coefficient of subgrade reaction C_z is defined by the condition of settlement equality evaluated by the theory of local deformations and an elastic halfspace as follows:

$$C_z = \frac{E_o A_r}{\omega(1-\nu^2)d_{av}} \ , \qquad (5)$$

where w – settlement coefficient depending upon the shape of the loading area and a foundation rigidity;

E_o – modulus of the general soil deformation;

A_r – empiric coefficient defined by results of the experimental data processing when the trial foundations testing;

ν – Poisson's ratio equal to 0.42 for clay, 0.35 for loam and 0.30 for sand and sandy loam;

d_{av} – average dimension of the side of the foundation section.

When FTH lateral load design, two cases are considered (fig.5):

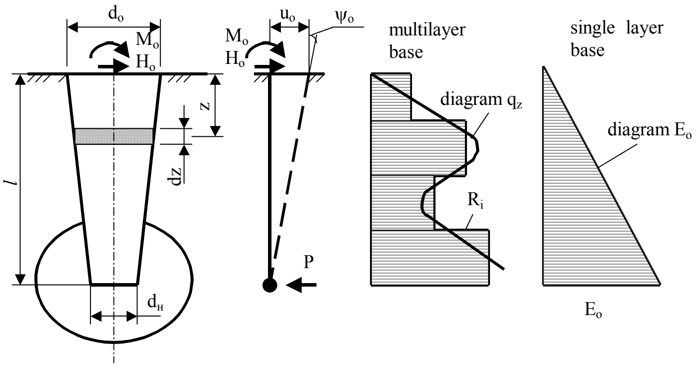

Fig. 5. Scheme of FTH lateral load design

a) C_z is calculated by CPT data; in this case the base is multilayer with the coefficient of subgrade reaction constant within separate layers;
b) C_z is calculated using the modulus of deformation; in this case the base is considered both as multilayer and as single-layer with the coefficient of subgrade reaction linearly increasing with depth.

In the first case the coefficient of subgrade reaction C_z is defined by formula

$$C_{zi} = \frac{18R_{iav}}{(1-\nu_i^2)d_{iav}} \ , \qquad (6)$$

where

$$R_{iav} = \frac{\sum_{j=1}^{m} q_{ij}}{1,25m} \quad , \tag{7}$$

m – a number of CPT points;

q_i – resistance of i-th layer under the probe tip by CPT data by the unit C-832;

d_{iav} – average dimension of the side of the foundation section in the i-th soil layer.

In the second case C_z is defined by formula

$$C_Z = \frac{2,7E_o}{(1-\nu^2)d_{av}} \quad , \tag{8}$$

By the condition of equilibrium of existing and reactive forces, formulas for the evaluation of displacements and forces in a foundation within accepted boundary conditions have been obtained

$$z = l \ , \quad u_l = 0 \ , \quad M_l = 0 \ , \quad Q_l = P \quad , \tag{9}$$

where P – reactive force at the level of a foundation base.

The displacement at the level of the soil surface u_0 and the angle of rotation ψ_0 are determined by formulas

$$u_o = M_o\delta_2 + H_o\delta_3 + \eta_1 \ , \ \psi_o = H_o\delta_3 - M_o\delta_1 + \eta_2 \ , \tag{10}$$

where

$$\delta_1 = I_1 / \zeta \ , \quad \delta_2 = I_2 / \zeta \ , \quad \delta_3 = I_3 / \zeta \ , \tag{11}$$

$$\eta_1 = (PI_5) / \zeta \ , \qquad \eta_2 = -(PI_4) / \zeta \ , \tag{12}$$

$$\zeta = d_o (I^2_2 - I_1I_3) / 12 \ . \tag{13}$$

The coefficients $I_1 – I_5$ are evaluated by formulas

$$I_1=12A - 6B\xi \ ; \ I_2=4C\xi - 6B; \ I_3=4C - 3D\xi \ ;$$

$$I_4=I_1l + I_2; \ I_5=I_2l + I_3 \tag{14}$$

$$A = \sum_{i=1}^{n} C_{zi} (z_i - z_{i-1}) \ ; \ B = \sum_{i=1}^{n} C_{zi} (z^2_i - z^2_{i-1}) \ ; \tag{15}$$

$$C = \sum_{i=1}^{n} C_{zi} (z^3_i - z^3_{i-1}) \ ; \ D = \sum_{i=1}^{n} C_{zi} (z^4_i - z^4_{i-1}) \ .$$

where z_i and z_{i-1} – distances from the design soil surface to the base and the roof of the i-th soil layer.

The value P is estimated by the condition

$$u_l = u_o - \psi_o \, l = 0 \quad , \tag{16}$$

where u_l – the lateral displacement of the foundation at the base level.

Substituting (10) into (16), taking into account the boundary conditions (3) and solving against P we obtain

$$P = (H_oI_5 - M_oI_4) / (I_5 + I_4 \, l) \quad . \tag{17}$$

For FTH without the tamped in ballast P=0.

The design values of the lateral force Q_z and the bending moment M_z at the depth z in the FTH section are evaluated by formulas

$$Q_z = H_o - (d_o / 12) (u_oI_{1z} + y_oI_{2z}) \ ; \tag{18}$$

$$M_z = M_o + H_oz - (d_o/12) \, u_o(I_{1z}z + I_{2z}) – (d_o/12)\psi_o (I_{2z} \, z + I_{3z}).$$

The values I_{1z}, I_{2z}, I_{3z} are evaluated by formulas (14) and (15), but summation in formulas (15) is done to a layer with a section Z.

In case of use of the modulus of deformation as the design parameter, the base is taken single-layer and the coefficient of subgrade reaction changes with depth by a linear law as:

$$C_Z = C_l \frac{z}{l} , \tag{19}$$

where C_l – the coefficient of subgrade reaction of soil at the level of a foundation base evaluated by formula (8).

Then the calculation is done by the above method, but in this case the values of I_{1z}, I_{2z}, I_{3z} are evaluated by formulas:

$$I_1 = C_l (6 \, l - 4 \, \xi \, l^2) \ ;$$

$$I_2 = C_l (3 \, \xi \, l^3 - 4 \, l^2) \ ; \tag{20}$$

$$I_3 = C_l (15 \, l^3 - 12 \, \xi \, l^4) / 5 \ .$$

The results of calculations show the satisfactory convergence with the test data (discrepancy does not exceed 20 %). This allows recommendation of this method for practical design use and its inclusion in the Code.

By the above algorithm a design computer program "HOLE" has been worked out, however, when it is necessary the "manual" calculation is possible.

CONCLUSIONS

1. The experimental data obtained, characterizing the stressed-deformed state of the laterally loaded FTH, shows that already with small horizontal displacements of the foundation (2-3 mm), the dependence "load-displacement" has a linear character. Here an enlarged base has a significant influence on FTH lateral load resistance.
2. Based on the analysis of the stressed-deformed state of the system "foundation-base", some standard value of the horizontal foundation displacement can be taken as the criterion of limit state considering assurance of FTH bending strength, and design parameters of the soil base can be chosen meeting this criterion.
3. The method of design of the laterally loaded FTH is suggested, that allows the evaluation of displacements and forces in a foundation. The coefficient of subgrade reaction can be estimated with use of the modulus of the general soil deformation or by CPT data.