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A Comparison between Russian and Western Approach for Deep Foundation Design

Saeed Delara, Kendra MacKay

***Abstract*—**Varying methodologies are considered for pile design for both Russian and Western approaches. Although both approaches rely on toe and side frictional resistances, different calculation methods are proposed to estimate pile capacity. The Western approach relies on compactness (internal friction angle) of soil for cohesionless soils and undrained shear strength for cohesive soils. The Russian approach relies on grain size for cohesionless soils and liquidity index for cohesive soils. Though most recommended methods in the Western approaches are relatively simple methods to predict pile settlement, the Russian approach provides a detailed method to estimate single pile and pile group settlement. Details to calculate pile axial capacity and settlement using the Russian and Western approaches are discussed and compared against field test results.

***Keywords*—**Pile capacity, pile settlement, Russian approach, western approach.

I. Introduction

SINCE more international companies have started working in Russia and former Soviet Union countries, a better understanding of geotechnical site investigation procedures and shallow and deep foundation design is becoming more necessary. Soil classification, field and laboratory testing procedures, and geotechnical design methods followed in Russia are slightly different from those followed in North America and Europe [14]. For projects in Russia, it is highly important to understand the key differences between Russian and Western design and properly follow the Russian approach for the entire design.

In this paper the Russian approach is discussed regarding pile geotechnical design procedures for axial compression and tensile capacity, as well as settlement of piles. A comparison of the Russian approach with the Western approach is presented to understand key differences. Field test results are also examined and compared to both the Russian and Western approaches. Details about pile lateral analysis are not discussed in this paper due to the complex procedure provided in the Russian approach. A separate paper is under preparation to compare pile lateral capacity procedures in the Russian and Western approaches.

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II. Pile Design

A. Axial Capacity

In general, pile axial capacity consists of two resistances: i) toe and ii) shaft resistances. However, the contribution of each resistance to the total pile capacity depends on pile characteristics and soil shear resistance and density.

Though both the Russian and Western approaches are in agreement to separate pile capacity into toe and shaft resistances, different methods are proposed to estimate these resistances.

1) Western Approach for Axial Capacity

Many manuals, practices, and specifications are used in North America for pile capacity and settlement calculations. In Europe, the Euro code provides general guidance to calculate pile capacity, and a national annex (for example BS8004:2015) is considered for further details. In Canada, the Canadian Foundation Engineering Manual (CFEM) [7] is considered for all geotechnical foundation design. In the United States, other manuals, practices, and specifications, including but not limited to reference materials provided by the Federal Highway Administration (FHWA) of U.S. Department of Transportation [8], [9], the American Association of State Highway and Transportation Officials (AASHTO) Specification [1], and the American Petroleum Institute (API) [2] are considered for pile design. These manuals and practices are also considered in many other countries to calculate pile capacity.

Some of the references followed in the Western countries such as CFEM provide unique procedures to estimate toe and shaft resistance of piles. These proposed methods vary, so it is the geotechnical engineer's responsibility to choose the methodology based on sound engineering principles for each design.

In general, for cohesive soils, shaft and toe resistances are a function of the undrained shear strength of the surrounding or base soil, while for cohesionless soils, soil friction angle is considered to estimate pile toe and shaft resistance. Equations for shaft friction, q_s, and pile toe bearing capacity, q_t, are presented below for cohesive and cohesionless soils.

$$q_s = \alpha . s_u \text{ for cohesive soils} \quad (1)$$

$$q_s = \beta . \sigma'_v \text{ for cohesionless soils} \quad (2)$$

$$q_t = N_t . s_u \text{ for cohesive soils} \quad (3)$$

$$q_t = N_t . \sigma'_t \text{ for cohesionless soils} \quad (4)$$

where: α is the adhesion coefficient for cohesive soils; β is the shaft resistance factor for cohesionless soils; N_t is the bearing capacity factor; s_u is the undrained shear strength of cohesive soils; σ'_v is the vertical effective stress adjacent to the pile; σ'_t is the vertical effective stress at the pile toe; q_s is the unit shaft friction; and q_t is the bearing capacity of the pile toe.

Though different methods are proposed in the Western manuals to estimate the adhesion coefficient (α), shaft resistance factor (β) and bearing capacity factor (N_t), relatively similar values are recommended for these factors.

2) Russian Approach for Axial Capacity

The Russian approach for pile design provided by the Ministry of Regional Development of Russia is SP 24.13330 [16] Pile Foundation, which is a revised edition of SNiP 2.02.03-85.

According to SP 24.13330, pile axial capacity consists of two resistances:

- Toe resistance (R); and
- Side frictional resistance (f_i).

The values of toe and side frictional resistances are defined based on the depth of the pile toe/shaft and grain size for sandy units, or liquidity index (I_L) for clayey units. The value of side frictional resistance varies along the pile length. Values of toe and frictional side resistances are provided in Tables I and II, respectively. As presented in these tables, both toe and side resistances are related to soil grain size in cohesionless soils and liquidity index for cohesive soil.

TABLE I
TOE RESISTANCE (KPA) BASED ON THE RUSSIAN APPROACH

Pile driving depth, m	Sandy units						
	gravel	coarse	—	medium	fine	silty/dust	—
	Clayey units with I_L equal to:						
	0	0.1	0.2	0.3	0.4	0.5	0.6
3	7,500	6,600 4,000	3,000	3,100 2,000	2,000 1,200	1,100	600
4	8,300	6,800 5,100	3800	3,200 2,500	2,100 1,600	1,250	700
5	8,800	7,000 6,200	4000	3,400 2,800	2,200 2,000	1,300	800
7	9,700	7,300 6,900	4300	3,700 3,300	2,400 2,200	1,400	850
10	10,500	7,700 7,300	5000	4,000 3,500	2,600 2,400	1,500	900
15	11,700	8,200 7,500	5600	4,400 4,000	2,900	1,650	1,000
20	12,600	8,500	6200	4,800 4,500	3,200	1,800	1,100
25	13,400	9,000	6800	5,200	3,500	1,950	1,200
30	14,200	9,500	7400	5,600	3,800	2,100	1,300
>35	15,000	10,000	8000	6,000	4,100	2,250	1,400

Notes: a. More details are provided in SP 24.13330, b. Where two values are provided, the top value is for sandy units and the bottom value is for clayey units.

Compared with the Western references, the Russian approach recommends higher toe resistance for clayey soils. Most of the Western references [1], [2], [7] recommend $9\times s_u$ for toe resistance of piles placed on clay units. The values recommended by the Russian approach for clayey soils (Table I) are significantly higher than $9\times s_u$, though the liquidity index was considered for clay consistency. For example, based on the Western approach, the toe resistance for soft clay at 15 m below ground surface with undrained shear strength of 60 kPa and liquidity index of 0.37 is 540 kPa. However, according to the Russian approach, a toe resistance of 3,200 kPa (interpolation between liquidity index of 0.3 and 0.4 at 15 m below ground surface) may be considered.

The bearing capacity of a single pile can be calculated using (5).

$$F_u = \gamma_c.\left(\gamma_{cR}.R.A_t + \mathrm{u}.\Sigma\gamma_{cf}.f_i.h_i\right) \tag{5}$$

where: γ_c is the soil service factor; γ_{cR} is a ground condition factor; γ_{cf} is a ground condition factor; u is the pile perimeter; A_t is the pile cross section area at the toe; R is the toe resistance; f_i is the frictional side resistance; and h_i is the thickness of the ith-layer along the pile length.

TABLE II
SIDE FRICTIONAL RESISTANCE (KPA) BASED ON THE RUSSIAN APPROACH

Average depth of soil layer, m	Sandy units								
	coarse and medium	fine	silty/dust	—	—	—	—	—	—
	Clayey units with I_L equal to:								
	≤0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
1	35	23	15	12	8	4	4	3	2
2	42	30	21	17	12	7	5	4	4
3	48	35	25	20	14	8	7	6	5
4	53	38	27	22	16	9	8	7	5
5	56	40	29	24	17	10	8	7	6
6	58	42	31	25	18	10	8	7	6
8	62	44	33	26	19	10	8	7	6
10	65	46	34	27	19	10	8	7	6
15	72	51	38	28	20	11	8	7	6
20	79	56	41	30	20	12	8	7	6
25	86	61	44	32	20	12	8	7	6
30	93	66	47	34	21	12	9	8	7
≥35	100	70	50	36	22	13	9	8	7

The soil service factor can be assumed to be equal to 1.0 for driven piles. For drilled piles, this factor varies between 0.8 and 1.0. The ground condition factors for driven piles depend on the pile driving method. For drilled piles, the ground condition factors depend on soil characteristics and pile type. The range of values for γ_{cR} and γ_{cf} are provided in Table III. More details to properly select these factors are provided in SP24.13330.

TABLE III
GROUND CONDITION FACTORS

Soils surrounding the piles	Ground Condition Factor, γ_{cf}	Ground Condition Factor, γ_{cR}
Driven Piles	0.5 - 1.0	0.7 - 1.2
Drilled Piles	0.5 - 1.0	0.9 - 1.3

It should be noted that the bearing capacity calculation of piles using the Russian approach (5) has reduction factors and cannot directly be compared with the ultimate bearing capacity calculated using the Western approach.

B. Reduction Factor

North American approaches consider a factored resistance methodology, such as load and resistance factor design (LRFD) to calculate allowable (design) pile capacity. In this approach, ultimate resistance is calculated using unfactored strength parameters and then multiplied by specified geotechnical resistance factors to obtain the factored geotechnical resistance at Ultimate Limit States (ULS) for design purposes. The recommended geotechnical resistance factors in North American manuals fall within a narrow range and vary based on soil conditions and resistance determination methods from 0.25 to 0.6 for axial compression capacity and 0.2 to 0.6 for uplift resistance on a single pile.

A relatively different approach is recommended in the Euro Code. According to the Euro Code [5], partial factors should be applied directly to the geotechnical strength parameters. The factored strength properties are used for the direct calculation of factored geotechnical resistance at ULS for design.

The reduction factors proposed in the Russian approach are smaller since the bearing capacity is already factored, as presented in (5). The Russian approach proposes the following equation to calculate allowable bearing capacity of a single pile:

$$F_a = \frac{F_u}{\gamma_n \cdot \gamma_{c,g}} \tag{6}$$

where: F_a is the allowable pile capacity; F_u is the pile capacity calculated using (5); γ_n is the structure reliability factor; and $\gamma_{c,g}$ is the design parameters reduction factor.

The structural reliability factor varies between 1.10 and 1.20 based on the structure level of responsibility. Details about structure level of responsibilities are provided in SP 22.13330 [15]. The design parameters reduction factor considers uncertainties associated with the design parameters calculation and varies between 1.20 and 1.50. More details to properly select $\gamma_{c,g}$ are provided in SP24.13330.

III. Pile Settlement

A. Single Pile Settlement

The proposed procedures in the Western approach to predict pile settlement are relatively simple. These procedures are based on empirical and theoretical methods to estimate required pile movement to mobilize axial shear or full end bearing resistances. In general, the required displacement to mobilize maximum soil pile adhesion or unit skin friction varies from 0.25 to 2.0 percent of the pile diameter (typical value of one per cent). While the maximum soil pile adhesion remains constant for sandy soils, it drops to 70 to 90 percent of the maximum value for clay soils. A relatively higher displacement, up to 10 percent of the pile diameter, may be required for full mobilization of the end bearing resistance in both sandy and clay soils [2].

More complicated approaches such as Fleming's method [10] are also proposed to estimate pile settlement. Fleming's method considers a hyperbolic function to predict single pile behaviour under maintained loading. The Fleming method is one of the proposed methods in British Standard BS 8004:2015 [3]-[6] to estimate pile settlement.

The Russian approach proposes the following series of equations to estimate settlement of a single pile with no toe enlargement:

$$s = \beta \frac{N}{G_1 . l} \tag{7}$$

$$\beta = \frac{\beta'}{\lambda_1} + 0.5 \frac{1-\beta'/\alpha'}{\chi} \tag{8}$$

$$\beta' = 0.17 \ln\left(\frac{k_v . G_1 . l}{G_2 . d}\right) \tag{9}$$

$$\lambda_1 = \frac{2.12\chi^{\frac{3}{4}}}{1+2.12\chi^{\frac{3}{4}}} \tag{10}$$

$$\chi = \frac{E_p . A_p}{G_1 . l^2} \tag{11}$$

$$k_v = 2.82 - 3.78v + 2.18v^2 \tag{12}$$

$$k_{v1} = 2.82 - 3.78v_1 + 2.18v_1^2 \tag{13}$$

$$v = \frac{v_1 + v_2}{2} \tag{14}$$

$$G = \frac{E}{2(1+v)} \tag{15}$$

where: N is the vertical load; β is the coefficient related stiffness; β' is the coefficient related to pile absolute stiffness; α' is the coefficient related to soil stiffness; χ is the pile relative stiffness; l is the pile length; E_p is the pile elastic modulus; A_p is the pile cross section; λ_1 is a coefficient related to settlement rate due to shaft compression; E is the soil elastic modulus; G is the soil shear modulus; G_1 is the soil shear modulus along the pile length; G_2 is the soil shear modulus below the pile toe through 1.5 times of pile length; ν_1 is the soil Poisson's ratio along the pile length; ν_2 is the soil Poisson's ratio below the pile toe through 1.5 times of pile length; k_v is the soil coefficient along the pile length; and k_{v1} is the average soil coefficient.

The settlement of a single pile with foot enlargement can be estimated from the following equation:

$$s = \frac{0.22N}{G_2 . d_b} \frac{Nl}{E.A} \tag{16}$$

where d_b is the diameter of pile enlargement.

B. Pile Group Settlement

Based on recommendations provided in the Western manuals, where a pile group is supported in and underlain by cohesionless soils, immediate settlement that occurs immediately as the pile group is loaded can be considered. Various empirical equations and graphs are proposed to

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estimate pile group settlement based on field tests results such as Standard Penetration Tests (SPT) or Cone Penetration Tests (CPT) [1], [2], [4], [7]-[9].

For cohesive soil, both immediate settlement and long term settlement should be considered. Long term settlement usually occurs over a relatively long period of time due to consolidation of normally consolidated clay. In most cases, consolidation settlement is the major foundation settlement.

The Russian approach for estimating pile group settlement is different from the Western approach. The following equations are provided in the Russian standard to estimate pile group settlement when the pile group consists of less than 25 piles.

$$s_{ad} = \delta \frac{N}{G_1.l} \tag{17}$$

$$\delta = \begin{bmatrix} 0.17\ln\left(\frac{k_v.G_1.l}{2G_2.a}\right) & if & \frac{k_v.G_1.l}{2G_2.a} > 1 \\ 0 & if & \frac{k_v.G_1.l}{2G_2.a} \le 1 \end{bmatrix} \tag{18}$$

where: s_{ad} is the additional pile settlement at distance of a; δ is the coefficient related to stiffness; and a is the centre-to-centre distance of piles in pile group.

To calculate an individual pile settlement within a pile group, the following equation can be considered.

$$s_i = s(N_i) + \sum_{j \neq i} \delta_{ij} \frac{N_j}{G_1.l} \tag{19}$$

Note that i and j are indices to describe the pile number within the group. For example, for a pile group of three piles (3×1), the settlement of the first pile can be calculated as:

$$s_1 = \beta \frac{N_1}{G_1.l} + \delta_{12} \frac{N_2}{G_1.l} + \delta_{13} \frac{N_3}{G_1.l} \tag{20}$$

It should be noted that the value of a changes to $2a$ when calculating δ_{13}.

For large pile groups (n>25 piles), the settlement consists of three terms as presented in the equations below:

$$s = s_{ef} + \Delta s_p + \Delta s_c \tag{21}$$

$$\Delta s_p = \frac{\Delta s_{p1}}{\frac{\Delta s_{p1}}{\Delta s_{p0}}\left(1 - \frac{E_1}{E_2}\right) + \frac{E_1}{E_2}} \tag{22}$$

$$\Delta s_{p0} = \frac{(1 - v_2^2)(1 - k)P}{d.E_2} \tag{23}$$

$$\Delta s_{p1} = \frac{\pi(1 - v_2^2)P}{4.E_2}(a - 1.5d) \tag{24}$$

$$\Delta s_c = \frac{P(l - a)}{E_p.A_p} \tag{25}$$

$$k = \sqrt{\frac{A}{\Omega}} \tag{26}$$

$$P = p.\Omega \tag{27}$$

where: p is the load on a single pile; Ω is the effective area of a single pile within a pile group; a is the centre-to-centre distance of piles in the pile group; n is the number of piles in a row within the pile group; s_{ef} is the virtual foundation settlement with a width of n×a at the bottom of the pile group (see Fig. 1), note that the settlement calculation for shallow foundations should be considered for this parameter; Δ_{sp} is the pushing settlement for a soil column around an individual pile; Δ_{sc} is the pile elastic deformation; k is an area coefficient; E_1 is the soil elastic modulus along the pile length; and E_2 is the soil elastic modulus below the pile toe through 1.5 times the pile length.

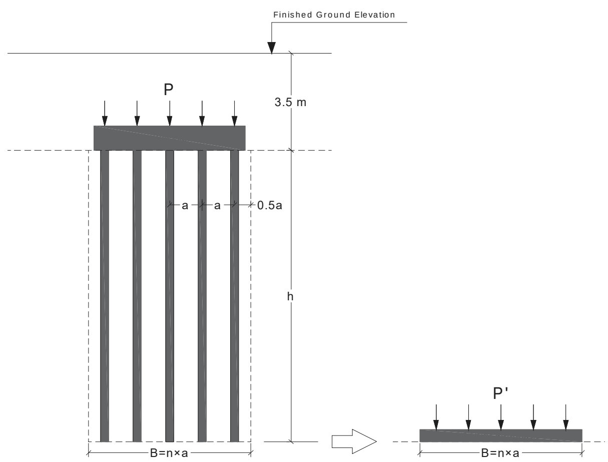

Fig. 1 Virtual foundation below the pile group for settlement calculation

IV. Frost Action on Piles

The procedure to calculate the depth of frost penetration is different in Russia [17] and North America [7], [18]. The objective of this paper is not to outline the calculation procedure for the depth of frost penetration however, as frost action on piles has been discussed previously by other researchers [13]. In general, the Russian procedure is more complex and considers more parameters, and the Western approaches provide slightly more conservative frost action values.

V. Seismic Design

Compared to the Western approaches, a relatively simple pseudostatic approach is provided for the Russian approach in SP24.13330 [16] to assess seismic effect on piles. Two seismic coefficients (γ_{eq1} and γ_{eq2}) are considered for calculation of toe (R) and side frictional (f_i) resistances provided in (5). The seismic coefficients are defined based on the project seismic zone and water content for granular soils and liquidity index for clay soils.

In addition to the seismic coefficient, for areas with seismic activity, the value of the frictional side resistance and soil lateral support should not be considered for a length of pile (h_d) below the ground surface. The value of h_d can be calculated using (28) with a maximum limit of $3/\alpha_c$.

$$h_d = \frac{a_1.(H+\alpha_\varepsilon.a_3.M)}{b_p.\left(\frac{a_2}{\alpha_\varepsilon}.\gamma.tan\varphi+c\right)} \quad (28)$$

$$\alpha_\varepsilon = \sqrt[5]{\frac{K.b_p}{E.I}} \quad (29)$$

$$b_p = \begin{vmatrix} d+1.0 & \text{for d} > 0.8\text{m} \\ 1.5d+0.5 & \text{for d} > 0.8\text{m} \end{vmatrix} \quad (30)$$

$$\mathrm{K} = \frac{K_I.l_I(2l_k-l_I)+K_{II}(l_k-l_I)^2}{{l_k}^2} \quad (31)$$

$$l_k = 3.5\mathrm{d} + \Delta \quad (32)$$

where: H is the pile length; b_p is the relative pile width; d is the pile diameter; M is the pile moment; E is the pile elastic modulus; I is the pile moment inertia; K is the soil coefficient as per Table V; K_I is the soil coefficient for the top layer; K_{II} is the soil coefficient for the bottom layer; Δ is a constant value equal to 1.5 m; $_k$ is the depth of interest; l_I is the depth of the top layer; $\gamma$is the soil unit weight at pile head; φ is the soil friction angle at pile head; c is the soil cohesion at pile head; α_ε is the deformation coefficient; and a_1, a_2, a_3 are dimensionless coefficients as per Table VI.

The depth, h_d depends on ground properties, pile dimension, and lateral load at the pile head. To calculate h_d, it is also recommended to decrease the internal friction angle of soil according to the seismic activity class as presented in Table VII.

TABLE IV
SERVICE FACTOR FOR SEISMIC CONDITION

Seismic Zone	γ_{eq1}						γ_{eq2}				
	dense Sands		medium dense Sands		Clay		medium dense to dense Sand		Clay		
	damp to wet	sat	damp to wet	sat	$I_L < 0$	$0 \le I_L \le 0.5$	damp to wet	sat	$I_L < 0$	$0 \le I_L \le 0.75$	$0.75 < I_L \le 1$
7	1	0.9	0.95	0.8	1.0	0.95	0.95	0.9	0.95	0.85	0.75
	0.9	0.5	0.85	0.4	1.0	0.9	0.85	0.5	0.9	0.8	0.75
8	0.9	0.8	0.85	0.7	0.95	0.9	0.85	0.8	0.9	0.8	0.7
	0.8	0.4	0.75	0.35	0.95	0.8	0.75	0.4	0.8	0.7	0.65
9	0.8	0.7	0.75	-	0.9	0.85	0.75	0.7	0.85	0.7	0.6
	0.7	0.35	0.6		0.85	0.7	0.65	0.35	0.65	0.6	-

Notes: a. More details are available in SP24.13330. b. Where two values are provided, the top value is for driven piles and the bottom value is for drilled piles. C. These values should be multiplied by 0.85, 1.0 or 1.15 for buildings and structures constructed in regions with frequency of occurrence, equal to 1, 2 and 3, respectively (except transport and hydraulic structures).

TABLE V
VALUES OF K

Soils surrounding the piles	Soil Coefficient K, kN/m^4
large Sand ($0.55 \le e \le 0.7$); hard Clay and Loam ($I_L \le 0$)	18,000 - 30000
Small Sand ($0.6 \le e \le 0.75$); medium Sand ($0.55 \le e \le 0.7$), sandy Loam ($I_L \le 0$); hard and semi-solid Clay and Loam ($0 \le I_L \le 0.75$)	12,000 - 18,000
Dusty/silty Sand ($0.6 \le e \le 0.8$); plastic sandy Loam ($0 \le I_L \le 0.75$); clay and soft-plastic Loam ($0.5 \le I_L \le 0{,}75$)	7,000 - 12,000
plastic loamy Clay ($0.75 \le I_L \le 1.0$)	4,000 - 7,000
gravelly Sand ($0{,}55 \le e \le 0{,}7$); coarse grain soils with sandy aggregate	50,000 - 100,000

TABLE VI
VALUES OF a_1, a_2 AND a_3

Foundation type	a_1	a_2	a_3
Free Head	1.5	0.8	0.6
Fixed Head	1.2	1.2	0

TABLE VII
PROPOSED REDUCTIONS OF SOIL FRICTION ANGLE

Seismic activity class	Reduction of soil friction angle (°)
7	2
8	4
9	7

VI. CASE STUDY

In this section, details of pile design using the Western and Russian approaches are discussed for a project in Russia. In addition, the predicted settlements using both approaches were compared with actual field static test results. For this project, concrete driven piles with cross sections of 300 mm×300 mm and 400 mm×400 mm and lengths varying between 8 m to 12 m were considered for all structures.

A number of static and dynamic field tests were conducted to validate predicted pile capacity. As per the Russian approach for static field testing (GOST 2686) [12], piles should be loaded to reach at least 40 mm settlement (20 mm for some certain conditions). If the lower end of the testing pile is placed on coarse grains, dense sand or hard clay, the load should be at least 1½ times (150%) the pile bearing capacity and not more than the structural capacity of the pile. While a static pile test is recommended to continue to pile failure, the Russian approach does not mandate it. As a result, no static pile test was conducted to failure.

Among the conducted pile tests, two tests were selected to compare the Western and Russian design approaches for pile design. One test was conducted on a pile penetrated in sand soils and the other test on a pile penetrated in mixed sand and clay units. No piles were driven fully within clay soils.

A. Ground Condition

In general, the project area consists of alluvial and alluvial-diluvial deposits. These deposits are represented mainly by sands of different grain sizes, lean clays and silty clays, and less often, clays and gravel in the form of low-thickness lenses. Based on the boreholes drilled close to the first test area, sand deposits with various grain sizes and compactness were encountered as shown in Fig. 2.

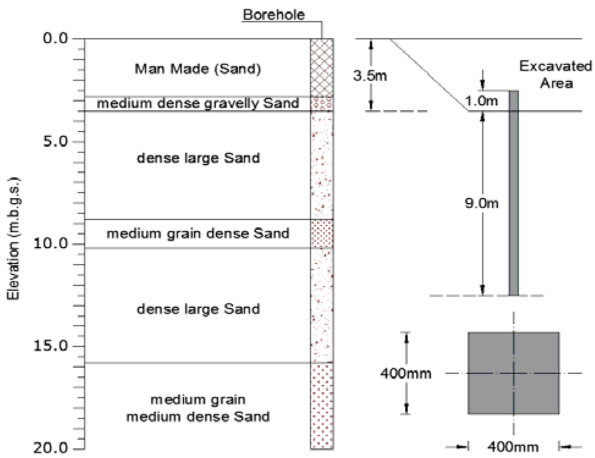

Fig. 2 Subsurface ground profile at the first pile test area

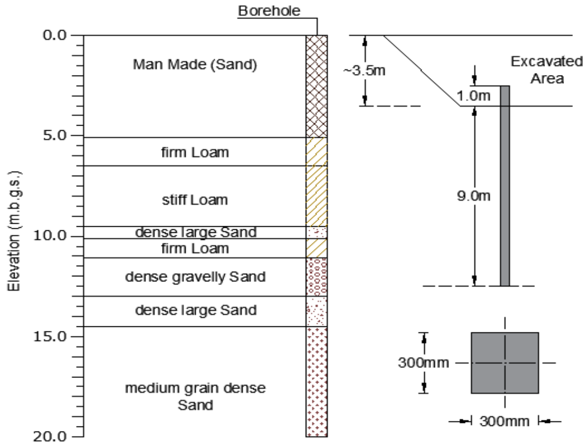

Fig. 3 Subsurface ground profile at the second pile test area

The second test was conducted in an area where relatively thick clay soils were encountered above sand deposits. The soil profile for the second test area is shown in Fig. 3.

The encountered sand deposits were classified based on a project soil classification system that is not discussed here. Soil characteristics were determined in the field and in the lab based on relevant Russian Standards (GOST). The geotechnical properties for sand deposits and clay units encountered at the test areas are provided in Table VIII.

TABLE VIII
GEOTECHNICAL PROPERTIES AT THE TEST AREAS

Soil Unit	Unit Weight, γ (kN/m³)	Internal Friction Angle, ϕ (°)	Cohesion, c (kPa)	Elastic Modulus, E (MPa)	Undrained Shear Strength, s_u (kPa)
Man Made (Sand)	17.9	31.0	0	20.0	-
stiff Loam	19.6	25.0	36	28.9	85
firm Loam	19.7	22.0	28	19.5	60
medium dense gravelly Sand	16.8	36.0	1	32.5	-
dense gravelly Sand	18.0	36.0	1	38.6	-
dense large Sand	18.0	37.0	4	39.1	-
medium grain dense Sand	18.0	37.0	2	38.7	-
medium grain medium dense Sand	17.2	35.0	1	29.6	-

Concrete piles with length of 10 m and cross section of 400 mm×400 mm (first test) and 300 mm×300 mm (second test) were penetrated 9 m below the bottom of the trench. As shown in Figs. 2 and 3, the top 3.5 m of the ground surface were removed to place a pile cap 3.5 m below the natural ground surface. An elastic modulus of 26 GPa was considered for the pile.

B. Field Static Test Results

Two static field tests were selected to compare Russian and Western approach for pile design. These two tests were conducted on 10 m 300 mm×300 mm and 400 mm×400 mm driven concrete piles. A 350 mm hole was drilled before driving the pile. The top 3.5 m of in situ soil was excavated to drive the pile. The top 1.0 m of pile was left out of the ground (9.0 m of penetration).

The first pile was loaded to 825 kN which is 1.5 times that of the structure load (550 kN) recommended by the Russian Standard (GOST 2686) [12]. The second pile was loaded to 597.4 kN, which is approximately 1.5 times that of the structure load (400 kN). As shown in Fig. 4, pile load-settlement graphs are relatively linear. Maximum settlements of 3.71 mm and 1.98 mm were recorded at the maximum test load for the first and the second piles, respectively.

C. Pile Axial Capacity

As mentioned in previous sections, various manuals and specifications are followed to calculate pile capacity in North America and Europe. Among them, the API practice [2] was considered to estimate geotechnical resistance of the tested piles. According to the API recommended values for toe and side friction resistances, the calculated design axial compression and tension capacities for each pile were calculated and are presented in Table IX.

The pile capacities were also calculated using the Russian approach. The calculated values using the Russian approach are also provided in Table IX. When compared to the API method, the axial compression capacity calculated by the Russian method is lower by about 20% for the first pile that was placed on "large" sand, while it is higher by about 20% for the second pile that was placed on "gravelly" sand. As presented in Table I, the toe resistance increases by about 80% if the pile toe is placed on "gravelly" sand compared to "large" grained sand. As a result, determination of grain size is highly important to estimate pile toe resistance. On the other side, the API and most other Western approaches rely of sand density and recommend the same toe resistance for these two units (large grain sands and gravelly sand).

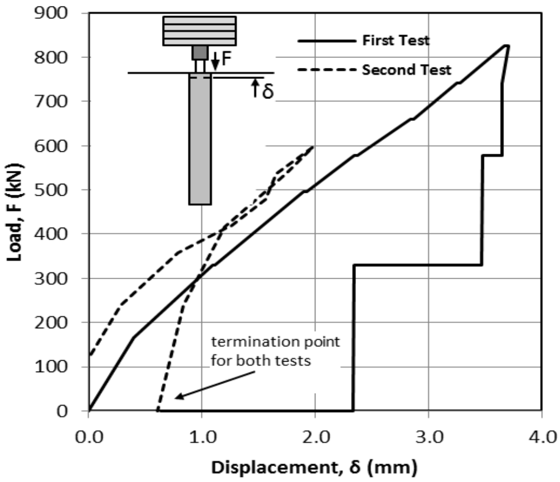

Fig. 4 Static pile test result

The uplift capacity calculated by the Russian approach is higher than the API method for both piles. The uplift capacity increased by 20% for the first pile (in sand soils) and by 40% for the second pile (in mixed sand and clay soils).

In general, where the pile toe is placed on gravelly sand (with grain size 1-2 mm) [11], the calculated pile capacity using the Russian approach is higher than the API method. For piles on other sand units, the Russian approach provides lower pile capacity.

When the pile toe is placed on a clay unit, the Russian approach always provides higher pile capacity when compared to all other Western methods.

TABLE IX
PILE CAPACITY FOR THE TESTED PILES BASED ON API AND RUSSIAN APPROACHES

Test	Method	Pile Width (mm)	Pile Length (m)	Compression (kN)	Uplift (kN)
Test I	API	400	10	961	291
	Russian			760	360
Test II	API	300	10	562	170
	Russian			664	238

D. Predicted Pile Settlement and Actual Field Data

Of the methods proposed by the Western approaches, the API method and Fleming's method [10] were considered to estimate pile settlement. Pile settlement was also calculated using the Russian approach explained in Section III. A comparison between Fleming, API and Russian methods with actual field data are provided in Fig. 5 for the first tested pile and in Fig. 6 for the second tested pile.

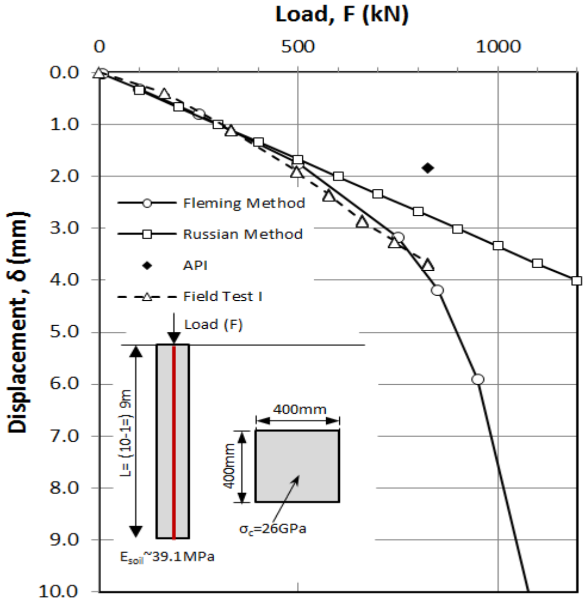

Fig. 5 Settlement comparison for the first tested pile

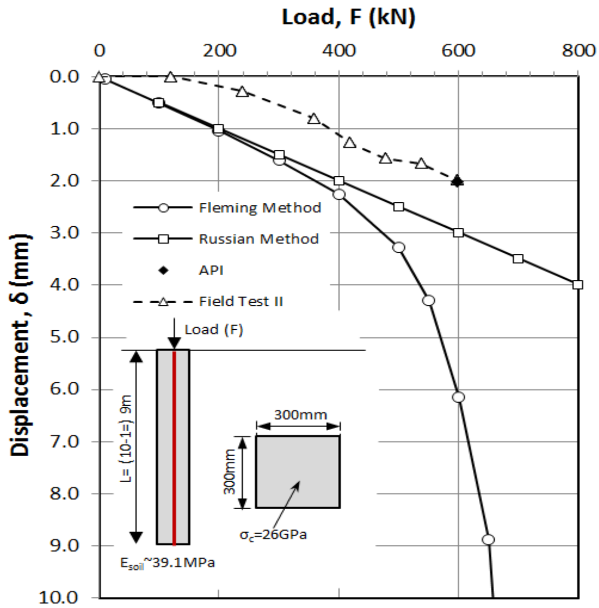

Fig. 6 Settlement comparison for the second tested pile

For the first pile, the Fleming method predicts pile settlement better than the Russian approach. Both the API and Russian approaches under predict the pile settlement. For the second pile, the Fleming and Russian methods over predict the pile settlement; however, the Russian approach provides a linear load-settlement graph that has better predictions. The API method provided the best estimate for this test. In general, the hyperbola model considered in the Fleming method provides high settlement when axial load reaches the ultimate capacity. The Fleming method may be considered for design since it predicts slightly more conservative settlement.

VII. Conclusion

Russian and Western approaches for pile axial capacity and settlement were compared to best understand the difference between the varying approaches. Both approaches rely on toe and shaft resistances to estimate pile axial compression and uplift resistance. However, the procedures to estimate these resistances are different. The Russian approach relies on the grain size of sand soils and liquidity index of clay soils, while the Western approaches rely on internal friction angle of sand soils and the undrained shear strength of clay soil. Compared with static field tests, the Russian approach provides a more conservative axial compression capacity if the pile toe is placed on fine to large sandy deposits, while the Western approach (API) provides more conservative uplift resistance. In the Russian approach, the recommended toe resistance for piles placed on fine to large sandy deposits is smaller than the Western approach. For piles placed on gravelly sand (more than 25 percent are larger than 2mm) or clay units, the Russian approach provides higher pile capacities.

Calculation of pile settlement was also compared between the Russian and Western approaches. Though most of the Western approaches provide relatively simple procedures to predict pile settlement, the Russian approach has more details. Among methods proposed in the Western manuals, the Fleming method provides a relatively more complete procedure to estimate pile settlement. Pile settlement was calculated by Russian, API and Fleming methods and their predicted settlements were compared with actual field results. The Russian and API methods may provide slight lower settlement values while the Fleming method provides more conservative results and is recommended for pile design.

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