Development of a new deformation forecast method

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Abstract. Deformations of foundations of buildings and structures are calculated by the second limit state, i.e. by limiting deformations. These calculations are relevant for specific soils, in particular loess soils. The paper suggests a method for determining the structural strength of the soil in compression tests and presents a method for predicting base foundation deformations when soaking and flooding built-up loess territories using approved equipment and software.

1. Introduction
When calculating uneven deformations of foundations composed of loess with subsidence soils having the structural strength, the following factors should be taken into account:
• for soils with the structural strength, the SP method [1] provides overestimated values of settlement and underestimated settlement unevenness;
• subsidence properties of soils manifest themselves when the required humidity is achieved (initial subsidence (critical moisture)). When predicting uneven deformations, subsidence soil properties should be taken into account at $S_i < 0.8$ [1–6].

2. Materials and methods
Methodology for prediction of foundation deformations:
1. Let us analyze adjacent foundations, under one of which the soil is in its natural state ($S_r < 0.5$), i.e. the soil does not exhibit subsiding properties. The second foundation is subject to 1) $0.5 \leq S_r < 0.8$ and 2) $S_r \geq 0.8$.

Relatively non-uniform deformation is determined in two cases: 1 - local soaking; 2 - local full soaking. When designing new objects, to predict uneven deformations, it is recommended to consider case 1 (local partial soaking), since full water saturation of the whole subsidence is unlikely which is confirmed by the data of field studies [2, 5, 7, 8].

2. Deformations of the foundation with a partially soaked base are defined as the sum of deformations caused by the settlement and subsidence

$$S = S_i + S_{sl}$$

where $S_i$ is determined by the SP method, the soil does not have $P_b$ (example 1). To determine $S_{sl}$, the SP method is used (example 2). The deformation of the completely soaked (water-saturated) foundation is determined according to the standard SP method (Example 3). Let us calculate $\sigma_s$ and $0.5\sigma_{sg}$ $\sigma_{zl} = \sigma_{zgi-1} + \gamma_{hi} \cdot h_i;$$

where $h$ – layer thickness; $\gamma$ – specific weight; $i$ – layer; $0$ – earth surface; $\sigma_{zg0} = 0; 0.5 \cdot \sigma_{zg0} = 0;\ T.1$ – at the level of the foundation base; $\sigma_{zg0} = \gamma_i \cdot h_1 = 29.25$ kPa; $0.5 \cdot \sigma_{zg0} = 14.63$ kPa; $T.2$ – at
the boundary of layers 1 and 2: \( \sigma_{zg1} = \sigma_{zg0} + y_1 \cdot h_1 = 29.25 + 19.5 \cdot 0.5 = 39 \text{ kPa} \); 0.5 \cdot \sigma_{zg1} = 19.5 \text{ kPa}; T.3 – at the boundary of layers 2 and 3; \( \sigma_{zg2} = \sigma_{zg1} + y_2 \cdot h_2 = 39 + 19.5 \cdot 1.5 = 68.3 \text{ kPa} \); 0.5 \cdot \sigma_{zg2} = 34.1 \text{ kPa}; T.4 – at the boundary of layers 3 and 4; \( \sigma_{zg3} = \sigma_{zg2} + y_3 \cdot h_3 = 68.3 + 18.2 \cdot 2 = 104.7 \text{ kPa} \); 0.5 \cdot \sigma_{zg3} = 52.4 \text{ kPa}; T.5 – at the boundary of layers 5 and 6; \( \sigma_{zg5} = \sigma_{zg4} + y_4 \cdot h_4 = 104.7 + 17.7 \cdot 2 = 101 \text{ kPa} \); 0.5 \cdot \sigma_{zg5} = 70.1 \text{ kPa}; T.6 – at the boundary of layers 6 and 7; \( \sigma_{zg7} = \sigma_{zg6} + y_5 \cdot h_5 = 140.1 + 17.4 \cdot 2 = 174.9 \text{ kPa} \); 0.5 \cdot \sigma_{zg7} = 87.5 \text{ kPa}; T.7 – at the boundary of layers 7 and 8; \( \sigma_{zg8} = \sigma_{zg7} + y_6 \cdot h_6 = 174.9 + 17.2 \cdot 2 = 209.3 \text{ kPa} \); 0.5 \cdot \sigma_{zg8} = 104.7 \text{ kPa}

1. Based on the obtained values of ordinates on a geological section, we constructed the plot of natural pressure \( \sigma_{zgi} \) and auxiliary plot 0.5\( \sigma_{zgi} \).

2. We determined the vertical pressure on the base along the foundation base: \( \sigma_{zp} = p = 150 \text{ kPa} \), where \( p \) – average pressure under the foundation base.

3. We divided the soil thickness under the base into elementary sublayers with a thickness \( \Delta_i=\frac{0.2-0.4}{b_f} \), where \( b_f \) – base width. Let us take \( \Delta_i=0.2=0.4 \text{ m} \).

4. Let us determine vertical normal stresses \( \sigma_{zp} \) by depth \( z \) from the foundation base: \( \sigma_{zp} = \sigma_i \cdot \sigma_{zp} \), where \( \sigma_i \) – stress dispersion coefficient for the corresponding soil layer depending on the shape of the base and the ratio \( \zeta = 2_i/b_f \) and \( \eta = i/h_i \), where \( z_i \) – depth of the i-th elementary layer from the base; \( z_i=\sum \Delta_i \) is determined by Table 4.5, \( \xi = 2 \cdot z_i/2 = z_i; \eta = 2/2 = 1 \).

**Table 1. Calculation of probable settlement**

| No | Name of the soil and its condition | Layer thickness \( h_i, \text{ m} \) | \( \Delta_i \) m | \( z_i \) m | \( \zeta_i \) m | \( \alpha_i \) m | \( \sigma_{zpi} \) kPa | \( \sigma_{zpi}^{cp} \) kPa | \( E_{zpi} \) kPa |
|----|----------------------------------|----------------|--------------|--------------|----------------|----------------|----------------|----------------|----------------|
| 0.4 | GEE-1 Refractory loam | 1.6 | 0.0 | 0.0 | 0.0 | 1.0 | 150 | 147 | 5000 |
| 0.4 | 0.4 | 0.4 | 0.96 | 144 | 132 |
| 0.4 | 0.8 | 0.8 | 0.8 | 120 | 105.5 |
| 0.4 | 1.2 | 1.2 | 0.606 | 90.9 | 79.2 |
| 0.4 | 1.6 | 1.6 | 0.449 | 67.4 | 58.9 |
| 0.4 | 2.0 | 2.0 | 0.336 | 50.4 | 44.5 |
| 0.4 | 2.4 | 2.4 | 0.257 | 38.6 | 34.4 |
| 0.4 | 2.8 | 2.8 | 0.201 | 30.2 | 27.1 |
| 0.4 | 3.2 | 3.2 | 0.16 | 24 | 21.9 |
| 0.4 | 3.6 | 3.6 | 0.131 | 19.7 | 18 |
| 0.4 | 4.0 | 4.0 | 0.108 | 16.2 | 15 |
| 0.4 | 4.4 | 4.4 | 0.091 | 13.7 | 12.7 |
| 0.4 | 4.8 | 4.8 | 0.077 | 11.6 | 10.9 |
| 0.4 | 5.2 | 5.2 | 0.067 | 10.1 | 9.7 |
| 0.4 | 5.6 | 5.6 | 0.058 | 8.7 | 8.2 |
| 0.4 | 6.0 | 6.0 | 0.051 | 7.7 | 7.7 |

5. According to the data obtained, we constructed a plot of the additional vertical stress \( \sigma_{zp} \) from the base.

6. Let us determine the height of the compressed base thickness \( H_c \), whose lower boundary BC is \( z=H_c \), where \( \sigma_{zp}=0.5 \cdot \sigma_{zg} \).

7. Let us determine the value of total settlement by formula

\[ \]
\[ S = \beta \sum_{i=1}^{n} \frac{(\sigma_{zpi} - \sigma_{zji}) \cdot h_i}{E}, \]
where \( \beta \) – dimensionless coefficient \( \beta = 0.8 \); \( \sigma_{zpi} \) – average value of vertical normal stress from the base in the \( i \)-th layer; \( \sigma_{zji} \) – thickness of the \( i \)-th layer; \( E \) – modulus of deformation of the \( i \)-th layer; \( n \) – number of layers.

8. Let us calculate settlements in a tabular form (Table 1)

\[ S_1 = 0.8/5000 \cdot (147 \cdot 0.4) = 0.94 \text{ cm} \]
\[ S_2 = 0.8/13500 \cdot 0.4(132 + 105.5 + 79.2 + 58.9) = 0.89 \text{ cm} \]
\[ S_3 = 0.8/26000 \cdot 0.4(44.5 + 34.4 + 27.1 + 21.9 + 18 + 15 + 12 + 10.9 + 9.7 + 8.2) = 0.24 \text{ cm} \]
\[ S_t = S_1 + S_2 + S_3 = 0.94 + 0.89 + 0.24 = 2.07 \text{ cm} \]

9. Let us compare settlement value \( S \) with the value of ultimate deformations of the base \( S_u \), depending on the structural system of the building according to [6, 8].

\[ S = 2.07 \text{ cm} < S_u = 12 \text{ cm}, \]
the condition is met.

**Figure 1.** Calculation of settlement of the foundation: \( BC \) – lower bound of compressible stratum; \( d \) – foundation depth; \( H_c \) – compressible thickness; \( \gamma, \sigma_{zg} \) and \( 0.5\sigma_{zg} \) – main and auxiliary diagrams of vertical stresses of the own weight of the soil; \( \sigma_{zp} \) – vertical stress plot from the bottom of the foundation

**Example 2.** It is required to calculate deformations of the base with dimensions 2.0\times2.0 \text{ m}, depth \( d = 1.5 \text{ m} \) with an average pressure along the sole \( pII = 150 \text{ kPa} \). The admissible settlement for this building is \( S_u = 12 \text{ cm} \). The construction site is composed of loess loams 8.0 \text{ m} in thickness underlain by non-subsiding soils. The main characteristics of soils are given in Table 1. The specific gravity of the soils is adopted for the soaked saturated water.

**3. Solution**

For a given base size, we determined the distribution of vertical stresses \( \sigma_{zg} \) and \( \sigma_{zp} \) over the depth of their base. The calculation results are summarized in Table 3. At the same time \( \sigma_{zg0} = \gamma \cdot d_f = 19.9 \cdot 1.5 = 29.85 \text{ kPa} \).

The thickness of the elementary soil layer is assigned from the condition \( h_i < 0.4 \cdot d_f = 0.4 \cdot 2 = 0.8 \text{ m}, \ h_i = 0.5 \text{ m} \).

Using Table 2, we determined the relative subsidence of the soil in the middle of each layer with an average actual stress \( \sigma_{zi,m} \). Results are presented in Table 2. From the comparison of stress values \( \sigma_{zi,m} \)
at the base with the value of initial subsidence \( p_a \), we established that total stresses over the entire thickness are greater than pressure \( p_a \).

Let us determine the amount of base subsidence by SP formula CR using the data from Table 1 and 3.

\[
S_{Sl} = \sum \varepsilon_{Sl} \cdot h \cdot K_{Sl,i} = 0.5 + 1.5(p - p_{Sl,i})/p_0 = 0.5 + 1.5(150 - 125)/100 = 0.88
\]

Values of coefficient \( K_{Sl,i} \) are calculated by SP formula for each layer. Results are presented in Table 3. The value of soil subsidence is

\[
S_{Sl} = 50 \cdot 0.88(0.068 + 0.052 + 0.047 + 0.033 + 0.028 + 0.024 + 0.021 + 0.024 + 0.015 + 0.014 + 0.017) = 50 \cdot 0.88 \cdot 0.343 = 15.11\, cm
\]

Let us calculate the settlement value for the foundation by the method of layer-by-layer summation according to CR 22.13330.2011

### Table 2. Soil characteristics for the construction site

| №  | Soil | Depth \( h \), m | Ground weight \( \gamma \), kN/m\(^3\) | Share of dry soil \( \gamma_{st} \), kN/m\(^3\) | \( \varepsilon_{Sl} \) at P, kPa | \( P_{Sl,i} \), kPa | \( E \), kPa |
|----|------|-----------------|----------------|-----------------|----------------|-----------------|--------|
| 1  | Loam | 1               | 19.9           | 15.5            | 0.02 0.046 0.09 0.117 | 105 | 4 |
|    |      | 2               | 19.9           | 15.8            | 0.02 0.046 0.09 0.117 | 105 | 5 |
|    |      | 3               | 19.5           | 15.6            | 0.011 0.021 0.036 0.056 | 125 | 7 |
| 2  | Loam | 4               | 19.5           | 15.9            | 0.011 0.021 0.036 0.056 | 125 | 7 |
|    |      | 5               | 18.2           | 15.2            | 0.007 0.014 0.024 0.043 | 130 | 12 |
|    |      | 6               | 18.2           | 15.2            | 0.007 0.014 0.024 0.043 | 130 | 15 |
| 3  | Loam | 7               | 17.7           | 15.0            | 0.007 0.014 0.024 0.043 | 135 | 15 |
|    |      | 8               | 17.7           | 15.2            | 0.007 0.014 0.024 0.043 | 135 | 22 |
|    |      | 9               | 17.4           | 15.1            | 0.004 0.009 0.017 0.032 | 140 | 140 |
| 4  | Loam | 10              | 17.4           | 15.5            | 0.004 0.009 0.017 0.032 | 140 | 30 |
|    |      | 11              | 17.2           | 15.8            | 0.003 0.008 0.009 0.009 | 150 | 150 |
|    |      | 12              | 17.2           | 15.9            | 0.003 0.008 0.009 0.009 | 150 | 150 |

### Table 3. Results of calculation of stress values for the base depth

| №  | \( z \), m | \( \sigma_{zi} \), kPa | \( \alpha \) | \( \sigma_{zp} = \alpha \cdot \sigma_{zi} \) | \( \sigma_{zi} = \sigma_{zi} + \sigma_{zp} \) | \( \sigma_{zi,m} \), kPa | \( \varepsilon_{Sl,i} \) | \( K_{Sl,i} \) |
|----|-------------|----------------|---------|-----------------|-----------------|-----------------|----------------|--------|
| 0  | 0           | 29.9           | 1.0     | 150             | 179.1          | -               | -               | - |
| 1  | 0.5         | 39.8           | 0.907   | 136.1           | 175.9          | 177.9           | 0.068          | 0.88 |
| 2  | 1.0         | 48.8           | 0.703   | 105.5           | 154.3          | 165.1           | 0.052          | 0.88 |
| 3  | 1.5         | 58.5           | 0.488   | 73.2            | 131.7          | 143.0           | 0.047          | 0.88 |
| 4  | 2.0         | 63.7           | 0.366   | 54.9            | 118.6          | 125.2           | 0.033          | 0.88 |
| 5  | 2.5         | 72.8           | 0.243   | 36.5            | 109.3          | 114.0           | 0.028          | 0.88 |
| 6  | 3.0         | 79.7           | 0.180   | 27.0            | 106.7          | 108.0           | 0.024          | 0.88 |
| 7  | 3.5         | 88.5           | 0.138   | 20.7            | 109.2          | 108.0           | 0.021          | 0.88 |
| 8  | 4.0         | 95.7           | 0.108   | 16.2            | 111.9          | 110.6           | 0.024          | 0.88 |
| 9  | 4.5         | 104.4          | 0.087   | 13.1            | 117.5          | 114.7           | 0.015          | 0.88 |
| 10 | 5.0         | 111.8          | 0.071   | 10.7            | 122.5          | 120.0           | 0.014          | 0.88 |
| 11 | 5.5         | 120.4          | 0.060   | 9.0             | 129.4          | 126.0           | 0.017          | 0.88 |
Figure 2. Schemes for calculation of the basement subsidence: 1 – vertical load from the own weight $\sigma_{zg}$; 2 – total vertical stress from the external load and own weight $\sigma = \sigma_{zp} + \sigma_{zg}$; 3 – change with a depth of initial drawdown pressure $P_{sl}$.

The total amount of settlement and complete subsidence is $S + S_{sl} = 2.07 + 15.11 = 17.18$ cm, that is more than the permissible value equal to $S_{u} = 12$ cm. Therefore, to ensure operational suitability of the building, it is necessary to eliminate subsidence properties of soils and reduce the amount of settlement.

Example 3. Calculation of the fully soaked (water-saturated) base.

As a result of analysis of the calculations, we have:
- at $S_s=0.31$: $S=0.66$ cm, $H_s=5.1$ m;
- at $S_{sl}=0.6$: $S=1.66$ cm, $H_s=4.7$ m;
- at $S_{war}>0.8$: $S=4.16$ cm, $H_s=4.7$ m.

When calculating settlement, the compressible strata is divided into three layers according to the degree of humidity. As a result of the calculation, the settlement turned out to be less than at $S_s>0.8$, and the depth of the compressible thickness did not change.

$S=3.84$ cm, $H_s=4.7$ m

Calculation of the settlement at $S_s=0.31$ ($d_{H}=1.5$ m)
Calculation results:
Settlement – 0.66 cm. The depth of the compressible stratum is 5.10 cm
Calculation of settlement at $S_{sl}=0.60$ ($d_{H}=1.5$ m)
Calculation results:
The settlement value is 1.66 cm. The depth of the compressible stratum is 4.70 cm
Calculation of settlement without $P_{sl}$ at $d_{H}=1.5$ m
Calculation results:
The settlement value is 3.84 cm. The depth of the compressible stratum is 4.70 cm

For the first foundation, we calculated the settlement using the formula by V.P. Dyba and O.N. Osipova [9] for soils with the structural strength. The settlement value was determined by formula

$$S = \beta \sum_{i=1}^{n} \frac{(\sigma_{zpi} - P_{str}) \cdot h_i}{E_i}$$

Example 4. The administrative and housing building in Klykov street, Elista.

The settlement value was calculated in Settlement 1.1 developed in the South SRPU (NPI) by G.M. Skibin, S.I. Evtushenko, and E.Yu. Anischenko.
Calculation of the settlement value ignoring and taking into account the structural strength of the soil:

Calculation of the settlement value without \( P_b \) at \( d_{b0}=1.5 \) m
Calculation results:
The settlement value \( = 1.43 \) cm. The depth of the compressible stratum is \( 5.10 \) cm
Calculation of the settlement value accounting for \( P_b \) at \( d_{b0}=1.5 \) m
Calculation results:
The settlement value \( = 0.89 \) cm. The depth of the compressible stratum is \( 4.3 \) cm
Calculation of the settlement value without \( P_b \) at \( d_{b0}=1.5 \) m
Calculation results:
The settlement value \( = 1.07 \) cm. The depth of the compressible stratum is \( 4.2 \) cm
Calculation of the settlement value accounting for \( P_b \) at \( d_{b0}=1.5 \) m
Calculation results:
The settlement value \( = 0.69 \) cm. The depth of the compressible stratum is \( 3.9 \) cm

When taking into account \( R_\text{s} \), the sediment value decreases \( 1.6 \) times, and the depth of the compressible depth \( 1.2 \) times.

4. For the second foundation, we determined \( R \) and \( F \) when the moisture regime of the base changes.
The forecast of the design resistance of the soil and the bearing capacity of the foundations

\[
K_{\varphi 1} = \frac{\varphi_{\text{wet}}}{\varphi_{\text{lowmoist}}} \quad K_{\varphi 2} = \frac{\varphi_{\text{watsat}}}{\varphi_{\text{lowmoist}}} \quad K_{c 1} = \frac{c_{\text{wet}}}{c_{\text{lowmoist}}} \quad K_{c 2} = \frac{c_{\text{watsat}}}{c_{\text{lowmoist}}}
\]

**Table 4.** The values of the strength parameters of loam \( c, \varphi \) and reduction factors \( K_\varphi \) and \( K_c \) when changing the moisture regime of the soil base

| Indicators                     | Loam                      | Reduction factors \( c \) and \( \varphi \) |
|-------------------------------|----------------------------|-----------------------------------------------|
|                               | Low moisture \((S<0.5)\)  | Water saturated \((0.5< S<1.0)\)              | \( K_{\varphi} \) | \( K_c \) |
| \( \varphi \), degree         | 26.4                      | 17.9                                          | 0.86            | 0.68    |
| \( c \), kPa                  | 42                        | 28                                            | 0.67            | 0.64    |

The results show that after soaking and water saturation, specific cohesion \( c \) decreases \( 1.5 \) and \( 1.6 \) times, and the angle of internal friction decreases \( 1.2 \) and \( 1.5 \) times, respectively.

For quantitative assessment of the calculated soil resistance of the base in low-moisture, wet and water-saturated states, let us consider examples of calculation.

**Example 5.** The foundation has a size \( 1 = b = 2 \) m and a depth of \( d_1 = 1.5 \) m; the base is composed of low-moisture loess loam. The specific weight of soils under and above the base in the low-moisture state are 17.5, and the wet and water-saturated state are equal to \( \gamma_p = \gamma_p' = 19.4 \) kN/m\(^3\). The building has no basement, therefore, \( d_{b0} = 0 \). We determined that \( \gamma_{c1} = 1.1, \gamma_{c2} = 1, K = 1, K_s = 1 \).

1. Let us determine the design resistance of the soil in the low-moisture state. As a result of experiments, we determined the angle of internal friction \( \varphi = 26.4^\circ \), and specific adhesion \( c = 42 \) kPa, \( M_g=0.85, M_g=4.44, M_c = 7.01 \). Then we calculated the soil resistance \[1\].

\[
R_1 = \frac{1.1 \cdot 1}{1.1} (0.85 \cdot 1 \cdot 2 \cdot 17.5 + 4.44 \cdot 1.5 \cdot 17.5 + 7.01 \cdot 42) = 440.72 \text{kPa}
\]

2. When determining the design resistance for the soaked base, we conducted experiments with soil samples in the wet state and determined the angle of internal friction \( \varphi = 22.7^\circ \) and specific adhesion at \( c = 28 \) kPa, dimensionless coefficients \( M_g = 0.65, M_g = 3.60, MS = 6.16 \). Then we calculated the soil resistance

\[
R_2 = \frac{1.1 \cdot 1}{1.1} (0.65 \cdot 1 \cdot 2 \cdot 19.4 + 3.60 \cdot 1.5 \cdot 19.4 + 6.16 \cdot 28) = 302.46 \text{kPa}
\]

3. Resistance in the conditions of water saturation of the soil is calculated after determining strength parameters of the soil \( M_g=0.42, M_g=2.71, M_c=5.27 \).

The resistance is calculated by formula:
\[ R_3 = \frac{1.14}{1} (0.42 \cdot 1 \cdot 2 \cdot 19.4 + 2.71 \cdot 1.5 \cdot 19.4 + 5.27 \cdot 27) = 237.45 \text{ kPa} \]

The results show that the soil resistance decreases 1.46 and 1.86 times, respectively, compared to the resistance of slightly wet soil. Thus, resistance values and reduction coefficients of resistance for base loams in low-moisture, wet and water-saturated states were determined experimentally (Table 5).

### Table 5. Values of the soil resistance \((R_1, R_2, R_3)\) and reduction coefficients of the resistance \((K_1, K_2)\)

| Soil       | Coefficient | Coefficient |
|------------|-------------|-------------|
|            |             |             |
|            | \(R_1\), kPa | \(R_2\), kPa | \(R_3\), kPa | \(K_1\) | \(K_2\) |
| Loam       | 440.7       | 302.5       | 237.4       | 0.69    | 0.54    |

where \(R_{\text{lowmoist}}, R_{\text{wet}}, R_{\text{watsat}}\) – calculated resistance values for the soil in low wet, wet and water-saturated states. The basis for the carrying capacity is calculated taking into account the following conditions:

\[ F \leq \gamma_c \cdot F_{u} / \gamma_{s} \]

where \(F\) – base load; \(F_{u}\) – strength of base resistance; \(\gamma\) – coefficient of working conditions taken for silty-clay soils in a stabilized state; \(\gamma_{s}=0.9\), for silty clay soils in unstabilized condition \(\gamma_{s}=0.85\); \(\gamma_{c}\) – reliability coefficient equal to 1.2; 1.15 and 1.10 for buildings and facilities of the first, second and third levels of responsibility.

The vertical component of forces of ultimate resistance \(N_u\) of the base composed of non-rocky soils in a stabilized state can be determined by formula (16) of CR 22.13330.2011.

\[ N_u = b^1 l^1 \left( N_p \xi_p b^1 y_1 + N_g \xi_g d y_1 + N_r \xi_r C_1 \right) \]

Designations are the same as in formula (16) CR 22.13330.2011.

**Example 6.** Calculation parameters from example 5.

### Table 6. Values of the bearing capacity of base soil \((F_1, F_2, F_3\) in kN) coefficients of reducing the bearing capacity of the soil \((K_{1F}, K_{2F})\)

| Soil       | Coefficient | Coefficient |
|------------|-------------|-------------|
|            | \(F_1\), kN | \(F_2\), kN | \(F_3\), kN | \(K_1\) | \(K_2\) |
| Loam       | 6585        | 3659        | 2956        | 0.56    | 0.45    |

where \(F_{\text{lowmoist}}, F_{\text{wet}}, F_{\text{watsat}}\) – the bearing capacity of the soil in slightly wet, wet and water-saturated states. The obtained results show that the bearing capacity of base soils composed of loess subsidence loams in wet and water-saturated states decreases 1.36 and 1.9 times compared to the bearing capacity of the slightly wet soil of natural density-humidity.

**Example 7.** Determination of the size of the bottom of the columnar foundation taking into account ultimate stress states. The foundations are located on loams with the following design characteristics: IL = 0.3; \(\gamma_1 = 17.5 \text{ kN } / \text{ m}^3\); \(\varphi = 26^\circ\); \(c = 0.042 \text{ MPa}\); \(E = 14.8 \text{ MPa}\). The foundation depth is \(d_1 = 1.5 \text{ m}\). The foundation transmits a load of \(N = 1.5 \text{ MN}\) to the base. Maximum allowable sediment is \(s_s=12 \text{ cm}\).

According to the CR method, \(\gamma_c=1.1\); \(\gamma_{c}=1.0\); \(\kappa=1.1\).

\[ R = \frac{3.14}{1} \left[ 0.85 \cdot 1 \cdot 2 \cdot 17.5 + 4.44 \cdot 1.5 \cdot 17.5 + 7.01 \cdot 42 \right] = 440.72 \text{ kPa} \]

Determining the sizes of the base from \(P \leq R\), we have \(b=l=2x2 \text{ m}\), where \(P=R=441 \text{ kPa}\).
The calculation of the foundation settlement by the method of layer-by-layer summation (Settlement software) gives the depth of the compressible thickness $H_c=5.1$ m and settlement $S_c=1.43$ cm. Since the calculated settlement is still far from the maximum allowable one, we will consider the foundation of 1.5 x 1.5 m with an average pressure in the base $P_{av}=0.382$ MPa.

For the foundation of 1.5x1.5 m in size, the resistance is

$$R = \frac{1.11 \cdot 10^{-1}}{1} [0.85 \cdot 1 + 1.5 \cdot 17.5 + 4.44 \cdot 1.5 \cdot 17.5 + 7.01 \cdot 42] = 433.3$ KPa
$$

$R=0.433$ MPa and $R_0=0.200$ MPa

By the method of layer-by-layer summation (Settlement) $H_c=4.2$ m, $S_c=1.17$ cm

The pressure limit is

$$p_{lim} = \frac{N_u}{b^2} = 1.5 \cdot 1.5(6.19 \cdot 0.75 \cdot 1.5 \cdot 1.75 + 11.26 \cdot 2.5 \cdot 15 \cdot 1.5 + 21.88 \cdot 1.3 \cdot 43)/2.25 = 1.98$ MPa
$$

Let us determine the following values

$$K_p = \frac{0.382}{0.200} = 1.91; \ K_p = \frac{0.198}{0.200} = 9.9; \ x = \frac{1.91 - 1}{9.9 - 1} = 0.10
$$

By $x=0.10$ and $\varphi=26^\circ$ using the CR method, we determine $K_p^{str} = 1.08$

AT $p=0.382<R=0.433$, the base settlement is

$$S_c = S_y \cdot K_p^{str} = 0.82 \cdot 1.17 = 1.26$ cm
$$

With the values of the calculated load $N=1.5$ MH and the bearing capacity $N_u=1.98 \cdot 2=3.96$ MH.

The reliability coefficient is

$$Y_n = Y_c \cdot \frac{N_u}{N} = \frac{3.96 - 0.9}{15} = 2.64 > Y_n = 1.2$ (by the CR method)
$$

The foundation with dimensions of 1.5x1.5 m can be used instead of the foundation with dimensions of 2.0x2.0 m. The example confirms that transition to the design of foundations for deformations of the foundations makes it possible to reduce the cost of foundations while maintaining the required bearing capacity.

4. For most buildings, uneven deformations (cracks) are dangerous, since all structures located on subsiding soils subside. The calculation must be carried out according to the second group of ultimate states, by deformations.

**Example 8.** Determination of uneven deformations

1. CR calculation:

$$\Delta S/L = (S_1 - S_2)/L = (15.11 - 2.07)/630 = 0.021
$$

2. Calculation by the Dyba and Osipova’s formula [9]:

$$\Delta S/L = (S_1 - S_{2str})/L = (15.11 - 0.89)/630 = 0.023
$$

Therefore, $S_{2CR} > S_{2str} \Rightarrow \Delta S_{str}/L > \Delta S_{CR}/L,

2.07 > 0.89 \Rightarrow 0.023 > 0.021

---

**Figure 3.** Determination of non-uniform deformations with local soaking

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8
where $S_2$ – foundation settlement f2 at the slightly wet base; $S_1$ – foundation settlement f1 at local soaking; $S_{str}$ – settlement taking into account the structural strength of soils; $S_{CR}$ – settlement calculated by the CR method.

4. Conclusion
1) Depending on the soil moisture, the settlement of the foundation increases from 0.66 cm at $S_r=0.3$ to 1.66 cm at $S_r=0.6$ and 3.84 cm at $S_r>0.8$; depth of the compressible thickness decreases from 5.10 m to 4.70 m;
2) Full water saturation of the whole subsiding strata does not occur.

![Figure 4. The degree of soil moisture base during: 1) slightly damp condition; 2) water-saturated state](image_url)

3) According to the Dyba and Osipova’s formula, uneven deformation of the foundations is more than according to the CR formula as the former takes into account the structural strength of soils.
4) The method can be used to predict uneven deformations of regional loess soils.

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