APPENDIX E - *SOFT SOIL TREATMENT*

E.1. Types of soil foundation needing treatment

1. *Evaluation by soil composition*

- Soft clay;
- Peat: organic content > 13%;
- Quick sand;
- Soil with the content of soluble impurity of chloride salt greater than 5%, of sulphate salt or chloride sunlphate greater than 10% in terms of weight.
- Alluvial soil, muddy soil, loamy soil (with low bearing capacity).

2. *Evaluation by forming origin*

- Types of mineral origin, can be mixed with organic matters during the deposition (organic content up to 10 - 12%);
- Soft soil can also be formed in the form of silt, fine silt in the valleys (void ratio e > 1.0, degree of saturation G > 0.8);
- Types of organic origin which usually formed in the mudflats;
- Soft soil in mudflats with peat.

3. *Evaluation by testing results*

- For non-cohesive soil: Compactness D<0.66, number of hammering in the dynamic penetration test N < 30.
- For cohesive soil, the values of viscosity criterion I_L are as follows:
  + Clayey sandy soil: $I_L \geq 0$.
  + Sandy clayey soil and clay: $I_L > 0,25$

4. *Requirements of the determination of physico-mechanical properties of soft soil*

a. Method of shearing-resistant criterion testing

- Dangerous case for dike stability is that when the phreatic level drops suddenly while completely saturated. Therefore, the non-consolidated, rapid-shearing and non-drainage diagram (UU diagram) should be chosen for the completely saturated prepared samples;
- In case of embankment on the natural foundation: triaxial compression test UU determining (c_{uu} and ϕ_{uu}) by the non-consolidated, non-drainage testing diagram;
- In case of embankment on reinforced foundation: using the rapid-shearing, consolidated and non-drainage testing diagram c, ϕ;
- In case of chronological embankment: Tria-axial compression test CU determining (c_{cu} and ϕ_{cu}) by consolidated, non-drainage diagram, measuring pore water pressure.

b. In order to measure the deformation of filling soil, the test determining the settling compression criteria a, modulus of deformation E and lateral (horizontal) swelling factor need to be conducted.

E.2. Computation of counter-pressure dimensions

1. Parameters of cross-section

- Dike cross-section

The parameters of preliminary dike cross-section are as follows:

+ Crest width: B_{crest}

+ Footing width: B_{foot}

+ Dike height: H

+ Seaward and landward slope: m_{tl}, m_{hl}

- Counter-pressure cross-section

Dimensions of the counter-pressure footing include thickness (h), width (L), slope angle (b). The width of counter-pressure footing on each side should exceed the range of critical sliding curve by at least 1-3m. The height of counter-pressure footing should not be too large to cause uplift sliding for the counter-pressure filling part which can be determined by the two methods presented below.

2. Method of determination by plastic deformation zone

Method of computing the dimensions of counter-pressure is based on the development of the plastic deformation zone within the soil foundation. With the dike foundation, load distribution has trapezoidal pattern and the plastic deformation zone below the structure has the oval shape which is concave in the middle (Figure E-1). The allowable plastic deformation development zone is half of the distance between the two outer edges of the counter-pressure footing. The width of plastic deformation zone is defined by the net of declination curve θ_M with equal values.

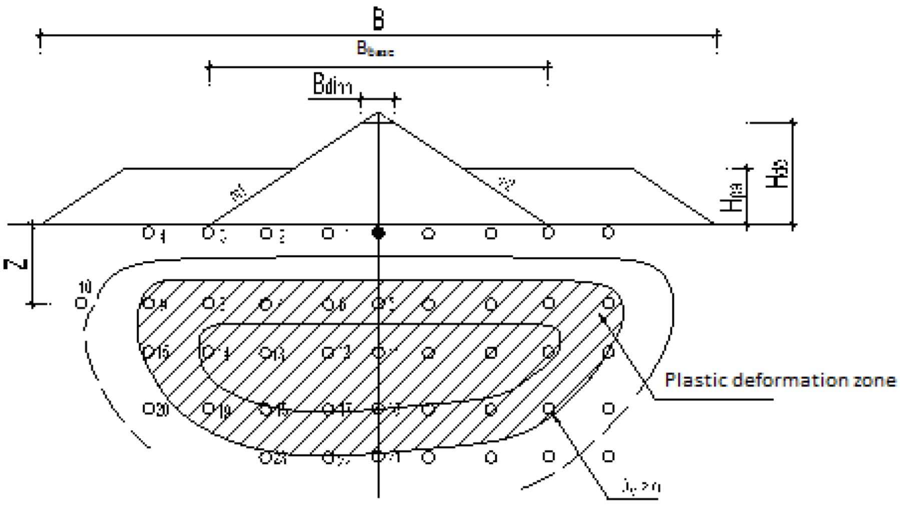


Figure E-1. *Counter-pressure footing*

The plastic deformation zone is drawn as followed: Divide the soil foundation into square net system and then determine the principal stress σ_1, σ_2 or σ_z, σ_y and τ_{xy} at the nodes of the net and calculate the corresponding angle of declination θ_M. Draw the isoline of θ_M. The region corresponding to the curve of $\theta_M = \varphi$ (φ - internal friction angle of soil foundation) is the plastic deformation zone. Empirically, the width of counter-pressure footing is twice of the width of plastic deformation zone.

The value of declination angle θ_M is determine by the following formula:

$$\sin\theta_M = \frac{\sigma_1 - \sigma_2}{\sigma_1 + \sigma_2 + 2\gamma(z + h_m + h_c)} \quad \text{(E-1)}$$

where σ_1, σ_2 - principal stresses at the surveying point within the soil foundation;

$$\sigma_1 = \mu p \quad \text{(E-2)}$$

$$\sigma_2 = \nu p \quad \text{(E-3)}$$

μ and ν - dependent coefficients $v = \frac{z}{b}$, $d = \frac{y}{b}$ given in Table 15-6, 15-7 in the Appendix.

p – average principal stress causing settling and distributed under the foundation base.

Or in terms of σ_z, σ_y, τ_{xy} applying the following formula:

$$\sin^2\theta_M = \frac{(\sigma_z - \sigma_y)^2 + 4\tau_{xy}^2}{\left[\sigma_z + \sigma_y + 2\gamma(z + h_m + h_c)^2\right]} \quad \text{(E-4)}$$

where σ_z, σ_y, τ_{xy} - vertical, horizontal compressing stress components and tangential stress at surveying point;

$$\sigma_z = k_z\, p \quad \text{(E-5)}$$

$$\sigma_y = k_y\, p \quad \text{(E-6)}$$

$$\tau_{zy} = \tau_{yz} = k_\tau\, p \quad \text{(E-7)}$$

$$p = \sigma_o - \gamma h_m \quad \text{(E-8)}$$

k_z, k_y, k_τ - dependent coefficients $v = \frac{z}{b}$, $d = \frac{y}{b}$ (given in the tables)

γ - volumetric weight of soil foundation;

z - depth of the surveying point;

h_m - placing depth of the foundation

$$h_c = \frac{c}{\gamma \mathrm{tg}\varphi} \quad \text{(E-9)}$$

c - unit cohesive force of soil foundation;

φ - internal friction angle of soil foundation ;

* The height of counter-pressure is decided on the basis of safety factor against sliding and satisfy the technical-economical conditions.

3. Method of nomogram

By means of nomogram method, the dimensions of counter-pressure footing are determined empirically and then the safety factor of the embankment with the counter-pressure footing is checked by using the nomogram.

By Chinese experience :

Height h > 1/3H

Width L = (2/3-3/4) length of the uplifting soil

By Pilot's nomogram :

Height h = 40÷50% of filling height of the dike H

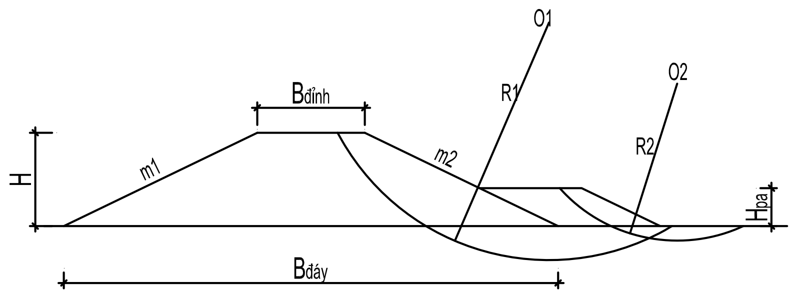
Width h = 2÷3 times of the thickness of soft soil D

The nomograms assisting the are presented in the reference books.

4. Stability computation

The computing diagram here include the computation of general stability for the dike profile with the improved stability by means of counter-pressure footing, and the computation of stability for the filling mass used for the pressure countering.

Figure E-2. *Computing diagram for the stability of dike profile treated by pressure countering.*



5. Settlement

The settlement of dike is computed as per the guideline on foundation design.

E.3. Excavating and replacing the base soil partially or completely

1. Designing the replacing sand buffer layer

When designing the sand buffer layer, the following conditions must be satisfied:

- The sand buffer layer is stable under the acting of structural load.

- Pressure on the surface of soil layer under the buffer layer caused by the structural load must be less than specified pressure on the surface of that soil layer.

- The total settlement of the buffer layer and the underlying soil layer as well as the irregular settlement of the foundation must be less than the limited value stipulated in the code for foundation design.

There are two trends in the computation of the dimensions of sand buffer layer that are most commonly used:

- Considering the sand buffer layer a part of the structural foundation and calculate it as a shallow foundation placed on a natural base. This is an approximate assumption but the computation is rather simple.

- Considering the sand buffer layer a part of the soil foundation, i.e. identifying the linear deformation. When computing the dimensions of the sand buffer layer, the distribution rules as well as the formulas determining stresses which presented in the lecture notes of Soil Mechanics can be applied.

Some approximate methods are presented below:

a. Determining dimensions of sand buffer layer based on the deforming conditions of soil foundation

According to this method, the dimensions of sand buffer layer are determined meeting the following condition: Total stress created by structural load and self weight of soil foundation and sand buffer layer exerting on the surface of the soft soil layer under the buffer layer must be less than or equal to the specified pressure on that soil layer, meaning:

$$\sigma_1+\sigma_2 \leq R^{tc} \quad \text{(E-10)}$$

where:

σ_1 - permanent stress created by self weight of foundation soil and sand buffer layer exerting on the surface of soft soil layer under the buffer layer;

$$\sigma_1 = \gamma_d h_d + \gamma h_m \quad \text{(E-11)}$$

γ_d, γ - volumetric weight of the sand buffer layer and foundation soil;

h_d, h_m - thickness of sand buffer layer and foundation placing depth;

σ_2 - stress created by structural load exerting on the surface of soft soil layer under the buffer layer;

$$\sigma_2 = \alpha_0(\sigma_0^{tc} - \gamma h_m) \quad \text{(E-12)}$$

α_0 – coefficient taking the stress variation along the depth, depending on the ratio

$$m = \frac{2z}{b} \text{ and } n = \frac{l}{b}$$

with z - depth calculated from the foundation base to the point where the stress is being considered;

l - foundation length;

b - foundation width;

σ_0^{tc} - average specified stress under the foundation base determined as follows:

- In case of centric load:

$$\sigma_0^{tc} = \gamma_{tb} h_m + \frac{\sum N^{tc}}{F} \quad \text{(E-14)}$$

- In case of eccentric load:

$$\sigma_0^{tc} = \frac{\sigma_{max}^{tc} + \sigma_{min}^{tc}}{2} \quad \text{(E-15)}$$

$$\sigma^{tc}_{max,min} = \gamma_{tb} h_m + \frac{\sum N^{tc}}{F} \pm \frac{\sum M^{tc}}{W} \qquad \text{(E-16)}$$

$\sum N^{tc}$ - Total vertical specified load created by structures exerting at the foundation base;

$\sum M^{tc}$ - Total specified moment created by structural load exerting at the foundation base;

F - Area of foundation base;

W - Section modulus of the foundation base;

γ_{tb} - Average volumetric weight of the foundation and soil exerting on the foundation, taking the value of 2 (T/m^3);

R^{tc} - Specified pressure on the surface of soft soil layer under the sand buffer layer, calculated by the formula given in the code for foundation design:

$$R^{tc}=[Ab_{mq}+B(h_m+h_đ)]\,\gamma^0_{tb} +DC^{tc} \qquad \text{(E-17)}$$

A, B, D - Dimensionless coefficients, depending on the specified internal friction angle, given in the tables of the code for foundation design;

b_{mq} – Conventional width of the foundation, determined as follows:

- For strip foundation:

$$b_{mq} = \frac{\sum N^{tc}}{\sigma_2 l} \qquad \text{(E-18)}$$

- For rectangular foundation:

$$b_{mq} = \sqrt{\Delta^2 + F_{mq}} - \Delta \qquad \text{(E-19)}$$

$$\Delta = \frac{l - b}{2} \qquad \text{(E-20)}$$

$$F_{mq} = \frac{\sum N^{tc}}{\sigma_2} \qquad \text{(E-21)}$$

γ^0_{tb} - average volumetric weight of the soil layers from the natural surface to the base of buffer layer, taking the uplifting pressure of water into consideration;

C^{tc} - Specified cohesive force of the soil foundation below the base of sand buffer layer.

For simplicity in computation, the thickness of the sand buffer layer $h_đ$ can be determined by the following approximate formula:

$$h_đ = Kb \qquad \text{(E-22)}$$

Where:

K – Coefficient depending on the ratios $\frac{a}{b}$ and $\frac{R_1}{R_2}$

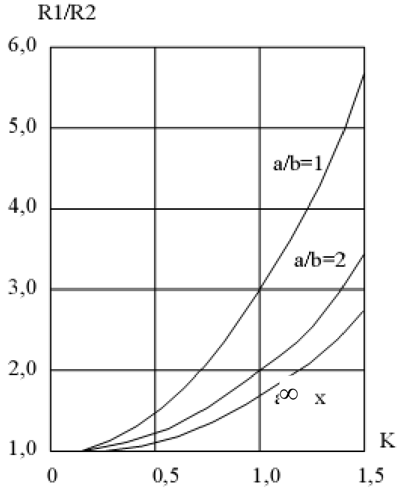
R_1 - Specified pressure on the surface of sand buffer layer, at the depth h_{m};

R_2 - Specified pressure on the surface of soft soil layer, under the sand buffer layer, above the depth of (h_m+h_d), determined by the nomogram given in Figure E-3;

b – Width of the foundation;

After determining the thickness of sand buffer layer by the formula (E-22), it is necessary to check the condition (E-10) and the settlement under the foundation of the structure. If the conditions are not satisfied, the thickness of sand buffer layer or the area of the foundation should be increased;

The settlement under the foundation of the structure is calculated by the following formula:



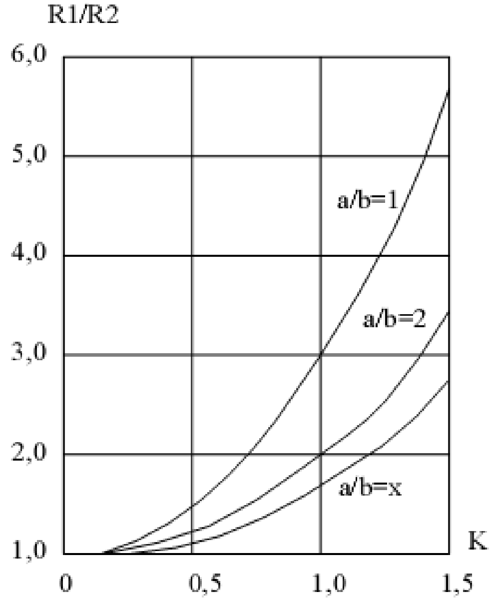


Figure E-3. *Nomogram for the determination of coefficient K*

$$S = S_1 + S_2 \leq S_{gh} \qquad \text{(E-23)}$$

where,

S_1 - settlement of the sand buffer layer;

S_2 - settlement of the soil layers under the sand buffer layer;

S_{gh} - Allowable limited settlement for each type of structure, determined as per the current codes for foundation design.

In order to ensure the stability of the soil surrounding the sand buffer layer, the width of buffer layer must be enough so that the lateral deformation caused by the structural load is not too large and in the allowable limit.

By the design experience, in order to ensure the aforementioned requirement, the angle of load exerting α is taken as the internal friction angle $\varphi_{đ}$ of the sand buffer layer ($\alpha=\varphi_{đ}$) or take the values in the range of $30\text{-}45^0$ ($\alpha=30\text{-}45^0$). The width of sand buffer layer is then calculated by the following formula:

$$b_{đ} = b + 2h_{đ}tg\alpha \tag{E-24}$$

b. Determination of dimensions of sand buffer layer using the method proposed by B.I. Đalmatov

When determining dimensions of sand buffer layer under the rigid foundation, B.I. Đalmatov has based on the condition of equilibrium between the lateral pressure created by the sliding prism and the soil pressure exerting surrounding the sand buffer layer. Assuming that the distribution of soil pressure exerting surrounding sand buffer layer due to the self weight is the same as the distribution pattern of hydrostatic pressure and the distribution of pressure under the base of buffer layer created by the structural pressure and self weight of the buffer layer is considered uniform. The aforementioned assumptions can be considered rational if the soil surrounding the buffer layer is soft soil in saturated state.

Based on those proposal, B.I. Đalmatov has investigated the two cases and established the formulae determining the dimensions of sand buffer layer for each case.

Case I: Sliding surface AD intersect the base of sand buffer layer (Figure E-4)

Taking the equilibrium condition of the sliding prism ADFC into consideration and from the closed force polygon, we get:

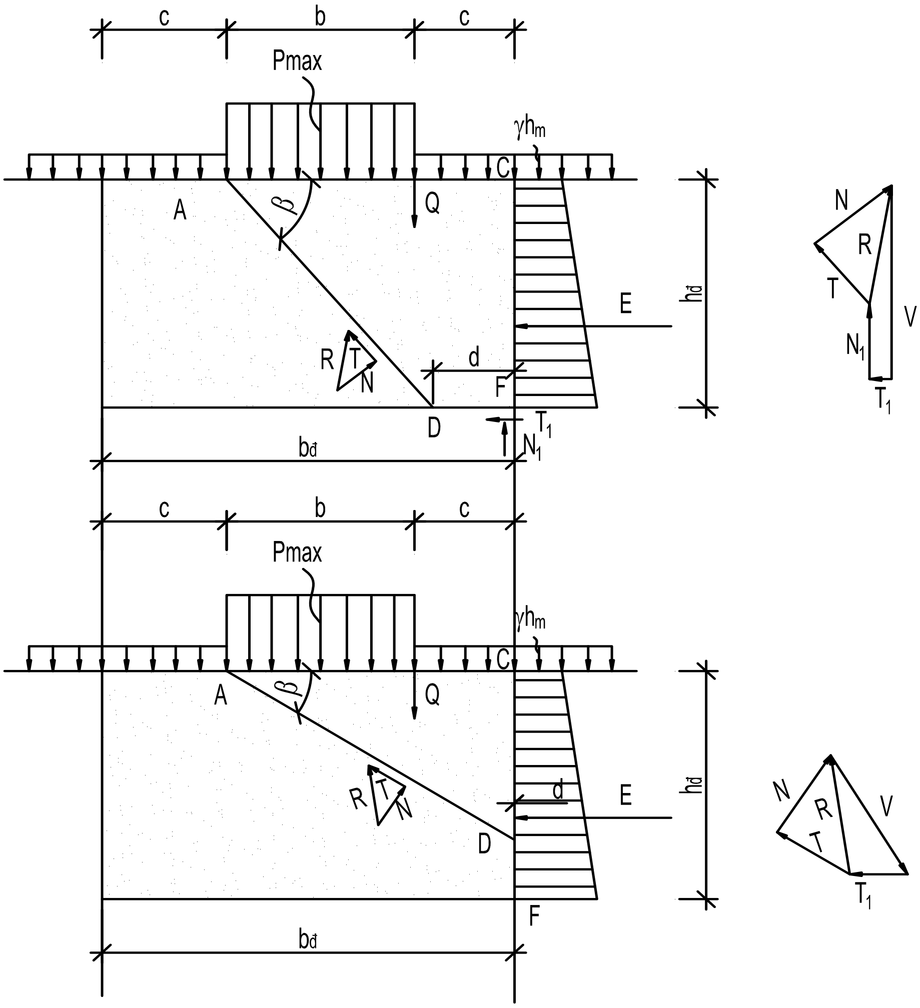


Figure E-4. *Computing diagram for the computation of sand buffer layer according to B.I.Đalmatov's proposal*

a – sliding surface AD intersecting the base of sand buffer layer;

b - sliding surface AD intersecting the vertical plane CF;

$$tg(\beta-\varphi_{đ})=\frac{T_1+E}{Q-N_1} \qquad \text{(E-25)}$$

$$E = \gamma h_{đ}(h_m+0{,}5h_{đ}) \qquad \text{(E-26)}$$

$$N_1 = \left[\left(P_{max}-\gamma h_m\right)\frac{b}{b_d}+\gamma h_m+\gamma_d h_d\right] \qquad \text{(E-27)}$$

$$T = N_1 tg\varphi_1 \qquad \text{(E-28)}$$

where:

β - inclination corresponding to the sliding surface AD;

$\varphi_{đ}$ - Internal friction angle of sand within the buffer layer;

P_{max} – Maximum (limited) pressure created by the design load exerting under the foundation base;

φ_1 - Internal friction angle of sand within the soil layers under the buffer layer;

γ, γ_d - Volumetric weight of the soft soil layer and the sand buffer layer.

designating:

$$n = \frac{b}{b_d}$$

$$k = \frac{\gamma_d}{\gamma}$$

also by transforming, the average value of the maximum pressure P_{max} exerting at the foundation base can be derived as follows:

$$P_{max} = \frac{\gamma}{n}\left\{\begin{array}{l} \dfrac{h_d(h_m + 0{,}5h_d) + a(h_m + kh_d - nh_m)\left[tg\varphi_1 + tg(\beta - \varphi_d)\right]}{(b_d - a)tg(\beta - \varphi_d) - atg\varphi_1} - \\ \dfrac{\left[hmc + 0{,}5(b + c + a)khd\right]tg(\beta - \varphi d)}{(bd - a)tg(\beta - \varphi d) - atg\varphi_1} \end{array}\right\} \quad \text{(E-29)}$$

Case 2: Sliding surface AD intersect the vertical plane CF. In this case, the values of N_1 and T_1 are zero, therefore:

$$P_{max} = \frac{\gamma(b + c)^2 tg\beta}{2b}\left[\frac{(b + c)tg\beta + 2h_m}{(b + c)tg(\beta - \varphi_d)} - \frac{2h_m c}{(b + c)^2 tg\beta} - k\right] \quad \text{(E-30)}$$

For convenience in calculation, formula (E-30) has been proposed to be rewritten in the simple form as follow:

$$p_{max} = \frac{\gamma}{2b}\left[m^2(A_1 - kD_1) + 2hm(mB_1 - c)\right] \quad \text{(E-31)}$$

Where:

$$M = b + c \quad \text{(E-32)}$$

$$A_1 = \frac{tg^2\beta}{tg(\beta - \varphi_d)} \quad \text{(E-33)}$$

$$B_1 = \frac{tg\beta}{tg(\beta - \varphi_d)} \quad \text{(E-34)}$$

$$D = tg\beta \quad \text{(E-35)}$$

The values of A_1, B_1, D_1 in Formula (E-31) depend on β and φ_d given in the tables of Foundation Design Handbook.

Consequently, the calculation of dimensions of sand buffer layer depends on the selection of P_{max} value. After changing the value of inclination b and based on the two formulae (3-19), (3-20) or formula (3-21), the minimum value of P_{max} can be obtained. This minimum value of P_{max} must satisfy the following condition:

$$\sigma_0^{tt} \le \frac{p_{max}}{1{,}1} \quad \text{(E-36)}$$

Where,

σ_0^{tt} - average design stress created by design load at the foundation base, determined as follows:

- In case of centric load:

$$\sigma_0^{tt} = \gamma_{tb} h_m + \frac{\sum N^{tt}}{F} \qquad \text{(E-37)}$$

- In case of eccentric load:

$$\sigma_0^{tt} = \frac{\sigma_{max}^{tt} + \sigma_{min}^{tt}}{2} \qquad \text{(E-38)}$$

$$\sigma_{\substack{max\\min}}^{tt} = \gamma_{tb} h_m + \frac{\sum N^{tt}}{F} \pm \frac{\sum M^{tt}}{W} \qquad \text{(E-39)}$$

$\sum N^{tt}$ and $\sum M^{tt}$ - Total vertical design load and total design moment exerting at the foundation base;

F - Area of foundation base;

W - Section modulus of the foundation base;

Determining the width of buffer layer by the method of B.I. Dalmatov will give reliable and rational results, especially in case of saturated soft soil foundation with high compressibility. In comparison with the other methods, however, the computing process is more complicated when determining the minimum value of P_{max}.

2. Settlement computation

Calculation of foundation settlement with the sand buffer layer is conducted under the same procedure as multi-layer soil foundation.

E.4. Usage of Geotextile

1. Placing geotextile as a lining in the excavated hole and inside the dike body

Due to the characteristic of soft soil, i.e. low bearing capacity and high settling compressibility, when excavating a part of the foundation and replacing with a better sand or soil layer, the settlement and stability will be improved partially. However, during the first stage of construction or operation, sinking settlement of the replacing soil layer into the soft soil foundation usually occurs pushing the underlying soft soil up to both sides. Taking advantage of seperating capcity of geotextile, one (or some) layer of this as a lining in the excavate hole in order to prevent the sinking settlement and also redistribute the structural load on the base.

In case the soft soil layer has been replaced partially but the dike still unstable, some geotextile layers can be placed inside the dike body in order to improve the shear resisting capacity of the filling soil and thus improve the dike stability.

2. Replacing soil mass can be covered by a geotextile bag

The replacing soil mass covered by a geotextile bag is a type of composite material which is very good for the treatment of foundation, due to the following causes:

- Shear resisting strength of soil + geotextile is much larger than foundation soil and therefore the bearing capacity will increase;

- The contracting compressibility of the replacing soil mass is lower and therefore the settlement of the structure will decrease;

- Good water drainage and guiding capacity of the geotextile bags will enhance the consolidation process of the foundation when bearing the external load.

3. Requirements of geotextile used for soil lining or coating

The geotextile used as a lining in the excavating holes replacing soil or covering the replacing soil must satisfy the following conditions:

- Good soil blocking capacity: The openings of geotextile must be small enough to prevent the soil particles of certain sizes in the replacing soil layer from moving into foundation soil.

The dimension of filtering geotextile openings is determined by means of the homogeneity C_u and d_{50} of soil. C_u is determined by the following formual:

$$C_u = \frac{d_{60}}{d_{10}}$$

where,

d_{60} – diameter of the soil particle which cannot be exceeded by 60% of the particle weight ;

d_{10} - diameter of the soil particle which cannot be exceeded by 10% of the particle weight ;

d_{50} - diameter of the soil particle which cannot be exceeded by 50% of the particle weight.

Depending on the soil particle characteristics and the soil homogeneity C_u, the dimension of filtering openings of geotextile can be determined.

- Good permeability: Geotextile placed inside the foundation must not change pattern of the seepage flow, i.e. the permeability of geotextile must be large enough for the water to go through and must not create uplift pressure beyond the allowable level.

Permeability coefficient of the geotextile must satisfy the following requirement:

$$k_g \geq \frac{t.k}{5.d_{50}}$$

where:

k_g – permeability coefficient of geotextile;

t - thickness of geotextile;

k - permeability coefficient of soil;

- Clogging resisting: The openings of geotextile must be large enough so that there is no clogging during working time. By the experience in other countries, for non-woven geotextile, the ratio of opening area to the total area of geotextile must be greater than 30%; for woven geotextile, this ratio must be greater than 4%.

- Durability: Geotextile used to improve the stability of soft soil foundation must be based on the calculation in section 2 and 3, the following criteria must also be satisfied:

+ Selecting the geotextile with the minimum disruptive tensile strength greater than 25 kN/m in order to ensure high efficiency of compaction at the first filling layer;

+ Elongation at break ≤ 25%;

+ Penetration-resisting capacity (CBR): 1500 + 5000N (BS 6906-4)

- Sewing thread for geotextile is the dedicated thread with diameter of, disruptive tensile strength > 40N per thread.

- The dedicated sewing machine must be used to sew the geotextile. Geotextile sewing machine is the professional one with the thread head distance of 7 to 10mm.

4. Structural design and computation

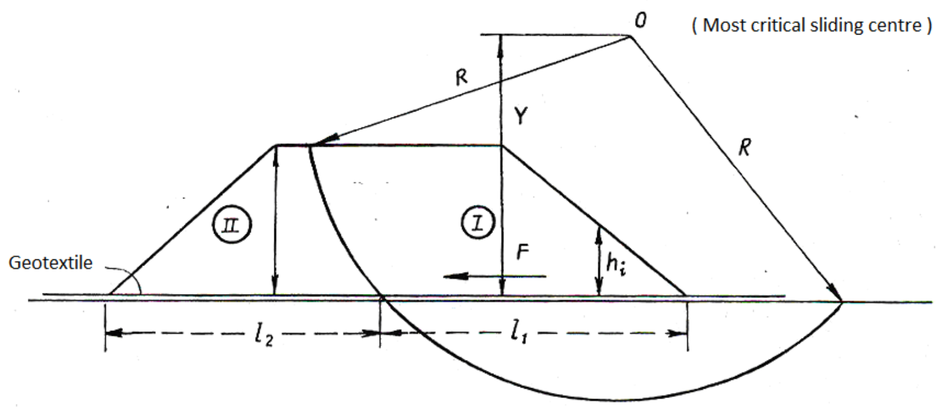


Figure E-5. *Acting mechanism of the geotextile layer*

Designating: **I** is active zone (sliding mass); **II** is passive zone (where geotextile acting as an anchor); F is tensile force borne by geotextile (T/m); Y is the moment arm of the force F with the most critical sliding center.

a) When placing geotextile between the soft soil and filling foundation as shown in Figure E-5, the friction between filling soil and the upper surface of geotextile will create the retaining sliding mass force F (ignoring the friction between soft soil and the lower surface of geotextile)

Applying this method, the following condition must be met in the design:

$$F \le F_{cp} \quad \text{(E-40)}$$

Where:

F – tensile force exerting on the geotextile (T/m);

F_{cp} – allowable tensile force for the geotextile with a width of 1m (T/m).

b) Allowable tensile force for the geotextile F_{cp} is determined by the following conditions:

- Condition of geotextile:

$$F_{cp} = \frac{F_{max}}{k} \qquad \text{(E-41)}$$

Where:

F_{max} – break resisting strength of geotextile of 1m size (T/m);

k – Safety factor,

k = 2 for geotextile made of polyester;

k = 5 for geotextile made of polypropylene or polyethylene.

- Condition of friction for the geotextile layer placing directly on the soft soil:

$$F_{cp} = \sum_{0}^{\ell_1} \gamma_d h_i f' \qquad \text{(E-42)}$$

$$F_{cp} = \sum_{0}^{\ell_2} \gamma_d h_i f' \qquad \text{(E-43)}$$

Where:

ℓ_1 and ℓ_2 - length of geotextile within active and passive zones;

γ_d - volumetric weight of filling soil;

f' - allowable friction coefficient between the filling soil and geotextile used in the computation;

h_i – filling height on the geotextile (varying in the range ℓ_1 to ℓ_2, from h_i = h to h_i = 0) (see Figure E-5).

Expression (E-42) and (E-43) is the total friction on the geotextile within the active and passive zones:

$$f' = k'.\frac{2}{3} tg\varphi \qquad \text{(E-44)}$$

where:

φ - internal friction angle of the filling soil determined correspondingly to the actual compactness of the filling foundation or available sand buffer layer (degree);

k' - reserve coefficient of friction, the value 2/3 can be used.

Determination of ℓ_1 and ℓ_2 is performed at the same time with the check of the above-mentioned stability level: assuming the force F in order to ensure that the minimum stability factor satisfies the aforementioned requirement and check the condition (E-41) in such a way that the conditions (E-41), (E-42) and (E-43) are all met;if these conditions are met, the geotextile with corresponding value of F_{max} can be selected on the basis of the minimum value of F_{cp} according to the aforementioned relations.

c) Geotextile used to improve the stability of the filling foundation on soft soil can be placed in one or many layers (1÷4 layers), if there is a demand for the improvement of drainage, arrange the geotextile and sand with the thickness of 30-50cm in alternate layers. Total disruptive tensile strength of these layers must be equal to F_{max} determined in (b) section.

Notes: In case of the upper geotextile layers placed inside the filling sand (upper and lower surfaces are in contact with sand), the value of F_{cp} calculated by (E-42) and (E-43) is multiplied by 2, from this the total allowable friction of the geotextile layers can be determined.

d. Computation of general stability of the structure:

Computation of the stability against deep sliding with the sliding stability factor requires K_{min} > 1,20 applying the classical method of fragmentation:

$$K_{min} \frac{M_{momengiu} + F.Y}{M_{momentruot}} \qquad \text{(E-45)}$$

E.5. Filling in layers with consolidation – chronological filling

1. Limited height

In case of the dike route which is not so high and the construction period is allowed to be extended, the effective methods is to divide the dike height into 2 or 3 layers with increasing height in many years facilitating the consolidation and therefore improving the bearing capacity.

Allowable limited height $[H_{gh}]$ of filling soil mass based on the bearing capacity of soft soil foundation $[H_{gh}]$ is calculated by the following formula:

$$[H_{gh}] = \frac{5,14.C_u}{K\gamma_®} \qquad \text{(E-46)}$$

or $$[H_{gh}] = \frac{5,14xC}{K\gamma_® x(1-5,14.tg\varphi_w)}$$

Where:

C_u- shear resisting strength of foundation soil determined by the triaxial compressing test with non-consolidating, rapid-shearing diagram without drainage;

C, φ - cohesive force and internal friction angle of soil determined with flat shearing machine and the non-consolidating, rapid-shearing test without drainage;

γ_d- specific weight of filling soil;

K- safety factor, K=1,25.

If the required height of the dike (h_{dike}) is lower than the allowable limited height, $h_d \leq H_{gh}$, the dike can be embanked directly on the natural foundation.

If $h_đ \geq H_{gh}$, i.e. the bearing capacity of foundation soil is not enough, certain methods must be applied in order to improve the bearing capacity of the soft soil foundation before embanking up to the required height.

2. Determining filling parameters

- Determining the safety height H_{at} of the filling soil mass with the natural soft soil foundation

Safety height H_{at} of the filling soil mass with the volumetric weight $\gamma_đ$ is:

$$H_{at}=\frac{3{,}14xC_u}{\gamma_{®}} \quad \text{(E-47)}$$

C_u – shear resisting strenght of foundation soil. If the testing data determining C_u on triaxial machine is unavailable, the data on φ, C of foundation soil determined on flat shearing machine can be used, applying the following expression:

$$H_{at}=\frac{3{,}14xC}{\gamma_{®}x(1-5{,}14.tg\varphi_w)} \quad \text{(E-48)}$$

- Selecting the thickness of the filling soil layer in first stage designated by h_1

In order to ensure the stability of the soft soil foundation under the dike, the thickness of the first filling soil layer (h_1) should not exceed the safety height (H_{at}) for the foundation soil, i.e.:

$$h_1 \leq H_{at} \quad \text{(E-49)}$$

If necessary, the value in the range $H_{at} \leq h_1 \leq [H_{gh}]$ can be chosen.

- Determining the time interval when the construction break is needed (T1):

After finishing filling up layer h_1, a waiting period T_1 is needed for the foundation soil to reach the required degree of consolidation U_t under the pressure created by the first soil layer $P_1=\gamma_đ h_1$;

$$T_1=t\,x\left(\frac{H_1}{h}\right)^n \quad \text{(E-50)}$$

where, n – consolidation index, depending on plasticity index (I_p) and consistency index I_L of soil. For muddy soil, cohesive soil in pasty or pasty plastic state, the value n = 2 can be used;

t – duration of compression of soil sample with the height h (h=2cm) under the pressure $P_1 = \gamma_đ h_1$ until the required degree of consolidation U_1 is reached in the laboratory;

H_1 – thickness of foundation soil compressed by the pressure $P_1 = \gamma_đ h_1$ created by the first filling soil layer.

In case the testing data on shear resisting strength with time and with the degree of consolidation is unavailable, the consolidation time for each filling layer can be determined by the following formula:

$$t = \frac{T_v h^2}{C_v} \quad \text{(E-51)}$$

T_v is determined by the relation between the degree of consolidation U and T_v.

- Investigating the stability of foundation soil after the time interval T_1 in order to determine the thickness of the second filling layer (h_2)

After the duration T_1, due to the consolidating compression, the shear resisting characteristics of foundation soil will reach the values of φ_{cu}, C_{cu}. In comparison with the value φ_u, C_u in the initial natural state, the shear resisting strength of foundation soil will increase:

$$\left.\begin{aligned}\Delta C_{cu} &= C_{cu} - C_u \\ \Delta\varphi_{cu} &= \varphi_{cu} - \varphi_{cu}\end{aligned}\right\} \qquad \text{(E-52)}$$

Investigating the safety height, allowable limited height of the dike after the time interval T_1 with the shear resisting strength determined by the average rate of increment:

$$\left.\begin{aligned}C_{cu}^{tt} &= C_u + \frac{\Delta C_{cu}}{2} \\ \varphi_{cu}^{tt} &= \varphi_u + \frac{\Delta\varphi_{cu}}{2}\end{aligned}\right\} \qquad \text{(E-53)}$$

- Safety height of the filling soil mass after the time interval T_1 is:

$$H_{at} = \frac{3{,}14xC_{cu}^{tt}}{\gamma_{®}x(1-5{,}14.tg\varphi_{cu}^{tt})} \qquad \text{(E-54)}$$

- Allowable limited height of the filling soil mass after the time interval T1 is:

$$[H_{gh}] = \frac{5{,}14xC_{cu}^{tt}}{K\gamma_{®}x(1-5{,}14.tg\varphi_{cu}^{tt})} \qquad \text{(E-55)}$$

If $H_đ \leq [H_{gh}]$, the necessary thickness of the second filling layer (h2) is:

$$h_2 = H_đ - h_1 \qquad \text{(E-56)}$$

If $H_đ > [H_{gh}]$, the necessary thickness of the second filling layer (h2) is:

$$h_2 = [H_{gh}] - h_1 \qquad \text{(E-57)}$$

After finishing filling up the second layer, a waiting time T2 is needed until the next filling period.

3. Performing the investigation of consolidation state of soft soil under the filling foundation applying the following methods:

- Measure the pore water pressure;
- Measure the settlement of soft soil layer;
- Determine the increase of cohesive force C_u by the vane shearing test;

Applying the computing method in order to inverstigate and to make sure that the construction can be started, and if not, the waiting period for the consolidation must be extended or the most appropriate method must be chosen.

E.6. Special methods

E.6.1- Method of foundation consolidation by the vertical drainage equipments (applying sand pile or vertical artificial drain)

Method of vertical drainage consolidation is applied when the soft soil layer is relatively thick, the drainage consolidation period of the foundation soil is long. In order to reduce the consolidation period, the drainage distance need to be shortened by the vertical arrangement of drainge passages, and at the same time by covering the surface of foundation soil with drainage sand layer and loading layer in order to accelerate the consoliation process. The vertical drainage passages can be sand pipes or vertical artifical drain.

a. Sand pipe is formed using steel pipes driven into the soil by the pipe driver. The sand is filled into the pipe and then the steel pipe is lifted up. Diameter of the sand pipes is normally in the order of 20÷30cm and 30÷40cm in water. Distance between the sand pipes is normally in the order of 8-10 times of the diameter, the length should not exceed 20m. Thickness of drainage sand layer on top of the sand pipes is in the order of 0,3÷0,5m on dry land and 1,0m in water .

b. Vertical artificial drain has the common section area of 100x4mm to 100x7mm. Vertical artificial drain is also inserted into the soft soil foundation using the dedicated equipments. The distance between the drains is 1,0÷1,5m. Nowadays, the driving depth into the soft soil foundation of the vertical artificial drain is less than 20m with a maximum of 23m.

Due to the vertical drainage means, water consolidating in the deep layers of soft soil under the action of filling load will flow into drainage passage and follow the horizontal drainage layer to reach the drainage pump. However, in order to ensure the improvement of the drainage efficiency, the minimum height of filling foundation should be 4m and in design the following conditions must be satisfied:

$$\sigma_{vz} + \sigma_z \geq (1{,}2 \sim 1{,}5).\ \sigma_{pz}\ ; \qquad \text{(E-58)}$$

$$\eta = \frac{(\sigma_{vz} + \sigma_z) - \log \sigma_{pz}}{(\sigma_{vz} + \sigma_z) - \log \sigma_{vz}} > 0{,}6\ ; \qquad \text{(E-59)}$$

where σ_{vz} – vertical stress (pressure) created by self weight of the soft soil layers at the depth z (MPa) ;

$$\sigma_{vz} = \Sigma\ \gamma_i\ .\ h_i\ ;$$

where γ_i and h_i are the volumetric weight and thickness of the soil layer i within the range from contact plane between the soft soil and the base of filling foundation (z=0) to the depth z in the soft soil ; note that for the soft soil layers lying under the phreatic level, the buoyant volumetric weight γ_i must be used.

σ_z = vertical stress (pressure) created by filling load (filling foundation and the available loading filling parts, but excluding the filling height hx converted from the vehicular load) exerting at the depth z in soft soil from the base of filling foundation (MPa);

σ_{pz} = pre-consolidating pressure at the depth z in soft soil (MPa); σ_{pz} is determined by the consolidation test.

Conditions (E-58) and (E-59) must be satisfied at all the values of the depth z within the range from the base of filling foundation to the driving depth of sand pipe or artificial vertical drain.

If the above-mentioned conditions are not satisfied, the method of pre-loading can be combined in order to increase the value of σ_z .

Methods of vertical drainage consolidation are usually applied when the soft soil layer is thick (thickness of soft soil layer exceeds the width of filling foundation base) and the filling foundation is hight. Due to the hight construction cost, these methods are only applied when the others cannot ensure the standard of remaining consolidating settlement ΔS during the stipulated construction time.

Upon applying the method of vertical drainage consolidation, the sand buffer layer must be used. If the sand pipes are used, the top of these pipes must be in direct contact with the sand buffer layer. If the artificial vertical drain is used, the penetration through the sand buffer layer is required and the minimum extra cut part is 20cm above the upper surface of sand buffer layer.

Sand used for sand pipes must also have requirements of filtering and clogging resistance.

Artificial vertical drain used for vertical drainage consolidation must satisfy the following requirements:

+ Dimension of the openings of the filtering cover (determined according to Standard ASTM D4571): $O_{95} \leq 75$ µm;
+ Permeability coefficienct of the filtering cover (ASTM D4491): $\geq 1.10^{-4}$ m/sec;
+ Drainage capacity of the drains with a pressure of 350 KN/m2 (ASTM D4716): $q_w \geq 60.10^{-6}$ m^3/sec;
+ Tensile resisting strength corresponding to the elongation less than 10% (ASTM D4595) against tearing during construction : ≥ 1 KN/drain;
+ Elongation (along the entire drain width) (ASTM D4632): >20%;
+ Width of the drains (to be compatible with standardized drain placing equipment): 100 mm ± 0,05 mm.

Determining the depth of sand pipe or vertical drain is a technical-economical problem requiring the designer to consider on the basis of the distribution of the settlement of the soft soil layers along the depth under the acting of filling load for each design case. It is not necessary to place within the entire influence range of filling load (range of settlement), only necessary to the depth where the consolidating settlement of the soft soil layer is small. From this depth upwards, the total settlement accounts for a ratio which is large enough compared with the forecast consolidating settlement S_c in such a way that if the rate of consolidation is accelerated in the range where the piles or drains are placed, it is adequate to meet the criteria of remaining allowable consolidating settlement during the required construction time.

Therefore, the design must propose different alternatives of sand pipes and vertical drains arrangement (in terms of depth and distance). Each arranging alternative in terms of depth must satisfy the aforementioned conditions.

When applying the method of vertical drainage consolidation, the pre-loading process must be combined and in all cases the maintaining duration of filling load should not be less than 6 months. Any type of soil (including organic soil) can be used to fill up the pre-loading layers, but sand is the best option. The slope of pre-loading filling is up to 1:0.75 and the degree of compactness is K = 0,9 (standard compaction).

E.6.2. Foundation treatment using soil-cement piles

Nowadays, the calculation of bearing capacity and deformation of soil foundation reinforced by soil-cement piles is still the problem needing more development researches. There are two main viewpoints as follows:

- From the viewpoint of pile foundation. From this viewpoint, the piles requires relatively large stiffness and their tips are driven to the load-bearing soil layers. The load acting on the foundation will then mainly exert on the soil-cement piles (ignoring the acting of soil under the foundation base). In case the piles cannot be driven to the load-bearing soil layer, the same calculating method for friction piles can be used. This viewpoint is adopted when calculating the arrangement of piles under the foundation base.

- From the viewpoint of equivalent base: the soil base is improved after treatment with the physico-mechanical properties of an equivalent base (γ_{td} ; φ_{td} ; E_{td});

Method of soil-cement pile design, given in the Shanghai Code (China) and TCXD 385-2006 (Ministry of Construction), is presented below.

1. Computation from the viewpoint of pile foundation

a) Allowable bearing force of single soil-cement pile should be determined by means of the test on the load of single pile, and it can also be estimated by the formula (E-60) or (E-61):

$$P_a = \eta .\ f_{cu} .\ A_P \qquad \text{(E-60)}$$

or $$P_a = U_P\ .\ \Sigma\ q_{si}\ \ell_i + \alpha .\ A_P .\ q_P \qquad \text{(E-61)}$$

Where,

P_a – allowable bearing force for single pile (kN);

f_{cu} – average value of indoor compression resisting strength (kPa) of the soil-cement test sample (a cube with the side length of 70,7mm) with the same mixing formula of soil cement for the pile body, after 90-day duration and in standard maintenance conditions;

A_P – cross section area of the columns (m^2);

η - reduction factor of column body strength, the values of 0,3 ~ 0,4 can be used;

U_P – column circumference (m);

q_{si} – allowable friction of the i[th] soil layer surrounding the column.

+ For silty soil: q_{si} = 5~8 kPa;
+ For muddy soil: q_{si} = 8~12 kPa;
+ For clay: q_{si} = 12~15 kPa;

ℓ_i – thickness of the i[th] soil layer surrounding the pile (m);

q_P – bearing force of the natural foundation soil at the tip of the pile (kPa);

α - reduction factor of bearing force of the natural foundation soil at the tip of the pile, the values of 0,4 ~ 0,6 can be used.

The bearing force of composite foundation soil with load-bearing soil-cement piles should be determined by means of the test with the load of combined foundation, and it can also be estimated by the following formula:

$$f_{sp} = {}^{m.P_a}\!/_{A_p} + \beta(1-m)f_s \qquad \text{(E-62)}$$

Where,

f_{sp} - allowable bearing force of combined foundation (kPa);

f_s - allowable bearing force of the natural foundation soil between the piles (kPa);

m - distribution ratio of pile area and soil area;

β - reduction factor of bearing force of soil between the piles.

+ In case of soft soil at the pile tips: β = 0,5 ~ 1,0;
+ In case of hard soil at the pile tips: β = 0,1 ~ 0,4.

It can also be based on the requirement of the structure to reach the allowable bearing force of combined foundation, the distribution ratio of the pile area to soil area is calculated by the following formula:

$$m = \frac{f_{sp} - \beta.f_s}{P_a \ell A_P - \beta.f_s} \qquad \text{(E-63)}$$

Arranging the lay-out of load-bearing soil-cement piles can be based on the requirement of bearing force and the deformation of foundation base in relation to the superstructures as well as their structural characteristics. Cylinder-typed, wall-typed, enclosing-typed or block-typed reinforcement can be applied and arranged only within the range of foundation base lay-out. The pile length must be considered on the basis of many factors such as the deformation requirement of the architecture (structure) block and the foundation structure.

For the treatment of pile foundation base, the piles can be arranged in square or equilateral triangular patterns, and the total number of piles required can be calculated by the following formual:

$$n = \frac{m.A}{A_p} \qquad \text{(E-64)}$$

Where,

n – total number of piles;

A – area of foundation base (m^2).

When the load-bearing soil-cement piles has relatively large distribution ratio of soil and pile (m>20%), and not arranged in single rows, cluster of soil-cement piles and soil between piles must be considered a conventional monolithic foundation. In order to check the strength of the soft soil layer under the conventional monolithic foundation base, the following formula is applied:

$$\mu = \frac{f_{sp}.A + G - A_s.q_s - f_s.(A - A_1)}{A_1} < f \qquad \text{(E-65)}$$

Where,

f_{sp} - compression force of the base surface of the conventional monolithic foundation (kPa);

G - weight of conventional monolithic foundation (kN);

A_s - area of the lateral surface of conventional monolithic foundation (m^2);

q_s - average friction on the lateral surface of conventional monolithic foundation (kPa);

f_s - allowable bearing force of soil near the edge of conventional monolithic foundation (kPa);

A_1 – area of the base surface of the conventional monolithic foundation (m2);

f – allowable bearing force of the foundation base after adjusting the base surface of the conventional monolithic foundation (kPa).

b) Calculation of deformation:

Calculation of the deformation of composite foundation soil with load-bearing soil-cement piles must include the total contracting deformation of soil-cement pile cluster and deforming contraction of the non-reinforced soil layer under the pile tip. The value of contracting deformation of the soil-cement pile cluster can be based on the structure of the upper part, pile length, pile body strenght etc. the values of 20-40 mm is taken by experience. The value of contracting deformation of the non-reinforced soil layer under the tip of soil-cement piles is calculated in the same way as non-reinforced natural base soil.

2. Computation from the viewpoint of equivalent base

This viewpoint is appropriate when the soil-cement piles for the base reinforcement of filling soil mass are used. The density of piles then should be approximately 12 ~ 20%, also the cement content should not exceed 300 kg/m^3 of pile. This is for the purpose of limiting the large difference between the foundation and the filling soil mass above.

Foundation after being reinforced is considered homogeneous with the improved strength data of $\varphi_{tđ}$, $C_{tđ}$, $E_{tđ}$ compared with the values of φ, C, E of foundation soil before the reinforcement. The equivalent conversion formula is based on the stiffness of soil-cement piles, soil and the area of soil replaced with soil-cement piles.

Designating m as the ratio of the area of replacing soil-cement piles to the area of foundation soil:

$$m = \frac{A_p}{A_s} \qquad \text{(E-66)}$$

$$\varphi_{tđ} = m\varphi_{pile}+(1-m)\varphi_{base} \qquad \text{(E-67)}$$

$$C_{tđ} = mC_{pile}+(1-m)C_{base} \qquad \text{(E-68)}$$

$$E_{tđ} = mE_{pile} + (1-m)E_{base} \qquad \text{(E-69)}$$

Where A_p – Area of foundation soil replaced with soil-cement piles (i.e., area of occupying soil-cement piles);

A_s- Area of foundation soil to be reinforced

By this computing method, the following two criteria are needed to check in the soil improvement problem:

- Criteria of strength: $\varphi_{tđ}$, $C_{tđ}$ of reinforced foundation must satisfy the condition of bearing capacity under the acting of structural load.

- Criteria of deformation: Deformation modulus of the reinforced foundation $E_{tđ}$ must satisfy the condition of settlement.

Analytical formulae and available geo-technical softwares can be used to solve this problem.

3. Input data used for computation

The input data requires not only the physico-mechanical properties of the non-reinforced base soil, but also the data on 90-day soil-cement piles. Necessary test should be conducted in order to provide the data for the consultants. The best way is to conduct the field test with in situ soil. However, by doing so, time and finance are required. Therefore, the standard allows taking in situ soil sample in order to mix the prepared samples indoors. The criteria used in the calculation is taken from the indoor sample test and appropriate reduction (usually 2 times) based on the experience of the designer.

In fact, in case the conditions of testing is not available (indoors and in situ), the designer can use the results of similar structures. However, field tests should be conducted before mass construction in order to adjust the design.