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The Influence of Changes of Soil Parameters due to Consolidation on the Interaction of Piles and Soft Soil Layer

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Abstract

The problem of determination of lateral earth pressure of the soft soil on piles related to the safe design of construction founded on deep piles is presented in the paper. The examples of lateral earth pressure acting against piles are described, as well as properties and response of non-cohesive, cohesive and organic soils forming a soft layer subjected to unsymmetrical loading. Current approaches related to the determination of lateral earth pressure loading the piles are shown. The influence of consolidation on the change of soil strength parameters is presented as well as the application of own model and laboratory test results to the Winter-Leinekugel proposal of earth pressure calculation. Many years research have been carried out in cooperation with Prof. Helmut Meißner from Kaiserslautern University, Germany under the common project entitled "The lateral earth pressure of soft soil acting against piles" as well as in the frame of grants of Polish Research Council.

Key words: piles, soil parameters, soft soil layer

1. Introduction

The problem considers reliable assessment of the distribution and magnitude of lateral earth pressure acting against the piles due to unsymmetrical loading caused by soft soil layer in the subsoil along part of the pile's length. Unsymmetrical load is usually induced by construction of high road embankment or deep excavation at one side of piling foundation, or due to the loading of the stacking yard by containers as well as by coal or ore dumps, Fig. 1. Improper calculations of later bearing capacity of piles or its complete excluding from the overall assessment of bearing capacity of piling foundations may lead to damages and failures of structures founded on piles or excavations protected by palisades.

In addition, soft soil existing in the ground, characterized by low strength and high compressibility as well as high deformability to the lateral deformation due to unsymmetrical load may reach such a state of stress, which induces a specific

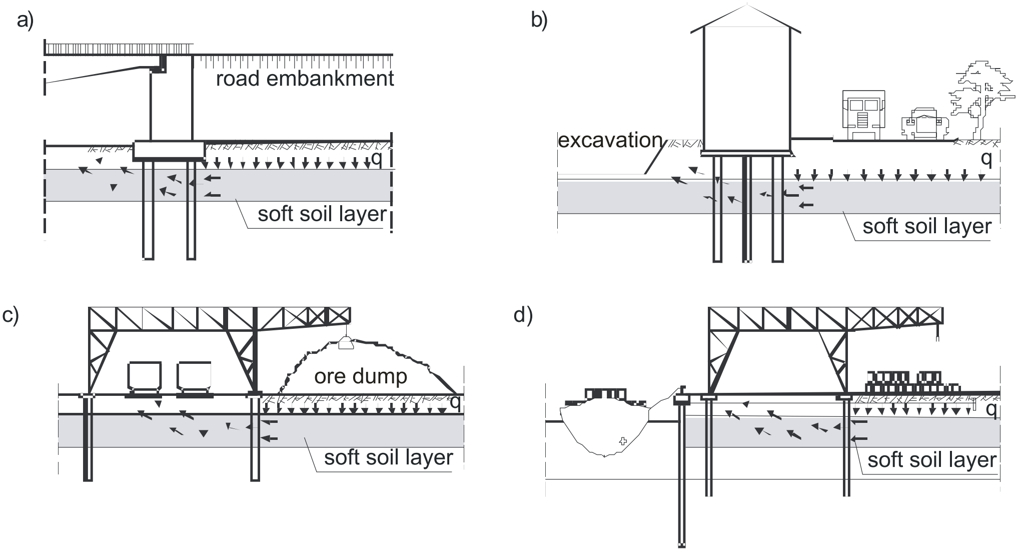


Fig. 1. Examples of lateral earth pressure induced by soft soil layer acting against the piles due to unsymmetrical subsoil loading: a) road embankment, b) excavation near the building on piles, c, d) dumping of the materials near crane railway subgrade

type of active earth pressure higher than passive earth pressure which, in turn, will cause displacement of the soil at the lower stress side.

In loose non-cohesive soil, an increase of vertical stresses in the roof of soft soil layer along one pile side (unsymmetrically) induces quick mobilization of lateral loading acting against the pile which causes its displacement together with supported structure. This phenomenon is accompanied by multidirectional deformation of soft soil layer with significantly higher vertical displacement than a horizontal one and mobilized negative friction.

Cohesive soil loaded unsymmetrically induces more complex loading against the pile. Soft soil layer is compressed and sheared, undergoing spatial deformation. In the plastic state, an increase of vertical stresses in the roof of soft soil layer causes lateral loading acting against the piles which changes its magnitude to a longer period. Initial value of additional earth pressure corresponds approximately to the value of earth pressure at rest and after some time becomes the earth pressure of creeping soil. Increased vertical stresses also cause negative friction and the thickness of soft soil layer becomes gradually lower. On the opposite side of surcharge loading, the process of soil uplift in front of the piles is observed which simultaneously causes initiation of friction forces on pile shafts with an oblique direction. To a certain depth, within contact zone along part of pile shaft the adhesion and passive earth pressure decay – some open voids (cracks) between the soil and the pile are created, Fig. 2. Deformation range is related to the depth of the crack in the subsoil and the magnitude of external loading. At certain depths

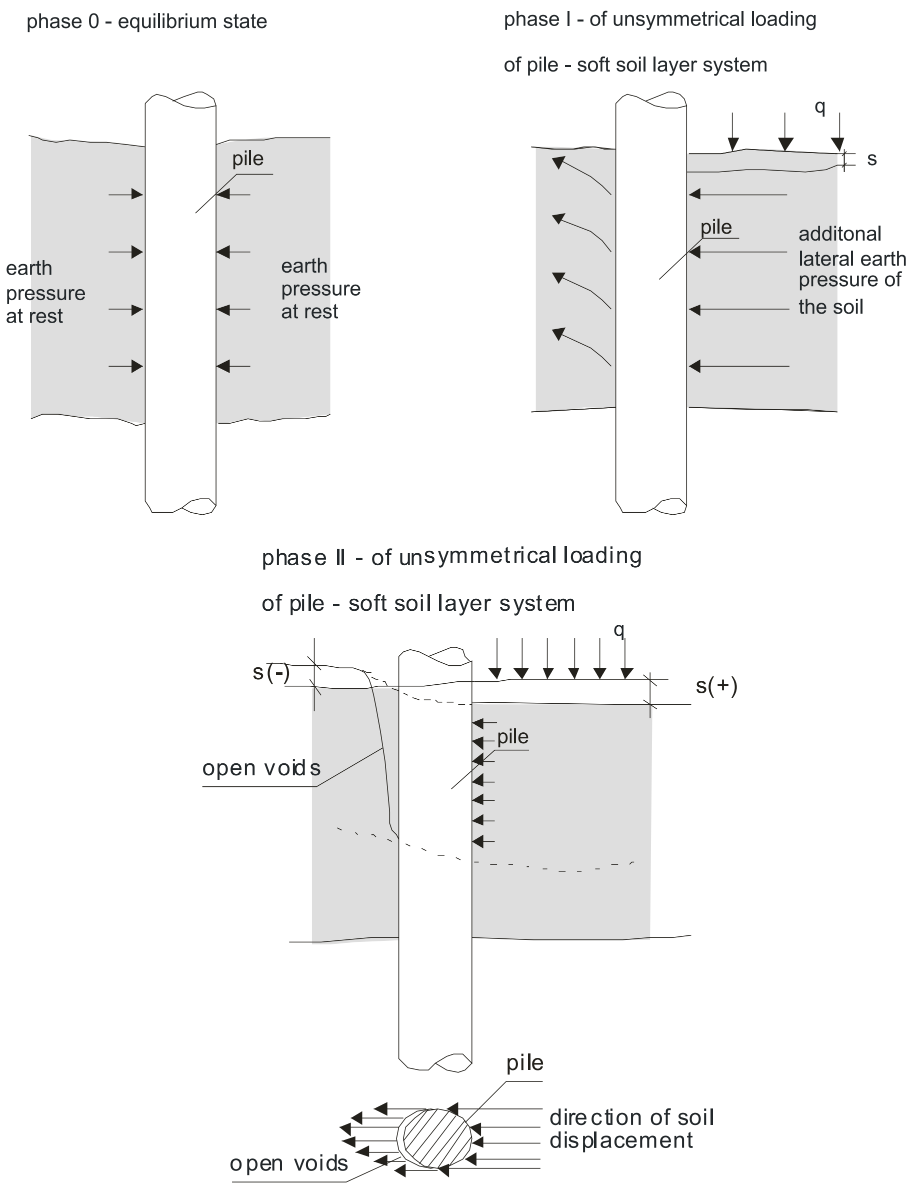


Fig. 2. Simplified deformation scheme of the soil layer with low shear strength around the pile

of the embankment the process described above does not occur, and at a specific relation between soft soil layer strength and height of embankment the initiation of additional lateral loading acting against the pile is observed.

For example, according to Peck et al (1957), for cohesive subsoil with the one-dimensional compressive strength q_u and bearing capacity, around $3q_u$, loaded by embankment, its maximum safe height amounts to

$$h_{\max} = \frac{1.5q_u}{\gamma} \quad [\mathrm{m}], \tag{1}$$

where:

γ – unit weight of the soil in embankment.

According to Terzaghi (1956), for organic subsoil the height is:

$$h_s = \frac{N_c \tau_{fu}}{F\gamma_n} \quad [\mathrm{m}], \tag{2}$$

where:

- N_c – bearing capacity factor for circular slip surface (recommended = 5.52),
- τ_{fu} – shear strength of soft soil in [kPa],
- γ_n – unit weight of the soil in embankment in [kN/m^3],
- F – factor of safety,

and according to Taylor (1956), the embankment height should not exceed

$$h_s = \frac{\tau_{fu}}{(FN_t\gamma_n)} \quad [\mathrm{m}], \tag{3}$$

where: N_t – bearing capacity factor.

Dembicki et al (1983) think that, when the initial shear strength of the soft soil is sufficient for assuring the stability at full loading of the embankment, it can be continuously constructed to the designed height. In the opposite case the embankment should be founded in stages. In the case of pre-consolidated organic soils (the effective stresses in the subsoil several times exceed the initial value of pre-consolidation pressure) it is recommended to apply a method given by Lechowicz (1994) which includes the shear strength change and the influence of the state and history of effective pressure.

In order to eliminate negative lateral impact of soft soil on piles the following solutions guaranteeing safety of the construction are proposed:

- consolidation of the subsoil with soft soil layer before the construction of piling foundation and embankment,

– restriction of embankment's height or dump of deposited material,
– in the case of palisade at the slope toe, in which stability loss can occur, the piles with high rigidity should be preventively installed.

Simultaneously, Steinfeld (1986) recommends excluding the influence of additional earth pressure on piles when the embankment's stability factor $n > 1.5$ (it corresponds to the $I_L < 0.5$ for soft soil). De Beer and Wallays (1972, 1977) relate the presence of additional earth pressure of soft soil with total stability of the embankment expressed by safety factor $F < 1.4$ or $F > 1.4$.

Schmiedel (1984) gives two cases, in which additional earth pressure on piles occurs:

1) where the factor of safety is lower than 1.5, soil moisture content is lower than 75% and unit weight after deep drying decreases to 15%,
2) where the factor of safety is lower than 1.8, soil moisture content is greater than 75% and organic matter losses are also greater than 15%.

2. Shear Strength Characteristics of Soft Soils Laterally Loading the Piles

According to Japanese (Ito et al 1975, 1982) and Russian (Luga 1962, 1971) recommendations, the lateral earth pressure on piles is caused by sand and gravel with various density, as well as soft and low plasticity clays, thus a majority of non-cohesive and cohesive soils. Different impact of these soils on piles results from different (short and long-term) mechanical properties – shear and compression strengths. For a determination of shear strength of sands, Chen et al (1993), Kirkpatrick (1957), Sutherland, Messdary (1979), Arnold, Mitchel (1973), Goldscheider (1979), Gudehus (1984) and others confirmed a suitability of Coulomb-Mohr criterion. Brinch-Hansen and Lundgren (1960) elaborated the formula for calculation of internal friction angle of loose sandy soils, (Eq. 4). However, in the Author's opinion, in the case of loose sands, application of this criterion as well as determination of friction angle values by formula (4) is merely an approximation and the formula requires a modification with regard to reliable determination of Φ_3 and Φ_4 components

$$\Phi=26^{\circ}+\Phi_1+\Phi_2+\Phi_3+\Phi_4, \tag{4}$$

where:

Φ_1 – shape of grains,
Φ_2 – dimensions of grains,
Φ_3 – degree of compaction and
Φ_4 – density index.

The lateral interaction between piles and the soil is influenced by both pile geometry (its rigidity, shape, space and arrangement of piles in the group) as well as soil properties and the depth of soft soil layer, degree of saturation, the stress difference along both pile sides, state of the soil and rate of unsymmetrical loading of the subsoil.

Additionally, in the case of:

- non-cohesive soils the important role is played by grain size distribution and porosity of the soil, negative friction along the pile shaft and the range of vertical deformation,
- cohesive soils – adhesion related to cohesion and viscosity of the soil and the shape and range of lateral soil deformation which changes in time.

The loading in granular soils induces two mechanisms of small deformations which finally cause large deformations of soil layer taking place in a relatively short time and influencing the lateral interaction and, in turn, the lateral loading acting against the piles. Two main mechanisms are the following:

- deformation and crushing of particular grains,
- relative movement between grains due to slipping and rotation.

In loose soils with large grains, angular and uniformly graded, the process of crushing and breaking results in smaller critical stresses at low strength of particular mineral grains. For critical stresses, the elastic deformation of single grains is initiated first and then the process of slipping (Whitman et al 1964). The larger initial stresses and smaller void ratio the larger stresses for initiation of slipping are required. According to Lambe and Whitman (1978), when the sand cannot laterally deform the following phases may occur:

- compaction (wedging) and crushing of grains at the contact surfaces with slipping for increased stresses,
- gradual breaking of grains combined with significant movement and size-reduction,
- further compaction of crushed soil and decrease of unit pressure on a larger number of contacts.

In the Author's opinion, in the case of occurrence of one or multi-dimensional lateral expansion of sand, depending on soil gradation, some slightly different phases take place, namely:

- phase of partial compaction – clear compression of soil layer combined with momentary disturbance of pore water pressure,

– gradual displacements of grains in soil layer combined with its rotation and slipping in the direction of a zone with lower stresses in a given plane and dissipation of pore water pressure.

Cohesive soils subjected to loading are characterized by another deformation process and for specified content of bound water its particles move between each other without breaking of continuity and interaction with piles in a long-term period. The cohesion manifests within moisture content range, when the dry soil looses its strength and does not reach the liquid state. Additionally, grain size distribution, among other factors (i.e. mineral composition), is one of the more important factors influencing the soil plasticity, the properties of which occur when there are particles of a diameter less than 5 μm.

Cohesive soils reveal highest strength at fast increase of loading. For a period of stress action shorter than relaxation time, elastic strains develop mainly in the soil whereas over a longer period – creep and yielding of soils take place, (relaxation time – period, in which the change of stresses is observed at constant strains). They can occur in the case of unsymmetrical loading of soft soil layer due to embankment weight. Thus, the cohesive soil behaves as a solid body or body resembling liquid, in which at small stresses steady flow initiates with constant viscosity which does not change during stress increase. That increase results in a decrease of plastic viscosity. In plastic soils, the creep process occurs already at small shear stresses of the order of several dozen of kPa. The creep is determined by apparent viscous slipping of one particle over an other, its re-orientation towards normal direction to the resultant shear stresses as well as the development of microcracks. Kinematics of creep process depends on stress and temperature magnitudes. In addition, it is also influenced by the change of compaction, strength increase in the phase of creep decay, etc. According to Maslow (1968), the creep limit (one of the parameters, besides viscosity, required in the creep prediction) represents such shear stresses, at which or above which an increase of shear strains is observed. It depends on the fabric and composition of the soil, temperature and stresses. For dense soils, the creep limit is higher than for looser ones. The increase of compaction causes an increase of viscosity and the breakage of clay structural bounds results in significant decrease of viscosity.

In cohesive soils with low permeability, total increase of stresses is overtaken by water whereas effective stresses in the soil skeleton remain unchanged and the shear resistance does not increase. Internal friction angle ϕ determined by Coulomb formula is lower than effective friction angle ϕ'. Pore water pressure increment does not influence the shear resistance of the soil. The increase of shear resistance is caused by the increment of effective stresses which for cohesive soils is strongly dependent on consolidation time. For a given normal load σ_n the shear resistance and effective stresses reach its maximum values and simultaneously pore water pressure minimum values at the end of consolidation process which is a function of time the external load is applied as is consolidation time causing the decrease of porosity.

3. Geotechnical Parameters used in Calculations of Lateral Load Acting against the Piles

3.1. Constants Describing the Lateral Soil Reaction

In order to better understand the lateral interaction between piles and the soil comprehensive, various scale investigations have been carried out for over 100 years. The investigations have aimed at determination of the values of coefficients of lateral interaction, deflection of piles, distribution of bending moments and the deformation of the ground around the piles due to surcharge pressure. Reliable description of the phenomenon allows the optimum design of the lateral bearing capacity of piles. First approach was related to a determination of c constant based on k elastic characteristics of the soil

$$c = kD \quad [\mathrm{N/cm^2}], \tag{5}$$

where: D – diameter of the pile cross section in [m].

The history of the investigations has been presented in Table 1.

Table 1. Collection of the investigation results of lateral soil reaction

Author	Results		Notes
Boussinesq (1885)	$C = \dfrac{8\pi\ G_B}{2 + \log 64\pi^4 \frac{E_p J}{D^4} - \log c}$		G_B – shear modulus
Forsell (1926)	$c = 1.0D \quad [\mathrm{kN/cm^2}]$		c determined for sands
Walter (1951)	$c = 0.43E_B$		E_B – soil deformation modulus
Terzaghi (1956)	$k = k_R z$		z – depth, k_R – empirical reaction coefficient
	Q_s [MN/m^2]	k_R [MN/m^2]	
	$5 \div 10$	2	
	$10 \div 15$	6.5	
	> 15	18	
Bergfelt (1957)	$c = 15 \div 25\tau$		τ– shear strength of clay
Rinkert (1960)	$c =< 8\tau$		τ– shear strength of soil
Sherif (1974)	$k = 160c_u$		k – empirical reaction coefficient c_u – undrained cohesion [kPa]
	State	k[MN/m^2]	
	plastic	8	
	of low plasticity	16	
	of very low plasticity	32	

Besides the soil reaction coefficients and deformation modulus, in the analysis of the interaction between pile and cohesive soil, many authors determined theoretically or experimentally bearing capacity factor N_c, Table 2.

Table 2. Collection of N_c factors

Pile cross-section	Brinch-Hansen (1961)	Wenz (1963, 1972)	Schenck, (Smoltczyk) (1955)	Horch (1980)	German KM Gi F (Fedders 1977)
Squared	$N_c = 7.5$	$N_c = 8.3$	$N_c = 3.4$	$N_c = 7.0$	$N_c = 7.5$
Circular	$N_c = 6.4$	$N_c = 7.0$	$N_c = 2.6$	$N_c = 7.0$	$N_c = 7.5$

3.2. Geotechnical Parameters used in Various Proposals for Calculation of Lateral Earth Pressure Acting against the Piles

In existing proposals for determination of lateral earth pressure acting against the piles (Δe) the authors incorporate various assumptions and sometimes different approaches regarding the values of geotechnical parameters. The most popular can be classified into three main groups:

1) proposals in which constant geotechnical parameters are used (γ, ϕ and c_u) for calculation of lateral earth pressure Δe as the difference between the active e_a and passive earth pressure e_p of the soil with linear distribution (Coulomb 1773);
2) proposals, in which variable values of soil parameters such as elastic modulus E_s along the pile shaft, soil deformation velocity, creep are applied;
3) proposals, in which mathematical models concerning many soil features and special computer codes for complex analysis of soil-pile lateral interaction as well as stability of constructions founded on piles, are applied.

An example belonging to the first group mentioned is the recommendation contained in Polish Piling Code PN-83/B-02482, according to which the additional earth pressure of the soil acting against the piles is induced by layer of silty sand in a loose state with density index I_D lower than 0.33 as well as mud and cohesive soil layers with liquidity index I_L within the range from 0.5 to 1.0, Fig. 3. Geotechnical parameters used in that proposal are mostly read out from the diagrams or determined during geotechnical investigations as constants:

$$\Delta e = \eta(e_a - e_p), \tag{6}$$

where: η – reduction coefficient depending on relative pile spacing in the direction perpendicular to earth pressure action.

Calculations of additional earth pressure according to Polish Code can be made by "PARGRUNT" numerical code elaborated in the framework of Polish Research Council grant (Kurałowicz, Janczewski 1995).

Additionally, according to Ito and Matsui (1975) and Ito et al (1982) proposals, in which soil partial yielding near the pile toe is assumed, Fig. 4, the unit earth pressure for cohesive and non-cohesive soils can be calculated according to the following formulae:

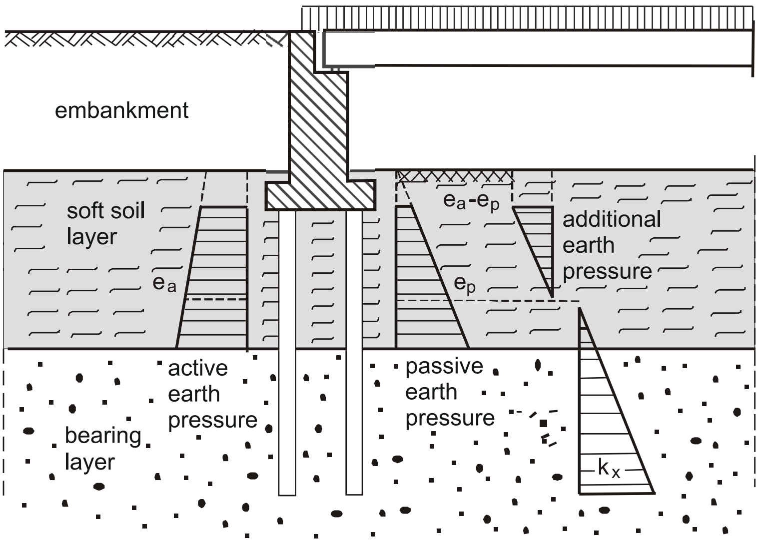


Fig. 3. Distribution of active and passive earth pressures and additional earth pressure acting against the piles, according to PN-83/B-02482

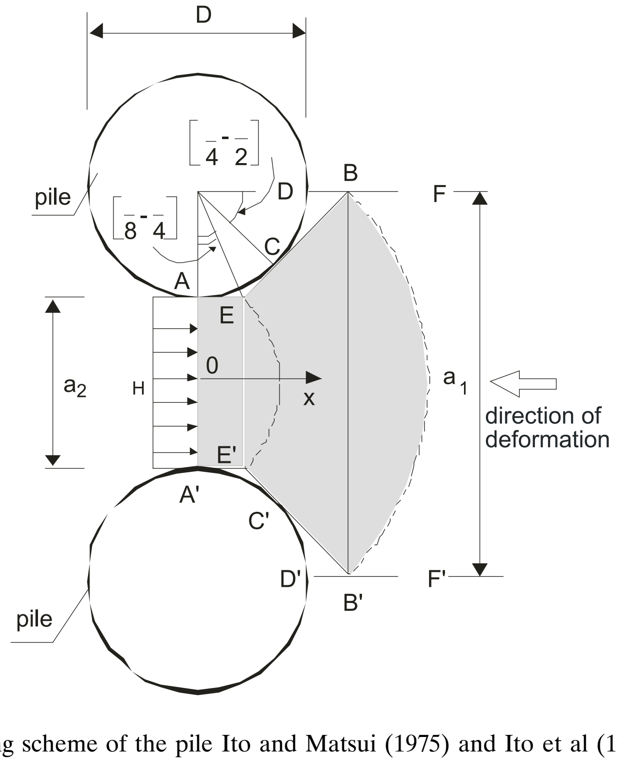


Fig. 4. Loading scheme of the pile Ito and Matsui (1975) and Ito et al (1982) proposal

for cohesive soils:

$$q = Ac_u\left(\frac{1}{N\phi^{*}\tan\phi}\left\{\exp\left[\frac{a_1-a_2}{a_2}N_\phi\tan\phi\tan\left(\frac{\pi}{8}+\frac{\phi}{4}\right)\right]-2N_\phi^{1/2}\tan\phi-1\right\}+\right.$$
$$\left.+\frac{2\tan\phi+2N_\phi^{1/2}+N_\phi^{-1/2}}{N_\phi^{1/2}\tan\phi+N_\phi-1}\right)-c_u\left(a_1\frac{2\tan\phi+2N\phi^{1/2}+N\phi^{-1/2}}{N_\phi^{1/2}\tan\phi+N_\phi-1}-2a_2N\phi^{-1/2}\right)+$$
$$+\frac{\gamma z}{N_\phi}\left\{A^{*}\exp\left[\frac{a_1-a_2}{a_2}N_\phi\tan\phi\tan\left(\frac{\pi}{8}+\frac{\phi}{4}\right)\right]-a_2\right\}, \tag{7}$$

for non-cohesive soils:

$$q = \gamma\cdot\frac{z}{N_\phi}\left\{A\exp\left[\frac{(a_1-a_2)}{a_2}\cdot N_\phi\cdot\tan\phi\cdot\tan\left(\frac{\pi}{8}+\frac{\phi}{4}\right)\right]-a_2\right\}, \tag{8}$$

and passive earth pressure p:

$$p = c\cdot a_1\left[3\cdot\ln\left(\frac{a_1}{a_2}\right)+(a_1\cdot a_2)\cdot\tan\left(\frac{\phi/8}{a_2}\right)\right]+\sigma_H\cdot(a_1-a_2), \tag{9}$$

where:

- c_u – soil cohesion in [kPa],
- a_1 – pile spacing between its axes in [m],
- a_2 – actual distance between piles in [m],
- z – depth from the ground surface in [m],
- γ – soil unit weight in [kN/m^3],
- ϕ – internal friction angle of the soil [$^\circ$],
- σ_H – active earth pressure in [kPa],
- q – unit later earth pressure in [kPa]

and

$$A = a_1\left(\frac{a_1}{a_2}\right)^{N_\phi^{1/2}\tan\phi+N_\phi-1}, \tag{10}$$

$$N_\phi = \tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right). \tag{11}$$

Apart from that, in the same group the following proposals can be mentioned:

- Steinfeld (1986), who determines earth pressure of the soil and corresponding small passive earth pressure (close to earth pressure at rest) assuming small soil displacements. He thinks that additional earth pressure acting against the piles cannot be higher than the difference between active earth pressure acting on the surcharge side and passive earth pressure on the opposite side (Δe) or than the value of earth pressure of creeping soil $p_f = (7 - 10)c_u D$. Besides standard parameters he also concerns shear strength S_u of soft soil and suggests the reduction of earth pressure depending on the embankment height and the distance between piles in the soil;
- De Beer and Wallays (1972, 1977) propose to include geostatic pressures and soil strength based on CPT tests;
- Morarieskul (1979) – distribution of load on piles determined on the basis of Bussinesq assumptions and solutions;
- Luga (1962, 1971) – who takes into account constant coefficients of horizontal stress distribution depending on type and state of the soil;
- Brinch-Hansen (1961) – who uses geotechnical parameters (internal friction angle in relation to relative depth of the pile z/D for calculation of K_q and K_c coefficients);
- Broms (1964, 1972) includes two cases of a subsoil with constant parameters $c_u\phi, \sigma_z'$;
- Wenz (1963) who gives the formula for maximum force acting against the single pile as a sum of constant lateral force P_0 depending on development of yielding zones around the pile with undrained constant shear strength S_u and on earth pressure of creeping soil P_f;
- Fedders (1977) who applies Prandtl-Hill's (1920) and Brinch-Hansen-Lungren's (1960) solutions for determination of earth pressure of creeping soil on the basis of known constant value of soil shear strength, s_u, N_c.

In the second group of proposals concerning changeable values of geotechnical parameters the following methods should be listed:

- Poulos (1980, 1995) who considers changeable elastic modulus E_s and Poisson's ratio ν along the pile. He assumes that lateral earth pressure varies in the soil layer and is limited by $P\gamma$ value i.e. known magnitudes of free soil displacement and its conformity with pile displacement, value of surcharge pressure and limit conditions;
- Marche (1973) who suggests to calculate the lateral earth pressure for the distributions shown in Fig. 5 according to the following formula

$$p = (y_z - \delta_z)k_{hB}, \tag{12}$$

where:

k_{hB} – modulus of soil lateral response at specified depth,
y_z and δ_z – soil and pile displacement.

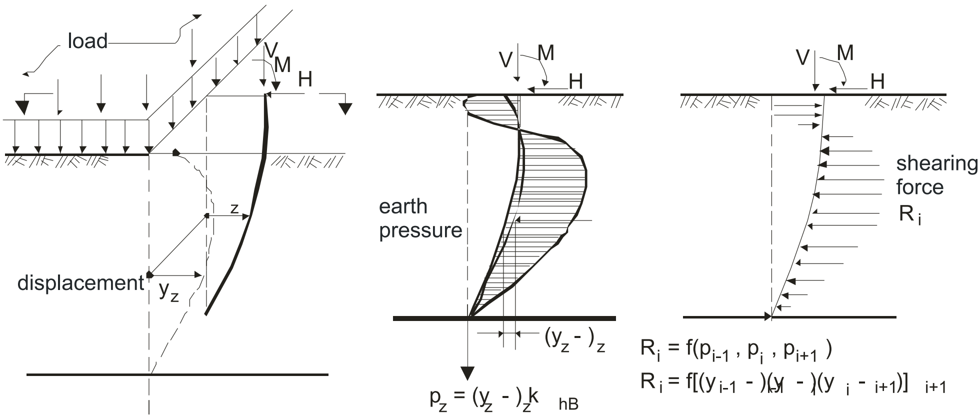


Fig. 5. Scheme of pile displacement and assumption for Marche method (1973)

– Frank (1981) – (French Bridges and Roads Laboratories – LPC method) who extended Marche's proposal applying "equilibrium equations in numerical analysis of beam" loaded by soil earth pressure determined for geotechnical parameters dependent on its deflection (deflection of piles)

$$\frac{dTMd^2}{dzEJdx^2} = -P(z) \quad \text{and} \quad \frac{M}{EJ} = \frac{d^2y}{dz^2}, \tag{13}$$

$$\frac{EJd^4y}{dz^4} + P(z) = 0, \tag{14}$$

where:

T – shearing load in [kN],
M – bending moment in [kNm],
EJ – pile (beam) flexural rigidity in [kNm^2],
y – pile displacement in horizontal direction in [m].

New element in this calculation proposal is related to the determination of earth pressure value p as a function of relative displacement $\Delta y = y - g$, which for linear distribution (Winkler's model) can be calculated based on the following formula

$$p = k_s(y - g) \text{ and } P = pD = E_s(y - g), \tag{15}$$

$$E_s = k_s D, \tag{16}$$

where:

k_s – classical coefficient of soil deformability,
D – pile diameter in [m],
g – "free" displacement of the soil,
E_s – being now the secant modulus on the reaction curve at a given depth.

A solution for each level requires an iteration method using following differential equation

$$\frac{EJd^4y(z)}{dz^4} + E_s^t y(z) = E_s^t g(z) - P_0, \tag{17}$$

where:

$E_s^t(y - g)$ – the tangent modulus,
$P_0(y - g)$ – the ordinate in the origin,

The coefficients E_s^t and P_0 of the layer at iteration i are determined from the displacement y at the centre of the layer calculated at iteration $i - 1$;

- French recommendations contain regulations regarding a determination of unbounded horizontal deformation of soft soil layer subjected to embankment loading. The calculation procedure of the displacements is a function of depth z and time t which in further calculation step are used for evaluation of earth pressure of the soil on piles. $t = 0$ corresponds to the end of embankment's formation while it is done relatively quickly whereas the stability factor is greater than 1.5 ($\Delta g_{\max}(t)$ depends on settlements along the embankment's axis);
- Goldscheider (1979) and Gudehus (1984) propose the following formulae for calculation of lateral earth pressure

$$p = 4\pi D s_u, \tag{18}$$

for $D/a < 0.59$:

$$s_u = \tau_{f0}(1 + \Sigma_0 \ln V), \tag{19}$$

and for $D/a < 0.6; 0.2 >$

$$\Sigma_0 \ln V = \left(\sigma_e \theta / \tau_{f0}\right) \ln v_0 / [(a - D)\varepsilon_0], \tag{20}$$

where:

Σ_0	–	material constant,
V	–	dimensionless creep rate,
θ	–	soil viscosity determined experimentally,
v_0	–	creep rate,
ε_0	–	soil deformation corresponding to soil creep rate;

– Leinenkugel (1976) and Winter (1979), suggest to include in calculations void ratio, moisture content and strain of creeping soil (ε_a) as a function of velocity

$$p_f = Dc_{u\alpha} k \left[1 + I_{v\alpha} \ln\left(\frac{v_0/\dot{\varepsilon}_\alpha}{a - D}\right)\right], \tag{21}$$

where:

$k = 4.83\,(2.76D/a + 1)$	–	Winter's empirical coefficient for *D*/*a* range from 0.1 to 0.5,
$c_u = c_{u\alpha}\left(1 + I_{v\alpha} \ln \dfrac{\dot{\varepsilon}}{\dot{\varepsilon}_\alpha}\right)$	–	undrained shear strength $\dot{\varepsilon}_\alpha$,
$I_{v\alpha} = 0.6 + 2.6 \ln(w_L/20)$	–	coefficient of viscosity corresponding to strain rate, depending mainly on the type of soil and liquid limit [w_L in %] (approximated value).

The third group which is based on modern approach related to comprehensive analysis of the problem, consists of the following proposals:

– Baguelin, Frank, Said (1977) assumed circular plane of *R* diameter describing the soil as elastic material around the pile, Fig. 6. Earth pressure as a function of unit pile deformation is given by the following relationship

$$\frac{p}{G} = \frac{\left(\rho/D\right)}{G} = fn(\nu, (R/D))(\delta/D), \tag{22}$$

where:

$p = (\rho/D)$	–	equivalent active earth pressure for 1.0 m length along the pile axis,
ρ	–	force acting on 1.0 m length along the pile axis,
G	–	shear modulus for elastic material,

ν	–	Poisson's ratio for elastic material dependent on drainage conditions,
D	–	pile diameter,
δ	–	relative pile displacement referred to plane edge,
R/D	–	coefficient assumed arbitrary on the basis of experiments;

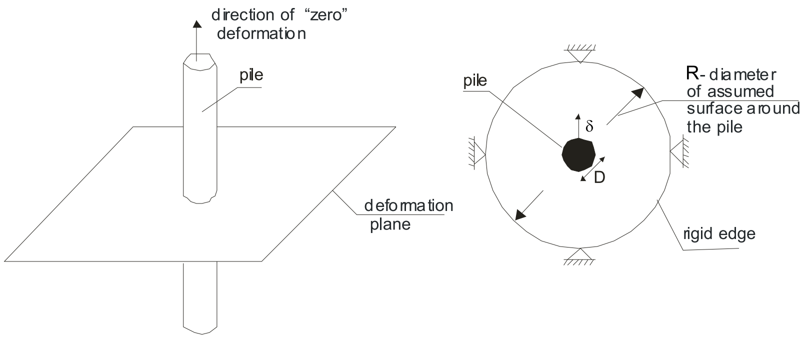


Fig. 6. Calculation model for pile and surrounding soil, acc. to Baguelin et al (1977)

– Randolph (1981) assumed equivalent system in the form of continuous sheet pile wall (Fig. 7) with equivalent rigidity EJ containing rigidity of piles and soil in between;

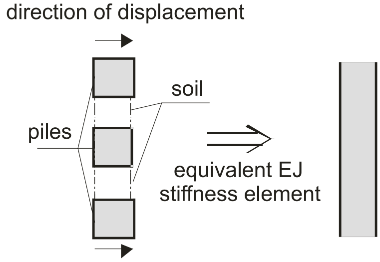


Fig. 7. Actual and equivalent Randolph's (1981) and Naylor's (1982) schemes

– Ratton (1985), in calculations of lateral earth pressure applied 3D analysis based on finite linearly elastic elements method. However, due to too many restrictions such as soil reology and change of geotechnical parameters he suggests to use 2D model, e.g. in the form of continuous sheet pile wall;

– relying on centrifuge tests (for the group of steel H piles), Steward, Jewell and Randolph (1993, 1994) proposed the following relationship for the determination of lateral earth pressure as a function of surcharge depth

$$p_m = \frac{q}{\frac{8DG_m}{(h_s-h_1)/2}\left[\frac{1}{5.33G_r} + \frac{n_r h_1 (h_s-h_1/2)}{8s\,h_s\,G_m} + \frac{F_1}{24EJ}\right]}, \tag{23}$$

where:

q	–	embankment load in [kPa],
D	–	pile diameter in [m],
G_m	–	reduced soil modulus in [kPa],
h_1	–	height of additional earth pressure in [kPa],
h_s	–	thickness of soft soil layer in [m],
n_r	–	number of piles,
s	–	pile spacing across the face of the embankment.

They recommend to calculate the length of loading section h_1 by lateral earth pressure acting along the pile using following formula

$$\frac{\left[(h_s-h_1)/2\right]F_2-(h_s-h_1)F_1}{24EJ} - \frac{(h-h_1)}{5.33G_r} = 0, \tag{24}$$

$$M_q = \frac{\Delta M_{\max}}{\Delta q\; dL_{eq}^2}, \tag{25}$$

$$y_p = \frac{\Delta y\, E_p I_p}{\Delta q\; dL_{eq}^4}, \tag{26}$$

$$K_r = \frac{E_p I_p}{E_s h_s^4}, \tag{27}$$

where:

L_{eq}	–	length of pile between supports in [m],
$L_{eq} = L$	–	for double sided fixed piles in [m],
$L_{eq} = 0.6L$	–	for pinned head piles (cantilever beam) in [m],
$L_{eq} = 1.3L$	–	for free-head piles in [m],
E_p	–	Young's modulus of a pile material in [kPa],
I_p	–	moment of inertia for pile's cross-section [m^4],
E_s	–	stiffness modulus of the soft soil in [kPa],
h_s	–	thickness of the soft soil layer in [m].

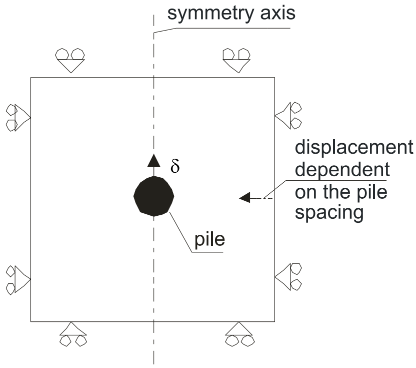


Fig. 8. Bransby and Springman's (1996) calculation scheme

– Bransby and Springman (1996) assumed squared area around the pile, Fig. 8. Next, basing on finite element analysis they proposed the following relationship between pile deformation and stresses for undrained conditions

$$q = x\varepsilon_q^y, \tag{28}$$

$$p = xX\,(\delta/R)^y\ , \tag{29}$$

where:

q	–	stress deviator,
ε_q	–	strain deviator,
x, y	–	parameters defining constitutive laws,
$X = a$	–	scale factor dependent on y and ν, a parameter relating stresses and strains or loads and displacements,
δ	–	pile displacement,
R	–	width and length of the assumed area around the pile,
p	–	coefficient relating loads and displacements;

– Wakai et al (1997) and Akai et al (1994) have used the 3D model presented in Fig. 9.

– Ellis and Sprigman (1997, 2001), in their analysis of lateral loading of abutment pile foundation have also applied 3D finite element method, Fig. 10.

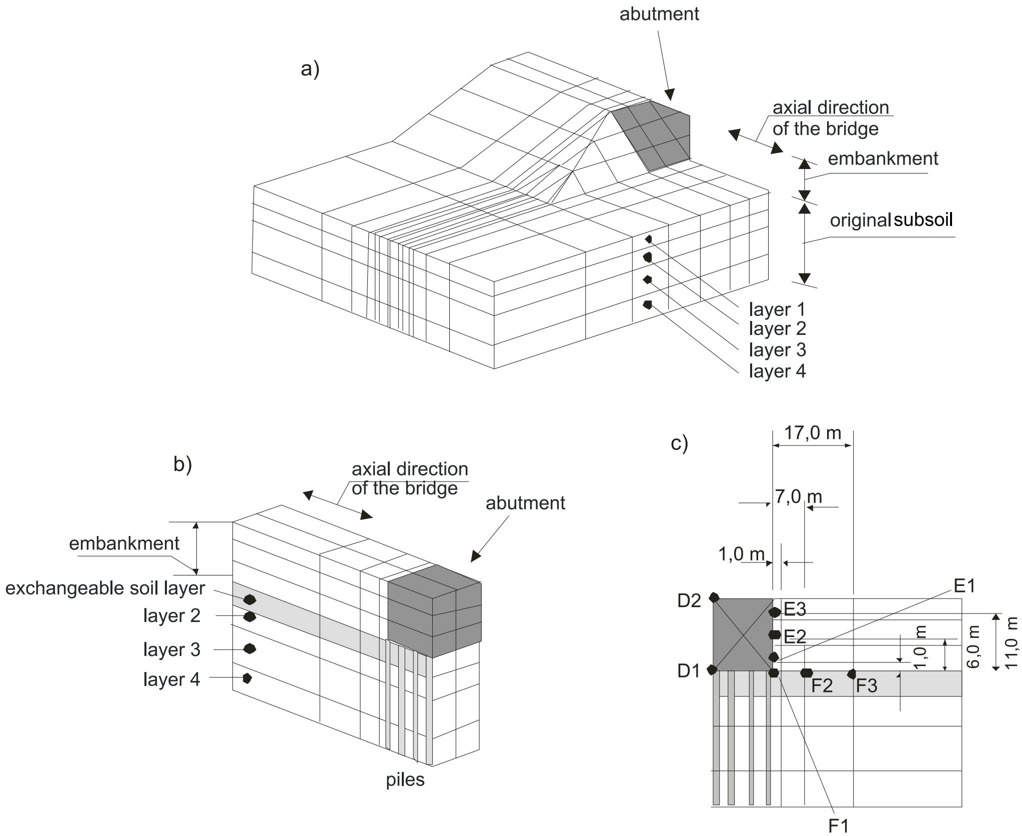


Fig. 9. 3D model for the embankment-subsoil-piles system, acc. to Akihiko et al (1997) and Akai et al (1994)

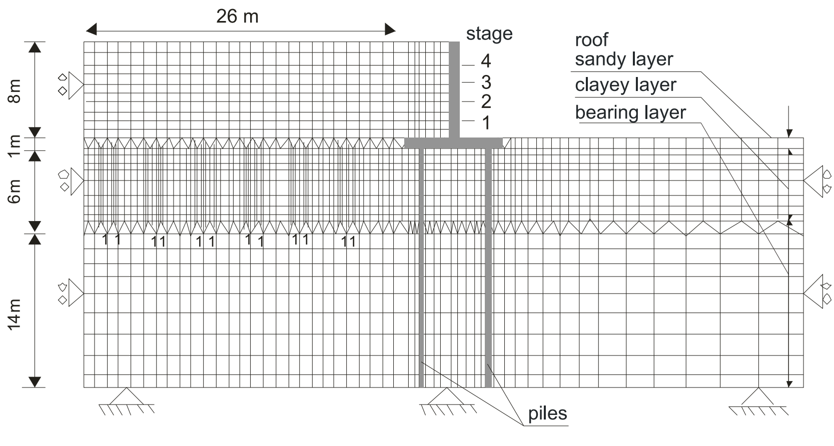


Fig. 10. Finite element mesh for embankment-piling foundation-subsoil system, acc. to Ellis and Springman (1997, 2001)

4. The Influence of the Improvement of Soft Soil Layer on Lateral Loading on Piles

4.1. The Influence of Consolidation on the Improvement of Soft Soil Layer and Earth Pressure Acting against the Piles

Steinfeld (1986) states that the consolidation process occurring in soft soil layer due to one-side surcharge, causing horizontal displacement larger than 3.0 cm, eliminates additional earth pressure acting against the piles. De Beer (1972), in turn, suggests to make the alternative calculations of the lateral earth pressure on piles and to assume values lower than soil earth pressure or than creeping soil earth pressure. For designed bearing capacity of piles he gives schematic process of additional earth pressure change of soil in time, Fig. 11.

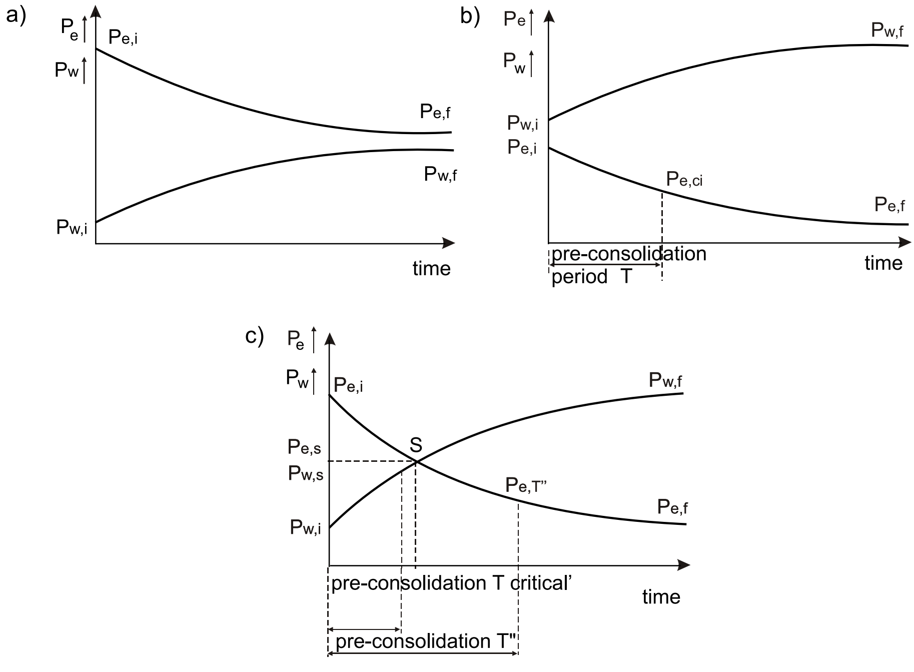


Fig. 11. Theoretical relation between total soil earth pressure and earth pressure of creeping soil and time, acc. to De Beer (1972)

He also think that the total soil earth pressure will decrease due to the increase of shear strength and consolidation process. Simultaneously, this increase will also cause the increase of the load from creeping soil earth pressure, Fig. 11. Additionally, based on theoretical analysis of various alternatives regarding the occurrence of lateral earth pressure acting against the piles he states that, depending on the state of soft soil layer and time, the consolidation process will not always positively influence the decrease of lateral earth pressure and assumption of undrained shear strength only is sufficient. In turn, in Schmidt's (1989) opinion, the factor deciding as to the initiation of lateral earth pressure acting against the piles is consolidation time. Schmiedel (1984) determines lateral loading Δq based on consolidation degree U and effective strength parameters ϕ' and c' of the soil.

Assumed deformation process of soft soil layer due to unsymmetrical external load near piles requires atypical theoretical description and solutions based on suitable calculation model. Basic models describe elastic and plastic properties of soil whereas its modifications – various deformation phases under applied load. Detailed characteristics and classification of these models one can find in works of Jardine, Hight (1987), Perzyna (1966), Mróz et al (1980), Glazer (1985), Gryczmański (1994, 1997), who presented theoretical fundamentals in geotechnics and showed directions of its further development as well as in works of Pietruszczak, Stolle (1986), Jarzębowski (1990), Niemunis (2003) and Lechowicz.

The basic condition enabling the improvement of soft soils is proper choice of loading and its rate which should assure safe consolidation of a subsoil. Assumption of too high values of loading and loading rate will cause the occurrence of an excess of pore pressure at relatively low increase of effective stresses. In consequence, the improvement of organic soil will not occur (Jamiołkowski et al 1981).

The main factors which influence the process and magnitude of the increase of organic soil shear strength due to surcharge are the following:

- loading scheme: magnitude, distribution and process of loading,
- ground geometry: thickness, layering and arrangement of layers,
- drainage conditions: boundary conditions and the length of filtration way,
- soil properties: type of the soils, initial shear strength and permeability, porosity,
- stress state and history: $(\sigma_v')_0$, $(\sigma_p')_0$, K_0^{nc}.

When analysing the interaction between piles and soft soil the initial state of the soil and prediction of shear strength changes with time in actual conditions should be considered. One way to describe the changes of undrained shear strength τ_{fu} is its decomposition onto an initial value τ_{f0} and its increment $\Delta\tau_{f'}$ caused by consolidation process (Lechowicz 1986):

$$\tau_{fu} = \tau_{fo} + \Delta\tau_f. \tag{30}$$

According to the French Laboratory for Roads and Bridges, total shear strength increment $\Delta\tau_{fc}$ can be calculated from the following relationship:

$$\Delta\tau_{fc} = \tan\alpha_{cu} \cdot \Delta\sigma_v', \tag{31}$$

where:

α_{cu} – angle of consolidation increment in [°],
$\Delta\sigma'_v$ – total increment of vertical component of effective stress in [kPa].

The relationship (31) is based on the assumption that shear strength increment for an arbitrary time of consolidation process, being the linear function of degree of consolidation U, is calculated according to Eq. (Dreyfus 1971):

$$\Delta\tau_{ft} = \tan\alpha_{cu} \cdot U \cdot \Delta\sigma'_v. \tag{32}$$

Przystański (1980) has determined empirical relationship for peat characterized by degree of decay $R = 19$–25% which enables calculation of shear strength increments as a function of time t in the following form:

$$\Delta\tau_{ft} = (A_0 + A_t\sqrt{t})\cdot\Delta\sigma'_v, \tag{33}$$

where:

t – consolidation time in [s],
A_0, A_t – empirical coefficients.

Using one of the above mentioned equations, Lechowicz suggests to determine the shear strength increment for an arbitrary consolidation time $\Delta\tau_{ft}$, depending on degree of improvement U_τ, using the following formula:

$$\Delta\tau_{ft} = \Delta\tau_{fc}\cdot U_\tau. \tag{34}$$

In presented relationships the influence of secondary compressibility on the shear strength has been neglected. This factor has, however, been included in the method proposed by Lechowicz (1994), which is based on reological model.

4.2. Test Results of the Improvement of Strength Parameters of Clay

In the period from 1994 to 2003, in the Soil Mechanics Laboratory at Kaiserslautern University, the Author had carried out comprehensive model tests of piles loaded laterally as well as long-term laboratory tests of some clay properties such as:

- oedometric compressibility,
- shear strength in direct shear apparatus,
- triaxial shear strength (TX) including possible response of natural subsoil such as:
  - soft soil around the piles is pre-consolidated with fully drained conditions (CD test – S method),
  - soft soil around the piles is unconsolidated and we deal with undrained conditions (UU test – Q method),

- ○ soft soil around the piles is pre-consolidated in undrained conditions (CU test with constant and alternate shear velocity – R method) – for determination of undrained shear strength and the change of viscosity coefficient with time.

The specimens have been taken directly from the model stand prepared for consolidation process of soft soil.

Since the test with gradually changing shear velocity allows a determination of the actual value of viscosity coefficient of clay $I_{v\alpha}$ depending on shear strength increment $\Delta\tau$ (Leinenkugel 1976) we have

$$\Delta\tau = I_{v\alpha} c_{u\alpha} \ln(\dot{\varepsilon}/\dot{\varepsilon}_\alpha), \tag{35}$$

thus the results of this test, supplemented by oedometric and direct box shear tests have been treated by the Author as main investigations in the analysis of the influence of the change of loading conditions on the magnitude of lateral load acting against the piles.

In the triaxial test made on cylindrical samples with 100 mm diameter and 120 mm height, the following values of axial strain $\dot{\varepsilon}_\alpha$, rate and time of shearing have been applied:

$\dot{\varepsilon}_\alpha = 0.1\%$ $H_0/h = 0.12$ mm/h $v = 0.002$ mm/min. $t = 1000$ min.
$\dot{\varepsilon}_\alpha = 1.0\%$ $H_0/h = 1.20$ mm/h $v = 0.020$ mm/min. $t = 100$ min.
$\dot{\varepsilon}_\alpha = 10.0\%$ $H_0/h = 12.00$ mm/h $v = 0.200$ mm/min. $t = 10$ min.

Relying on the analysis of CU triaxial tests with changing shear rate, the shear strength increments for clay have been determined. Next, using Leinenkugel formula the relation between shear strength $c_{u\alpha}$ and viscosity coefficient $I_{v\alpha}$ and time and magnitude of consolidation load, applied in model tests, was derived. Respective non-linear curves have been shown in Fig. 12 (Kurałowicz 2003). The charts can be used for calculation of additional earth pressure of creeping soil acting against the piles by Leinenkugel-Winter formula for the case of pre-consolidated soft soil layer.

5. Calculation Examples of Lateral Earth Pressure against the Piles for Improved Clay Parameters

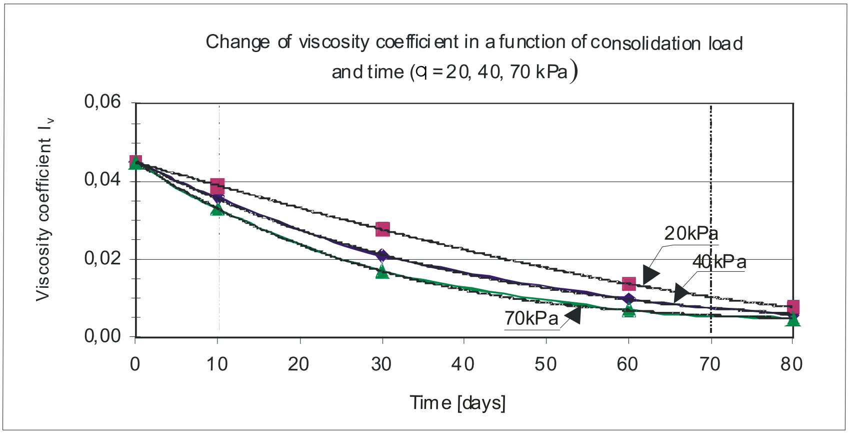
Example 1

Using Leinenkugel-Winter formula the lateral earth pressure p_f induced by clay layer acting against the two piles for three strain rates v_0 has been calculated. In the calculations the following data have been assumed:

- soft soil layer characteristics:
  $v_0 = 0.5$ cm/month, 0.4 cm/month, 0.3 cm/month, $\dot{\varepsilon}_\alpha = 7.2$ mm/month,

– consolidation time of soft soil layer: 20 days,
– consolidation load q: 20, 40, 70 kPa,
– pile characteristics: diameter $D = 1.0$ m, axial spacing $a = 2.5$ m, $a - D = 1.5$ m,
– calculated coefficient of the influence of pile spacing $k = 8.0294$.

a)



b)

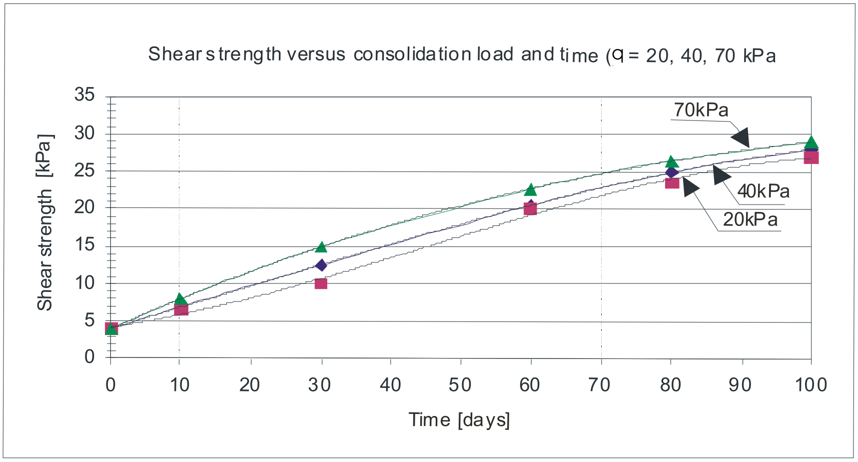


Fig. 12. The influence of consolidation load and time on viscosity coefficient (a) and shear strength of clay (b), Kurałowicz (2003)

Lateral earth pressure induced by creeping soil p_f, calculated by Leinenkugel--Winter formula, for consolidation load q in time t according to the charts proposed by the Author (Fig. 12) is equal to:

$$\left.\begin{array}{l} t = 20 \text{ days} \\ q = 20 \text{ kPa} \end{array}\right. \quad c_{u\alpha} = 7 \text{ kPa} \quad v_0 = 0.5 \text{ cm/month} \quad I_{v\alpha} = 0.035$$
$$p_f = 63.46 \text{ kN/m}$$

$$\left.\begin{array}{l} t = 20 \text{ days} \\ q = 40 \text{ kPa} \end{array}\right\} \Rightarrow c_{u\alpha} = 10 \text{ kPa} \quad v_0 = 0.4 \text{ cm/month} \quad I_{v\alpha} = 0.027$$
$$p_f = 92.55 \text{ kN/m}$$

$$\left.\begin{array}{l} t = 20 \text{ days} \\ q = 70 \text{ kPa} \end{array}\right\} \Rightarrow c_{u\alpha} = 12 \text{ kPa} \quad v_0 = 0.3 \text{ cm/month} \quad I_{v\alpha} = 0.024$$
$$p_f = 111.43 \text{ kN/m}$$

Example 2

In order to show the influence of time on the change of shear strength parameters and lateral earth pressure for natural conditions some numerical calculations with the help of PLAXIS v.8.1 code have also been carried out (Table 3). The results of these calculations are shown in Fig. 13.

Table 3. Soft soil layer parameters for calculations in 2D plane strain (Soft-Soil Creep model)

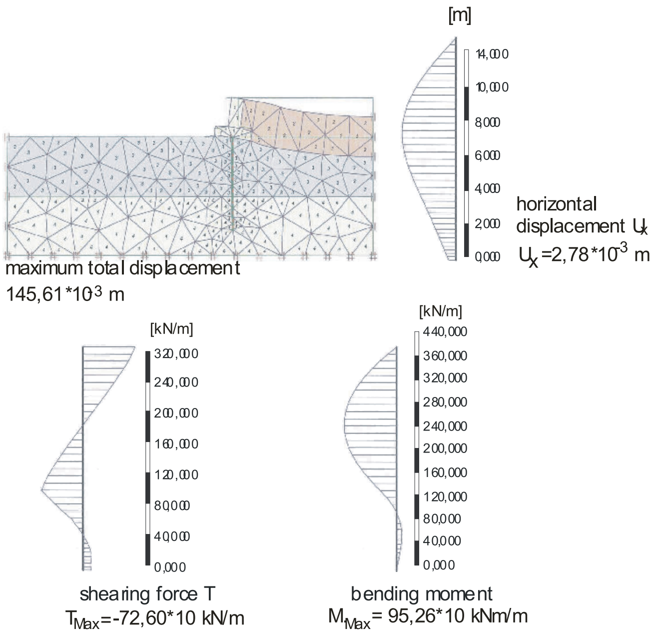
Layer	Type	γ [kN/m^3]	γ' [kN/m^3]	k_x	k_y [m/day]	λ^*	κ^*	μ^*	ν
soft	perm.	17.0	17.0	1.0000E-5	1.0000E-5	0.0430	0.0152	0.0016	0.15

K_0^{nc}[–]	M [–]	c^{ref} [kN/m^2]	φ[°]	ψ[°]	R_{inter}
0.935	0.37	5.0	10.0	0.0	1.00

6. Summary

Soft layer made of loose non-cohesive, plastic cohesive or organic soil has low shear strength and undergoes deformation at minimum external loading, i.e. caused by embankment surcharge. Long-term loading of soft soil layer influences its shear strength parameters and viscosity coefficient of the soft soil. The lack of unique solution for lateral soil-pile interaction does not allow the optimum design of bearing capacity of piling foundation including the lateral earth pressure acting against the piles. The consolidation process of soft layer does not induce significant reduction of the additional earth pressure on piles. After the consolidation, the interaction between piles and the soil changes and in subsequent loading phases changes also the magnitude and localization of maximum bending moments, as well as horizontal position of pile's head. Depending on consolidation conditions both the decrease as well as increase of additional earth pressure against the piles can occur. Proposed charts, presented in the paper, which relate the shear strength and viscosity coefficient changes with time enable calculation of the lateral earth pressure by Winter-Leinenkugel formula. The Author compared the changes of tested clay parameters with the test result made by Lechowicz (1992)

a) Directly after construction of the embankment



b) 1000 days after embankment construction

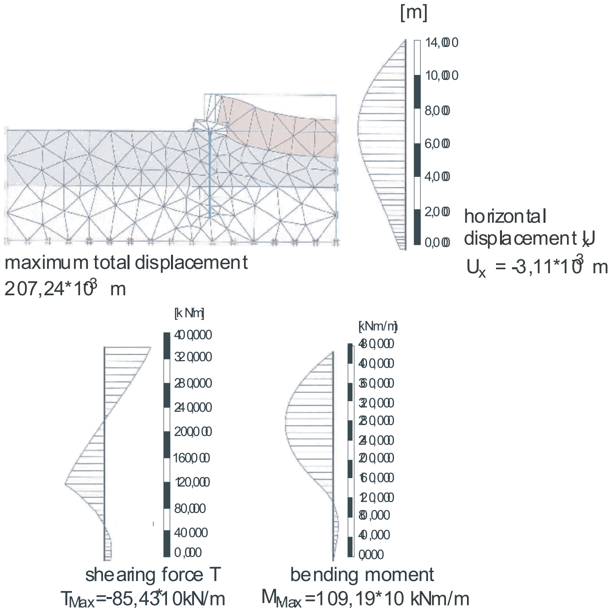


Fig. 13. Calculations of earth pressure induced by creeping soil, acting on abutment piles by PLAXIS 2D code for soft soil layer model – Soft Soil Creep (Geotechnical Department of the Technical University of Gdańsk)

and Przystański (1980). The improvement process of organic soils given by authors mentioned has the same character as for clay, tested by the Author.

Additionally, the following conclusions can be drawn:

– for design purposes, for small scale engineering structures, in the analysis of loads plane strain state can be applied as well as constant values of shear strength parameters of the soil, independent of the consolidation time of soft soil layer,
– for more complex foundation conditions and more responsible structures, in the analysis of lateral soil-pile interaction in time the influence of the improvement process on horizontal component of stress, as well as change of subsoil geometry should be considered and 3D analysis applied (see also Yong et al (1984, 1988), Wolski (1988), Lechowicz, Szymański (2002)),
– change of stress state in whole subsoil, including soft soil layer in which deformation process is initiated, together with soil-pile interaction depend mainly on the initial shear strength and stress at the roof of soft layer,
– improvement of soft soil due to consolidation process reduces the value of lateral earth pressure acting against the piles in the range of external load lower than the limit stress state in this layer. Further increase of loading will cause essential change of the soil-pile interaction and in consequence larger lateral loading acting against the piles.

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