THE EFFECTS OF MINE FIELD SUBSOIL DEFORMATIONS ON CONSTRUCTION'S SUPPORTS[1]

by

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Abstract: The interaction between construction's supports (supported slope) and ground medium in mine-field subsoil deformation conditions is discussed. Basingon the results of analog model investigation, the phenomenon of the soil thrust on the vertical elements of building structures in mining areas was found to be entirely different from classical soil pressure in view of Coulombe's theory. The novel theoretical concept of this phenomenon is based on the granular media mechanics and variable values of the earth pressure coefficient in the conditions described by horizontal unitary compacting strains in the subsoil.

Additional Key Words: analog medium, earth pressure, compacting strain, subsoil,

Introduction

The interaction between support constructions and ground medium constitutes one of the oldest civil engineering problems, first discussed over 300 years ago (Dembicki 1979,Whitman 1969). Strict analytical determination of all parameters of the soil-construction interaction is virtually impossible because of a considerable number of factors contributing to this process, including:

- variable geometrical parameters of the construction (height, tilt, plane view and cross-section view),
- geotechnical parameters of the ground medium, hydrological conditions,
- deformability of constructions under given load (permanent, periodic or emergency loads),
- specific function of structural elements subjected to the soil thrust (basement, retaining wall, bridge head, tunnel- culvert, sheet pile wall, etc.), together with strength and material parameters of a given structural element.

The phenomenon of the soil pressure has already been widely recognized , thanks to the availability of various calculation methods, especially as far as the assessment of the boundary values of this pressure on constructions in active and passive states is concerned, (Figure 1A).

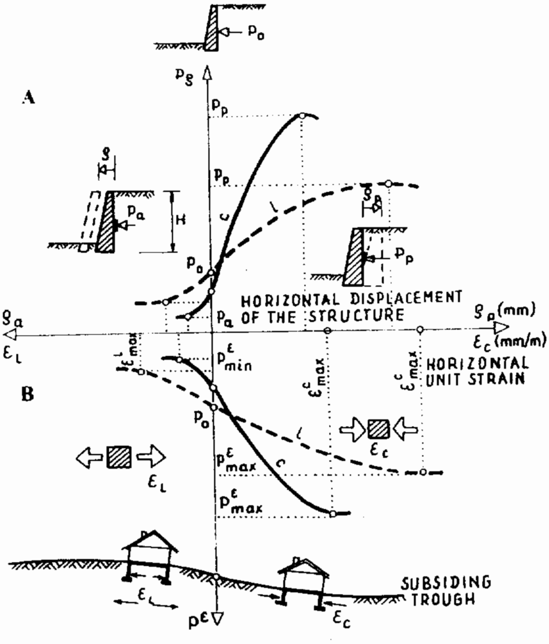

Figure 1. Influence of displacements or mine-induced strains ε on the engagement of soil pressure P (soil thrust) for loose soil.
A-displacement of the wall in relation to the passive medium; B-horizontal unitary loosening (compacting) strains acting in the medium in relation to the construction; ε_l-loosening state; ε_c- compacting state.

[1] Paper presented at the 1997 National Meeting of the American Society for Surface Mining and Reclamation Austin, Texas, May 10-15, 1997.

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Proceedings America Society of Mining and Reclamation, 1997 pp 382-392
DOI: 10.21000/JASMR97010382

https://doi.org/10.21000/JASMR98010382

Table 1. Compilation of most important expressions

Author	Proposed method of calculating the soil pressure
Wasilkowski F. (1980) Boczkaj B.(1994)	for underground structures: $\sigma_{22} = \gamma\times H\times tg^2(\pi/4 - \phi/2) - 2\times c\times tg(\pi/4 - \phi/2) + E_h\times(1+ L/1,5\times H)\times \varepsilon\leq\sigma_{22}^P$ where: $E_h = 0,4$ E; for sandy soil; $E_h = 0,5$ E; for clayey soil
Król W. (1980)	soil thrust on single, detached structure $\sigma_{22} = \gamma\cdot H\cdot tg^2(\pi/4 - \Phi/2) + P_o\cdot(1 + L/f)$; $f = 1,5$ B or $f = 3$ H soil thrust on the neighboring structures ($d< f$) $\sigma_{22}^* = \gamma\cdot H\cdot tg^2(\pi/4 - \Phi/2) + P_o\cdot(1 + L/f + L\cdot d/f^2)$; soil thrust from the exterior $\sigma_{22}^{**} = \sigma_{22}^* + H_o/H\cdot(1- d/f)$; soil thrust from the interior
Muller R.A (1980)	empirical expression based on field test measurements $\sigma_{22} = \varepsilon/(0,6\cdot\varepsilon + 0,002)$ analytical solution: $P_1 = 0,5L/\rho_p\times\varepsilon\times(P_p - P_a) + P_a$ for $\varepsilon\times0,5L \leq \rho_p$ $P_2 = P_p$ for $\varepsilon\times0,5L > \rho_p$ where: $\rho_p = P_p\times tg(\pi/4 + \phi/2)/E_h$; $E_h = (0,4 \div 0,5)\times E$
Kwiatek J.(1993)	for retaining constructions (one-sided pressure) $\sigma_{22} = \frac{\nu}{1-\nu}\cdot(\gamma\cdot H+q)+\frac{E}{1-\nu^2}\varepsilon \leq (\gamma\cdot H+q)\cdot tg^2(\pi/4+\phi/2)+2\cdot c\cdot tg(\pi/4+\phi/2) = \sigma_{22}^P$ for tunnel construction (two-sided pressure) $\sigma_{22} = \frac{\nu}{1-\nu}\cdot(\gamma\cdot H + q) + \frac{E}{1-\nu^2}\cdot\varepsilon\cdot(1 + L/3\cdot H) \leq \sigma_{22}^P$
Rosikoñ A. (1979)	empirical expression based on laboratory measurements: $U = S/S_p = \rho/\rho_p = \varepsilon L / 0,18H$ where $P_1 = e\times P_p$ where: $e = U/(-0,102\times U^2 + 0,997\times U + 0,05)$ or, according to the figure in (Rosikoñ 1979) $P_p = H\times B\times(0,5\times\gamma\times H\times\lambda_R + 2\times c\times\lambda_k)$

Notations:

σ_{22}, σ_{22}^P, P_p, P_o, P_a – horizontal strain, boundary strain, passive boundary pressure, static pressure, active pressure (Figure 1);

E, E_h, ϕ, c, γ, ν – vertical soil deformation module, horizontal module, internal friction angle, soil cohesion, bulk density, Poisson`s coefficient;

q, h, H, B, L – ground load, depth-height-width-length of the construction;

ε–horizontal unitary compacting strain [mm/m] or [‰]

ρ, ρ_p, S, S_p – displacement of the construction, transitory and boundary (Figure 1);

d – length between two neighboring structures.

Yet, according to several authors (Dembicki 1979,Klosek 1983), a wide range of changes in the values of the resultant forces reaction of the subsoil) P_{max}/P_{min} = $P_p/P_a = 5 \div 15$ makes it only possible to apply the value of the soil pressure, but not to calculate it accurately.

In view of this, the 'active' soil thrust on building structures in mine- induced deformation conditions is a novel concept, hardly ever discussed (Drumm 1988, Klosek 1978, Kratzsch 1988,Kwiatek 1993,Speck 1990, Rosikoñ 1979). Most analytical methods applied to describe this phenomenon are based on the solutions obtained for the subsoil which has not been subjected to mining deformations, Table 1. These methods identify both processes of kinematics to be of the same character, disregarding a difference in their origin. It should be verified if such approach is justified and acceptable if safe functioning of constructions is to be secured. Therefore, in the course of the discussed model investigation, the mechanics of the soil-construction interaction was represented for the subsoil which has not been subjected to the impact of mining (Figure 1A) as well as for the subsoil subjected to horizontal strains ε in the boundary and lowest zones of the land subsidence trough, Figure 1.B.

Model Investigation

Assuming a multi-parametric distribution of the mining subsoil indices (Klosek 1994) two of the parameters are regarded as the most decisive:

- horizontal strain ε of a compacting ε_c (or loosening ε_l) character, which is a derivative of horizontal displacements: $\varepsilon_{c,l} = du/dx_2$,
- land inclination $T=dw/dx_2$, causing the displacement of construction's supports towards the backfill ground or in the opposite direction, Figure 2 B - C.

The influence of local curvature $K = d^2w/dx_2^2$ is practically insignificant.

The above mentioned deformations activate the subsoil in relation to the passive reaction of the construction itself. To represent such complex kinematics of this phenomenon a prototype research stand was designed (Klosek 1996). The analog ground medium of the Taylor-Scheebelly type was applied, constituting a set of two-diameter Φ 4 and Φ 6 mm short bars, which were 50 mm long and the bulk density of which complied with the criteria of mechanical correspondence. Thus, a complete simulation of the boundary state comparative to the relevant mechanical condition was achieved for:

- parallel displacement of the vertical wall ρ towards the immobile (passive) medium, Figure 3A-A*
- horizontal strains ε homogeneously compacting the medium interacting with the model of the construction's support, Figure 3B.

The displacement field of the selected grains of the analog medium recorded in the course of the investigation are presented in Figure3. Following the author's expectations, for a classical case of the soil pressure (Figure 3A) the combination of slide lines (in relation to the construction) proves the existence of a potential slide plane and triangular solid of the soil wedge displaced from the subsoil. A different condition was observed for the combination of the curving lines of the slide recorded in the course of the compacting strains e in the mine field subsoil, where neither a solid of the soil wedge, nor the respective slide plane were observed. Accordingly, in view of these observations, there is a difference in the subsoil-construction interaction conditions, which certifies that the two discussed cases of mechanics should not be considered as identical and equally possible to account for by means of classical (or modified) calculation methods, mostly founded on Coulomb's model and Rankine's diagram (Boczkaj 1994, Drumm 1988, Kratzsch 1988, Speck 1990).

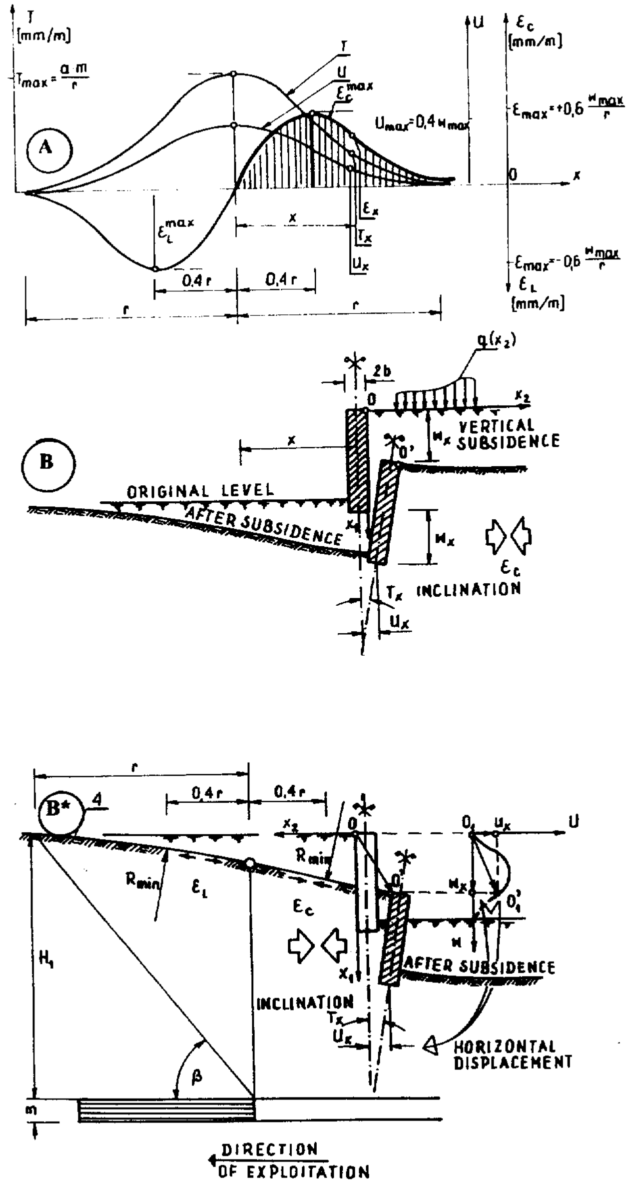

Figure.2. Mine-induced subsoil deformations in the construction's support zone.
A - main deformation components,
B - kinematics of the construction-compacted soil system interaction.

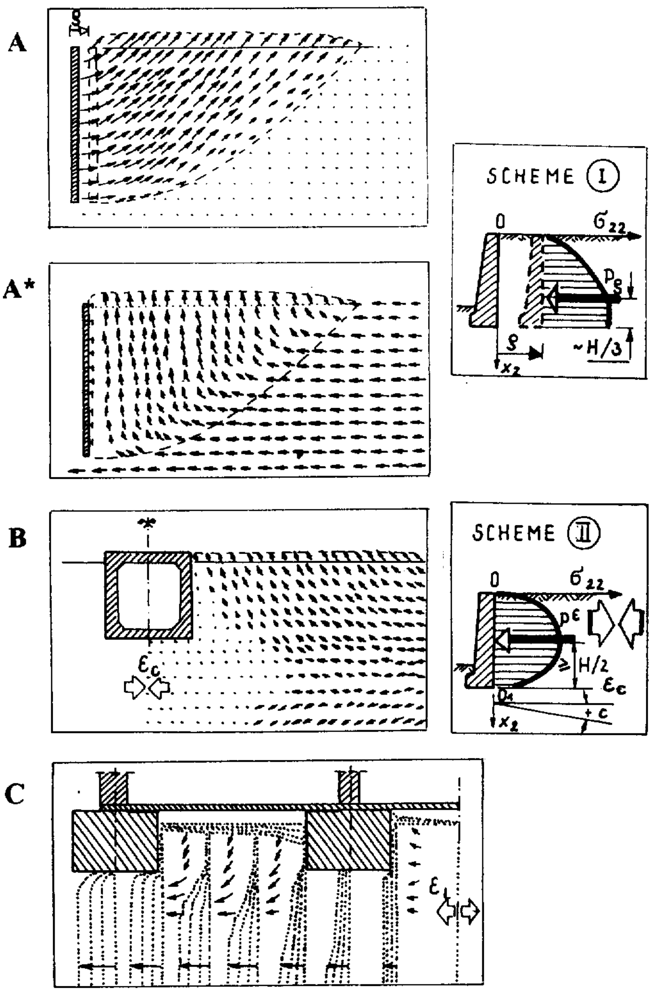

Figure 3. Vectorial displacement fields in the ground medium of Taylor-Schneebelly type.

A/A* - for the displacement of the construction tin relation to the passive medium (A - frame of reference to the ground, A* - frame of reference to the construction),

B - mining subsoil compacting deformations ε_c in relation to the construction,

C - mining subsoil loosening deformations ε_l in relation to the foundation with local compacting zone between long strip footing.

According to the measured ground pressure and ground thrust values, the rise of horizontal compacting strains in the mining subsoil was accompanied by the engagement of the resultant thrust force P, from height $H_o = 1/3$ H (scheme I) for the initial state to $H_{max} = H/2$ (scheme II) for the boundary state. The rising torque of the construction in relation to its foundation base further enhances the tilt of supports, loaded in a non-symmetrical way, Figure 3 A.
This phenomenon is often observed in field test measurements of the existing structures (Drumm 1988, Speck 1990).
The distribution of the stresses corresponding to the engagement of classical (passive) soil pressure and stable position of the resultant force P (Figure 3B-C), make an essential difference in comparison with the corresponding minefield subsoil deformation conditions (scheme II).
Therefore, a new method of determining the soil thrust in mining areas should be designed, accounting for the discrepancy discussed above.

Calculation Model

The analytical results compiled in Table 1 are based on Coulombe`s theory assuming the engagement of soil shear resistance along the shear plane which emerging in the course of the triangular soil wedge formation. Such phenomenon, as it has been proved by model field investigation, does not occur in mine-induced deformation areas. The deformation of the ground medium is characterized by predominantly homogeneous horizontal strains ε, related to the components of the stresses which lead to elastic volumetric strains of the soil semi-space interacting with the construction.

Assuming that soil porosity is dependent only on the total number of the main stresses acting on the structural frame (Klosek 1978,1983) the function of the earth pressure coefficient variation was determined for the compacted medium:

$$K_\varepsilon = \sigma^\varepsilon_{22}/\sigma_{11} = K_o + E_\varepsilon\,(1+\xi\nu)\,\alpha_c\,\sigma_{11}^{-1}\,\varepsilon_c \le K_{max} \qquad (1)$$

where:

K_o – earth pressure coefficient in the geo-static conditions;
K_{max} – boundary state criterion for loose soil;
$K_{max} = tg^2(\pi/4+/2)$
σ_{22}^ε – horizontal strains in mine deformation conditions;
E_ε – soil horizontal susceptibility (elasticity) module;t

$\xi = \varepsilon_{33} / \varepsilon_{22}$, relation of horizontal strains in the main directions

α_c – soil compressibility variation coefficient.

Further analysis was reduced to the case of a plane strain state in which the construction's support is located perpendicularly to the directions of maximal compacting strains ε_c.

Following the principles of the granular media mechanics, the stress state components were calculated by means of a basic system of differential equations of a parabolic type:

$$\begin{cases} \dfrac{\partial \sigma_{11}}{\partial x_1} = K_\varepsilon \cdot x_1 \cdot \dfrac{\partial^2 \sigma_{11}}{\partial x_2^2} + \gamma \\ \sigma_{12} = -K_\varepsilon \cdot x_1 \cdot \dfrac{\partial \sigma_{11}}{\partial x_2} \\ \sigma_{22} = K_\varepsilon \cdot \left(\sigma_{11} + K_\varepsilon \cdot x_1^2 \cdot \dfrac{\partial^2 \sigma_{11}}{\partial x_2^2} \right) \end{cases} \tag{2}$$

The shape of the construction, the ground load and the roughness at the edge of contact with soil D = 1/RH and the pressure of the acting forces are described by the equations:

$$\begin{cases} q(x_2) = \sigma_{11}(0; x_2) = 0 \\ \partial \sigma_{11}(x_1; 0) + D \cdot \left[\gamma \cdot x_1 - \sigma_{11}(x_1; 0) \right] = 0 \\ \dfrac{\partial \sigma_{22}(x_1; \infty)}{\partial \sigma_{11}(x_1; \infty)} = K_\varepsilon \end{cases} \tag{3}$$

For $x_2 = 0$, a complete system of the equations taking account of all the stress state components at any point of the ground semi-space may be obtained, as illustrated by the example in Figure 4.

The exemplary distribution of the contact stresses for the one-sided soil thrust is presented in Figure 5, and for the two-sided thrust in Figure 6.

The discussed model investigation is consistent with the results of numerical calculations, indicating good consistency further proved by a comparative analysis of the two procedures. It should be indicated that a curving character of the horizontal stresses is quite different from the results obtained on the basis of the traditional solutions and Coulombe's theory, compiled in Table 1.

If the above equations are solved for $x_2 = 0$, it is possible to arrive at a system of equations describing all the stress state components at the edge of contact between the construction and the backfill ground:

$$\begin{cases} \sigma_{11} = \gamma \cdot H \cdot K \cdot \sqrt{\pi/2 \cdot K_\varepsilon} \cdot \Big[1 - \exp\left(0{,}5 \cdot K_\varepsilon \cdot \eta_1^2 / R^2\right) \cdot \\ \quad \cdot \operatorname{erfc}\left(\eta_1 / R \cdot \sqrt{0{,}5 \cdot K_\varepsilon} + \dfrac{\aleph \cdot E_\varepsilon^c \cdot b \cdot \varepsilon}{\gamma \cdot H^2 \cdot R \cdot \sqrt{0{,}5 \cdot \pi \cdot K_\varepsilon}} \right) \Big] \\ \sigma_{22} = K_\varepsilon \cdot \sigma_{11} - \gamma \cdot H \cdot K_\varepsilon \cdot \sqrt{0{,}5 \cdot \pi \cdot K_\varepsilon} \cdot \eta_1^2 / R \cdot \\ \quad \cdot \exp\left(0{,}5 \cdot K_\varepsilon \cdot \eta_1^2 / R^2\right) \cdot \operatorname{erfc}\left(\eta_1 / R \cdot \sqrt{0{,}5 \cdot K_\varepsilon} \right) + \dfrac{\aleph \cdot E_\varepsilon^c \cdot b}{H} \cdot \varepsilon \\ \sigma_{12} = \gamma \cdot H \cdot \eta_1 \cdot \sqrt{0{,}5 \cdot \pi \cdot K_\varepsilon} \cdot \Big[\exp\left(0{,}5 \cdot K_\varepsilon \cdot \eta_1^2 / R^2\right) \cdot \\ \quad \cdot \operatorname{erfc}\left(\eta_1 / R \cdot \sqrt{0{,}5 \cdot K_\varepsilon} + \dfrac{2 \cdot \aleph \cdot E_\varepsilon^c \cdot b \cdot \varepsilon}{\gamma \cdot H^2 \cdot \eta_1 \cdot \pi \cdot K_\varepsilon} - 1 \right) \Big] \end{cases} \tag{4}$$

where :

$\eta_{1,2} = x_{1,2}/H$ – non-dimensional coordinates;

exp – base of a natural logarithm;

$\operatorname{erfc} = 2/\sqrt{\pi} \int_x^\infty \exp(-x^2) dx$ integral function, error function complement;

$D = 1/R \times H$ – wall roughness coefficient;

$$R = \frac{1}{1{,}25 \cdot f_0 \sqrt{\dfrac{1}{1 + 2 \cdot f^2 + 2 \cdot \sqrt{(1+f^2) \cdot (f^2 - f_0^2)}} + \dfrac{\sqrt{3} - f}{\sqrt{3} + 2 \cdot f} - tg^2(\pi/4 - \Phi/2)}} \tag{5}$$

$f = tg\,\phi$; ϕ – angle of soil internal friction;

$f_o = tg\,\phi_o$; ϕ_o – angle of soil friction with the wall ($\phi_o = 0{,}2 - 0{,}5\,\phi$).

The upward displacement of the resultant thrust force occurs when the compacting strains ε_c are magnified, which, in turn, increases the bending moment of the construction around the edge of its foundation base.

For the two-sided soil thrust, the maximal value of the bending moment is reached at the central zone of the underground part of the construction, which is also divergent from the calculation schemes commonly accepted for the discussed phenomenon.

However, the curves illustrating the rise of soil pressure- soil thrust on the construction, expressed by means of the non-dimensional coordinates $P/P_p = \rho/\rho_p$ (Figure 1A) and $P^\varepsilon/P_{max}^\varepsilon = \varepsilon/\varepsilon_{max}$ (Figure 1B), have a similar course, see Figure 7.

The boundary values of displacement ρ and strains ε are strictly dependent on the initial condition of the soil interacting with the construction. In the case of loose soil, the boundary state is reached for higher displacement (strain ε) values, and the soil pressure (soil thrust) value is lower than in the case of pre-consolidated soil. Correspondingly, these aspects should be taken into consideration in the design and building the constructions' supports, especially in mine- induced deformation areas.

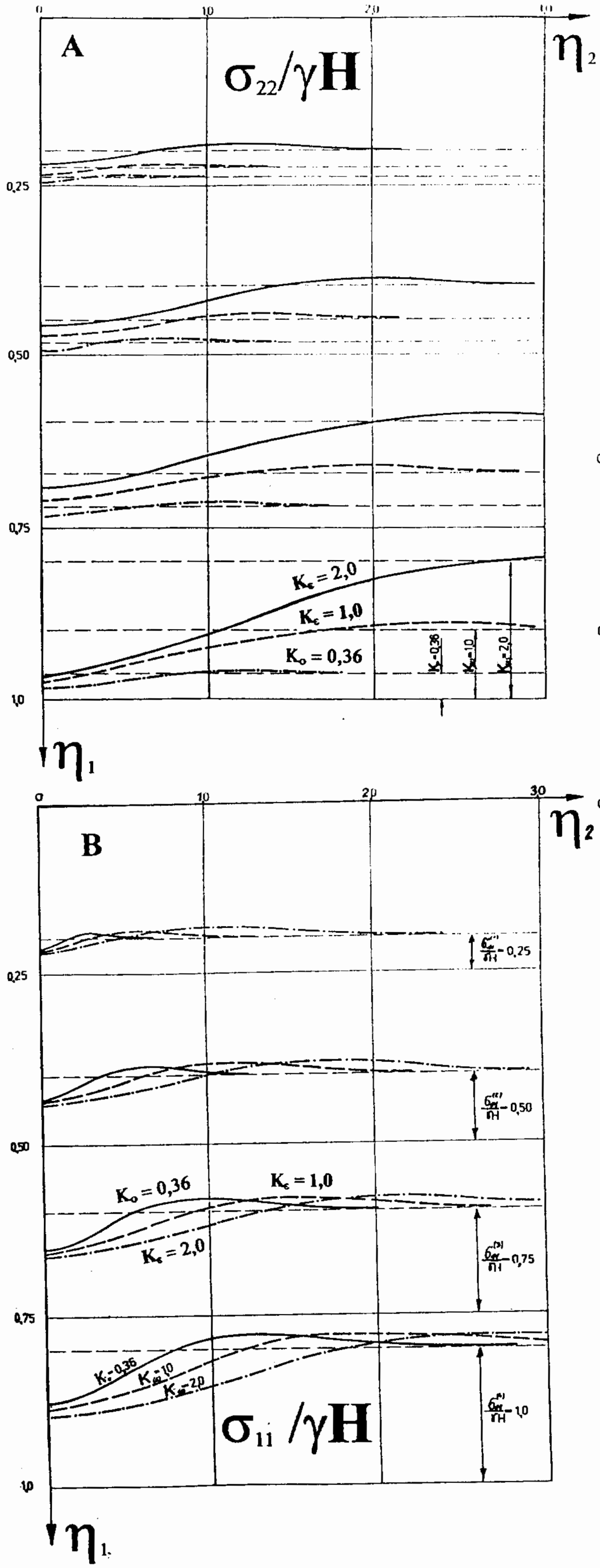

Figure 4. Stress state components in the horizontally compacted semi-space interacting with the construction`s support in mine-induced deformation area.

A - horizontal stress (non-dimensional);

B - vertical stress;

C - shear stress.

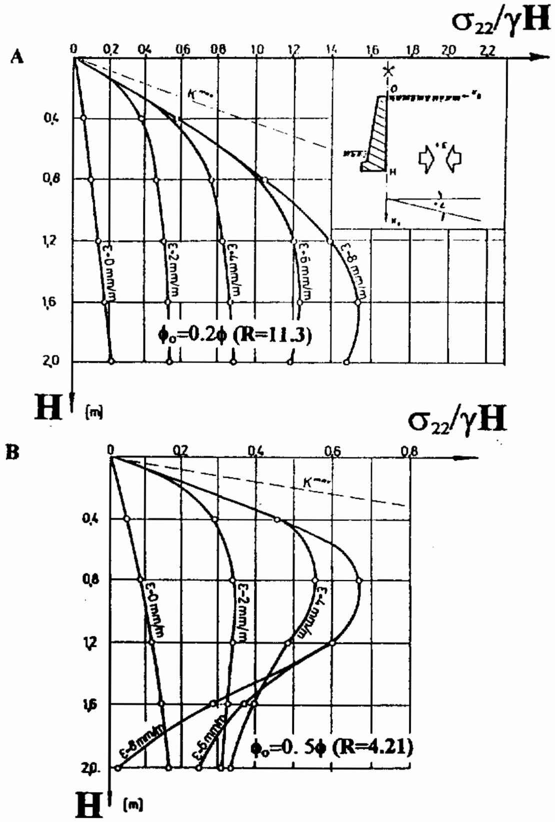

Figure 5. Theoretical increase of the one-sided thrust on the construction : A- smooth wall, B-rough wall

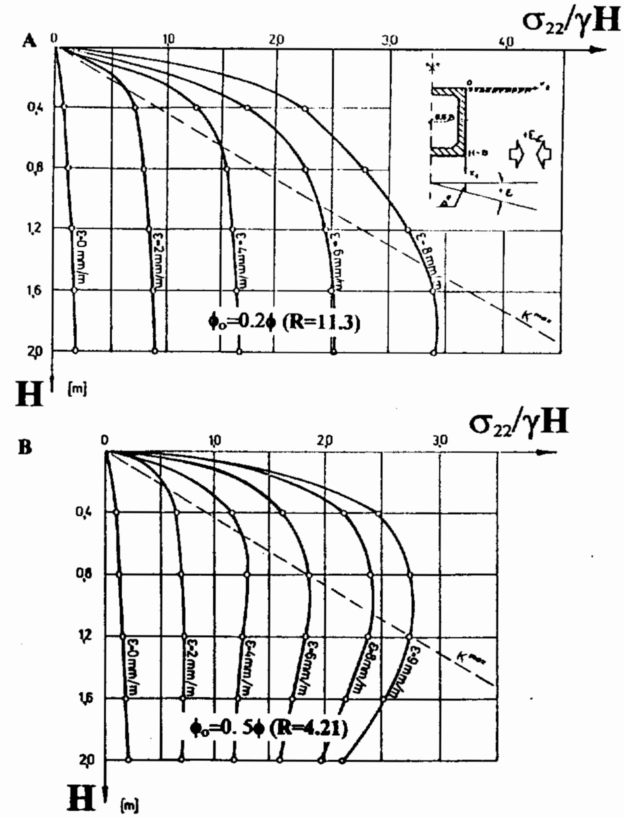

Figure 6.Theoretical increase of the two-sided soil thrust on the construction: A- smooth ,B- rough wall

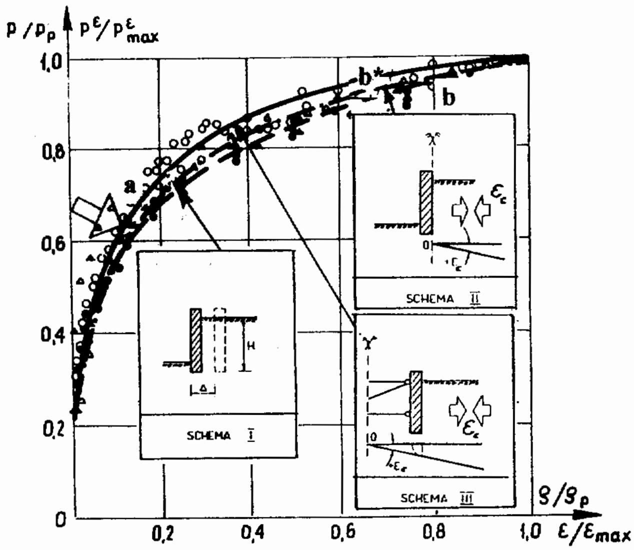

Figure 7. Engagement of the resultant soil pressure - soil thrust force on construction's supports for the non-dimensional coordinates. Diagrams : I-passive soil pressure (a), II- mine-induced compacting strain (b), III- two-sided soil thrust (b*).

Conclusion

The interaction of support structures with the compacted mining subsoil poses a complex geo-technical problem. The established analytical concepts so far identified the soil thrust and the passive pressure of the construction on the geostatic ground medium to be of the same character. The difference between the two kinematic cases of the soil-construction` support system discussed in the paper demonstrates the inconsistency of such identification, verified by both model field investigation and theoretical explication.

The assumed analytical interpretation is based on the mechanics of discrete ground media for variable earth coefficient, leading to a credible description of this complex problem.

The discussed solution may have its practicalapplication, contributing to better prevention measures taken in the design and building of constructions` supports in mining areas.

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