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Procedia Engineering 14 (2011) 1705–1712

Procedia Engineering

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The Twelfth East Asia-Pacific Conference on Structural Engineering and Construction

Phenomenon of Influence Zone in Civil Engineering Practise

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Abstract

When erecting upper structure the man manufactured materials are used. That is the reason of their excellent material properties. In the case of foundation the nature situation must be almost respected. The geostatical stress plays significant role on the subsoil behavior. The geostatical stress state is de facto natural form of the soil compaction. The soil has ability memorized the highest stress that has been ever loaded. The soil can be considered incompressible if the magnitude of a surcharge is lower. The excavation makes space for upper structure surcharge. The influence zone depth is defined like region where the geostatical pressure reduced by excavation together with upper structure loading exceed the original geostatical pressure. The Kantorovich method together with strategy of convolution was used to reach dimensional reduction when deriving analytical formulas. Derived formulas and presented graphs for influence zone depth estimation have considerable importance for civil engineering practice.

Keywords: pre-consolidation pressure; influence zone; Kantorowitch method; layered subsoil; geostatic stress state.

1. INTRODUCTION

It has been experimentally confirmed that a soil substantially changes its material properties when subjected to external loading. Apart from that, the soil, when subjected to a certain loading history, has the ability to memorize the highest level of loading mathematically represented by the over-consolidation ratio, and initial void ratio. In its virgin state, the soil deformability is relatively high. On the contrary, following the unloading/reloading path shows almost negligible deformation until the highest stress state the soil has experienced ever before is reached (Terzaghi et al. 1996). To study this behavior of soil, we performed several small laboratory tests. Transport processes were observed carrying out isotropic

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doi:10.1016/j.proeng.2011.07.214

consolidation in triaxial test apparatus. Employing genetics algorithms, soil parameters were determined comparing the pore pressure course (measured and calculated) (Janda et al. 2004). In large scale, the effect of over consolidation was simultaneously investigated by the way of rigid plate and piles working diagrams analysis. Both the finite element technique and elastic layer theory were employed in back analysis of the measured data.

2. PRECONSOLIDATION IN LABORATORY TESTING

The test was arranged in two runs, loading/reloading. Readings were taken of the highest level of effective mean stress. Referring to experimental measurements carried out in the triaxial apparatus, the isotropic consolidation can be viewed as one phase flow in a fully saturated deforming medium undergoing small deformation. When neglecting the body forces, the hydrostatic state of stress maintained during the experimental measurement gives

$$\sigma_x(x,y,z,t) = \sigma_y(x,y,z,t) = \sigma_z(x,y,z,t) = \sigma_m \tag{1}$$

where σ_m *is* the total mean stress. Following the Terzaghi-Fillunger concept of effective stresses, this quantity can be expressed in terms of the pore pressure p^s and the effective stresses between grains σ_m^{eff} as

$$\sigma_m = \sigma_m^{eff} - p^s \tag{2}$$

Assuming full saturation (S_w = 1) the pore pressure p^s equals the pressure in the liquid phase p^w. Referring to experimental conditions, the total mean stress remains constant throughout the consolidation process. The assumed stress homogeneity together with Eq. (2) then provide

$$\dot{\sigma}_m = \dot{\sigma}_m^{eff} - \dot{p}^w = 0 \tag{3}$$

where $(\dot{\ })$ represents the time derivative $\partial(\ )/\partial t$. Transport of the liquid phase throughout the soil sample can be described by the following set of equations:

Transport equation

$$J^w = -\frac{K\rho^w}{\gamma^w} \cdot grad\ p^w \tag{4}$$

where J^w is the mass flux of pore water, $\gamma^w = g\rho^w$ is the specific weight of water, ρ^w is the intrinsic mass density and K represents an instantaneous coefficient of permeability.

Balance equation reads

$$\rho^w \dot{\varepsilon}_V + div\, J^w = 0 \tag{5}$$

The volumetric strain ε_V follows from the

Constitutive equation

$$\varepsilon_V = \frac{e-\bar{e}_0}{1+\bar{e}_0} = -\frac{\kappa}{1+\bar{e}_0}\ln\left(-\sigma_m^{eff}\right),\ \sigma_m^{eff} > \bar{\sigma}_m^{eff} = -p_c \tag{6}$$

$$\varepsilon_V = \frac{e-e_0}{1+e_0} = -\frac{\lambda}{1+e_0}\ln\left(-\sigma_m^{eff}\right),\ \sigma_m^{eff} < \bar{\sigma}_m^{eff} = -p_c \tag{7}$$

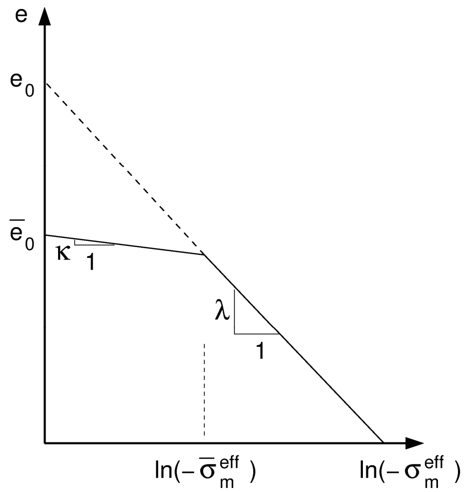

Figure 1: Bilinear form of the normal consolidation line.

They were derived for the case of the modified Cam clay model from the bilinear consolidation line, Fig. 1. The initial branch, often referred to as κ - line, gives evidence of the previous stress history and represents the effect of preconsolidation. The slope discontinuity between the κ and λ - lines can be identified with the structural strength of the soil given in terms of a certain level of the effective mean stress $\overline{\sigma}_m^{eff} = -p_c$ (p_c is termed the preconsolidation pressure).

Differentiating Eq. (7) with respect to time gives the rate of volumetric strain in the form

$$\varepsilon_V = \frac{\dot{e}}{1+e_0} = -\frac{\lambda}{1+e_0}\frac{\dot{p}^w}{\sigma_m^{eff}} \tag{8}$$

Substituting Eqs. (4) and (8) into Eq. (5) and taking into account the actual triaxial set-up, in which only the bottom face of the cylinder is drained, lead to

$$-\frac{1+e_0}{\gamma^w \lambda}\sigma_m^{eff}(z,t)\frac{\partial}{\partial z}\left(K(z,t)\frac{\partial p^w(z,t)}{\partial z}\right)-\dot{p}^w(z,t) = 0 \tag{9}$$

It has been verified experimentally that in the case of isotropic consolidation a mere power law written as (see Kuklík et al. 1998).

$$\frac{K(z,t)}{K_0} = \left(\frac{e(z,t)}{e_0}\right)^m \tag{10}$$

represents the soil behavior fairly well. The dependence of the actual void ratio *e* on the effective mean stress, Eq. (7), together with Eq. (10) provide

$$\frac{\partial K(z,t)}{\partial z} = -\frac{mK(z,t)\lambda}{e(z,t)\sigma_m^{eff}(z,t)}\frac{\partial p^w(z,t)}{\partial z} \tag{11}$$

Introducing Eq. (11) into Eq. (9) finally yields.

$$\dot{p}^w(z,t) = -\frac{K(z,t)(1+e_0)}{\gamma^w \lambda}\left[-\frac{m\lambda}{e(z,t)}\left(\frac{\partial p^w(z,t)}{\partial z}\right)^2 + \sigma_m^{eff}(t)\frac{\partial^2 p^w(z,t)}{\partial z^2}\right] \tag{12}$$

A similar equation can be derived for the unloading branch when replacing λ by κ and e_0 by $\overline{e}_0$. The main objective of these steps is to introduce a simple yet accurate numerical technique for extracting material parameters of clayey soils that behave according to the Cam clay model, from a simple one-dimensional consolidation test. To introduce this task, recall that reproducing laboratory data requires supplying the structural strength parameter p_c, the initial void ratio $\overline{e}_0$, the initial coefficient of permeability K_0, the swelling index κ and the compression index λ. A simple solution is offered. In particular, matching experimentally obtained data with those derived numerically might be the simplest view at a complex optimization procedure that provides the desired results. To that end, recall Eq. (12) describing the excess pore pressure variation during consolidation. In view of the optimization problem, the used material and structural strength parameters $(\overline{e}_0, K_0, \kappa, \lambda . p_c, m)$ now become the search variables to be found by minimizing the following function describing the bottom face pore pressure evolution (information in Matouš et al. 2000)

$$F = \sum_{k=1}^{\kappa} (p^w(t_k) - \overline{p}^w(t_k))^2 .$$

Example of result is listed bellow in Tab. 1.

Table 1: Optimal material parameters

κ	λ	$\overline{e}_0$	K_0 [ms^{-1}]	p_c [kPa]	m
0.012	0.074	0.56	1.53 e^{-9}	28.45	4.7

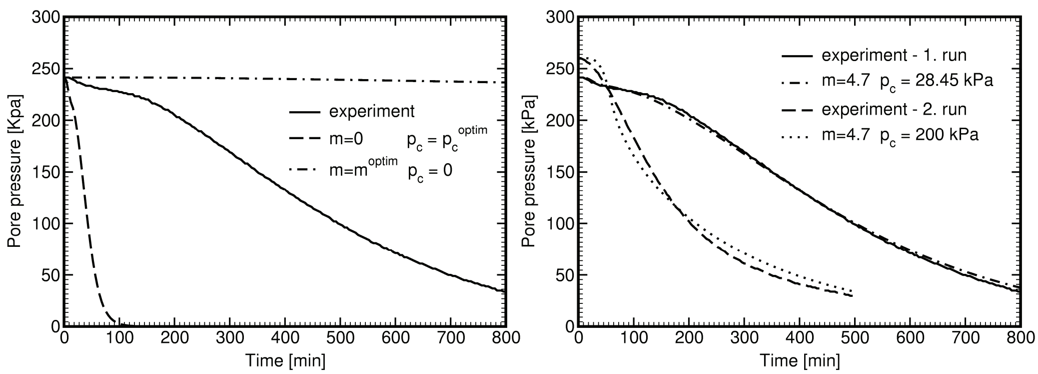

Figure 2,3: Time variation of pore pressure

An influence of the preconsolidation p_c and influence of the parameter m, describing the change of the permeability coefficient, are clearly presented in Fig. 2. Finally, the proposed theory is highlighted in Fig. 3. It is seen, that theoretical and measured loading/reloading curves fit fairly well (all details in Janda et al. 2004).

3. VALIDATION OF PRECONSOLIDATION IN SITU AND ITS TESTING

The pre-designed Brazilian capital Brasília, located in the Federal District of Brazil, was designed to house only the main Governmental administrative institutions and its public employees. Nevertheless, it has increased four times more than what was initially forecasted and is still expanding. Construction is advancing through distinct (geological) zones of this same district, and allowing the use of distinct techniques for deep foundation deployment and design. Hence in 1995, the University of Brasília started a major research project, in order to enhance the knowledge on the behavior of the typical deep foundations that are founded in the stratified, tropical subsoil of the Federal District. It was decided to carry out horizontal and vertical field loading tests on locally used foundation types. These foundations had full-scale dimensions and were placed within the University of Brasília campus at the experimental research site of the Geotechnical Post-Graduation Program. The geotechnical profile of this campus is composed by the typical unsaturated "porous" clay of Brasília. Field loading tests on full-scale instrumented foundations outside campus were also carried out. In all cases, the tests were performed with the cooperation of local engineering companies and University staff.

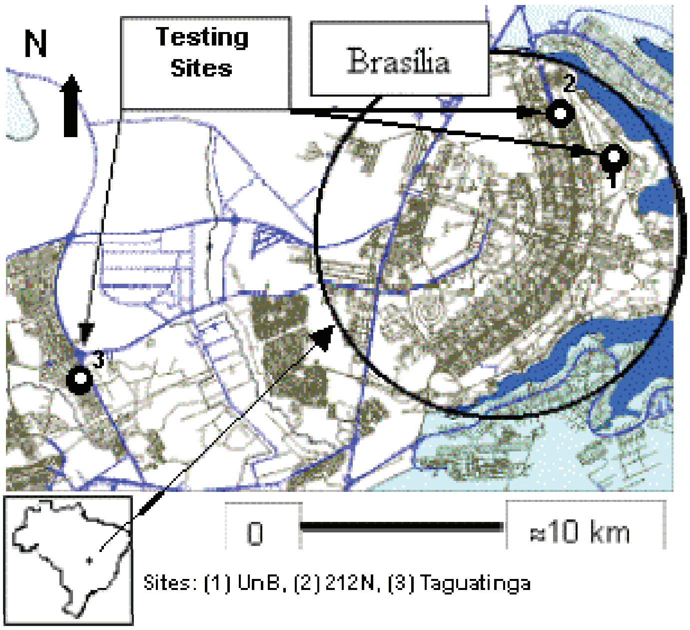

Figure 4: Location of the Studied Sites.

A mechanically bored pile founded in the University of Brasília campus (UnB research site) was analyzed together with instrumented piles from two other engineering sites within this same District.

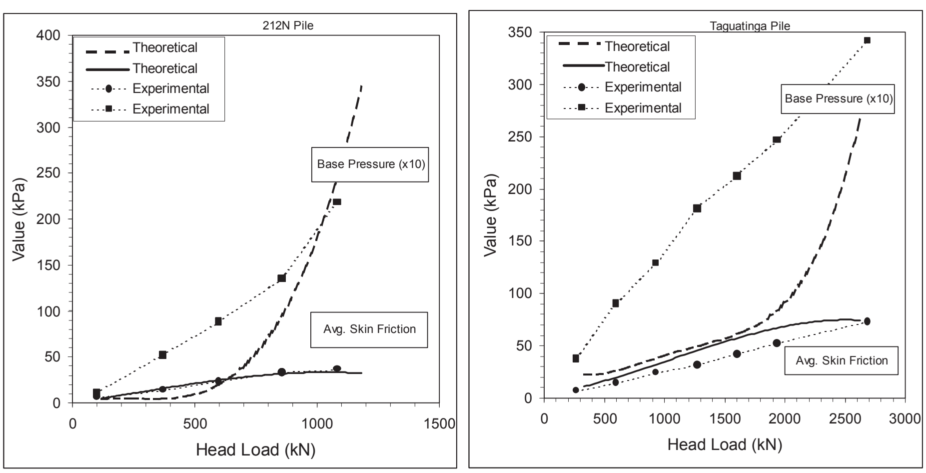

Figure 5,6: Comparative Results for the 212N Pile, for the Taguatinga Pile

One of them is a continuous flight auger type pile located in the North Wing of Brasília (212N site), close to the University Campus, and founded in the same geological material. The other is a mechanically bored pile excavated with bentonite mud, located in a site around 25 km from the University Campus (Taguatinga site), and founded in a foliated and stratified material which originates from slate decomposition. The experimental curves of pile load versus displacement, structural load transfer along the pile's shaft, and average skin friction and base pressure versus loading level were compared to numerical predictions from a semi-analytical procedure. This procedure is coded in the industrial software denominated GEO4 Pile modulus that was developed in cooperation between the Department of Mechanics, Faculty of Civil Engineering, CTU in Prague and Fine Ltd. Software Company. The lack of agreement for the base pressure is directly related to the fact that the analyses lacked to properly model the last layer or the foundation layer, where extremely high values of unit pressures took place and were indirectly measured. In numerical terms, a unique and particular soil spring was assigned to this last layer. A fine readjustment of the stiffnesses for both situations, increasing towards more representative values, would certainly improve the matching quality of the load transfer curves of Fig. 5 and Fig. 6, especially at the deepest pile sections. The experience gained with these analyses has demonstrated that a deep stratum, stiffer than the overlying layers and leveled to the pile's base, shall be incorporated in the numerical back analysis (more information in Cunha and Kuklík 2003).

4. DEFINING THE DEPTH OF INFLUENCE ZONE AND ITS ESTIMATION

We introduce the subject considering the distribution of vertical stresses according to Fig. 7.

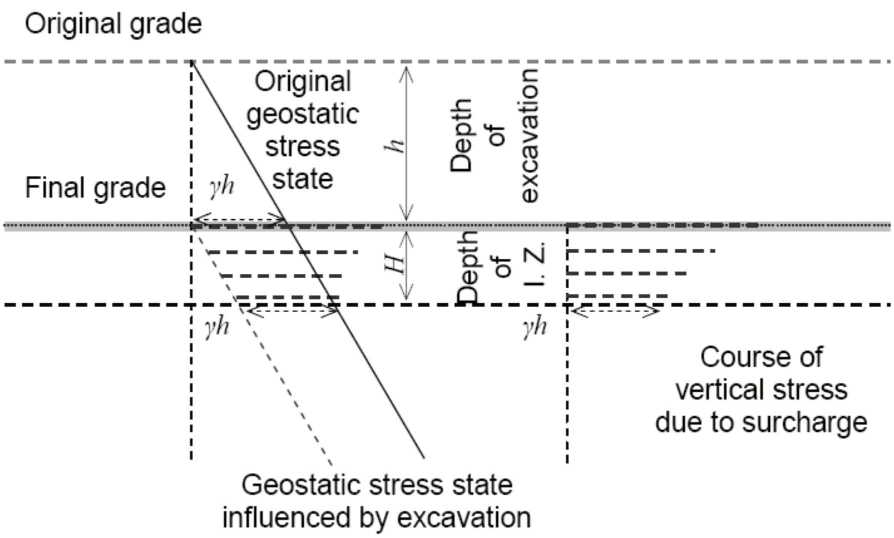

Figure 7: The governing idea of the influence zone calculation

Due to excavation to a certain depth h, the original geostatic stress state, which sets the initial compaction of soil represented by the preconsolidation pressure (see Janda et al. 2004; Kuklík 2006), the highest stress level in the soil recorded during the prior loading history, is reduced. Subsequent surcharge at the footing bottom gives further redistribution of the vertical stress. It is assumed that in the region where the vertical effective stress due to surcharge at the footing bottom combined with the reduced geostatic effective stress (by excavation) does not exceed the original geostatic effective stress, the skeleton deformations are negligible. This condition, in our sense, describes the depth of the influence zone H. If the uniform load is acting in an infinite strip (width of the strip is $2a$), the maximum of the stress function at the bottom ($z = H$) of the influence zone depth is (details in Kuklík et al. 2009).

$$\sigma_{zz}(0,H)=\frac{2f_z}{\pi}\arctan\sinh\alpha a \quad , \quad \alpha=\frac{\pi}{2H}\sqrt{\frac{2-2\nu}{1-2\nu}} \tag{13}$$

The influence zone depth is estimated by means of the quality

$$\sigma_{zz}(0,H)=\gamma h \tag{14}$$

where γ is the specific weight of soil and h is the depth of the excavation. Denoting

$$\beta=\frac{2\alpha a}{\pi}=\frac{a}{H}\sqrt{\frac{2-2\nu}{1-2\nu}}\ ,\ F_{strip}(\beta)=\frac{2}{\pi}\arctan\left(\sinh\frac{\beta\pi}{2}\right)$$

We can eliminate

$$\begin{aligned}H\ &=\frac{\pi a}{2}\sqrt{\frac{2-2\nu}{1-2\nu}}\frac{1}{\sinh^{-1}\left(\tan\dfrac{\pi\gamma h}{2f_z}\right)}=\frac{\pi(2a)}{4}\sqrt{\frac{2-2\nu}{1-2\nu}}\frac{1}{\sinh^{-1}\left(\tan\dfrac{\pi\gamma h}{2f_z}\right)}\\ &=\frac{\pi a}{2}\sqrt{\frac{2-2\nu}{1-2\nu}}\frac{1}{\ln\left(\sin\dfrac{\pi\gamma h}{2f_z}+1\right)-\ln(\cos\dfrac{\pi\gamma h}{2f_z})}\end{aligned} \tag{15}$$

This closed formula can be effectively used in civil engineering practice. Similarly, we can estimate the depth of influence zone in the case of rectangular footings. Let us introduce the transmission function

$$\begin{aligned}F(\beta)=&\frac{2}{\pi}\left(\arctan\frac{\beta\pi b}{2a}+\arctan\frac{\beta\pi}{2}\right)-\frac{4}{\pi^2}\int_0^1\frac{1}{\sqrt{\left(\dfrac{a}{b}\right)^2+1-t^2}}\arctan\sinh\left(\frac{\dfrac{\beta\pi b}{2a}}{\sqrt{1-t^2}}\sqrt{\left(\frac{a}{b}\right)^2+1-t^2}\right)dt\\ &-\frac{4}{\pi^2}\int_0^1\frac{1}{\sqrt{\left(\dfrac{b}{a}\right)^2+1-t^2}}\arctan\sinh\left(\frac{\dfrac{\beta\pi}{2}}{\sqrt{1-t^2}}\sqrt{\left(\frac{b}{a}\right)^2+1-t^2}\right)dt.\end{aligned} \tag{16}$$

The transmission function $F(\beta)$ describes the maximum layer bottom vertical stress $\sigma_z(0,0,H)$ by the unit uniform load acting in the rectangular region $2a\,x\,2b$. The depth of the influence zone is described by the point where the vertical stress due to surcharge reaches the value γh. The maximum stress below the centre of rectangle in the depth H can be expressed in following form

$$\sigma_z(0,0,H)=f_zF(\beta)=\gamma h \tag{17}$$

As the value of preconsolidation γh and the level of surcharge f_z are known, the value β is obtained as the inverse of the function $F(\beta)$. The following formula describes this statement and the idea how to calculate the depth of the influence zone

$$\frac{\gamma h}{f_z}=F(\beta)\rightarrow\beta=\frac{a}{H}\sqrt{\frac{2-2\nu}{1-2\nu}}\rightarrow H \tag{18}$$

Functions $F(\beta)$ for some quotients a/b are presented in Fig. 8.

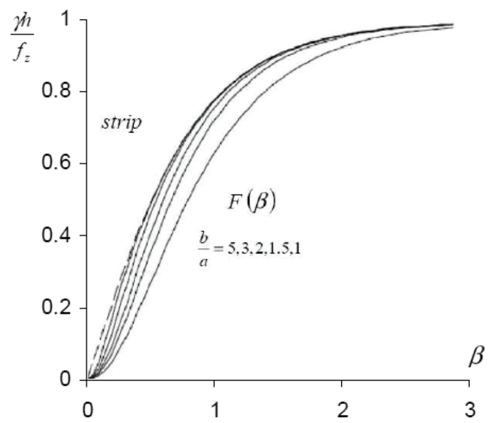

Figure 8: Courses of $F(\beta)$ function

5. CONCLUSIONS

In the scope of interest was mainly preconsolidation, special kind of the soil memory. In the laboratory was considered the significant influence of over consolidation on pore pressure evolution during isotropic consolidation in the triaxial test apparatus. Vice versa it was shown, employing the genetic algorithms, the soil parameters can be effectively investigated; namely initial parameters like void ration and preconsolidation effective mean pressure. In collaboration with UnB Brazil, provider of in situ tests, we observed significant effect of preconsolidation in the pile heel. The measured working diagrams can be explained only by the progress of the influence zone depth below the pile heel. The presented close formula and the graphs for the influence zone depth estimation give the way how to fit the experiment and numerical model.

6. ACKNOWLEDGMENTS

Financial support was provided by the project no. MSM 6840770001.

REFERENCE

[1] Cunha RP, Kuklík P (2003). Numerical Evaluation of Pile Foundations in Tropical Soils of the Federal District of Brazil by Means of a Semi-Analytical Mathematical Procedure. SOLOS E ROCHAS, SUELOS Y ROCAS, SOILS & ROCKS. 2003, vol. 26, no. 2, p. 167-182.

[2] Janda T, Kuklík P, and Šejnoha M. (2004). Mixed Experimental and Numerical Approach to Evaluation of Material Parameters of Clayey soils. International Journal of Geomechanics. 2004, vol. 4, no. 3, p. 199-206.

[3] Kuklík P, (2006). Several Comments on Influence Zone Depth Progress in Deep Hole Foundations. Proceedings of the GeoShanghai Conference 2006, Reston.

[4] Kuklík P, Šejnoha M, Mareš J (1998). Dimensional Reduction Applied to Specific Problems of Consolidation. Application of Numerical Methods to Geotechnical Problems. Wien: Springer, 1998, p. 337-346.

[5] Kuklík P, Kopáčková M, and Brouček M (2009). Elastic Layer Theory and Geomechanics. 1. ed. Praha: CTU Publishing House, 109 p.

[6] Terzaghi K, Peck RB, and Mesri Ch (1996) Soil Mechanics in Engineering Practice. Wiley, Chichester, NY.

[7] Matouš K, Lepš M, Zeman J, and Šejnoha M (2000). Applying genetic algorithms to selected topics commonly encountered in engineering practice. Comput. Methods Appl. Engrg., 119, 1600-1620.