An assessment of construction state and the subsoil conditions by loading tests and back analysis

Evaluation de 1'état de construction et de sous-sol par la méthode d'essai de charge et d'analyse inverse

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ABSTRACT: The long-term behaviour of a foundation under dynamic and cyclic loading due to the work of gantry machine is analysed. In order to evaluate true behaviour of a structure and to verify its design parameters together with the subsoil conditions assumed the load test method was used. The method applied is based on survey measurements of long-term displacement trends, the measurements of the foundation behaviour due to the live loads by high accuracy clinometres and the measurements of the immediate foundation response. In the back analyses the measured displacements were used to model the behaviour of subsoil under the foundation subjected to cyclic loading due to the work of gantry machine by application of FEM. The FEM prediction of structure behaviour for different boundary conditions was used for evaluation of structure destruction and necessary soil improvement.

RÉSUMÉ: On analyse la portance à long terme la poztance de foundation sous une charge dynamique et cyclique du poztique roulant pendant son travail. L'essai de charge est utilisé pour la détermination de la capacité réelle du portique et pour la contrôlé des paramètres du portique et ceux du sons-sol. La méthode appliquée est basée sur : la tendance de déplacement à long terme controlée par les mésures géodésiques, la portance de la fondation chargée controlée par les mésures de nivellement direct et la portance immédiate de la fondation sous la charge utile. L'analyse inverse basée sur les déplacements mésuréset l'implémentation de la métode des éléments finis (EF) ont servis la détermination du compostement du sons-sol chargé cycliquement par la fondation du portique. Une prédiction par la méthode des élements finis du compoztement de la structure du portique pour les conditions aux limites différentes a été utilisée pour évaluer les effets d'influence sur la structure du portique et le sol retorcé.

1 INTRODUCTION

High quality requirements for foundations under high class of accuracy gantry machines induce the same requirements for a subsoil and its long-term behavior. For example, for the machine with movable portal crane, the admissible longitudinal and transversal deformations due to machine's self-weight, processing parts and neighboring loads should not exceed γ_{adm} = 0,005 mm/m. (longitudinal inclination of bending line for the part of foundation with bed guides) and γ_{max} = 0,005 mm/m. (longitudinal and transversal inclination of bending line for the part of slab foundation for anchoring). In order to assure the required accuracy of work, proper functioning of processing table and stand sledge guides the foundation displacements can not exceed respective bending strains both in longitudinal as well as transversal direction. The higher linear, spatial, angular, parallel accuracy of manufactured elements the lower admissible bending strains have to be. The values presented here give the view about high requirements to be satisfied by foundation and a subsoil. Such factors as environmental influences of temperature and room air-conditioning have not been taken into account and the analysis has been focused on the geotechnical aspects, exclusively.

1. SITE CHARACTERISTICS

The foundation was designed as the beam with changing cross-section in longitudinal and transversal direction with dimensions of 45 m in length, 13 m in width and 7,5 m in height. It consists of two parallel channels at the top, two chambers at the beginning and the end and the gallery at the bottom part what gives the complex shape with different rigidity in longitudinal and transversal cross-sections.

Soil profile is shown in Figure 1. To the depth of 7,8 m below the surface there are fills and Holocene alluvial deposits built of mud, peat and fine sands with admixtures of humus. Below the foundation a subsoil consists of the layer of sandy clay with I_L = 0,23 and gravel and sandy gravel in medium dense and dense state, respectively. The density of soils increases with depth changing from $D_r = 0.5 \div 0.6$ to the value close to $D_r = 0.8$ for underlying layer of Pleistocene sands. The phreatic surface of groundwater varied between 1.5÷2.0 m below the subsoil the surface. General view and the cross-sections are presented in Fig. 2.

The foundation is enclosed by sheet pile wall with isolation of poliuretan foam in between. The loads are transmitted into the subsoil under the foundation base only without participation of the walls. The base was formed as a slab 45 m long and 13 m wide. Under the foundation there is a slab being the cap for 90 piles of TITAN 73/53 type with the length of 8 m, in regular net. According to the design the foundation was treated as a beam with different rigidity, resting on elastic subsoil. The history of the subsoil was rather complex due to former loadings from large foundations destroyed during the foundation work. The concrete class B30 W8 was placed in 2 m thick layers together with the main reinforcement made of ribbed steel 34GS and anti-shrinkage armouring.

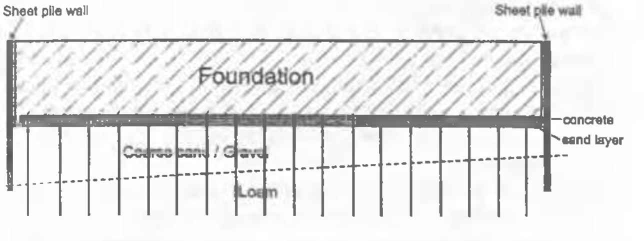

Fig. 1. Geotechnical profile

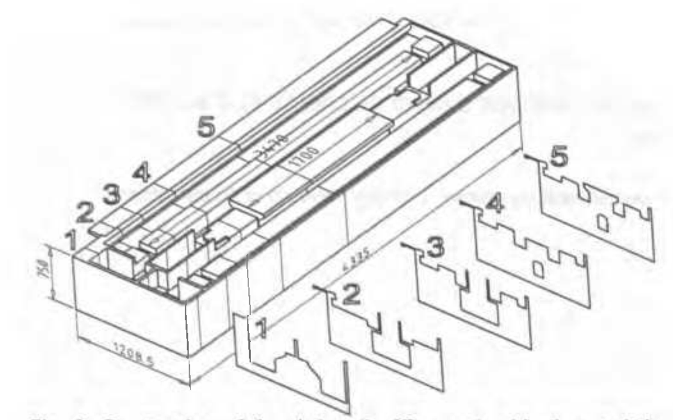

Fig. 2. Construction of foundation (in 3D space) with characteristic cross-sections

The foundation construction is symmetric with respect to the middle transversal cross-section. The exceptions are the foundation edges where various rooms and containers for processing wastes have been designed (Fig. 2).

From the view point of working requirements and correct work of the machine the important role plays the foundation deformation along the processing table and railway subgrade for the gantries (cross-sections 3÷5). Due to the large dimensions of the construction and differentiated loading the interaction of the foundation block and a subsoil is of the three-dimensional character. Along the foundation length the calculations may be simplified by the assumption of the beam resting on elastic subsoil what allows the evaluation the order of magnitude of the gantry railway deflection. However, the analysis of the foundation bending requires an application of more sophisticated methods including 3D character of the foundation work.

The deformations observed can be divided into two main groups i.e. the deformations caused by non-uniform settlement of the subsoil and the deformations caused by self-weight of the foundation and external live loads. The first group consists of the foundation transversal bendings, longitudinal tilts and longitudinal arc deflections as well as the deformations of more flexible parts of the foundation edges. The attempt of numerical modeling of this type of deformations has been carried out in 3D space. Second group consists of observed horizontal deformations of gantry railways occurring in the direction of processing table. Beside the external loads, the reason of the deformations can be related to a degradation of strength parameters of the concrete. In this case, the static analysis has been made in plane strain state for to selected cross-sections.

The load tests method was used for investigation of true behaviour of the structure and for verification of its design parameters and soil conditions. The applied method was based on:

1. Long-term observation of displacement trends controlled by survey measurements (7 series of measurements of displacements for 19 observation points stabilised in foundation). The analysis of the displacements shows clear, general tendency of the foundation to the rotation in perpendicular direction. In longitudinal axes there was no significant difference of displacements in particular regions observed. The maximum deviation measured on tracks amounted $\Delta H = 0{,}090$ mm. During the measurements the gantry machine was loaded with the elements of 106 t mass. The estimation of settlements was made on the basis of known values of loading induced by overhead crane, with geotechnical parameters assumed from geotechnical investigations. The estimated settlements for static load coming from fully loaded gantry machine were in the range from +0,039 to -0,063 mm.
2. The observations of the foundation behaviour under the live loads controlled by the measurements of high accuracy clinometres, (8 stabilised points along the foundation edges). These measurements gave the answer about the trend of displacements.
3. Measurements of rotating table by the high accuracy clinometers.
4. The measurements of the immediate foundation response due to the live loads. The measurements were carried out by LVDT transducers in eight series, in which many points were simultaneously measured for the same point of time.

The investigations were performed in three investigation stands localised on the gantry rail. The observations were being carried out simultaneously inside the beam (stand 1) and on both foundation sides (stand 2 and 3). The gauges were localised on the beam with measuring bases fixed to the floor at distance of 1.5 m from the foundation edge.

The displacements were measured with LVDT transducers connected by the bridge to PC. The accuracy of measurements was 10^{-3} mm. For all stands the same scheme of subgrade has been assumed which corresponded to ordinary work of gantry machine.

The program of each series included the measurements during the travel of the gantry machine within its ordinary working area, covering usually three to six travels.

The results of the measurements have shown that there was possible to determine the location of cross-section, in which change of the construction behaviour occurs. The change could be related to the change of the rigidity as well as to the discontinuity surfaces (e.g. the cracks). General behaviour of the foundation during the travel of the system of gantries i.e. the values of the vertical displacements for particular observation points during the increase and decrease of loading due to passing of the gantries is characterised by three groups:

Initial – displacement measurements of steel corps and differential displacements between elements of machine and foundation. The measurement was taken every 0,5 sec. (for each channel) and stored directly on the floppy-disc with the accuracy of 3×10^{-3}[mm].

Main – displacement measurements of the foundation edges and changes in a distance of the chosen parts of the foundation

Supplementary – displacement measurements of the foundation edges and changes in a distance of the chosen parts of foundation during the load test under the 1060 kN and live movement of the gantries.

Additional measurements such as: power consumption due to the crane movement, air temperature and groundwater level were also taken.

The results obtained show the differentiation of displacements in particular sections of track. At short intervals the increase of permanent displacements was of the order of 0.10 mm. The parts near the cracks deformed more than other foundation elements. It was a consequence of an essential decrease of the beam's rigidity (Fig. 2). The measurements indicate that the longitudinal displacements are of the secondary importance and transversal displacements do tend to increase and are of the plastic-elastic character. The location of observation points is presented in Fig. 3.

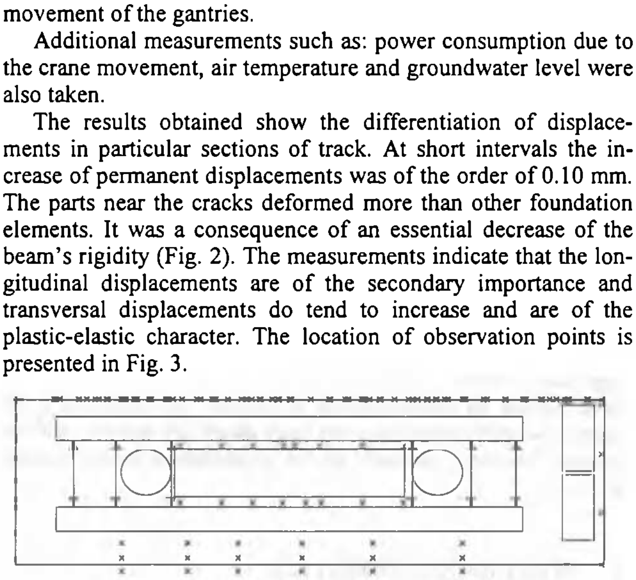

Fig. 3. Lay-out of observation points.

2 SUPPLEMENTARY CALCULATIONS FOR SELECTED ELEMENTS OF REINFORCED CONCRETE FOUNDATION

The calculations aimed at the verification of the selected factors which might influence the behavior of the construction, in particular the possibility of the existence of shrinkage cracks and thermal effects, foundation settlements and deformations related to the alternate loads and changes of groundwater table. Additionally, the analysis of loads at the level of foundation base in subsequent construction phases together with the foundation work and evaluation of the foundation deflection along the longitudinal axis during load tests has been carried out. Numerical simulations have been made with the help of Plaxis ver. 7.11 numerical code (Vermeer, Brinkgreve, 1998). The calculations were performed for plane strain conditions concerning both longitudinal and transversal cross-sections of the foundation. The calculations were to model all working phases starting from construction of the foundation up to its present state and to predict the character and the magnitudes of potential displacements in the future. Isolines of the total displacements are shown in Figs. 4 and 5.

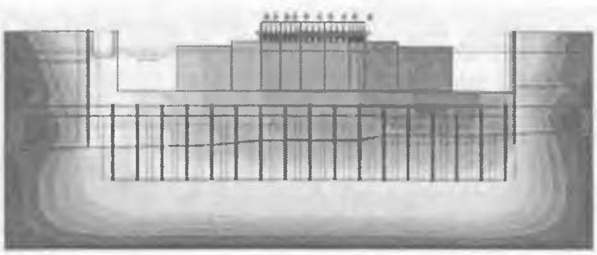

Fig. 4. Isolines of the displacements – longitudinal cross-section

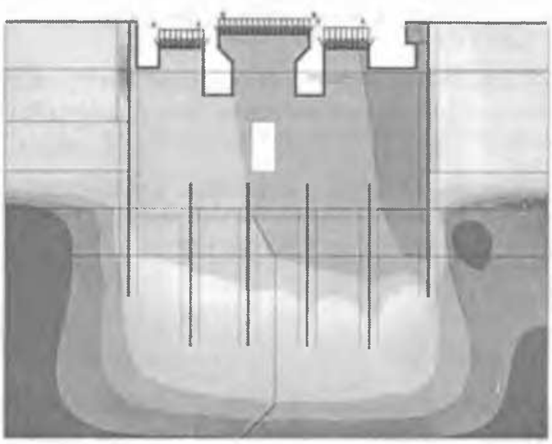

Fig. 5. Isolines of the displacements – transversal cross-section

3 PRELIMINARY STATIC ANALYSIS IN 3D STATE

The calculations of the boundary value problem regarding the interaction of the foundation block and the elastic subsoil have been carried out using FEM non-linear analysis. Non-linearity was related in this case to the geometry (gradual increase of loads in respective order and taking into account strain state for stress calculations). In the calculations the updated Lagrange system and elastic material (Hook's linear isotropic elasticity with E=2,5e7 kPa, ν=0,25, ρ=2,5 t/m^3) was assumed.

In order to avoid the mesh refinement for thin construction elements the geometry of foundation block has been simplified. After all, it concerned the walls at the foundation edges. These elements were not capped at its tops and had some cracks due to concrete shrinkage thus it was assumed that the elements are of the secondary influence on the general work of the foundation. Removed construction elements have been replaced by additional, uniformly distributed load. The comparison of the true and simplified geometry of the foundation is presented in Fig. 8. The mesh was built of 14945 linear tetrahedral elements with the average dimension of 1.0 m.

The subsoil has been replaced by elastic supports located in the nodes of foundation base. Measured tilt of the foundation, the results of the analysis of soil settlements made by *Plaxis* and studies of the loading history allowed the determination of respective differentiation of the subsoil rigidity. Due to this fact higher rigidity of the subsoil in north-west part of the foundation base (28500 kN/m^3 along 1/3 of the width × 2/3 of the length = 2/9 of total base surface) and for the rest of the foundation area 30% lower rigidity (20000 kN/m^3 have been assumed). East upper edge of the foundation has been blocked in the horizontal direction (extending slab rested on the floor of the hall).

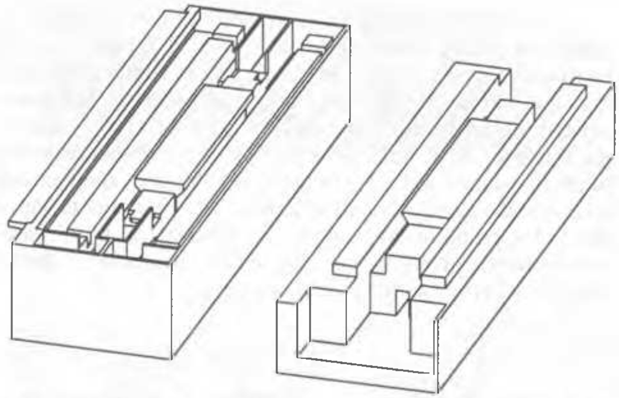

Fig. 8. Simplification of the foundation geometry for calculation in 3D state

In the initial phase of loading the gravitation (g=9.81 m/s^2) and subsequently live loads from gantries (2×650 kN + 2×770 kN) and from treated elements on the processing table (1060 kN) were introduced. The interaction of the hydrostatic forces has been excluded from the analysis. Finite element mesh scheme of the construction together with the results of the calculations performed are presented in Figs. 7 i 8.

a) b)

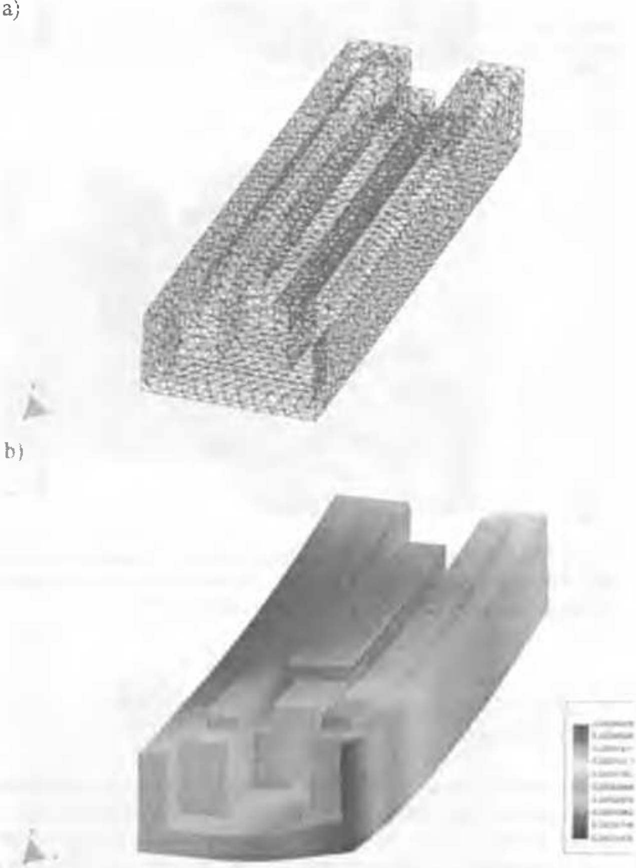

Fig. 7. The calculation results of the foundation in 3D state: a) finite element mesh assumed b) izolines of displacement valus. | u | [m]

General deformation of the foundation block for the assumed differentiation of the subsoil qualitatively corresponds to the measurements of the displacements (longitudinal deflection and bending of the block in the east direction). Deflection of the railway bed is $\Delta u_y \approx 0.5$ mm, whereas maximum difference of the settlements for eastern and western parts of the foundation is approximately $\Delta u_y \approx 0.3$ mm. In the case of the horizontal displacements maximum values of have been obtained for railways in the place of gantries stop $u_x \approx 3.0$ µm and for western edge of the foundation $u_x \approx 4.0$ µm.

Low displacement values are related to the assumption of simplified subsoil model and linear elastic constitutive law for concrete elements. However, calculated maximum values of principal stresses (<500 kPa) are much smaller than tensile strength of the concrete. It indicates the existence of cracks of the concrete which were confirmed by careful visual inspection of the foundation surface. The origin of the cracks is not directly related to the action of static external loads but rather to shrinkage and swelling of the concrete due to chemical reactions between cement components and quartz aggregates, thermal stresses and errors during construction.

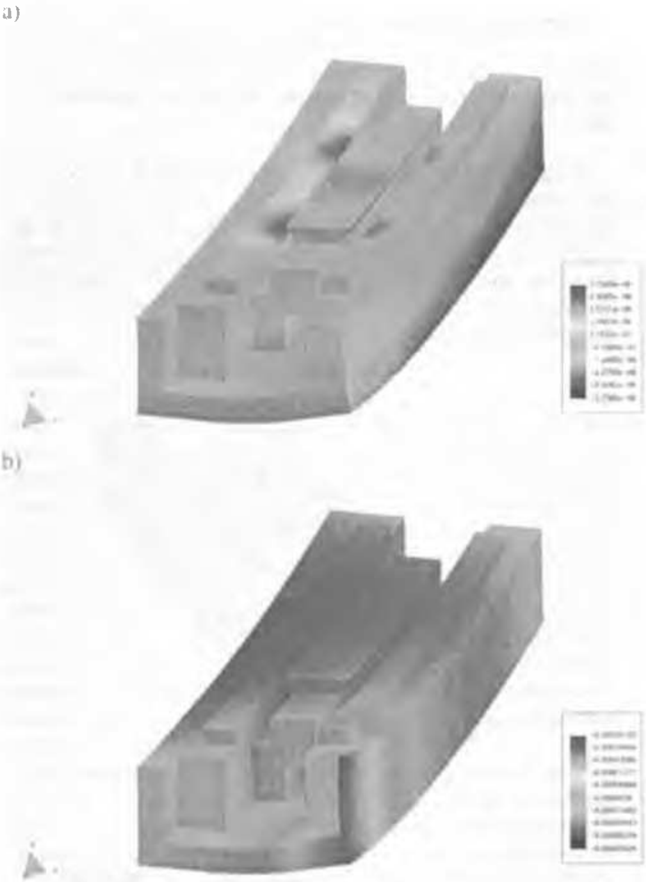

Fig. 8. The calculation results of the foundation in 3D state a) izolines of the horizontal displacements u_x [m], b) izolines of the vertical displacements u_y [m], (the foundation deformation scaled x5000)

4 STATIC ANALYSIS OF TRANSVERSAL CROSS-SECTIONS IN PLANE STRAIN STATE

More careful static analysis concerning better modeling of the concrete has been carried out for cross-sections No. 3 and 5 in plane strain state ($\sigma_{zz} \neq 0$). Besides linear elasticity, three additional plasticity criteria and simple law for concrete failure have been introduced. In formulation of the boundary value problems the attention has been focused on the explanation of the observed horizontal displacements of the railway subgrade in the direction of processing table. To do that the moveable vertical supports along the foundation base width and single fixed vertical in the middle of the foundation base have been assumed. It allowed to obtain fixed reference system for occurring displacements.

First two plasticity criteria regarded compressive and tensile strength of concrete, respectively, the yield surfaces of which are described by the following formulae:

$$\|\sigma\| - f_c = 0, \qquad \|\sigma\| - f_t = 0, \tag{1}$$

where

$$\|\sigma\| = \sqrt{\sigma_1^2 + \sigma_2^2 + \sigma_3^2}\ , \tag{2}$$

denotes stress norm expressed in terms of principal stresses. In the case of first plasticity criterion for the calculations of the norm negative values of principal stress components are assumed, only, whereas for second plasticity criterion positive values, only. Symbols f_c i f_t are compressive and tensile strengths of concrete, respectively which were assumed as f_c=30000 kPa, f_t=3000 kPa.

Third plasticity criterion is related to concrete shear strength. In this case Drucker-Prager model was considered in the following form:

$$3\alpha p + \|\sigma\| - K = 0$$

$$\alpha = \frac{2\sin\phi}{\sqrt{3}(3-\sin\phi)},\quad K = \frac{6c\cos\phi}{\sqrt{3}(3-\sin\phi)},\quad p = \frac{1}{3}\sigma_{kk} \tag{3}$$

where ϕ denotes an angle of internal friction (ϕ=45°) and c is the cohesion which was assumed equal to tensile strength of the concrete f_t=3000 kPa. In the calculations associated low rule was adapted.

Failure state of the concrete structure was described by the parameter d=(0÷1), the value of which influences the components of stress tensor in the following way:

$$\sigma_{ij}^d = (1-d)\sigma_{ij} \tag{4}$$

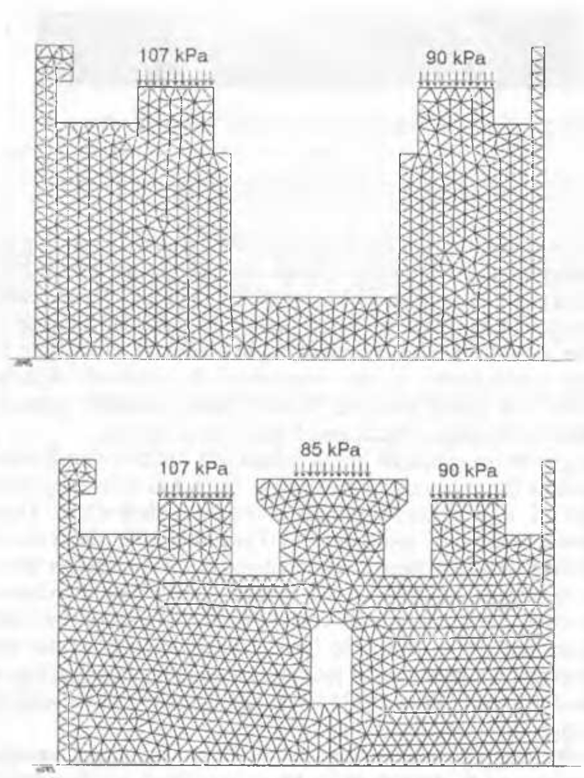

Fig. 9. Finite element mesh for cross-sections No. 3 and 5 in plane strain state with a scheme of boundary conditions.

The changes of the damage parameter d can be calculated form the following formulae:

$$d = d_t \alpha^{\beta} + d_c (1-\alpha)^{\beta},$$

$$d_t = 1-(1-a_t)\frac{\varepsilon^0}{\varepsilon^{eq}} - a_t e^{-b_t\left(\varepsilon^{eq}-\varepsilon^0\right)}, \tag{5}$$

$$d_c = 1-(1-a_c)\frac{\varepsilon^0}{\varepsilon^{eq}} - a_c e^{-b_c\left(\varepsilon^{eq}-\varepsilon^0\right)}, \quad \alpha = \frac{\varepsilon^{eq}}{\varepsilon}$$

where ε^0 denotes the strains (norm), at which failure occurs; ε^{eq} the norm of positive principal components of strain; ε the strain norm; β, a_t, b_t, a_c, b_c are material parameters. For calculations the following values of model parameters were assumed: ε^0=2e-4, β=1, a_t=1.5, b_t=1000, a_c=1.1, b_c=100.

In first calculation stage the analysis concerned stress and strain state for cross-sections No. 3 and 5, in which non-symmetric loads from gantries, dead load on the processing table and self-weight of the construction have been assumed (107, 90, 85 kPa and (9.81 m/s^2)x(2.5 t/m^3), respectively). Schematic finite element mesh (linear triangular elements) is presented in Fig. 9.

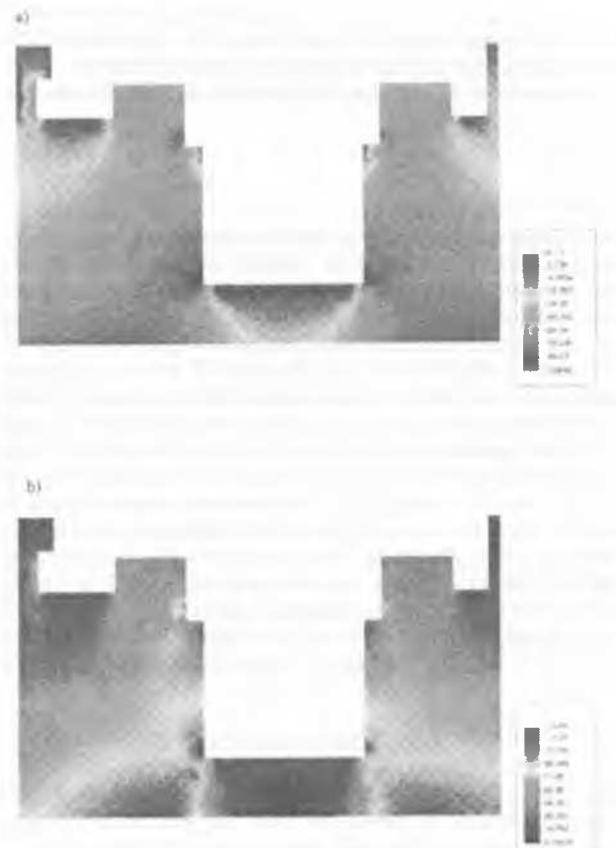

Fig 10. Calculation results in plane strain state (cross-section No. 3, stage 1)
a) izolines of mean stress p; b) izolines of deviatoric stress q

The goal of first calculation stage assuming plane strain state was the verification of the possibilities of the occurrence of inelastic deformations and cracks under action of ordinary live loads and that degradation process of the concrete strength did not occur. Additionally, the verification of horizontal values of railway deformations with respect to the calculated values in 3D space has been carried out. The examples of the calculation results are presented in Fig. 10.

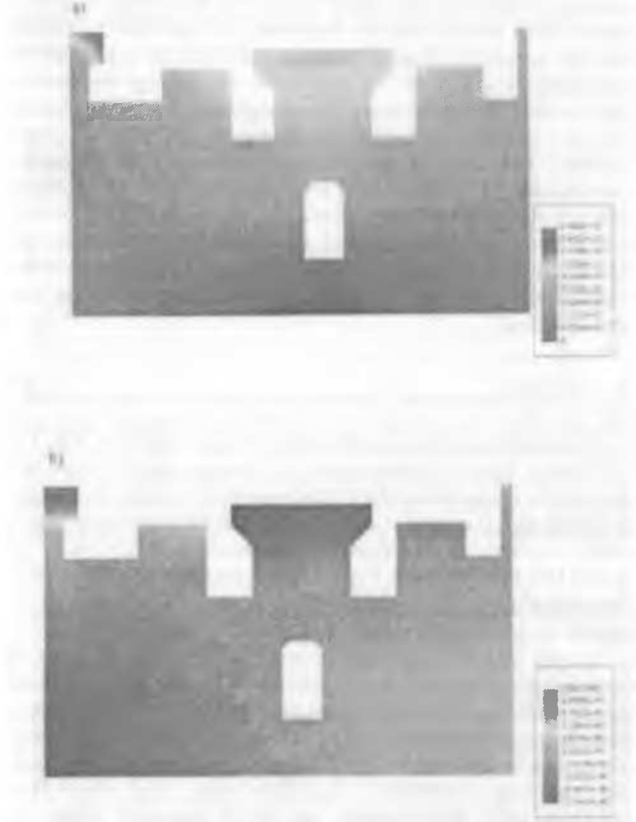

Fig. 11. The calculation results of the foundation in plane strain state (cross-section No. 5, stage 2) a) izolines of displacement values $|\mathbf{u}|$ [m] b) izolines of the horizontal displacements u_x [m],

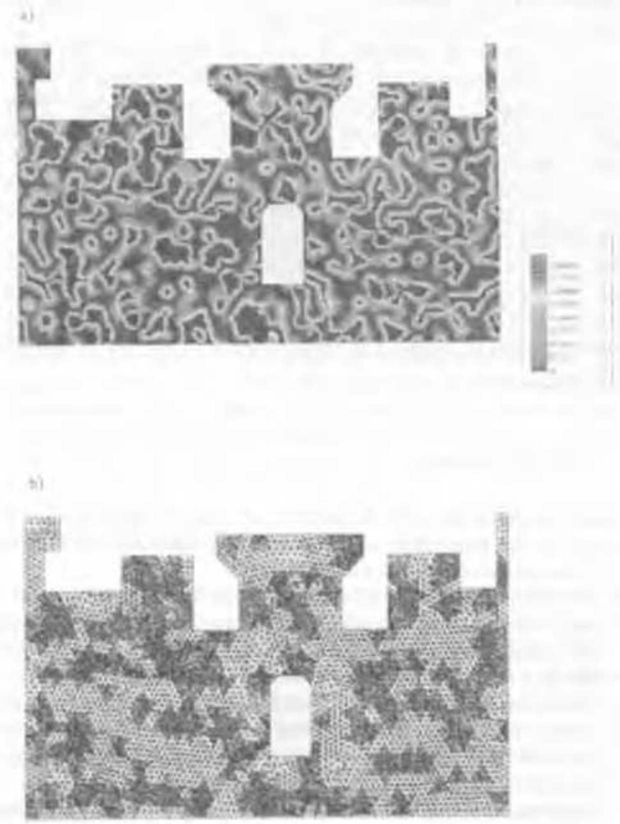

Fig. 12. The calculation results in plane strain state (cross-section No. 5, stage 3) a) izolines of the damage parameter b) finite element mesh for last calculation step.

In second stage of static analysis for plane strain state the loads from gantries have been replaced by substituted displacements which were to invoke the failure of construction system and the horizontal displacements of railways of the order of magnitude observed during measurements. During this procedure the ratio of displacement rate of right and left track equal to the ratio of respective loads from the gantry has been preserved. (107/90). The range of calculations was limited to the middle cross-section of the foundation (cross-section No. 5). In order to increase the accuracy within the areas of localization of plastic deformations the finite element mesh has been refined in the subsequent calculation steps. For the control of the finite element mesh change after its refinement the following parameter has been assumed:

$$\kappa = \sqrt{0.5\varepsilon_{ij}^{p}\varepsilon_{ij}^{p}} \tag{6}$$

The examples of the calculation results are shown in Fig. 13. For forced vertical displacement of approximately 4 mm the horizontal component of the displacement of left track was equal to $u_x \approx 0.8$ mm. It caused the increase of vertical stress component within the area of the displacement applied to the value of $\sigma_{yy} \approx 25\,000$ kPa, so over two hundred times larger than loads from the gantry. It indicates that the origin of deformation of foundation bearing elements is not related to live loads.

In third calculation stage for plane strain state the analysis of degradation of concrete and its influence on self-deformations of the foundation subjected to the ordinary live loads was performed. In the calculations the uniform distribution of strength parameters within the area of transversal cross-section analysed was applied (variance: $s_E = \pm 0{,}5e7$ kPa, $s_{fc} = \pm 5000$ kPa, $s_{ft} = \pm 1000$ kPa). Additionally, the initial non-zero values of damage parameter d together with uniform distribution were introduced (variance: $s_d = \pm 0{,}6$). For the control of the finite element mesh change after its refinement the parameter d has been assumed. Although relatively high degradation of strength parameters of the foundation was applied no plastic deformation due to ordinary live loads was observed.

The increase of parameter d cause the reduction of the internal stress and external forces may cause local increase of deformation, whereas in the areas adjacent to the localized failure zone the increase of stresses is observed what is related to the static equilibrium condition.

The example of the calculation results obtained in stage 3 is shown in Fig. 12. Maximum values of horizontal displacement component in the region of left track were equal to $u_x \approx 0.03$ mm. The increase of this value in numerical simulations can be possible only when assuming large local failure at the base of the railway bed (i.e. in the areas of localization of plastic deformations reported in stage 2) or concerning such factors as thermal changes.

5 CONCLUSIONS

Based on the numerical simulations performed regarding the behavior of the foundation under processing machines the following conclusions can be formulated:

1. The tilts and bending of the foundation block occurring during initial phase of its work were caused by consolidation of the subsoil related to complex hydrogeological conditions and loading history.
2. Assuming the correct construction of the foundation and its proper founding the degradation of its strength parameters is impossible – the values of stresses are much lower than the compressive, tensile and shear plastic limits of the concrete.
3. Significant degradation of concrete strength parameters does not cause the plastic deformation in transversal cross-sections of the foundation analysed. Such deformation might occur at the presence of deep transversal cracks of the foundation block what would require more advanced numerical analysis in 3D space. However, such cracks have not been observed during visual inspection of the structure.
4. Additional factor which may have some influence on the value of self-deformations of the foundation are thermal changes from the surrounding environment together with thermal changes resulting from its specific working conditions. However, the verification including the influence of thermal factor on stress and strain states of the foundation requires an implementation of more complex thermodynamic analysis.

6 REFERENCES

Vermeer at all (1998) Plaxis. Finite Element Code for Soil and Rock Plasticity. A.A. Balkema, Rotterdam, Brookfield.