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THE REINFORCEMENT OF TUNNEL LINING OF FORMER SERVICE TUNNEL DUE TO THE LOAD CAUSED BY A NEW UNDERGROUND STRUCTURE

1. Geological conditions in a locality

These are typical for the city Brno (Fig. 1). Locality is covered by continuous and quite huge layer of anthropogenous materials (layers more than 6 meters thick). Under this layer is situated a complex of quarternary fluvial embedded strata. The upper is comprised of clay and mould-clay and lower is formed by sandy gravels whith addition of mould. This layer is a relict of a river-bed. Pre-quarternary subbase in wide surrounding is formed by miocene calcic clay of baden era, stiff consistence. Underground water level is slightly pressured and is situated in depth of 8 to 9 meters under the surface (Fig. 2). Geological subbase of miocene clays is considered to be impermeable.

2. Construction of the former service tunnel

The effected primary (deep) service tunnel „Malinovského square" in the city Brno was built in year 1991 by company Subterra a.s. according to the design of Interprojekt Praha. Tunnel lining is created by multiple layers of shot-crete SB 25 (by Schmidt method was estimated pressure strenght of average value 38.4 MPa) and two layers of wire meshes (100 × 100 × 6.3 mm). Immediate supporting of bored sequence before the lining was created had been done by stiff-flexile frame consisted of K24 armatures. Complete thickness of lining in the top is and in the sides is 350 mm and in the bottom it is 640 mm. The shape of tunnel's crosssection is a horse-shoe type with inner meassures: width 5660 mm and height 5280 mm. In present is this section of service tunnel used by technical infrastructures only with low intensity.

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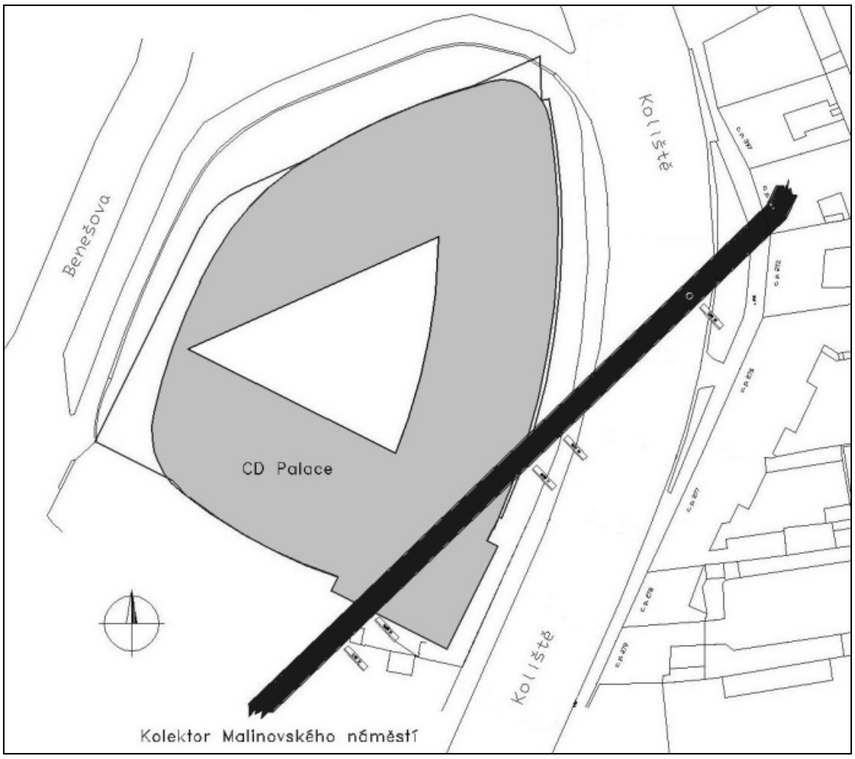

Fig. 1. Situation of CD Palace (grey) and service tunnel (black)

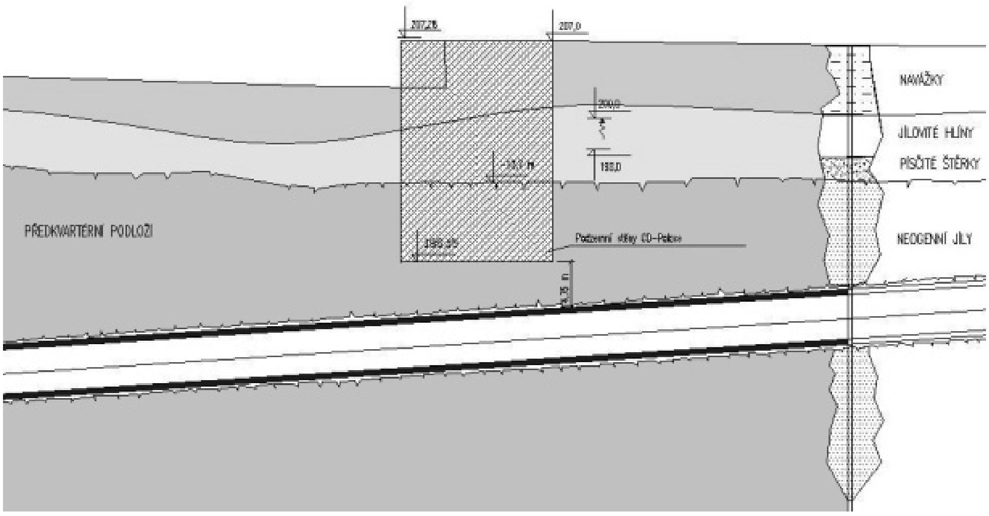

Fig. 2. Geological conditions underneath CD Palace. Position of service tunnel Malinovského square and underground founding walls of CD Palace is shown

3. Mathematical modeling and its results

Modeling was done by programmes ANSYS and PLAXIS 8.2

1) Calculation of two-dimensional model by FEM programme ANSYS show that construction of former service tunnel will be probably seriously effected by additional load caused by undergroung foundations of the CD Palace. Tunnel lining and tunnel as a whole is pressed into the subsoil and behaves as elastic structure. In vertical direction the lining is compressed (maximal value is 6 mm), in horizontal direction it is pressed into the soil (2.5 mm at both sides). The bottom of the tunnel is relativly elevated — it buckles upwards by 2 mm. This causes dangerous tension stresses of peak values around 5 MPa. These stresses are very unfavourable because the bottom of the tunnel consists of un-reinforced concrete and is localy weakened by service-train tracks and by drainage duct.
2) Calculation by FEM programme ANSYS 3D confirmed the results of two-dimensional analysis. Consequently was possible to estimate the lenght of effected section of service tunnel, where will be necessary to reinforce the tunnel lining. On basis of the results obtained from 3D model was estimated that the tunnel has to be reinforced 5 meters outwards and 8 meters inwards the intersection of foundations of CD Palace and the service tunnel. 3D model shown that the construction of tunnel will be probably deformed even along longitudinal axis of tunnel. This concerns mainly the bottom of the service tunnel.
3) For FEM calculation of the plain model in PLAXIS 8.2 in typical cross-section was used Mohr-Coulomb material model. Seven calculation phases were set up.

Phase 0: that is a result of pre-processor. Pore pressures and effective geostatical stresses are generated. This phase is basic and all other steps beginns from it.

Phase 1: cross section of tunnel is „excavated". Plastic calculation is used to estimate deformations and stresses. To calculate the influence of immediate support of cross-section the value Mstage was set to 0.5 (it means that only 50% of created unbalanced forces are used). By this way the technology of excavation is reflected. Whole cross-section is excavated and is supported by the face and immediate supports.

Phase 2: completion of construction. Whole lining is created and it is bearing all loads, thus Mstage is set to 1.0.

Phase 5: by continuous iterations the defomations are stabilized. The calculation is set up as consolidation. That means that stop criteria is the value of excessed pore pressures. According to the permeability of materials used in model the pore pressures continuously return to the hydrostatical values. When the difference is sufficiently small the calculation is stopped.

Phase 6: another plastic calculation. The most important phase of calculation. The reinforcement lining is activated and additional load of 200 kPa applied. Through numerous linear steps is performed non-linerear calculation.

Phase 7: again consolidation. After stabilization of excessed pore pressures the calculation is over.

Even according to the PLAXIS 8.2 calculation the tunnel lining as a whole is pressed to the subsoil (Fig. 3). Maximal compresion of the crown is 35 mm inwards, maximal deformation of the sides of the lining is 13.4 mm outwards. These values are higher than values obtained from ANSYS calculations.

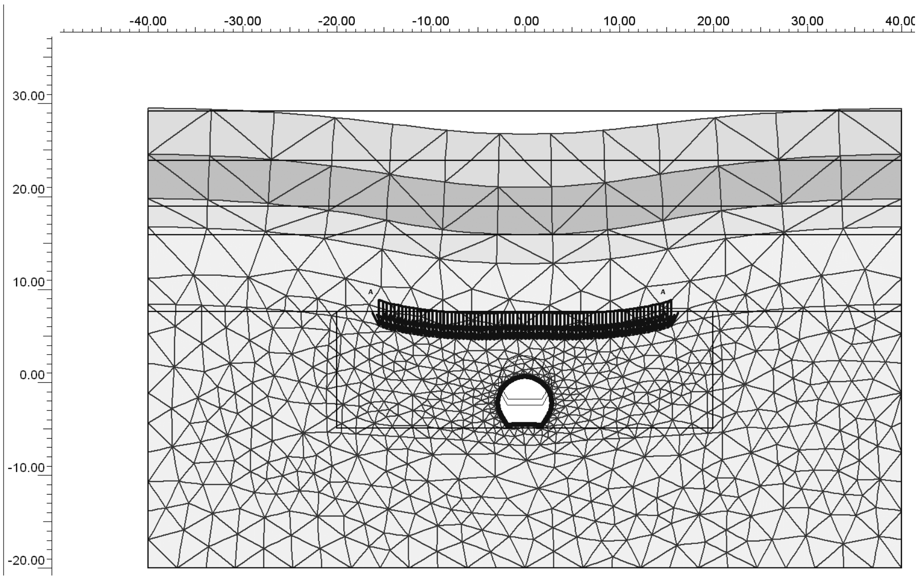

Fig. 3. FEM model created in PLAXIS 8.2 — cross-section (deformed, after final phase 7)

4. Design of reinforcement

According to the fact that capacity of service tunnel Malinovského square is used by very small part (service tunnel was constructed too big) the consultation with the controller of the tunnel led to the disgne of additional inner reinforcement to be used. Values of inner forces from 2D PLAXIS model were taken to the design. Reinforcement was designed in two alternatives (Figs 4 and 5):

— alternative 1:
  - crown and sides: one layer of wire mesh (100/100/6.3 mm), truss supporting arc girders ANKRA-ASTA 50 by span of 800 mm and additional tension reinforcement rods R25 per 200 mm. All covered in the layer of shotcrete 150 mm thick.
  - bottom: pair of rolled profiles I 220 placed into bored tranverse slot (300 mm wide, 420 deep) every 800 mm. These profiles will be bolted together with truss girders form upper part of reinforcement. Slot will be filled by concrete C25/30 afterwards.

— alternative 2:
  - crown and sides: one layer of wire mesh (100/100/6.3 mm), truss supporting arc girders ANKRA-ASTA 50 by span of 800 mm and additional tension reinforcement rods R20 per 200 mm. All covered in the layer of fibre reinforced shotcrete 100 mm thick.
  - bottom: pair of rolled profiles I 220 placed into bored tranverse slot (300 mm wide, 420 deep) every 800 mm. These profiles will be bolted together with truss girders form upper part of reinforcement. Slot will be filled by concrete C25/30 afterwards.

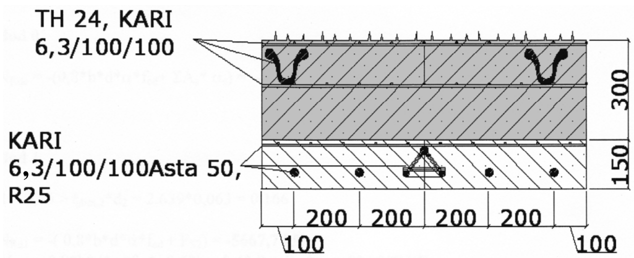

Fig. 4. Design of reinforcement — crown and sides

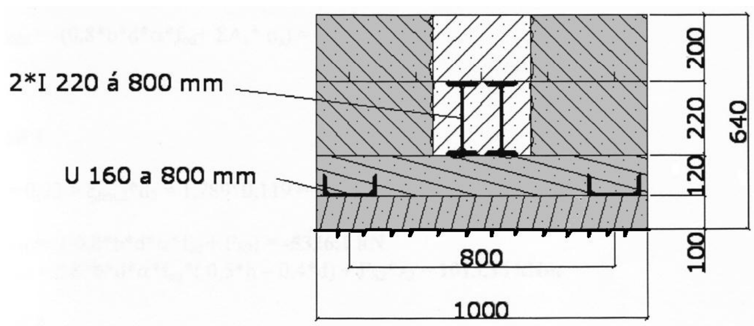

Fig. 5. Design of reinforcement — bottom

5. Conclusions

Difficult problem of additional loading of former underground structure in such a general case as was solved could by calculated only with use of mathematical modelling (in this case FEM was used). Software used and compared was ANSYS 2D, ANSYS 3D and PLAXYS 8.2 2D. Calculation with PLAXIS 8.2 2D showed bigger effects of loading (deformations, stresses, …). Using its results design was done by method of additional inner reinforcement. Usage of fibre reinforced shotcrete seems to be favorable if implementation is done by precise way.

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