Refurbishment and Underground Space Development of Moscow P.I. Tchaikovsky Conservatory

Restauration et extension souterraine du musée P.I. Tchaikovsky de Moscou

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ABSTRACT: Superficial channels on very soft clay deposits undergoing consolidation processes can generate tension zones that potentially can induce semi-vertical cracking. During construction of any underground works, such as tunnels, these cracks can be reactivated, especially if the construction process causes significant changes in the initial stress state of the ground, and then generates important deformation of the tunnel lining from confining loss around the tunnel, especially if dowels rings are used as lining. On the other hand, it is also possible to generate significant lining deformations if there are changes in the state of stress in the ground's surface due to the dredging of channels. This paper presents a case history about the behavior and numerical modeling of the primary tunnel lining during and after tunneling with an EPB machine in Mexico City soft clay deposits subjected to decompression stresses caused by the dredging of channels. Total displacements induced during tunneling under superficial channels were high but less than 1% of the tunnel diameter. After dredging, such channels' additional deformations were induced in the lining because of a reactivation of pre-existent cracks in the clay deposit. Numerical modeling was carried out to study the optimal solution. Based on numerical results, two solutions were applied: lining reinforcement and soil improvement.

RÉSUMÉ : Canaux superficiels sur les dépôts d'argile très douces en cours de processus de consolidation peut générer des zones de tension qui peut potentiellement induire des semi-verticale fissuration. Lors de la construction des ouvrages souterrains, tels que les tunnels, ces fissures peuvent être réactivés, surtout si le processus de construction entraîne des changements importants dans l'état initial des contraintes du sol, puis génère une déformation importante du revêtement du tunnel de la perte de confinement autour du tunnel, surtout si les chevilles des anneaux sont utilisés comme doublure. Cet article présente une étude de cas sur le comportement et la modélisation numérique du revêtement du tunnel principal pendant et après un tunnel avec une machine EPB dans les dépôts de Mexico argile molle soumis à une décompression contraintes provoquées par le dragage des chenaux. Déplacements totaux induits lors des tunnels sous canaux superficiels étaient élevés, mais moins de 1% du diamètre du tunnel. Après dragage ont été produites déplacement supplémentaire relance revêtement se craquelle. Les modèles numériques ont été utilisés pour étudier ces facteurs et déterminer la solution optimale. Avec ces résultats, nous proposons deux solutions: augmenter le revêtement et l'amélioration des sols.

KEYWORDS: tunneling in soft soils, soil fracture, decompression stresses, Mexico city tunnels.

1 INTRODUCTION

The Túnel Emisor Oriente (TEO, Spanish acronym for Eastern Emitter Tunnel) will be the new drainage system for Mexico City. It is located to the north of the city and it is a circular tunnel 62 km long, of 7 m inner diameter, set at variable depths between 30 and 155 m. It crosses all types of soils along 97% of its length, from very soft to hard, with the rest of the length crossing volcanic rock. For its construction, Earth Pressure Balance (EPB) tunnel boring machines are used, with a primary lining formed by dowels rings with sections 0.35 and 0.40m thick (COMISSA 2010). Almost the entire tunnel is under the groundwater level, with pore pressures of up to 0.8MPa.

The project's first trajectory, approximately 8 km long, is located at a zone of very compressible clays with low shear resistance, with water content in the order of 300%, running parallel to a surface channel. A particular aspect of this section is that on land near the channel surface cracks have been observed, and in the zone where the tunnel crosses under the channel (1+032 to 1+300) it has been observed that before the crossing (0+920 to 1+032) important primary lining deformations have occurred, with a tendency to their stabilization. This anomalous behavior of the tunnel has been caused by a diversity of factors, among which stand out the channel's dredging and the presence of intense fracturing at the zone of that channel.

The objective of this work is to evaluate the effects on the tunnels of the unloading induced by dredging surface channels located on cracked clayey deposits, and as a particular case the TEO project is presented.

2 GEOTECHNICAL CONDITIONS

<u>Stratigraphy</u>. Subsoil conditions at the zone where the atypical deformations occurred on the tunnel's primary lining are (Fig 1):

i. *Superficial Crust* (0 to 3m). It is a stratum formed by interspersions of sandy silts and hard silty sands, and on occasions fills up to 2m thick.
ii. *Superior Clayey Series* (3 to 26m). These are clays and silts of high plasticity with thin lenses of volcanic ash and sandy silts.
iii. *Hard Layer* (26 to 28 m). These are interspersions of sandy silts and silty sands (tunnel is located at the inferior part of the Superior Clayey Series resting on the Hard Layer).
iv. *Inferior Clayey Series* (28 and 42 m). It is a very compressible clayey deposit.

<u>Conditions of subterranean water</u>. At this zone the groundwater level is located at 3m depth, and the pore pressure measured at the tunnel's axis is in the order of u_{axis}=145kN/m^2, which is 65kN/m^2 less than the hydrostatic pressure.

the refurbishment and restoration activities in the Big Hall (Figure 3) and in the lobbies were practically completed.

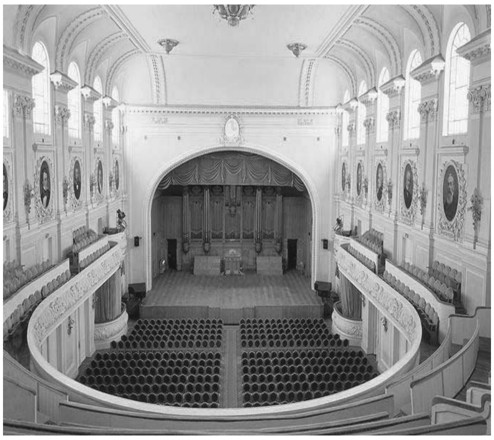


Figure 3. Conservatory Big Hall after refurbishment

This largely complicated the underground work below the building, because even negligible footing settlements during geotechnical operations could generate cracks in the structures and to destroy decorations and restorations, since long-term monitoring of the Conservatory building structural health prompted that 10 mm footings settlements would generate cracks in the superstructure parts.

In view of the foregoing, the design solutions for the underground portion of the Moscow P.I. Tchaikovsky Conservatory were developed that accounted for the following requirements of the technical assignment:

- underground development operations below the Conservatory building, including the Big Hall premises, were to be performed to 4,5 m depth below the basement floor, i.e. 3,5 m below the footings;
- 6 months deadline for the underground space development was assigned;
- extra settlements of the Conservatory building footings should not exceed 10 mm.

The geological section includes contemporary, upper and mid-Quaternary deposits as well as Upper Jurassic and Upper Carbon deposits (Figure 4).

Contemporary deposits – antropogenic soils (1), represented by a mixture of sand-sandy loam-clay loam soils, compacted and not-compacted, with low moisture content and moistened 0,8…4,6 m thick. They are underlain by Upper Quaternary alluvial deposits (2), represented by sands of various grain-size composition and loose (in the lower part of the section), with low and high moisture content and water-saturated 6,0…14,3 m thick.

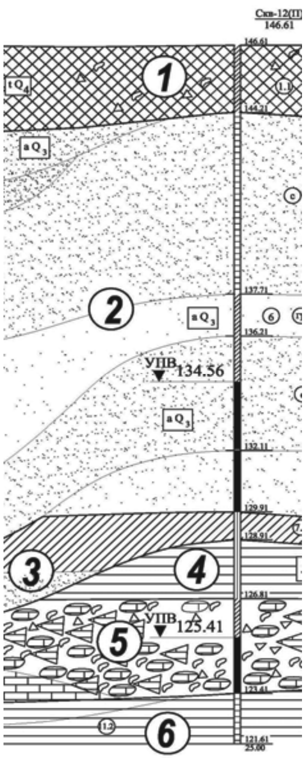


Figure 4. Geological section

Below Mid-Quaternary deposits, there were discovered fluvioglacial deposits (3) 4,8 m maximal thick, represented by sands and sandy loams. The sands are fine-grained of medium density, water-saturated, sandy clay loams are high to low plastic. Below there occur Upper Jurassic of Oxford tiers (4), represented by silty low-plastic clays. The bed maximum thickness is 7,6-7,8 m.

Deeper below Upper Carbonic deposits were found, represented by up to 3,4 m thick bed of Izmailovskyi limestone (5), crushed to powder or gravel; Mescherinskaya 3.3-6.0 m thick bed (6), represented by dusty low-plastic, medium hard and hard clays; Perkhurovskaya low-strength limestone bed, moistened and water-saturated.

In terms of hydrogeology the terrain is characterized by occurrence of three aquifers: phreatic, Super Jurassic at 5,0-15,7 m below the surface; Izmailovsky at 14,5-21,36 m depth; Perkhurovsky at 24,35-25,5 m depth. The terrain is naturally waterlogged.

The surveyed terrain features no karst or washout risk. No other unfavorable processes and events were found on the terrain.

Investigation of soil stress and strain behavior was numerically simulated, using FEM and non-linear soil models in PLAXIS 2D for a characteristic section along ducts 1,0…4,5 m deep. 3D analysis of structures was made with the help of MicroFe 2008 software (Figure 5). The analysis covered all work stages from soil stabilization to soil excavation down to design depths. Based on technological and architectural requirements as well as on structures' strength, stability and crack resistance in interaction of subsoil with the Conservatory building there were established the ultimate values of joint deformations equal to 10 mm. Prior to the project design development various options were analyzed that would enable arrangement of service ducts without auxiliary retaining structures.

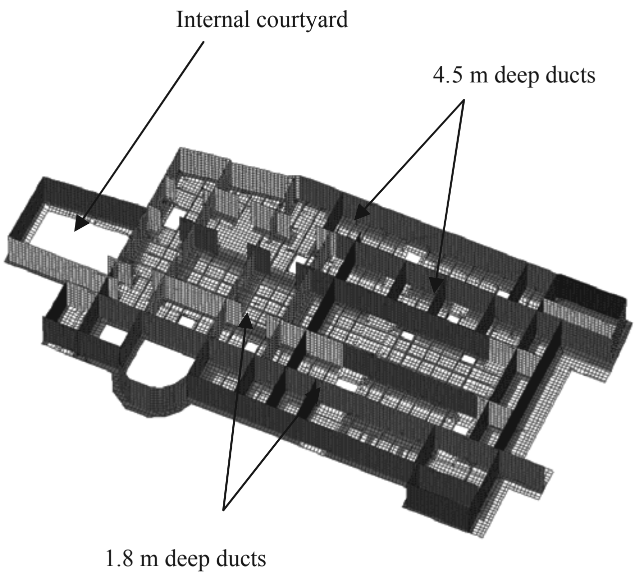


Figure 5. Conservatory underground model with 1.8 and 4.5 m deep service ducts

The analyses demonstrated that in such geological conditions 1,8 m deep ducts would hold with no support if they are strengthened with piles. But in order to exclude extra settlements of existing footings due to subsoil softening and walls caving, which could not be simulated in the analysis, there was designed and implemented multiple (up to 5 times) cement mortar compensation grouting behind the concrete walls in accordance with the method, developed by NIIOSP (Shulyatjev O.A. et al, 2008). The analysis showed that 4,0…4,5 m deep unsupported excavation of duct trenches would cause to extra settlements of existing footings, exceeding the 10 mm admissible value.

It was decided not to take into account existing piles made in 2005. This decision was based on two facts.

Firstly, the static load tests on existing piles showed great scatter in results. In fact, bearing capacity was two times lower then its design. Secondly, the connection of reinforced concrete capping beam to existing foundation made of crushed stone was questionable. Assesment of existing foundation and its structural integrity confirmed that the major part of the building load still transfered through existing strip foundation despite the presence of underpining piles.

With regard to aforementioned facts, a decision was made to cancel piling from the analyses to ensure a safety margin.

There were considered various trench retaining options: root piles, pressed-in piles, subsoil stabilization that could be technically possible in congested basement premises. However, none of the existing methods could resolve the above issues and ensure an adequate safety margin. Fast fabricated root piles usually have technological settlements unacceptable for the building in question. Pressed-in piles, having no such disadvantage, were used for the Bolshoi Theater refurbishment project, but they proved to be very labor intensive, and their installation

required much more time than the time, remaining before the P.I. Tchaikovsky Competition. Subsoil stabilization is not a sufficiently safe solution, as in such soils it was difficult to ensure adequate quality of respective construction operations.

In view of all above mentioned circumstances a decision was made to support 4.0...4.5 m ducts with root piles, reinforced with steel pipes.

Realization of the project required a technique and a sequence of the underground operations, which would minimize the construction the impact of construction works on the building structure (Petruknin V.P. et al. 2011). At the initial stage the soil was grouted with Microdur suspension, then the supporting root piles were erected (Figure 6), followed by stagewise soil excavation to design depths and raft concreting, only then the duct walls and the floor were erected.

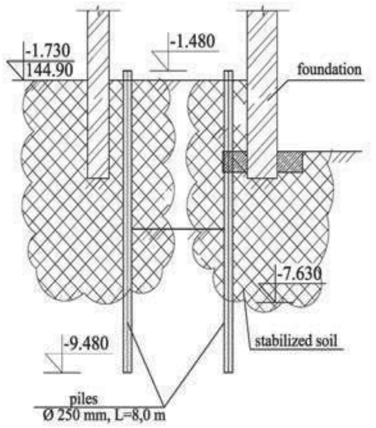


Figure 6. Soil stabilization layout and retaining piling

The reafter, in order to change the subsoil stress and strain behavior multiple compensation grouting was done behind the trench lining (Figure 7). Geodetic monitoring of the footing settlements showed that the resulting upheaval of markers was up to 2…3 mm.

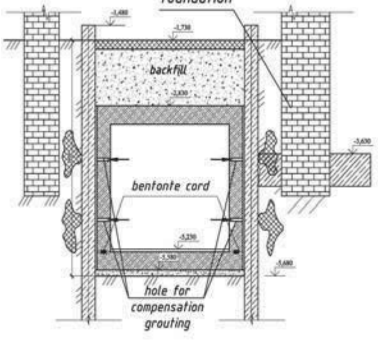
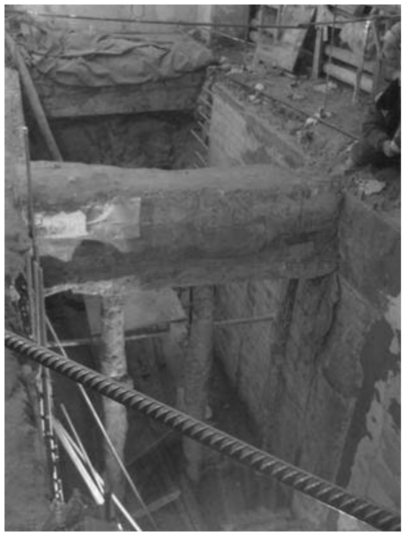


Figure 7. Duct concreting layout with the compensation grouting scheme

As is known the root piles' advantage is their low cost and fast erection, the only drawback is the respective technological settlements of footings, which can be as much as several centimeters (Shulyatjev O.A. et al, 2008; Petrukhin V.P. et al, 2008). In order to exclude technological settlements due to root piling there were performed tests, and a drilling set-up was developed, protected by a RF patent (Petrukhin V.P., Popsuenko I.K., Shulyatjev O.A., 2011), that compensated soil stress-strain behavior variation due to drilling by stagewise vertical pressurizing together with filling the bore hole with cement-sand grout, whose composition excluded sedimentation. The operations were performed gradually by 2 m work segments.

According to the above mentioned patent the hole drilling was accompanied by compaction and hole walls troweling to prevent water-saturated liquefied soil falling in the borehole. Figure 8 shows a photograph of a duct segment under the Big Hall, dug out manually to the design depth.



At the forefront root piles, made in 2005, are visible, they are joined together by a concrete raft, they are a sort of struts. Soil excavation from ducts is a rather labor-intensive process, therefore, as dimensions of the premises were small, the soil was dug out manually.

Figure 8. 4.5 m deep service duct

In order to reduce footings settlements during ducts mining operations the soil under bearing walls was grouted with "Microdur" suspension.

Therefore, the subsequent soil excavation showed that soil stabilization had been done adequately, however, practically no traces of grouting were discovered at some points in spite of the customer permanent strict control and designer supervision. Thus, it indirectly proved that the safer selected option i.e., retaining piles, was correct.

Soil mining under bearing walls of the building was a complicated issue. As is mentioned above, it is wrong during soil mining to rely on earlier arranged root piling. Therefore, a steel frame set-up was elaborated for 4,5 m deep ducts, which supported a part of the wall, under which the soil was dug out.

Mining 1,8 deep ducts was even more difficult. Notably, shallow ducts mining was performed not only in soil, it was often done along the body of rabble stone footing (the footing width was up to 1,5 m), therefore, soil excavation looked as non-mechanized mining (all operations were manual).

Underground development project involved various geotechnical operations that required on-line integrated geotechnical monitoring, performed by a specialized company in the most optimal way. Moreover, the designer company (Gersevanov NIIOSP) carried out their own supervision of the vertical and lateral displacements of the footings and of the Conservatory building structure. The congested conditions of the refurbishment operations, numerous labor force, multiple material storage sites, etc. restricted installation of an up-to-date on-line settlements monitoring system. Nevertheless, geodetic monitoring of root piling operations, soil excavation and grouting operations was done daily, for 4,5 deep ducts it was done twice a day. Figure 9 shows geodetic markers layout, whose number exceeds 90 pieces to ensure registration of all relevant displacements of the structures.

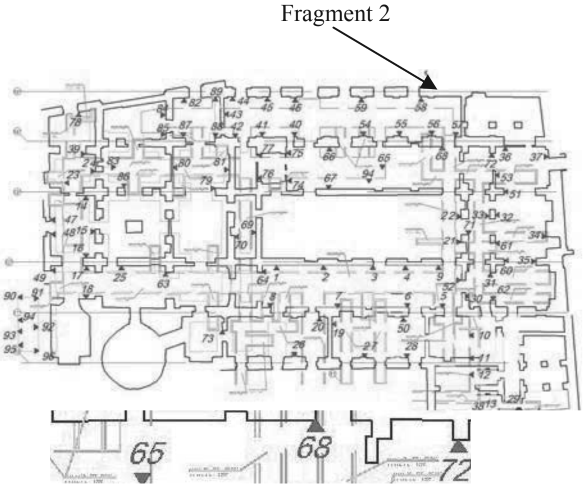


Figure 9 Geodetic markers layout and check markers 56, 57, 58 layout fragment

Geodetic monitoring data, including foundation settlement data precisely, confirmed analytical results and showed that most settlements stayed within 2…4 mm range (maximum 8 mm), the maximum differential settlement was 0,001 (Fig. 10). Foundation settlements due to root piling operations were zero. 2D analysis showed that at the project design stage subsoil grouting reduced extra settlements to 3 -5 mm.

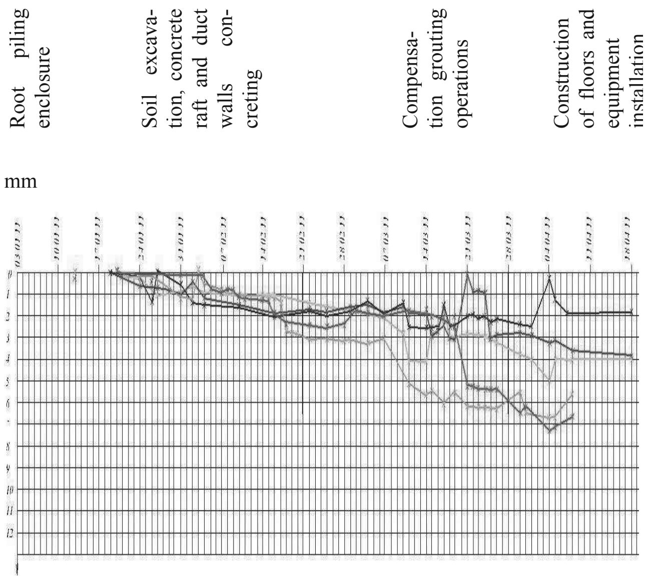


Figure 10 Markers settlement diagram along duct walls under the Big Hal

The diagram shows that during soil excavation from the trench there were settlements up to 6-7 mm while during grouting there was observed an increase of those values to 3 mm, followed by settlement stabilization. The check measurements in 2012 showed total absence of extra settlements of the Conservatory building footings.

The uniqueness of this effort consisted in that the large-scale historic building underground development operations (very sensitive to deformations) were carried out with practically no extra settlements, and during these operations no cracks, even hair-wide, were observed in the already refurbished part of the Conservatory. These operations took just 4 months that let the XVIIIth Jubilee P.I.Tchaikovsky Competition to be held in due time.

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