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Geotechnical and structural design of primary support for shaft and tunnelling construction works at Santos station-Lisbon subway - a 3D FE modelling approach

Conception Géotechnique et Structurelle du Support Primaire pour les Travaux de Construction de Puits et de Tunnels à la Gare de Santos - Métro de Lisbonne. Une Approche de Modélisation aux EF 3D

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ABSTRACT: As part of the extension of the Yellow and Green lines of the Lisbon Subway, the underground alignment of lot 2 starts at Santos Station and runs southwards ending at the terminal of the Cais do Sodré Station. The design of a new underground station – Santos Station has been considerably demanding. The highly complex geological and geotechnical challenges and the constraints related to surface occupation in a densely populated part of the city, where ground movement assessment is paramount, played a significant role in the selection of available design options for enabling the shaft and tunnelling works of the station. The geological and geotechnical challenging conditions are expressed by highly complex multi-layered terrain with formations of diverse nature such as Made Ground, Miocene formation, the upper cretaceous Lisbon Volcanic Complex, and the underlying Cretaceous formation. The scope of the article is to provide an insight of the achievements made by means of the design tool that has been adopted for the assessment of the stress and strain conditions of the surrounding ground due to the shaft and tunnelling construction works, as well as the assessment of forces and displacements on the primary support, enabling the carrying-out of stability checks and design optimization. As such, for this purpose, 3D FE models (Plaxis 3D 2022.01) have been developed. The ground and groundwater conditions, the constitutive modelling of ground behaviour, the structure-ground-structure interaction and the construction sequence of the shaft and tunnelling works have been comprehensively considered in these analyses.

RÉSUMÉ: Dans le cadre du prolongement des lignes Jaunes et Vertes du Métro de Lisbonne, le tracé souterrain du lot 2 commence à la gare de Santos et se dirige vers le sud pour se terminer au terminal de la gare de Cais do Sodré. La conception d'une nouvelle station de métro – la gare Santos – a été très exigeante. Les défis géologiques et géotechniques très complexes et les contraintes liées à l'occupation de la surface dans une partie densément peuplée de la ville, où l'évaluation des mouvements de terrain est primordiale, ont joué un rôle important dans la sélection des options de conception disponibles pour permettre les travaux de puits et de tunnels de la station. Les conditions géologiques et géotechniques difficiles s'expriment par un terrain multicouche très complexe avec des formations de nature diverse telles que terrain remblayé, la formation du Miocène, le Complexe Volcanique de Lisbonne du Crétacé supérieur et la formation sous-jacente du Crétacé. Le but de l'article est de donner un aperçu des réalisations réalisées grâce à l'outil de conception qui a été adopté pour l'évaluation des conditions de contrainte et de déformation du sol environnant dues aux travaux de construction de puits et de tunnels, ainsi que les évaluation des efforts et des déplacements sur le revêtement primaire, permettant de réaliser des contrôles de stabilité et d'optimiser la conception. À cet effet, des modèles Plaxis 3D 2022.01 ont été développés. Les conditions du sol et des eaux souterraines, la modélisation constitutive du comportement du sol, l'interaction structure-sol-structure et la séquence de construction des travaux de puits et de tunnel ont été largement prises en compte dans ces analyses.

Keywords: Constitutive modelling of ground behaviour; structure-ground-structure interaction; staged construction; 3D FE modelling; tunnelling design.

1 INTRODUCTION

The primary support works at Santos Station may be categorised into 4 parts: Central shaft, access shaft, underground works - north side, and underground works - south side.

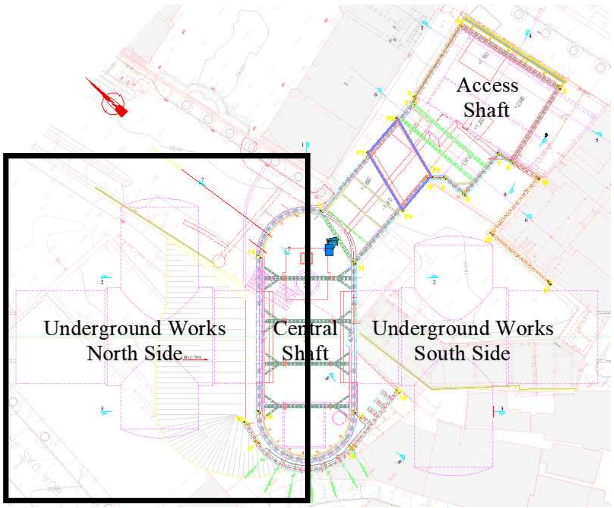

Figure 1. Plan view of primary support works at Santos Station.

The present article will be focused on the modelling procedures that have been developed for enabling tunnelling design check on the northern side of the underground works.

2 MODEL OVERVIEW

The 3D FE model considers the northern half of the central shaft, the central gallery, and the transversal gallery of the underground works on the northern side of Santos Station.

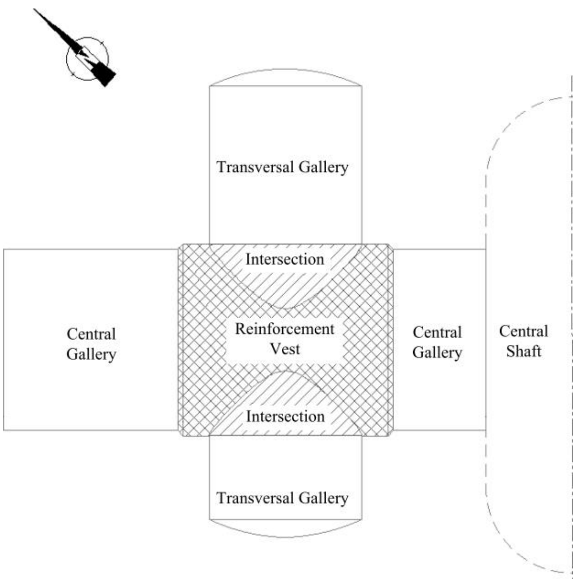

Figure 2. Plan view of the northern side of Santos Station.

The works started with the construction of a local pile wall, consisting in the installation of 0.8m diameter and 1m spaced contiguous piles, for enabling the creation of the working site at level 21.6mAOD from where the shaft construction will take place.

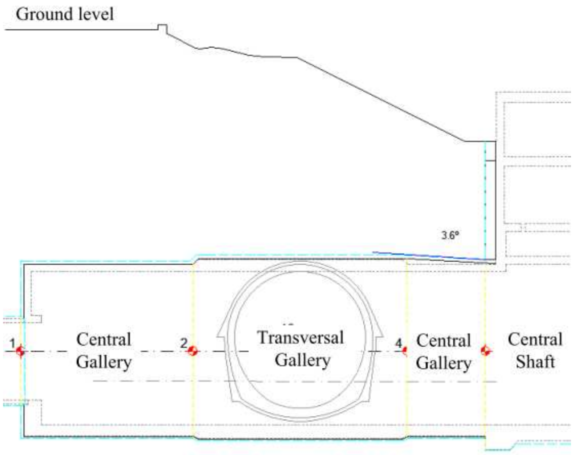

Figure 3. Longitudinal profile of the northern side of Santos Station.

The 45m long, 16m wide (total) and 30m deep central shaft comprises the installation of 1m diameter reinforced concrete piles spaced by 1.5m, a sequence of excavation down to design levels (20.3mAOD, 14.2mAOD, 9.23mAOD, 4.03mAOD, -1.07mAOD and -8.84mAOD) followed respectively by the installation of propping and/or anchoring sub-structures (21.1mAOD, 15.0mAOD, 9.9mAOD, 4.8mAOD, -0.5mAOD and -0.3mAOD).

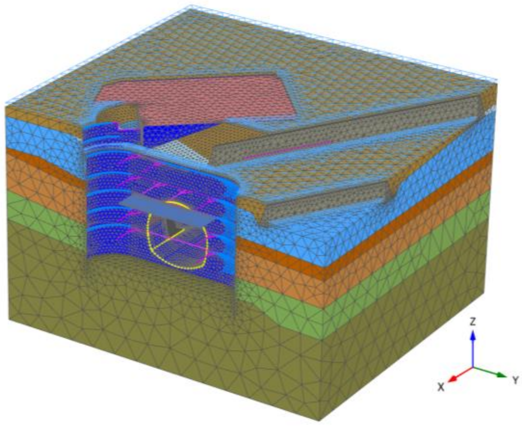

Figure 4. Plaxis 3D FE model of the primary support works of the northern side of Santos Station (Plaxis 3D).

The size of the 3D FE model has been established for ensuring that forces and displacements on the primary support are not impacted by the effects of the boundary fixity conditions. As such, the dimensions of the model are approximately 100m by 100m in plan view and 65m in height ($Z_{máx}$=28.7mAOD and Z_{min}=-35mAOD).

3 DESIGN STRATEGY

The tunnelling design strategy consisted in the development of a 3D FE model that is implicitly and explicitly capable of providing reliable output results for performing safety check calculations, according with the NP EN 1992 (EC2), NP EN 1993 (EC3) and NP EN 1997 (EC7), for all the failure mode mechanisms that have been identified. In order to accomplish such goal, the 3D FE model considered the following features:

- the heterogeneity of geological and geotechnical ground conditions;
- groundwater conditions and respective dewatering effects in the primary support;
- spatial distribution of loading conditions at the ground surface;
- ground-structure interaction and boundary conditions;
- explicit modelling of reinforcement elements of the primary support;
- ageing of shotcrete along the staged construction;
- comprehensive modelling of construction sequence of shaft and tunnelling works.

3.1 Modelling of ground conditions

The geological conditions at the Santos Station site, shown below in Figure 5, are characterized by a top Made Ground layer overlying a sequence of Miocene formation (blue layers) and the upper cretaceous Lisbon Volcanic Complex (CVL) which is composed by weathered and fractured Basalt (dark brown layer) overlying Tuffs (light brown layer). Underlying the CVL formation it has also been identified the presence of the Bica Limestone formation (green layers).

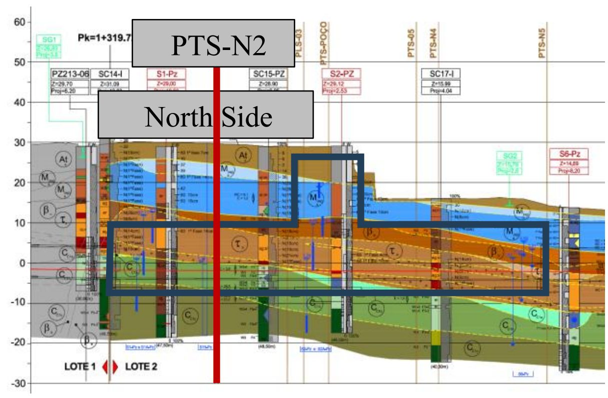

Figure 5. Geological and geotechnical longitudinal profile at Santos Station.

As it can be seen above in Figure 5, the tunnelling works are mainly developed through CVL materials i.e., generally highly fractured, and weathered Basalts and Tuffs. With greater expression on the northern side of the tunnelling works, the underlying cretaceous Limestone is also foreseen to be intersected throughout the excavation of the lower bench.

The idealised ground model on the northern side of the tunnel considers a critical geological cross-section (PTS-N2) while maintaining constant the thickness of strata along the longitudinal axis of the tunnel.

While the behaviour of the cretaceous Limestone (Bica Formation) has been modelled by means of the Hoek-Brown constitutive model for the remaining strata (Soils and Weathered Rock) a Hardening Soil like behaviour has been adopted, given that these materials are deemed to exhibit stress dependent stiffness.

Provided the temporary nature of the works under assessment, undrained behaviour has been assigned to the cohesive Miocene Formation.

3.2 Groundwater conditions

The piezometric readings and the observed waterflow concentrated in the region of Santos Station suggests that the underground works are likely to occur partially below the groundwater table. As such, for design purposes an initial groundwater table (GWT) at 14.8mAOD has been assumed.

During the tunnelling works a system of systematic drainage holes have been installed along the contour of excavation, especially in formations like the Cretacic where the presence of groundwater is likely. This has been thus implicitly considered by means of imposing localised dewatering conditions at the tunnel location down to the desired levels (Z=-0.5mAOD for top heading excavation and Z=-9.5mAOD for bench excavation).

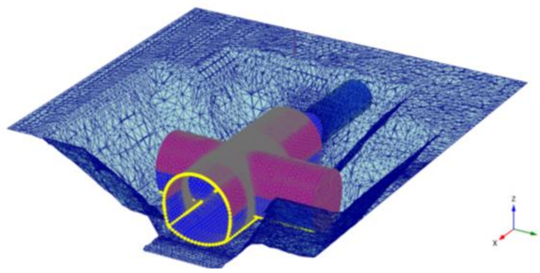

Figure 6. Groundwater surface condition for dewatering at level Z=-9.5mAOD (Plaxis 3D).

3.3 Loading conditions

Permanent and variable loading conditions have been considered such as the self-weight of the ground, groundwater, primary support, and existing structures. In places where ground surface circulation is possible a nominal live load of 10kPa has been assigned, with its magnitude conveniently adjusted for Ultimate Limit State (ULS) calculations.

3.4 Ground-structure interaction

The volume elements that are modelling existing structures at the ground surface level and the plate elements modelling the shaft have been provided with interface elements for mobilising an interface strength reduction of 0.7 (R_{inter}~2/3φ′). The plate elements modelling the primary support of the tunnel have been provided with interface elements with R_{inter}=1.0 for simulating a rigid condition between the shotcrete and the surrounding ground.

3.5 Boundary conditions

In terms of the boundary conditions of the model, the bottom boundary is fully fixed, the lateral boundaries are all normally fixed and the upper boundary is free for deformation.

The connectivity conditions between structural elements with limited capacity for loading transfer has been considered between the central shaft and the tunnel, the top heading and the temporary invert, the bench, and the invert. In such connections the degree of freedom for rotation has been assigned (sections with no capacity for bending moment transfer).

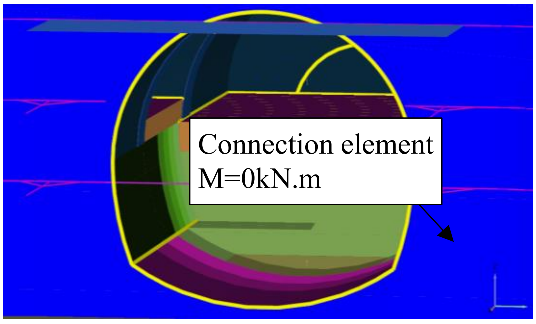

Figure 7. Connection elements for modelling limited capacity for load transfer (Plaxis 3D).

3.6 Primary support and reinforcement

The primary support is composed by a 300mm thickness shotcrete layer reinforced with steel fibres and 1000mm spaced TH-29 profile girders. The primary support has been modelled by means of isotropic plate elements with combined effects of shotcrete and steel girders.

At the intersection between the central and transversal galleries an additional 100mm thickness reinforcement vest has been introduced by means of a linear elastic volume element with a dummy plate element placed at the middle fibres of the volume element.

The 12m long forepole umbrellas to be installed both at the portal and tunnel intersections have been installed with 0.4m spaced 76mm diameter, 14.6mm thickness steel tubes. These rigid bodies have been modelled with embedded beam elements only capable of mobilizing lateral resistance, the value of which has been conveniently assessed by means of the Bustamante and Doix empirical formulation.

The 1.5m by 1.5m layout of 76mm diameter grouted holes reinforced with 28mm diameter fibre-glass nails, to be installed at the top heading of the tunnel face, have been modelled with beam elements. At their installation stage their length is 12m. After 4 advancements with step of 2m, their length is reduced from 12m to 4m, the moment of which another set of 12m long ground nails is installed. This trimming and installation sequence is repeated along the whole excavation of the top heading both for central and transversal galleries.

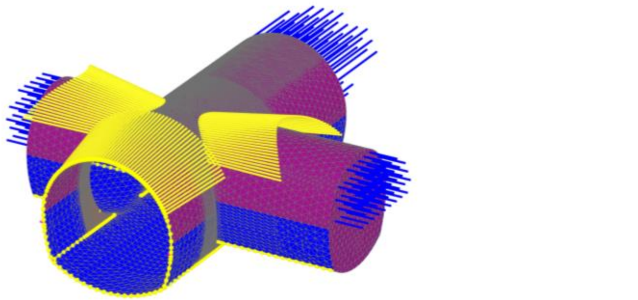

Figure 8. Primary support and reinforcement elements of north side tunnel (Plaxis 3D).

3.7 Ageing of shotcrete

The ageing of shotcrete has been modelled by means of stiffness increments which have been based on the curve extracted from the CEB-FIP Model Code 1990 shown below in Figure 9. For each sequence of excavation followed by lining installation a time increment of 1 day has been assumed.

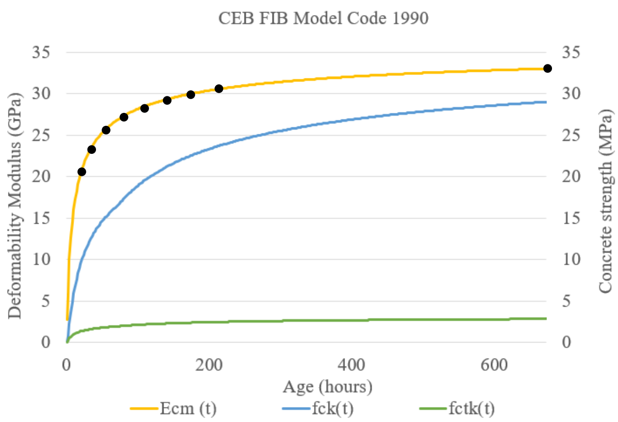

Figure 9. CEB-FIP Model Code 1990 Curves.

For reducing the modelling working time without compromising the validity of safety checks, the ageing of shotcrete has been modelled in steps of 1 day (24 hours) until 8 days (192 hours) from which the age of shotcrete is set to 28 days. As shown in Figure 9, the stiffness variation of shotcrete between 8 and 28 days is just about 3GPa, as such, refinement along these ages was deemed to bring negligible gains.

3.8 Staged construction

The modelling of the staged construction may be subdivided in 7 main parts, which are briefly described below:

- Initial conditions (phase 0 to phase 2) – Initial stress calculations with KO-procedure, plastic-nil phase, and application of loading pre-conditions at the ground level.
- Shaft construction (phase 3 to phase 18) – Shaft lining installation, staged excavation followed by activation of propping structures.
- Top heading of the central gallery (phase 19 to phase 109) – Installation of the starting forepole umbrella and tunnel face nails, excavation in advancement steps of 2m and installation of the primary support.
- Top heading of the transversal galleries (phase 110 to phase 141) – Installation of the reinforcement vest, activation of the intersection forepole umbrellas and tunnel face nails, excavation in advancement steps of 2m, installation of the primary support.
- Shaft Construction (phase 142 to phase 143) Excavation down to the minimum shaft design level followed by the installation of the propping substructure.
- Bench of central gallery (phase 144 to phase 233) – Excavation in advancement steps of 2m followed by primary support installation.
- Bench of transversal galleries (phase 233 to phase 265) – Installation of the reinforcement vest, excavation in advancement steps of 2m followed by primary support installation.

The central and transversal galleries of the north tunnel have been partialized has shown below in Figure 10.

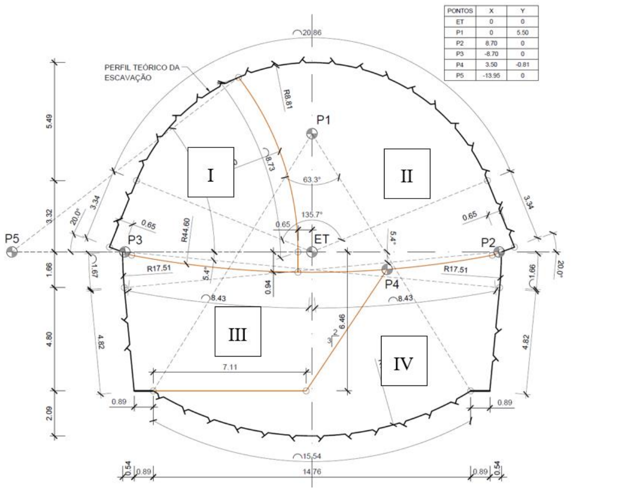

Figure 10. Partialization of the tunnel excavation.

The top heading works consists of excavation and support of sectors I and II with a maximum out-of-phase excavation of 8m. The sept that separates sectors I from II is uninstalled in each 4m of advancement of sector II, the moment of which temporary shotcrete invert is applied. Once the tunnel faces of sectors I and II are in-phase, works in sector II stops and in sector I restarts, with the sequence above described being repeated until tunnelling completion.

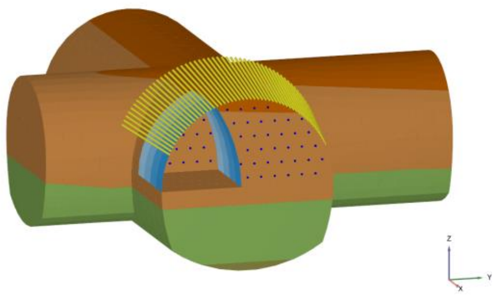

Figure 11. Excavation and support of sector I (Plaxis 3D).

Likewise, the bench excavation is done in two stages (sectors III and IV) with an out-of-phase excavation of 8m. Shotcrete invert is applied in each 4m of advancement.

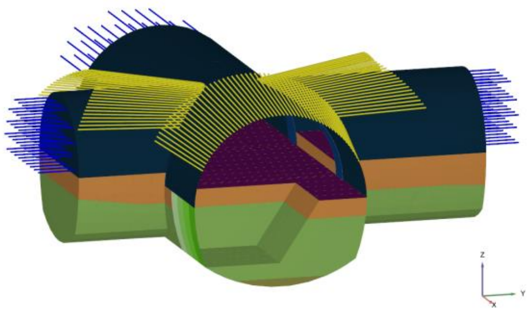

Figure 12. Excavation and support of sector III (Plaxis 3D).

4 SAFETY CHECKING

The output results of the 3D FE model enabled performing the required ULS and SLS safety checks which have been carried out according to the EC2, EC3, and EC7 (Design Approach 1 Combinations 1 and 2 – C1 and C2).

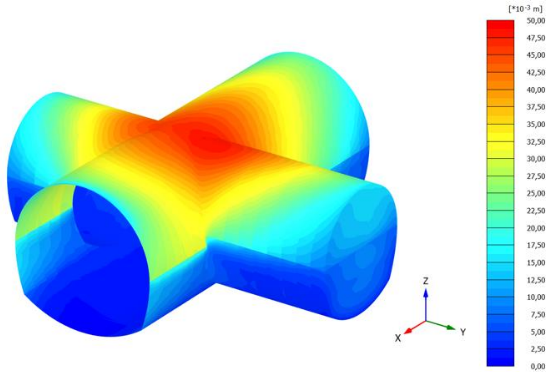

Figure 13. Total displacements (Plaxis 3D).

ULS safety checks:

- Shotcrete lining (*):
  - Bending resistance (C1)
  - Shearing resistance (C1)
  - Compression/tension resistance (C1)
- TH-29 steel girders:
  - Bending resistance (C1)
  - Shearing resistance (C1)
  - Bearing resistance of foundation ground (C2) (**)
- Forepole umbrellas:
  - Bending resistance (C1)
  - Shearing resistance (C1)
- Tunnel face:
  - Global stability analysis (C2) (***)

SLS safety checks:

- Displacements and convergences of tunnel section.
- Settlements of existing structures located in the region of influence of the tunnelling excavation.

(*) The safety check of both the shotcrete lining and shotcrete lining reinforced with steel fibres, has been performed at the early age of 1 day and for 28 days of age.

(**) The bearing resistance of foundation ground has been performed by means of a P2D model conveniently calibrated with the results obtained from the P3D model.

(***) The global stability analysis has been performed by means of the strength reduction method with ground parameters and loading conditions set according to C2.

5 CONCLUSIONS

The 3D FE model provided the necessary output results (forces and displacements) for performing the required safety checks against all identified failure mechanisms of the tunnelling works. Given the comprehensive modelling of the staged construction and mesh refinement that has been duly implemented, the present limit state analysis with FEM solution likely approximates the exact solution, as such, optimised design is plausible. On the other hand, a 3D FE modelling design approach for complex problems, as was the case, is recognized to be time consuming, machine demanding and requires expertise FE modellers with sound geotechnical engineering background.

Finally, the author hopes that the present article may have contributed to improve the awareness and acknowledgement of 3D FE modelling as a compelling tool for the future of geotechnical design.

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