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3D modelling of cast-in-place anchored beamless trench wall with edge elements of different topology

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ABSTRACT: The article presents the analysis of an anchored beamless cast-in-place trench wall. In the usual practice, the analysis in question is 2D and sees trench wall as a continuous longitudinal structure, failing to consider the vertical joints between work zones, the non-interaction between horizontal longitudinal reinforcement of adjacent frames, the reducing stiffness of the trench wall due to two-directional cracking, and the inelastic behavior of the compressive zone of the concrete. Performed as part of a development project in Moscow, the calculation of the anchored beamless cast-in-place trench wall have shown that failure to take into account the above features of its behavior may lead to a largely distorted stress and strain state of the entire shoring during the erection and operation, and hence wrongly selected reinforcement of the shoring. The Midas GTS NX 2018-assisted structural calculations involved elements of various topology (2D and 3D) and used a three-dimensional problem. Alon with shoring design diagram, description is given in the article of the geoengineering and hydrogeological conditions of the construction site. The calculation data is presented in the form of isofields of horizontal and vertical displacements of the trench wall, and normal stresses in its planes for different simulation cases. Based on the results of the numerical modelling, a cast-in-place shoring design model is recommended that makes allowance for the actual stress and strain state.

1 INTRODUCTION

In the analysis presented herein, we calculated the behavior of an anchored beamless trench wall.

The practice of modeling the designs of shoring of excavations with different software shows that a number of important behavioral parameters remain neglected, leading to a major distortion of the actual stress and strain state of the trench wall during erection and operation and, hence, wrongly selected reinforcement options.

One such shoring of excavation was calculated as part of a development project in Moscow. This excavation is shored with a cast-in-place ferroconcrete trench wall (concrete grade B40) with a thickness of 600 mm, depth of 21.5 m, reinforced with space frames of A500C rods. The trench wall has its bottom end embedded in a waterproof layer to the depth of 3.85–5.95 m, which makes it work like an advanced cutoff design. The top of the wall abuts a 600 × 700(h) mm cast-in-place ferroconcrete framing beam. The stability of the trench wall during excavation is ensured by embedding it 5.95 m deeper than the bottom of the excavation and installing of 3 tiers of temporary prestressed injection anchors with horizontal spacing of 1.5 m.

2 MATERIALS AND METHODS

Geologically, the excavation site is composed of modern soil, recent industrial deposits, upper quaternary covering deposits, middle quaternary fluvioglacial deposits of the Moscow horizon, and middle quaternary morainic deposits of the Moscow and Don horizon.

The modern soil is a 0.2 m thick layer of soil and vegetation.

The recent industrial deposits (EGE-1) occur at a depth of 0.3–3.2 m and are composed of filled soils—the stiff, uncaked clay loam with 5% construction waste.

The upper quaternary covering deposits are composed of 0.4–3.2 m thick semi-solid, fissile clayey loam interlain with stiff, solid clay loam (EGE-2).

The middle quaternary fluvioglacial deposits of the Moscow horizon are composed of clayey soil and sand:

- 0.5–3.7 m thick, medium density, medium and highly water saturated sandy silt, with layers of elastic clay loam and clayey sand and rare occurrences of gravel (EGE-3);
- 0.6–5.2 m thick, highly elastic sandy loam with 10% gravel and landwaste (EGE-4);
- 0.4–6.5 m thick, stiff sandy loam with layers of semi-solid clayey soil and 10% gravel and landwaste (EGE-5).

The middle quaternary morainic deposits of the Moscow horizon are composed of clayey soil and sand:

- 0.5–9.5 m thick, semi-solid sandy loam, with layers of solid clayey soil, semi-solid clay, and 10% gravel and landwaste (EGE-6b); and
- 0.3–6.0 m thick, dense, water saturated gravel sand with layers of silty sand and lenses of gravel (EGE-7b).

The middle quaternary morainic deposits of the Don horizon are composed of 3.5–18.6 m thick, solid calcareous clay loam with 30% gravel and carbonous landwaste.

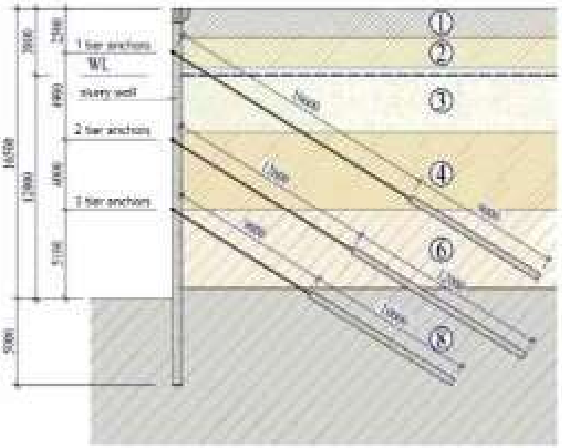

Figure 1. Excavation shoring design.

Hydrogeologically, the site has three groundwater reservoirs of Intermoranial and Jurassic period. The Intermoranial groundwater occurs at a depth of 5.5–13.9 m and the Jurassic one at 28.0–31.7 m.

The geological profile of the site with installed trench wall is shown in Figure 1.

To analyze the influence of the above mentioned factors on the results of trench wall design calculation, we performed a series of 3D calculations with Midas GTS NX 2018, which used 4-node planimetric rectangular and 8-node volumetric prismatic finite elements to model the cast-in-place ferroconcrete trench.

Stage one involved modelling the trench wall design according to the conventional method which used planimetric finite elements based on plate bending theory and made allowance for the initial modulus of elasticity of concrete due to the crack opening and the inelastic behavior of concrete in its compressive zone, which is known to reduce the bending stiffness and distribution-sharing capacity of shoring designs. The dimensions of the design model, inclusive of the soil (41 × 36 m), were assumed so that calculation results could not be influenced by the boundary conditions. The trench wall model consisted of plate elements (0.25 × 0.25 m). Anchors were modelled with rod finite elements. The design model had a width of 9 m. The resultant model allowed the stress and strain state to be analyzed across the "anchor-trench wall-soil mass" system, taking into account its three-dimensional behavior and the key stages of excavation work. The isofields of the displacements and the normal vertical and horizontal stresses occurring across the structural planes after excavation are shown in Figures 2–9.

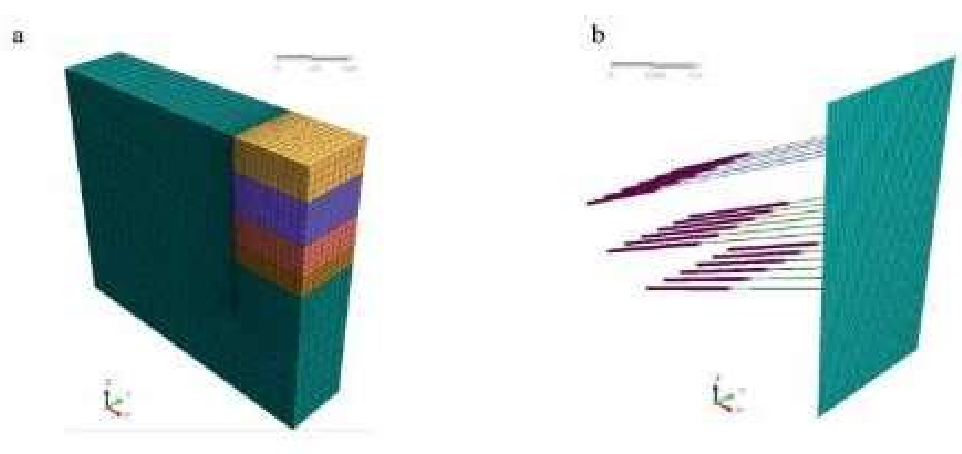

Figure 2. Trench wall design scheme: a – general configuration, b – an image of anchored trench wall.

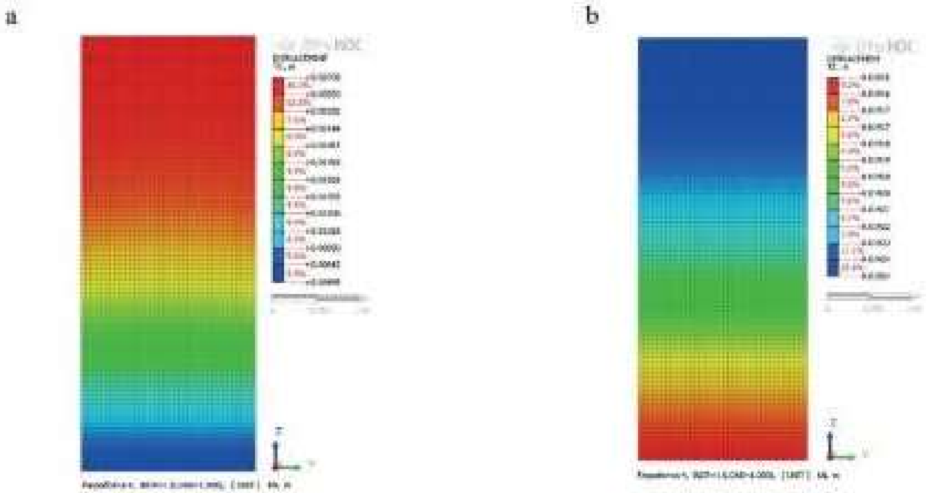

Figure 3. Isofields of displacements in the trench wall: a – horizontal displacements, b – vertical displacements.

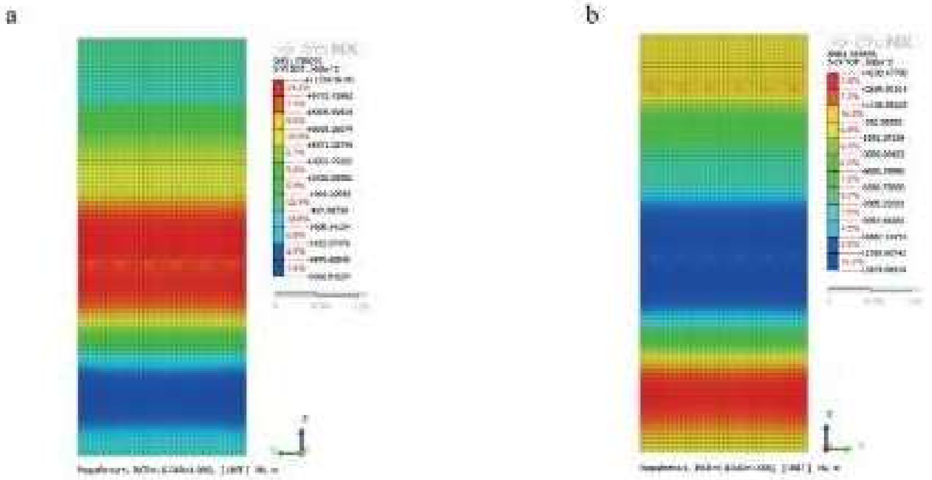

Figure 4. Isofields of normal vertical stresses across the planes of the trench wall: a – due to soil, b – due to excavation.

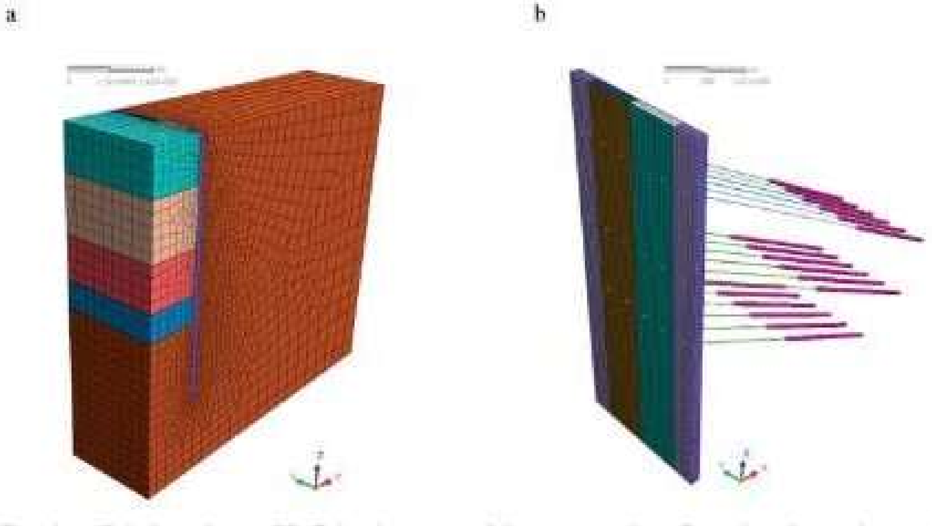

Figure 5. Trench wall design scheme (3D finite element model): a – general configuration, b – an image of anchored trench wall.

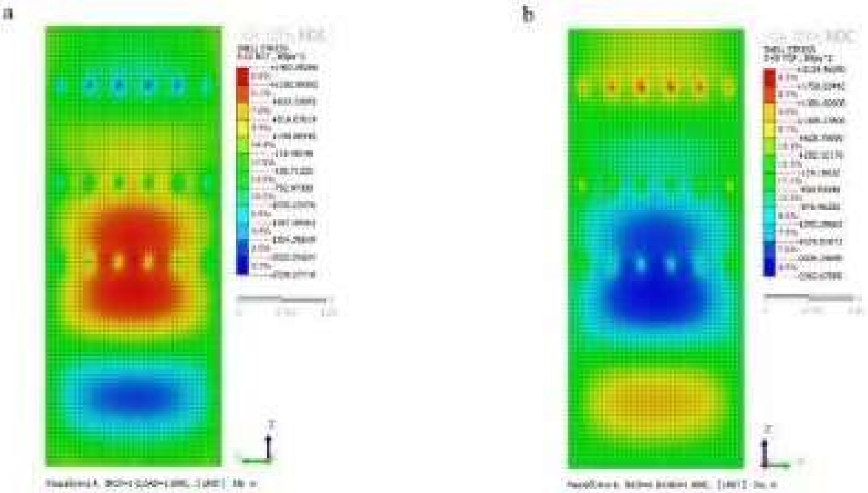

Figure 6. Isofields of normal horizontal stresses across the planes of the trench wall: a – due to soil, b – due to excavation.

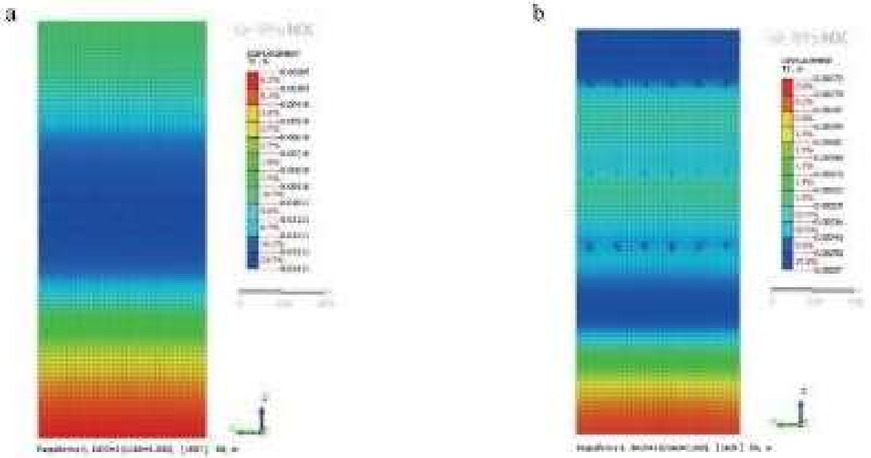

Figure 7. Isofields of displacements in the trench wall (3D fi-nite element model): a – horizontal displacements, b – vertical displacements.

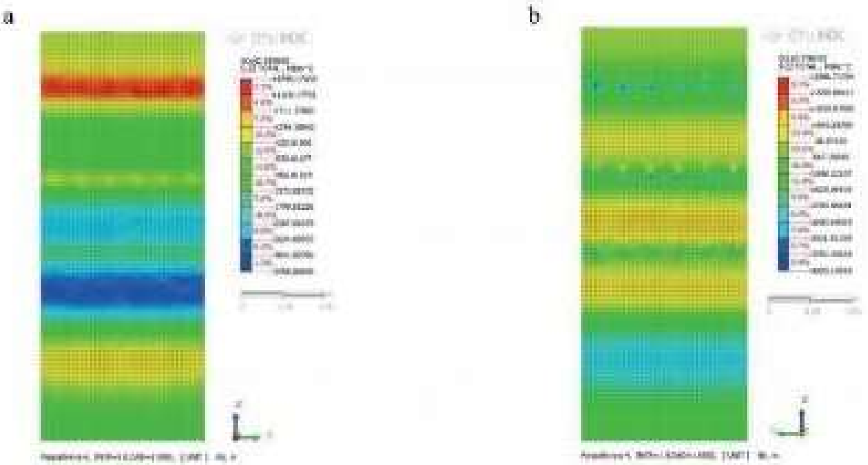

Figure 8. Isofields of normal vertical stresses across the planes of the trench wall wall (3D finite element model): a – due to soil, b – due to excavation.

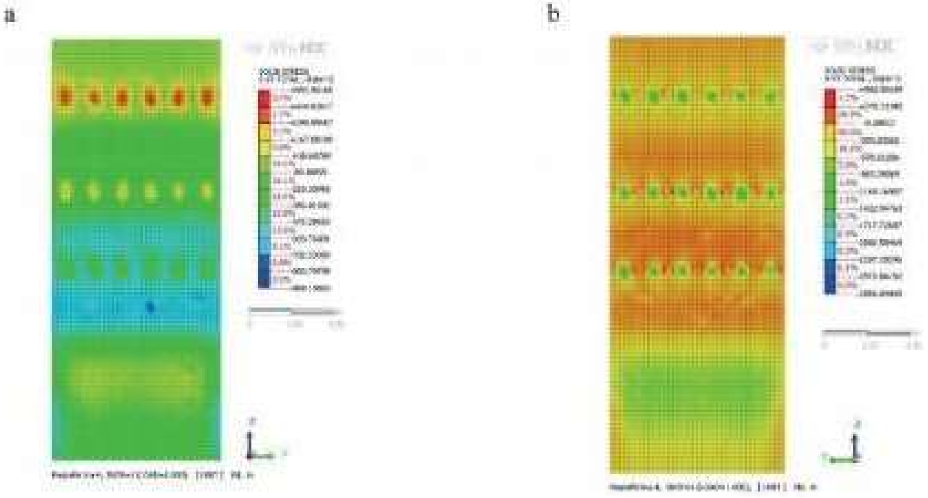

Figure 9. Isofields of normal horizontal stresses across the planes of the trench wall wall (3D finite element model): a – due to soil, b – due to excavation.

Since our design did not use spreader beams and for this reason the resultant vertical and horizontal internal stresses of the trench wall turned out high, of special concern was the quantity of horizontal reinforcement in space frames and its complete absence in middle part of work zones.

3 RESULTS

The 3D calculation of the strength and stability of the trench wall that used planimetric (2D) finite elements did not result in the actual stress and strain state of the shoring of excavation with regard to all of the drawbacks mentioned above. So, a new series of calculations was conducted.

In the subsequent design model, the cast-in-place trench wall modeled with volumetric (3D) finite elements and at the parameters analogous to the those used in the previous model. The dimensions of the 3D finite elements were 250 × 250 × 50(t) and 250 × 250 × 250(t) mm.

The analysis of the stress and strain state and the comparison of the calculated results have shown that the maximum horizontal displacements obtained with 2D finite element model (Fig. 3a) equaled 27 mm and were significantly higher than those obtained with 3D finite element model (Fig. 7a), which equaled 14 mm. Moreover, their isofields indicate a great difference: The zones of maximum horizontal displacements do not coincide in the structural height.

Significant qualitative and quantitative differences are also evident between the normal stresses across the planes of the wall, as can be seen from Figures 4 and 5 (2D finite element model) Figures 8 and 9 (3D finite element model).

It should be noted that the ferroconcrete shoring structure under analysis is classified, according to subsection 3.15 of Building Code 63.13330.2012, as a massive ferroconcrete design (1 m^2/0.6 m^3 = 1.67 ≤ 2) and therefore requires, under the provisions of subsections 5.1.2 and 5.1.13 of Building Code 63.13330.2012, the ultimate and serviceability limit state analyses. At the same time, the stress and strain state is to a large extent described by the yield in the vertical plane perpendicular to the horizontal longitudinal axis of shoring, which is in many respects similar to the behavior of a flat-slab multi-span continuous deck with unidirectional beams (horizontal spread beams) or a similar beamless design experiencing the evenly or (occasionally) abruptly increasing load due to the lateral pressure of soil and groundwater and the unilateral displacement of its anchors. In this regard, the comparison of the results obtained with 2D and 3D finite elements analyses that make allowance for the flexural components of the stress, is a highly relevant task.

4 CONCLUSIONS

1. When calculating the designs of cast-in-place ferroconcrete trench walls devoid of horizontal spreader beams, it is important to take into consideration their three-dimensional behavior and, specifically, that they bend in two directions and that 2D FEM and 3D FEM will produce two different pictures of their stress and strain state.
2. The normal stresses in the vertical and horizontal planes of the trench wall that are obtained with 2D modelling (Figs. 4 and 5) differ largely from those obtained with 3D modelling (Figs. 8 and 9) – due to the assumptions of the plate

bending theories (static and kinematical hypotheses). Also, a significant influence on the results of the shoring design calculations that are workflow-based can be caused by geometrical nonlinearity. The stress and strain state of the anchors (higher in the 2nd tier and lower in the 3rd tier) which has been obtained with 3D modeling and as compared to the 2D model ling results, evidences the occurrence of the bending moments in the above-support areas which tend to decrease rapidly in the areas near the anchors.

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