Numerical Analysis of Foundation for Underground Bridge Project in Moscow

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ABSTRACT: The effects of non-linear behavior of soil-structure interaction are illustrated in the paper with the example of underground bridge construction in Moscow. The results of numerical analysis of the structure are presented. A comparison of results obtained with staged construction analysis and conventional analysis is demonstrated and their deviation is discussed.

1 INTRODUCTION

The Third Circular Highway is the largest transportation project completed last year in Moscow. The so-called "Third Circle" is intended to discharge traffic in the central part of the city. The Third Circle contains several tunnels and bridges along its 50 km length.

One of the most complicated parts of this project is the junction at Gagarin Square. Two-sectional highway tunnel combined with railway tunnel and underground parking space was constructed at the bed of the previously existing Andrew ravine with cut-and-cover method. The tunnel is crossing on its way the underground metro station that was built about 30 years ago. Direction of the tunnel is nearly normal to direction of the metro station. The distance between top of the station and bottom of the designed tunnel generally does not exceed 3 m. A possible trade mall is planned to be constructed at this joint above the tunnel.

Construction of the tunnel is demonstrated in Fig. 1 while Fig. 2 illustrates contemporary situation.

2 GROUND CONDITIONS

Ground conditions of the site are the following. The upper layer consists of backfill from 1 to 14 m thick. Fill is followed deeper by layers of Quaternary fine and medium sands with total thickness up to 15 m. Calcareous dense fine sands up to 10 m thick underlay Quaternary deposits. Jurassic deposits that situated at the depths from 25 to 65 m are represented with fine sands and generally with stiff clays. Carbonian clays and limestone are situated deeper.

Three aquifers are met at the depth up to 80 m. The first Quaternary aquifer is unconfined. There are heads up to 12 m in the second aquifer found in sandy lenses in the Jurassic clays and up to 10 m in the third Carbonian aquifer. Free groundwater level is about 4.0 m below the ravine surface.

Figure 1. Construction of the tunnel with cut-and-cover method.

Figure 2. Contemporary view of the site.

The geological profile is presented in Fig. 1. Soil properties are summarized in Table 1.

Table 1. Soil properties.

№	Soil type	γ (kN/m³)	e	I_L	E (MPa)	R_c (MPa)
1	Backfill	19.8	0.6	-	14	-
2	Medium sand	19.6	0.6	-	30	-
3	Fine sand	19.4	0.7	-	23	-
4	Very fine sand	19.0	0.7	-	11	-
5	Loam	19.6	0.6	0.3	28	-
6	Dense fine sand	20.2	0.5	-	26	-
7	Very fine sand	19.5	0.5	-	28	-
8	Loam	20.7	0.7	0.7	10	-
9	Medium stiff clay	18.6	0.8	0.1	24	-
10	Stiff clay	17.7	1.2	0.0	21	-
11	Stiff clay	18.2	1.2	0.0	17	-
12	Stiff clay	20.4	0.6	0.0	24	-
13	Medium stiff clay	20.6	0.6	0.1	43	-
14	Weak limestone	21.7	0.3	-	1000	7.2
15	Lime-stone	24.1	0.1	-	1500	18.1

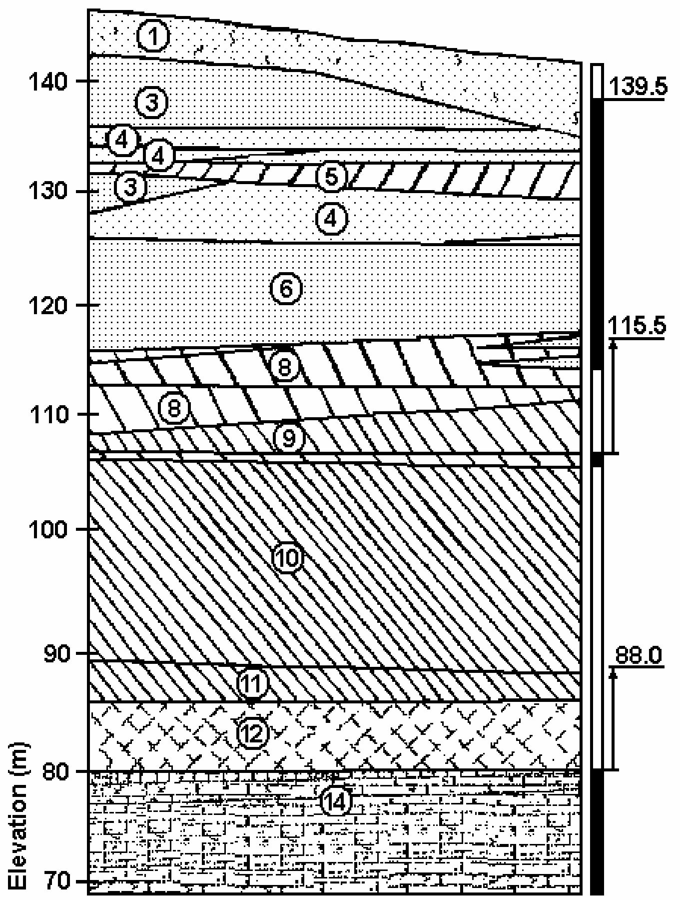

Figure 3. Geological profile.

3 DESIGN

The intersection of the designed tunnel with the existing underground metro station "Lenin's Prospect" presented many complications to be sold. Three sections of highway tunnel (sections B, C, D) and section of railway tunnel (section A) overlay the metro station normally to its direction. The spacing between the designed tunnel and station structure is very small and varies from 85 to 380 mm. Designed underground space for transfer zone is situated between railway and highway tunnels. After the completion of the tunnel it is planned to construct over it a four-stored trading center. The perspective underground metro station will adjoin to the tunnel from north. Fig. 4 illustrates the plan position of the tunnel and metro station.

The investigation of the existing metro station demonstrated that its serviceability does not allow applying any additional loads and influences to the structure. Supplementary deformations of the metro station due to tunnel construction should also be excluded or minimized. Thus the decision was taken to construct this part of the tunnel as an underground bridge above the metro station. The bridge consists of four separate parallel sections with maximum length 72 m and width 28 m. Each section is separated from others, behaves as a span structure and has four supports. The span structures of all sections are designed in prestressed reinforced concrete with rectangular cross-section. The values of loads applied to supports are very high. Maximum design load on a single support reaches 100 MN. Cross-section of the tunnels is shown in Fig. 5.

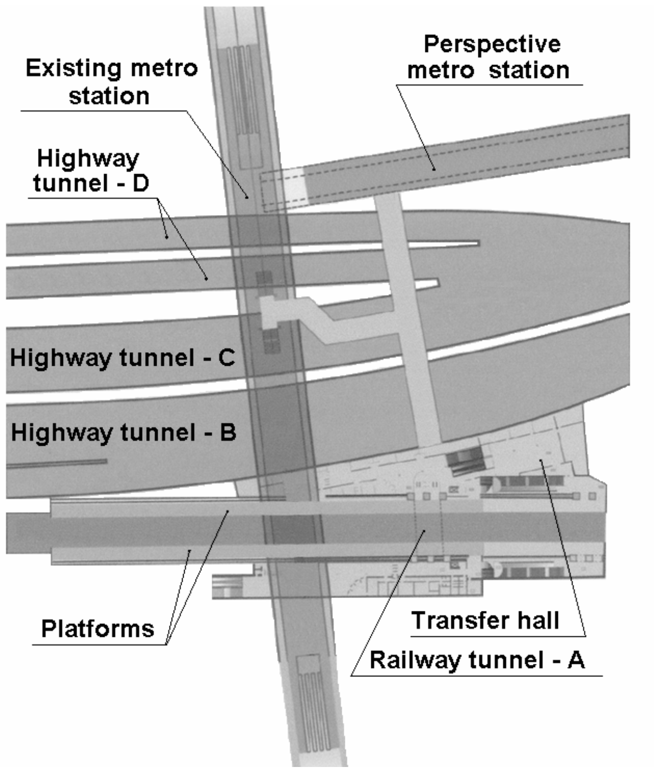

Figure 4. Plan of the tunnels.

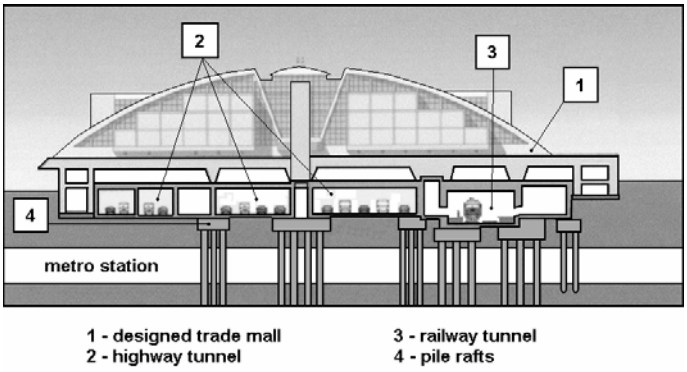

Figure 5. Cross-section of highway and railway tunnels.

It was obvious that deep foundations should be designed due to constraint of support area and large span of the bridge. More than twenty variants of foundations were proposed and analyzed. All of them may be classified as:

- Moderate deep foundations based on Quaternary or Calcareous sands;
- Intermediate deep foundations with tip in Jurassic clays;
- Extremely deep supports transmitting loads to Carbonian deposits.

Driven and bored piles, diaphragm walls and shafts were considered for construction of deep foundations. Bored piles were chosen finally because of technological and economical reasons.

4 NUMERICAL MODELLING OF FOUNDATIONS

4.1 *Objectives*

Within the design process the numerical study was conducted. The numerical modeling with FEM should help to solve following problems:

- To predict influences on the metro station due to construction for different types of foundations;
- To define the required depth of foundations with respect to safety and serviceability of the bridge as well as the metro station;
- To clarify demanded constructional parameters of the supports;
- To find the rational plan position of piles in groups considering base bearing capacity;
- To calculate inner forces acting in piles;
- To determine stresses in pile rafts.

The first step was done in order to choose the required depth of foundations. Finite element modeling was done with the help of *PLAXIS* software (1998) for the plain strain problem. Numerical analysis gave evidence that guarantee of serviceability of the metro station is a decisive factor dictating foundation depth. Initial modeling gave an opportunity to compare roughly the predicted settlements of the metro station caused by the construction for all of proposed variants of bridge foundations. It became clear that only extremely deep foundations based on Carbonian limestone satisfies serviceability requirements of the metro station.

This result of numerical analysis permitted to finalize the choice of foundation type. Thus the supports of each section of the bridge consist of pile group joined with raft 2.5 m thick. Bored piles 1.5 m in diameter are embedded in limestone at the depth of about 65 m.

Further analysis was aimed on more thorough study of behavior of the chosen foundation.

4.2 *Non-linearity of the problem*

To understand better factors that should be considered with geotechnical numerical modeling close cooperation with structural engineers was required. The sophisticated technology of underground bridge construction demanded an adequate numerical procedure able to predict inner forces in pile groups and rafts.

The sequence of construction stages for each section of tunnel includes:

1) Construction of pile groups;
2) Adjoining of pile groups with rafts;
3) Placing of temporary plumbic inserts on the rafts;
4) Mount of temporary trusses and suspended framework;
5) Casting of the bottom and walls of central span section of the tunnel;
6) Pretension of reinforcement;
7) Casting of support sections of the span structure;
8) Casting of tunnel head and its pretension;
9) Adjoining of foundation rafts and support sections of the span structure;
10) Dismantling of temporary inserts;
11) Construction of road cover inside tunnel;
12) Soil backfill;
13) Construction of a building upward.

The necessity of temporary plumbic inserts in combination with antifrictional materials between span structure and foundation rafts was dictated by prestressing of the span reinforcement. The inserts gave possibility not to transmit additional lateral forces to the supports. Further behavior of the structure requires frame adjoining of foundation rafts and support sections of the span structure. Idealized scheme of the joint between raft and span structure is shown in Fig. 6.

Thus a numerical modeling had to consider that loading of piles was affected by changing span stiffness and altering construction of span support joint. Along with that non-linear soil behavior should be taken into consideration although influence of this factor was not very significant as the piles were based in limestone. It was clear that constructional as well as physical non-linearity of soil structure interaction had to be taken into account when modeling.

4.3 *Methodology*

Three variants of numerical simulation were done. The initial conventional 3D finite elements analysis was completed with *MicroFe* software (2002) on the basis of constant spring base model. This calculation was performed for the final structural scheme of underground bridge without taking into consideration non-linearity of the problem due to construction sequence. Its results proved to be not quite reliable.

The 2D stage-by-stage finite elements analysis for plain strain approximation was done with the help of *PLAXIS* software with respect of non-linear soil behavior and load-stiffness dependence for the span structure. This analysis demonstrated that consideration of construction sequences was quite essential. Non-linear numerical study for plain strain problem helped to define load-displacement relations for the soil base to be used further as a characteristic of spring model for 3D study.

Since the available software were not able to solve non-linear 3D problem it was necessary to make its linearization. On the basis of 2D nonlinear analysis the initial 3D problem was divided on sequence of linear problems. Linearization of the problem is illustrated on example of the most loaded highway tunnel B. Plan of rafts and pile groups of tunnel B is shown in Fig. 7.

Linearization of the problem caused by technology of construction was the following:

Stage 1 – the model includes side walls and bottom of the tunnel. Slide hinge joints between rafts and span structure are applied. Load applied is deadweight of structure.

Stage 2 – head of the tunnel is added to the model. Slide hinge joints between rafts and span structure are still applied. Load is deadweight of the head structure.

Stage 3 – tunnel structure as in Stage 2, but frame joints between rafts and span structure are inserted. The applied load includes deadweight of road cover, soil backfill, live load in tunnel, weigh of perspective building over the tunnel.

The distribution of loads between calculation stages was: Stage 1 – 30% of total, Stage 2 – 10% and Stage 3 – 60 %.

Finite elements models for each of linearization stages are illustrated schematically in Fig. 8.

Linearization of the problem for properties of elastic springs modeling soil reactions according to preliminary calculations with *PLAXIS* was done. Thus springs constants were taken equal for calculation Stages 1 and 2 and were reduced for calculation Stage 3.

Finite elements analysis completed with *MicroFe* software was performed for free linearization stages. Combination of partial linear solutions gave the solution for 3D nonlinear problem.

4.4 *Modeling results*

Results of 3D numerical modeling are presented for Group-1 (see Fig. 7) of the highway tunnel B.

Distribution of inner forces in piles is illustrated for all of calculation stages in Fig. 9. It is vividly seen from the plot why the consideration of construction stages was so important.

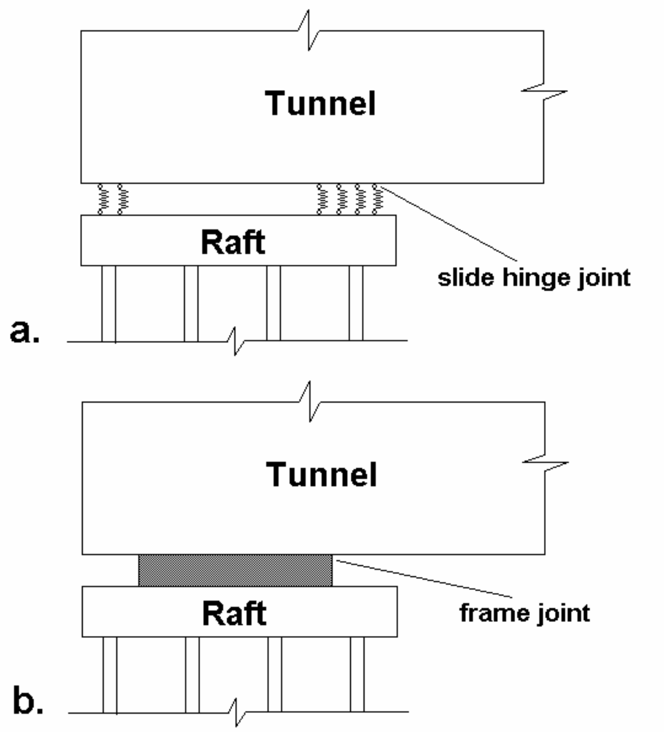

Figure 6. Idealized scheme of the joint between raft and span structure: a. for stages 1 and 2; b. for stage 3.

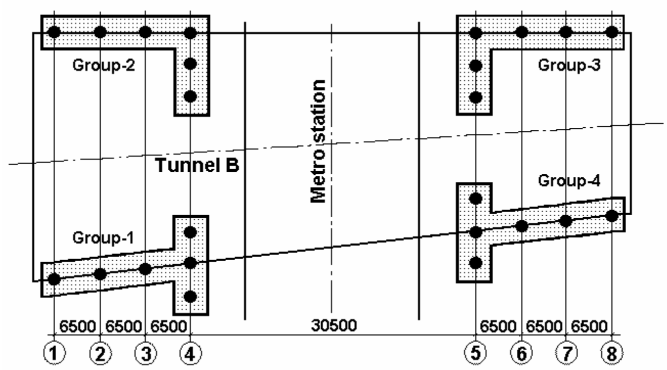

Figure 7. Plan of rafts and pile groups for tunnel B.

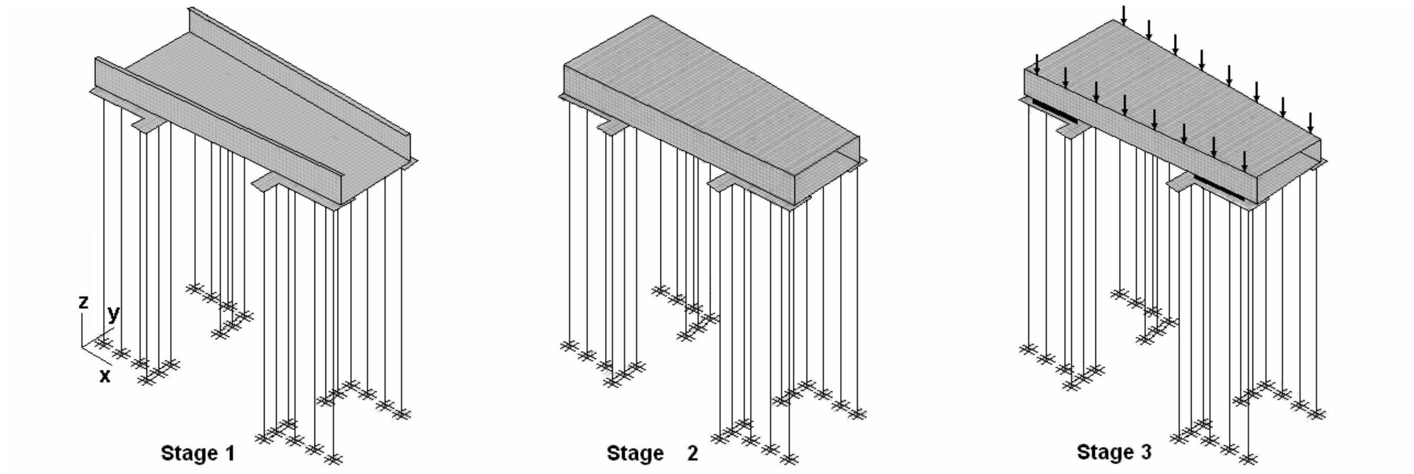

Figure 8. FE models of the tunnel B for linearization stages.

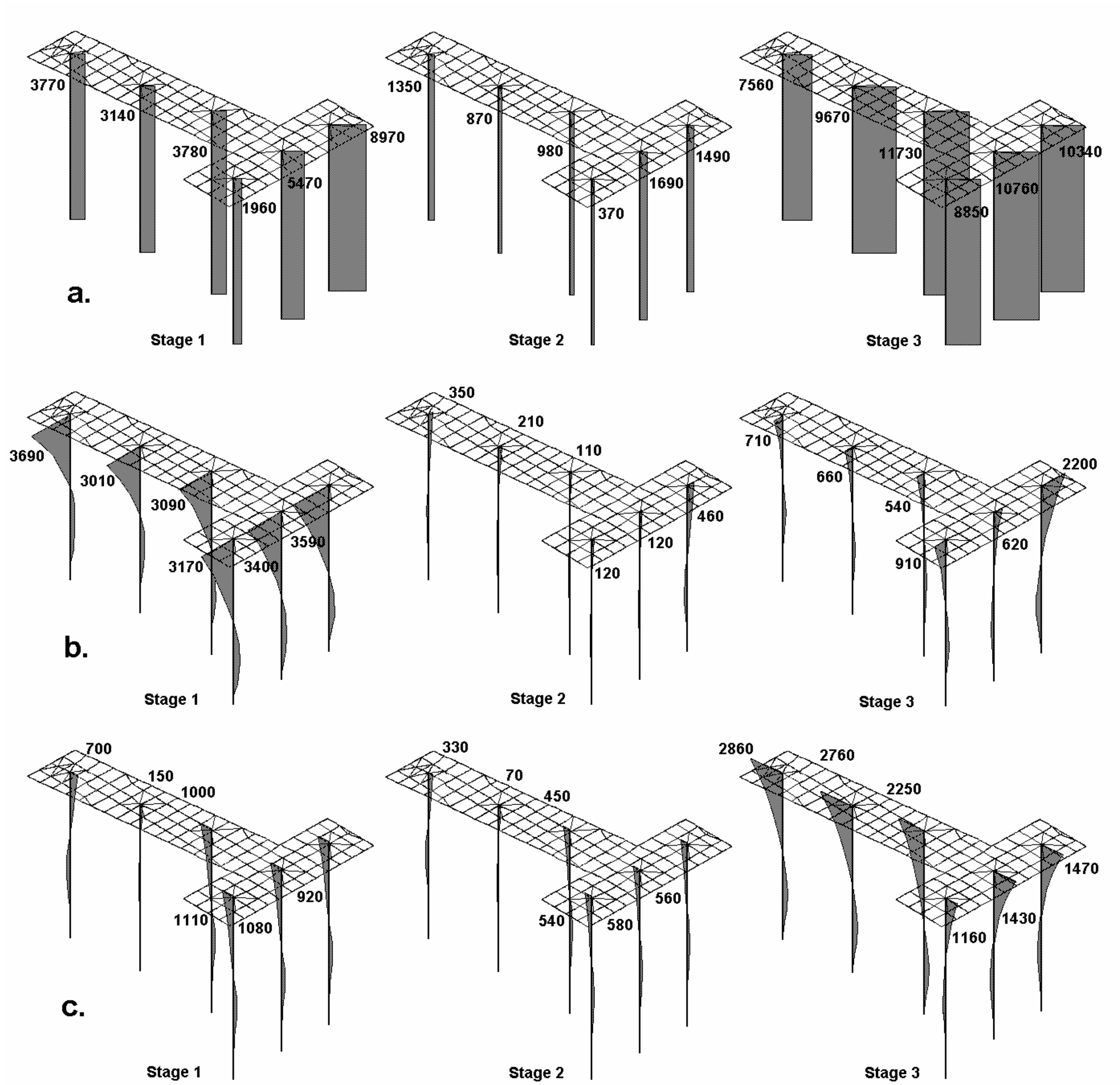

Figure 9. Distribution of inner forces in Group-1 of tunnel B: a. normal forces N, kN; b. bending moments M_x, kNm; c. bending moments M_y, kNm.

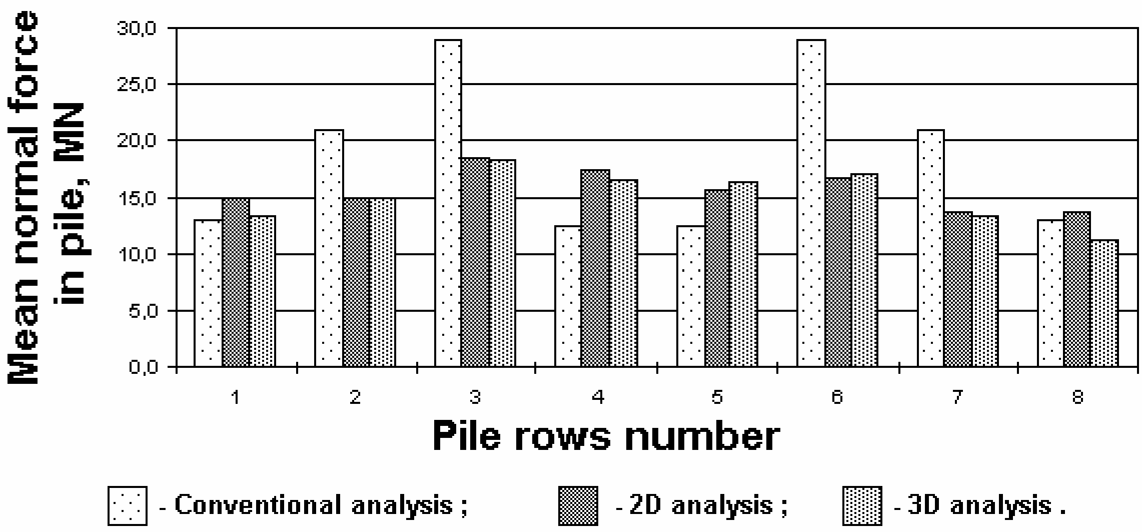

Figure 10. Comparison of the mean values of normal forces in piles for tunnel B according to conventional and non-linear finite elements analysis.

Distribution of normal forces in piles within group is altering according to stiffness of the span structure and conditions of its adjoining to support. Bending moments M_x in pile heads generally change their sign comparing calculation Stage 1 and Stages 2-3. Also change of sign in M_y values is seen if compare Stages 1-2 and Stage 3.

Comparison of the mean values of normal forces in piles for tunnel B obtained in numerical modeling is presented in Fig. 10. Conventional analysis neglected non-linear behavior of soil-structure interaction. It is obvious that neglecting of construction stages sequence leads to incorrect prediction of loads transmitted on piles and correspondingly to wrong design position of piles in pile groups.

5 CONCLUSIONS

Design of the underground bridge for highway and railway tunnels at Gagarin square faced geotechnical engineers with sophisticated soil-structure interaction problem. The numerical study of soil-structure interaction should take into consideration constructional as well as physical non-linearity under the decisive role of the first. The objectives of numerical modeling demanded close cooperation between geotechnical and structural engineers.

Analysis of the numerical modeling results demonstrates the necessity of thorough study of technological process of construction and its sequence in order to obtain reliable geotechnical prediction.

Construction of the underground bridge at Gagarin square was completed while the construction of trading mall over it is not started yet. Monitoring of the metro station "Lenin's Prospect" has shown that serviceability of the station was provided. The additional settlements of the station due to construction of the tunnels did not exceed 10 mm.

ACKNOWLEDGEMENTS

The authors acknowledge the general design organization MOSINGPROECT and the designers of span structure from PROMOS for fruitful cooperation in progress of the presented study. Special regards for research engineers from NIIOSP involved in design of Gagarin square junction.

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