Effects on adjacent buildings from diaphragm wall installation

Effets sur des bâtiments adjacents liés à l'installation de parois moulées

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ABSTRACT: A new approach for simulating the excavation and construction of subsequent panels is proposed to investigate the effects from the installation of diaphragm walls on the surrounding soil and adjacent buildings. The method has been combined with a 3-D nonlinear analysis and a constitutive law providing bulk and shear modulus variation, depending on the stress path (loading, unloading, reloading). The effects on an adjacent building have been investigated by applying a full soil-structure interaction analysis including the whole building. Contrary to lateral movements, which mostly take place at the panel under construction, it was found that the effect of settlements covers a larger area leading to a progressive settlement increase. The effect highly depends on the distance from the panel under construction. Settlement profiles and settlements at specific points as increasing with subsequent panels installation are given providing the ability of specific monitoring guidelines for the upcoming construction of the diaphragm wall in front of the building.

RÉSUMÉ : Une nouvelle approche pour simuler l'excavation et la construction des panneaux subséquents est proposée pour étudier les effets liée à l'installation de parois moulées adjacents sur les bâtiments et le sol adjacents. La méthode a été associée à une analyse 3-D non linéaire et une loi de comportement qui permet la variation des modules de déformations en fonction des chemins des contraintes. Les effets sur un bâtiment adjacent ont été étudiés en appliquant une analyse d'interaction sol-structures pleine, qui inclut l'ensemble du bâtiment voisin. Contrairement aux mouvements latéraux, qui principalement prennent lieu à partir du panneau en cours de construction, il a été constaté que l'effet des tassements couvre une plus grande région, conduisant à une augmentation progressive de tassements. Les effets dépendent fortement de la distance au panneau en cours de construction. Les profils des tassements et tassements aux points spécifiques augmentant progressivement avec l'installation des panneaux sont donnés en face de l'immeuble où la paroi moulée est en cours de construction.

KEYWORDS: diaphragm walls, soil-structure interaction, multi-stage analysis, buildings settlements.

1 INTRODUCTION

It is widely accepted that the process of installing diaphragm walls can result in potentially significant soil displacements and cause substantial reductions in horizontal stress. Depending on the soil profile, the diaphragm wall configuration (length and construction sequence) and the close existence of adjacent buildings with poor foundations may render the effects of diaphragm wall installation considerable. Field monitoring confirms that ground movements resulting from diaphragm wall installation could be a significant component of the overall displacement (Burland and Hancock 1977, Tedd et al. 1984, Symons and Carder 1993), while centrifuge tests verified the development of the effect as well (Powrie and Kantartzi 1996). Recent field evidences recorded during the on going construction of subway stations in Thessaloniki demonstrated that the component of ground movements resulting from the diaphragm wall installation may be higher than 50% of the overall displacements. It is therefore evident that the simplistic assumption of a 'wished-in-place' wall (installation without any change in stress and cinematic field) commonly applied for design purposes is rather questionable.

The aim of the present paper is to investigate the effect of a diaphragm wall installation to adjacent buildings with relatively poor foundations. The sequential installation of each individual diaphragm wall panel installation was simulated by a substitution of the parameters of excavated elements with those corresponding to the bentonite slurry and later on by the concrete tremied into the panel. Valuable qualitative and quantitative conclusions regarding the variation of the effects to the adjacent building have been drawn.

1 INSTALLATION PROCEDURE MODELLING

With the aim of minimising disturbance and increase stability during the excavation process, rotary drilling machines for slot excavation have been used in Thessaloniki's underground stations with poor soil conditions. Figure 1, on the left side, shows a rotary drilling machine equipped with cutting wheels and a reverse circulation system. On the right side of Figure 1 the numerical simulation of the excavation process is illustrated. The soil from the surface level down to the upper limit of the rotary wheels (line A), is replaced by a material simulating the bentonite slurry. Appropriate, very small values are attributed to the bulk and the shear modulus of the material. Within that zone the stresses are initialised to the values hydrostatically defined from the weight of bentonite slurry. This simulation process ensures that stresses within this zone remain always equal to the hydrostatic conditions no matter the deformation level. However, in the area occupied by the rotary cutters (area between line A and line B) the development of static hydrostatic pressure is not evident. For this reason, in that zone the stresses are not initialised hydrostatically and only internal gravitational stresses are considered. Within this zone the material (cuttings with bentonite slurry) has higher unit weight and is stiffer than bentonite slurry. The zone undertakes the pressure from the surrounding elements depending on the internal gravitational stresses, the stiffness and the shear resistance of the surrounding soil elements, and the arching developed around the trench. This complicated mechanism provokes a redistribution of stresses and the surrounding soil elements undergo some deformation. As a result horizontal displacements at the wall/soil interface are governed by the ability of the soil to move in response to the reduction in lateral stresses during the wall installation. The above mechanism leads to a temporary reduction of the horizontal stresses in the surrounding excavation faces, which however increase to the hydrostatic bentonite slurry pressure in the next stage of excavation. When the excavation of a panel is accomplished concrete is cast in place using tremie pipes. The same numerical process is applied to simulate the panel completion, i.e. appropriate values are attributed to the bulk and the shear modulus of the material simulating wet concrete, while, stresses are initialised to the values hydrostatically defined from the weight of wet concrete. When equilibrium is attained, regular concrete values are attributed to bulk and the shear modulus to the panel. The above simulation process is repeated over the entire depth of the panel.

The aforementioned simulation process reflects the construction of a single panel and is applied to all panels in a diaphragm wall. However, the response of each particular panel is greatly influenced by the construction sequence. Obviously when constructing a subsequent panel, with already completed adjacent panels, the effect of arching is strengthened due the high resistance of these elements. As a result a stress increase is observed not only at the adjacent soil, but also on neighbouring panels that have already been casted. Thus over the period of wall construction there will be a progressive transferring of load back and forth laterally, either from a primary panel to the adjacent soil or, as the wall progresses, from new panels to panels previously casted. It can be realised that when accurate prediction of displacements and stresses redistribution are demanded, a profound 3-D nonlinear multi-stage numerical analysis is required.

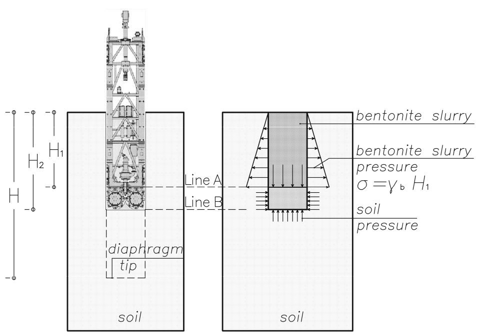


Figure 1. Schematic illustration of the proposed approach for simulating a single panel excavation

2 NUMERICAL SIMULATION

2.1 *Project description*

The station of Analipsis, 210 m long and 16.4 m wide, is considered as one of the most critical of the underground of Thessaloniki. With the exception of the surficial layer the soil conditions are relatively good. However, the fact that the diaphragm wall is located very close to adjacent buildings with poor foundations, in many cases, renders the construction of the diaphragm wall extremely demanding. According to the guidelines of the German code DIN 4126, the critical zone around the trench excavation extents up to a distance of 70% of the pile length. For this reason a relatively small typical panel length L = 2.8 m was applied and a rotary cutting machine was selected to perform the ongoing excavation of the panels. The thickness of the panels is t = 1.20 m, its depth is H = 44.0 m and the basement of the station is 28.0 m below the ground surface.

2.2 *Soil model and material properties*

The ground conditions at the site together with the soil properties of each soil layer, derived from the carried out geotechnical investigation and the evaluation of in-situ and laboratory tests are presented in Table 1. The groundwater level was encountered at 5.0 m below the ground level. Pressuremeter tests were carried out at the area to assess the in situ horizontal stresses and, according to the evaluation of the results, a constant value of K_0 = 0.54 has been adopted.

Bearing in mind the crucial effect and the necessity for settlements predictions to the adjacent buildings, a constitutive law with double yielding (FLAC 3D) has been applied in the present study. The model includes a volumetric yield cap surface in addition to Mohr-Coulomb shear and tensile failure envelopes. The cap surface is independent of the shear strength and it consists of a vertical line on a plot of shear stress vs mean stress with a trace on the mean stress axis defined as cap pressure p_c. Any violation of the cap surface produces volumetric plastic strain following a piecewise-linear law prescribed in a user-supplied table. The tangential bulk and shear moduli evolve as plastic volumetric strain takes place according to a special law defined in terms of a constant factor, R, which is the ratio of elastic bulk modulus, K_c, to plastic bulk modulus, K_t. The relevant values adopted are given in Table 1.

The concrete diaphragm wall behaviour was considered as an isotropic linear elastic. Linear elastic behaviour was attributed to the bentonite slurry with infinitesimal deformation values. The shear strength of bentonite slurry with unit weight of 11 kN/m^3 is of the order 50 Pa (DIN4126). A reasonable value for the slurry shear modulus is three hundred times the shear strength, G_{sl} = 15 kPa, while the Poisson's ratio was taken equal to 0.49. The application of these values to the analysis produced stresses within the bentonite computational domain equal to hydrostatic gravitational state, ensuring that appropriate hydrostatic pressures were developed at the trench faces. A higher value of unit weight (12.5 kN/m^3) has been attributed to cutting products mixed with bentonite slurry and similarly the shear modulus has been taken equal to 25 kPa. Taking into account that the construction schedule, the time period between adjacent panels installation, particularly the primary panels, is quite enough for any excess pore dissipation an effective stress analysis was applied.

Table 1. Geotechnical properties of soil layers.

Layer	Fill	A1a	A1b	A1c	B
Depth (m)	0 – 3	3 - 10	10 – 35	35 – 40	40 -60
Effective cohesion, c' (kPa)	3	3	5	40	50
Effective angle of friction, φ' (deg)	30	25	25	25	25
Poisson's ration, v	0.3	0.3	0.3	0.3	0.3
Plastic bulk modulus, K_t (kPa)	4,000	5,000	8,500	10,000	10,000
Ratio of elastic to plastic bulk modulus, R	5	6.5	10.5	12	12
Cap pressure, p_c (kPa)	100	100	NC*	NC*	NC*

Remark: NC means that cap pressure is equal to the in-situ mean stress

2.3 *Simulation procedure*

The effective numerical simulation of typical construction procedure for a cast in situ diaphragm wall must reflect the stages and the mechanisms developed during the excavation and throughout the completion of the wall. The first step was to establish the in-situ state of stresses. The construction of a single panel was simulated in 22 stages during which the excavation was advanced in 2.0 m. Within each stage the soil in

the excavation zone was replaced by cutting-bentonite, while the area above the zone being excavated was replaced by bentonite. The end of the excavation was followed by wet concrete placing and the value of E_{wc} = 1,000 MPa was attributed to Young's modulus and v_{wc} = 0.49 to Poisson's ratio. The last stage of analysis corresponded to concrete hardening. The same process was applied to all panels under consideration.
The most critical location in the area of the station corresponds to poor building foundation conditions very close to the diaphragm wall. The analysis is therefore focused on that. Prior to the currently presented full soil-structure interaction analysis including a 6-storey building, numerical analyses of a single panel construction and of a wall and an adjacent foundation verified the proposed simulation process as well as the constitutive law and the values for the parameters. Figure 2 shows the foundation plan of the adjacent building together with the location of the diaphragm wall and a curtain of micropiles used to minimize the effect of panels' installation. Further to the bay number of each panel the figure shows the panel type (primary, P, or secondary, S) and the order of installation in the circles on the right side of each panel. The foundation consists of individual footings connected with 0.20 m × 0.50 m reinforced concrete beams. The foundation level is at 3.0 m from the ground surface. The F.D. mesh included 89,000 3-D elements, 4,272 shell elements and 225 beam elements. The dead weight of the building has been explicitly introduced by the gravity of each element while a uniform load of 5 kPa has been applied to each slab to simulate all other permanent and variable loads. After the establishment of the initial stresses, the installation of the micropiles was introduced followed by the installation of the 9 panels according to the previously described approach. The sequence of installation is presented in Figure 3.

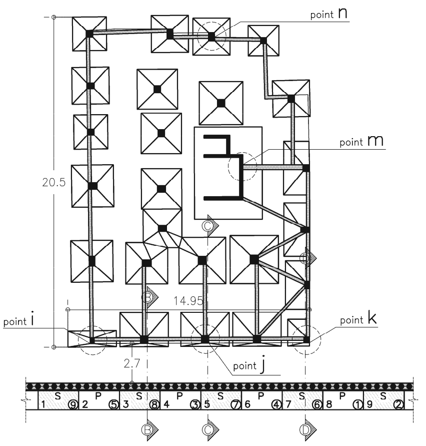


Figure 2. Individual footings of a 6-story building together with the diaphragm wall and the micropiles

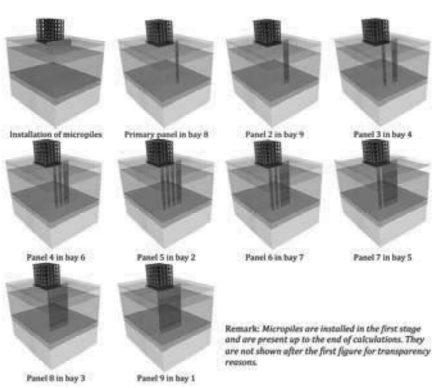


Figure 3. Sequence of panel installation

3 NUMERICAL RESULTS

The contour values for soil settlements, the building floors' settlements and the axial forces of the building columns developed after the completion of the first element (element no 8) are illustrated in Figure 4. For visibility reasons the figure is given in a section at the building face and a cross section at the middle of the building. It can be seen that the maximum soil settlement is located around the excavated panel and is of the order of 2.4 mm. The maximum settlement of the building is located at its corner nearby the excavated panel and the contours show a uniform reduction with distance from that point.
The sequential construction of the next panels provokes the maximum effect in front of each panel, as it has been expected, but at the same time contributes to a progressive increase of settlements in a widespread zone. When the primary panels are installed, an increase of settlements to the value of 4.2 mm is occurred. The soil settlements progressively decrease with the distance form the diaphragm wall and are almost zero at the backside of the building. The completion of the wall with the rest 4 secondary panels does not encounter significant increase to the maximum value of the soil settlements. The final value of maximum settlement is 5.3 mm and the same value is developed at the external side of the building close to the diaphragm wall. From the comparison of the axial forces variation throughout the construction of the panel arises that the panels' installation does not practically affect them.

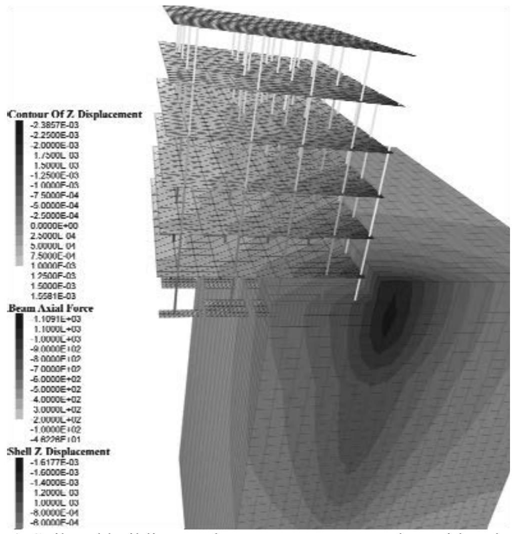
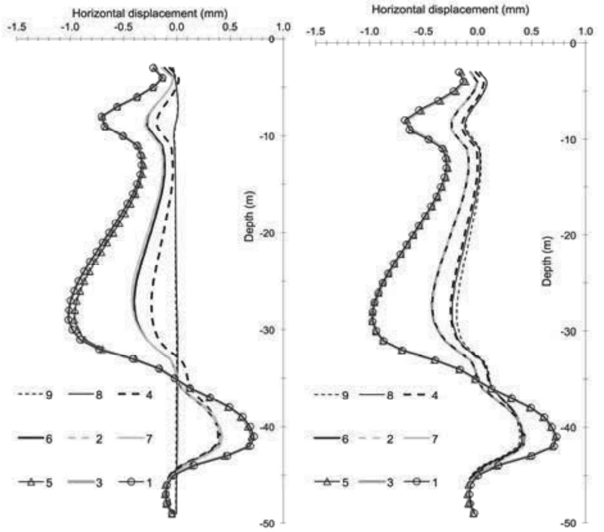


Figure 4. Soil and building settlement contours together with column axial forces after the completion of the first panel (bay no 8)

Figure 5a illustrates the variation of the horizontal displacements with depth below the external boundary of a footing at the front side of the building (cross section 'C-C'). The values are not exceeding the order of 1.0 mm and this is mainly due to the existence of the micropiles. The construction of the panels with bay no 8 and 9 (first and second in construction sequence) are relatively too far from that point and they do not provoke any horizontal displacement at the point under consideration. The panel with bay no 5 is just in front of the point and this explains the important movement of the displacement field during the construction of this panel. Similar are the results in the case of the point below the footing at the edge of the external footing at section 'D-D', Figure 5b.
The most important effect to the adjacent building is the anticipated settlements, the angular distortion that will develop to the foundation and if that last could be capable of provoking any notable bending moment to the foundation elements. Figure 6 illustrates the progressive increase of the settlements across the section 'C-C'. On the same figure the location of the diaphragm wall and the foundation of the building are shown.

The construction of every panel contributes to a progressive increase of settlements, with the maximum influence experienced when the primary panel close to the cross section is installed. This explains the maximum difference observed when panel no 4 is installed. The maximum settlement is developed at the end of the construction of all panels, its value is of the order of 5.5 mm and occurs at the front side of the building.



(a) (b)

Figure 5. Profile of horizontal displacements below the external footing at (a) the mid-face, point j, and (b) the end-face of the building, point k

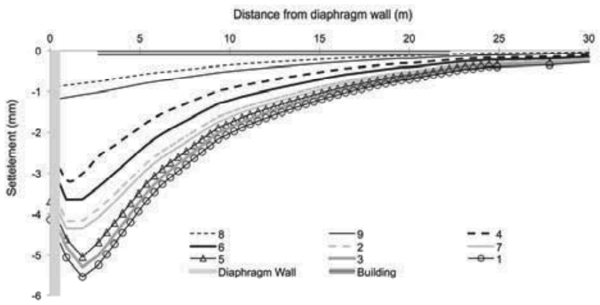


Figure 6. Development of settlement profile with panel sequence construction at cross section 'C-C'

It is worth noticing that the settlement values estimated from the 3-D analysis are leading to an angular distortion of 1:5,000. This value is considerably lower than the limits provided by CIRIA and the CFEM.

An effective design of complex retaining structures, with closely adjacent buildings, includes instrumentation and monitoring to ensure the safety of the construction and control the effect on the adjacent buildings. These data will be available when the diaphragm wall at this area will commence and histograms giving the contribution to cumulative settlements of each particular panel can be drawn. It is therefore extremely helpful to give these histograms resulting from the 3-D analysis and follow up the values as the wall is constructed. Figure 7 illustrates the numerically established cumulative settlements after the completion of each panel, at the characteristic points, i, j, k, m and n. The location of each panel corresponds to relative position from left to right ,while the installation sequence is given on the top of the histograms.

It can be seen that the final settlements at the front face of the building (points i, j and k) are of the same magnitude and that the values provided for the points far from the diaphragm wall (points m and n) are drastically lower and with no practical effect on the building. It is clearly evident that Figure 7 can be efficiently used to compare settlements during the up coming construction and provide alarm signal in case of significantly higher settlements values.

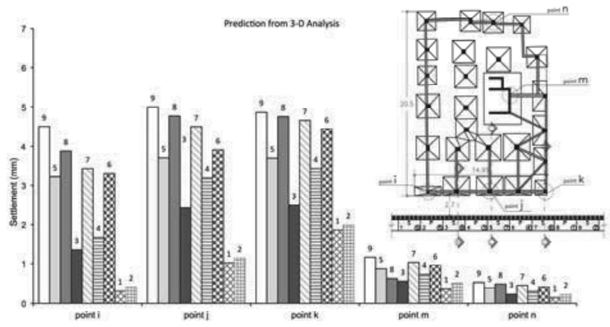


Figure 7. Predicted development of cumulative settlements at points i, j, k, m and n at the end of each panel construction

4 CONCLUSION

In this paper the effects from the installation of diaphragm walls have been investigated using a new approach for simulating the excavation and construction of subsequent panels. The method has been combined with a 3-D nonlinear analysis and a constitutive law providing bulk and shear modulus variation, depending on the stress path (loading, unloading, reloading). It has been observed that the most significant effect in front of a given panel occurs during the installation of that panel and that the effect on stress reduction and lateral movements in front of the subsequent panels is rather limited. The method has been used to estimate the effects on an adjacent 6-storey building by applying a full soil-structure interaction including the whole building. Progressively increased with subsequent panels installation settlement profiles are given along the building foundation. Moreover, settlements at specific points where leveling captures have been installed are given in cumulative form. The predictions indicate that the angular distortion of the building remains under the required limits of serviceability and at the same time provide the guidelines for the monitoring of the upcoming construction of the diaphragm wall in front of the building.

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