International Conference on Case Histories in Geotechnical Engineering

(2004) - Fifth International Conference on Case Histories in Geotechnical Engineering

16 Apr 2004, 8:00am - 9:30am

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Recommended Citation

Horodecki, G. A.; Bolt, A. F.; and Dembicki, E., "Deep Excavation Braced by Diaphragm Wall in Gdańsk (Poland)" (2004). *International Conference on Case Histories in Geotechnical Engineering*. 27.
https://scholarsmine.mst.edu/icchge/5icchge/session05/27

Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering
New York, NY, April 13-17, 2004

DEEP EXCAVATION BRACED BY DIAPHRAGM WALL IN GDAŃSK (POLAND)

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ABSTRACT

Protection of deep excavation by cast in situ concrete diaphragm walls for Shopping Center in Gdańsk (Poland) is described. The structure, was designed as five-storey-building with three additional underground floors and founded on diaphragm walls. The excavation was made by "half-floor" method with temporary supports of floor in the form of steel columns. Static calculations of excavation bracing were carried out by PLAXIS numerical code for both design stage (for two calculation schemes: anchored and supported by floor ring) as well as after its construction (back analysis). The calculations served for an assessment of predicted wall displacements, deformations of soil surface around the excavation and internal forces in subsequent stages of the excavation deepening and for different working schemes (supports) of the construction. During construction works vertical displacements of the soil surface around the excavation and surrounding buildings as well as horizontal displacements of diaphragm walls were monitored and compared with the results of corresponding calculations. Some exemplary distributions of calculated and measured values of the wall and soil surface deformations are presented. Subsoil unloading effects and the range of impact zones on the vicinity are also analyzed.

INTRODUCTION

New shopping and service centers require essential number of parking lots. In the city centers, when usually one deals with very compact infrastructure the only solution are multi-layered underground garages. For execution of such constructions, deep excavations are usually supported by the retaining walls. In order to minimize landtake and displacements of walls and nearby structures, the retaining walls, often in the form of diaphragm walls, are supported by anchors, props or parts of permanent floor slabs, at one or more levels.

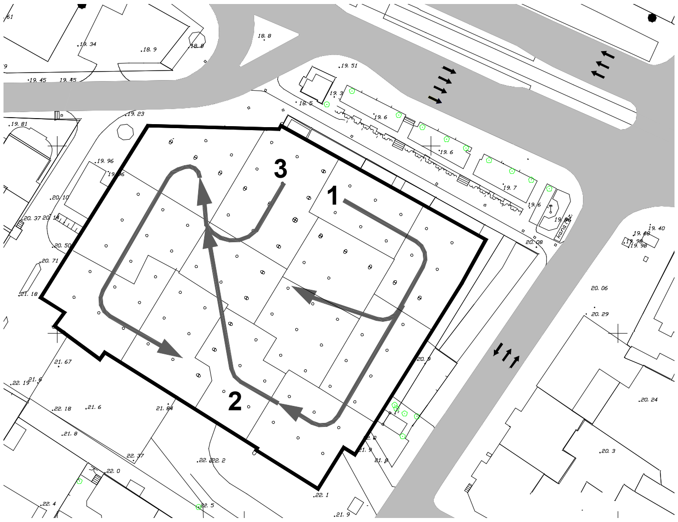

Fig. 1. Excavation layout with the sequence of deepening under the floor "-1"

Due to safety assurance of neighboring buildings and in order to avoid potential claims of its owners it is extremely important to determine the influence zones and to define the monitoring program of this constructions as well as ground surface nearby, prior to commencement of earth works, (Horodecki et al. 2003).

As an example, the structure of "Manhattan" shopping center in Gdańsk, which is localized very close to main city route and other communication roads and existing structures nearby is described. The area of site was covered by ruins underground of old buildings destroyed during the war. Perimeter, cast in situ concrete diaphragm walls which supported the sides of nearly squared, 85 m wide and over 12 m deep excavation were analyzed and designed, (Fig. 1). Influence zones due to excavation were defined and surveying program was assumed.

SUBSOIL CONDITIONS

The area is somewhat inclined West at the elevations varying from 19 ÷ 22.0 m a.s.l. From the morphological point of view it is a part of Pleistocene accumulation terrace. Near the surface, up to the depth of 50 ÷ 60 m there are quaternary formations in the form of sandy-gravel complex separated at the depth of 5.0 ÷ 18.0 m by clays and clayey sands. Under it there are clayey-sandy tertiary formations.

Soil types and their geotechnical parameters are summarised in Table 1. Typical geological profile is presented in Fig. 5.

Table 1. Values of characteristic geotechnical parameters

Soil	Plasticity or Density index	Moisture content [%]	Unit weight [kN/m^3]	Cohesion	Angle of internal friction [°]	Modulus of virgin compressibility [MPa]	Material coefficient
Clayey sand	0.28	16.37	20.5	15.9	14.8	28	1 ± 0.1
Sandy clay	0.31	17.46	20.5	18.2	14.2	26	1 ± 0.1
Fine sand	0.41	16.42	17.0	–	32.4	52	1 ± 0.1
Fine, sand	0.73	22.47	19.5	–	36.2	82	1 ± 0.1
Medium sand	0.76	12.47	18.3	–	30	128	1 ± 0.1
Gravel	0.77	12.52	20.5	–	40	195	1 ± 0.1

The groundwater table was found at the depth of 16.5 ÷ 17.8 m which is approximately equivalent to 4.0 to 5.3 meters below the excavation bottom.

DESIGN

Design Assumptions

Due to the significance of city route, as well as the constructions localised near excavation and economical aspects, in the design stage concrete diaphragm walls as retaining structures and foundations of shopping centre were assumed.

The structure consists of five floors and three additional underground floors. Zero level ±0.00 of the building corresponds to the elevation of +20 m a.s.l. The level of excavation under the foundation slab ("–3 level") corresponds to 7.50 m a.s.l. At this level there was local deepening to the elevation of 6.20 m a.s.l.

Finite Elements Analysis

The finite element analysis was carried out by PLAXIS v. 7.2 (Vermeer at al., 2000) numerical code taking into account two general alternative solutions of walls support i.e. by temporary anchors or by parts of permanent floor slabs. For each case four cross-sections of differed geometry of ground surface and applied loads behind the wall were analyzed, one for each side of the excavation. Static calculations aimed at a prediction of bearing capacity and serviceability limit states for final construction as well as for particular phases of the construction work during the progress. The numerical analysis was also to help in the assessment of potential displacements of the wall and the soil surface around the excavation as well as of the value of internal forces for the subsequent stages of the excavation, and for various working schemes of the construction. The calculation results allowed to assume maximal loading applied on the floor and corresponding displacements, too. The basic finite element meshes used in these studies are shown in Fig. 2 and 3, respectively.

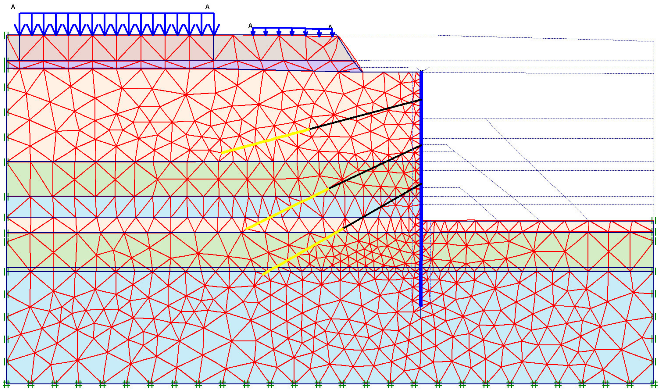

Fig. 2. The FEM mesh for the option of anchoring of the wall.

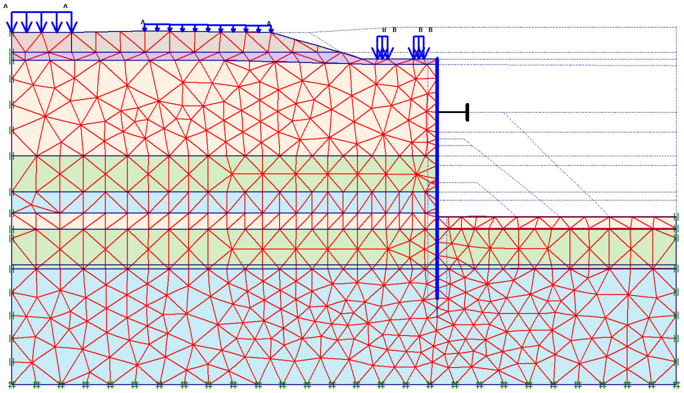

Fig. 3. The FEM mesh for the option of the support by „-1" floor.

For the case of anchored wall three levels of ground pre-stressing anchors were obtained. For second case analysed a part of permanent floor slab (ring shaped) on the level –1, supporting the wall was sufficient to meet the limit states conditions.

Due to the short time assigned for the excavation and taking into account other economical aspects, the support by the part of permanent floor slab was selected for final design.

Construction phases

The underground part of structure consists of three-level-floors. The basement is approximately 80 m x 90 m in plane and excavation depth 12.1 m (locally up to 13.4 m). The adjacent ground is retained by 17.5 m deep and 0.8 m thick perimeter concrete diaphragm wall with 0.6 m high capping beam. The walls are supported by the part of ring shape permanent floor slab localised at the level –1 (see Fig. 4). The floor was in turn supported by temporary H-shaped steel column in most cases situated exactly in position of final concrete structural columns. Only four of temporary columns which were installed in other places were cut out after finished concrete works of permanent columns. Columns were positioned on a variable nearly square grid of approximately 8 m x 8 m.

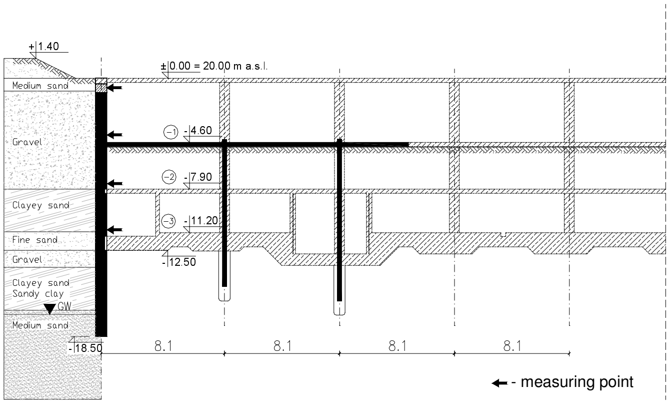

Fig. 4. Underground part of the structure (cross section).

The main stages of construction were as follows:
1. Construction of the diaphragm wall.
2. Construction of the capping beam.
3. Excavation to the "–1" level.
4. Construction of temporary H-shaped columns using diaphragm wall technique.
5. Casting of "–1" level slab (part of floor ring shaped; Figs. 5, 6).
6. Installing steel struts in local holes on "–1" slab.
7. Excavation to "– 3" level (bottom).
8. Casting of level "–3" slab (foundation slab).
9. Construction of permanent columns with H-shape steel pile inside, up to "–2" level.
10. Casting of level "– 2" slab.
11. Construction of permanent columns with H-shape steel pile inside, up to "–1" level.
12. Casting of remaining part of "–1" slab.
13. Removal of all struts.
14. Construction of permanent columns up to "0" level.
15. Casting of level "0" slab.

General overview of excavation at the "–1" and "–3" levels' stages are presented in Figs. 5 and 6, respectively.

Fig. 5. General overview of excavation at "–1" level stage. The "–1" parts of permanent floor slabs (the rest of floor has been carried out in further phase of works).

Fig. 6. General overview of excavation at "–3" level stage.

FIELD OBSERVATIONS

Influence Zones

The borders zones of predicted influence of excavation on adjacent areas were obtained in the design process (Fig. 7). The distances of influence zones I and II from the excavation edges were assumed to be 0.5 H and 2 H, respectively (where H is excavation depth). First zone corresponds to maximum width of wedge block behind the wall, whereas the second one includes the area where excavation induced soil deformations may cause damage of buildings, however without threat of its bearing capacity loss (Kotlicki and Wysokiński, 2002). Third zone refers to the area in which monitoring of deformations served for the assessment of the range of excavation impact.

Fig. 7. Excavation layout with indicated observation points, II zone, displacement isolines and the floor part supporting the walls.

Measured Displacements

In particular zones measurement points were installed on adjacent structures and on ground surface around the excavation. Horizontal movements of diaphragm walls were measured at four levels (Fig. 4). Vertical movements of capping beam were measured, too. Totally, there were 10 measuring points on structures, 17 points to measure ground surface deformations, installed at the depth 1.2 m and 43 points localised in 12 vertical axes to measure horizontal displacements of walls. Subsequent measuring points on the walls were installed after execution of next excavation stages. Monitoring frequency was adjusted to progress of excavation and construction works, once a week on average.

During the excavation stress relief of a subsoil was observed and related to it uplift of the walls and soil around the excavation together with small settlements of a subsoil in more distant areas.

Isolines of maximal soil displacements around the excavation at full subsoil unloading are presented Fig. 7. Figure 8 shows the change of vertical displacements of observation points 0 m, 5 m, 20 m, 27 m, 41 m distant from the walls for subsequent excavation phases. Base measurement refers to the “3” phase – i.e. achievement of “–1” level whereas the last measurement corresponds to “15” phase – finalising of “0” level. This demonstrates unloading effect and the range of influence zone due to excavation process. The “A”, “B”, “C” on Fig. 8 means the sequence of the digging works on the same level.

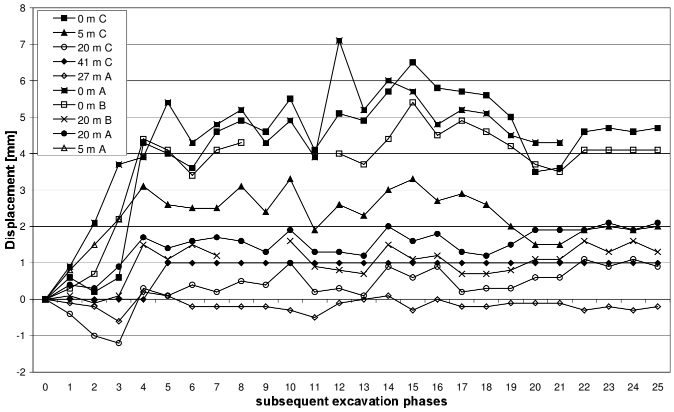

Fig. 8. Vertical displacements of observation points 0, 5, 20, 27 i 41 m distant from the wall.

BACK ANALYSIS

After execution of excavation, back analysis by PLAXIS v. 8.2 (Vermeer at al., 2002) numerical code was carried out. In the analysis the range of excavation influence on ground surface movements was being particularly considered. Soil unloading modulus were being assumed in such away in order to obtain approximately the same values of measured and calculated displacements. Other soil parameters were left without changes. The calculated and measured horizontal movements of wall after “7” stage (excavation to “-3” level) are compared in Fig. 10.

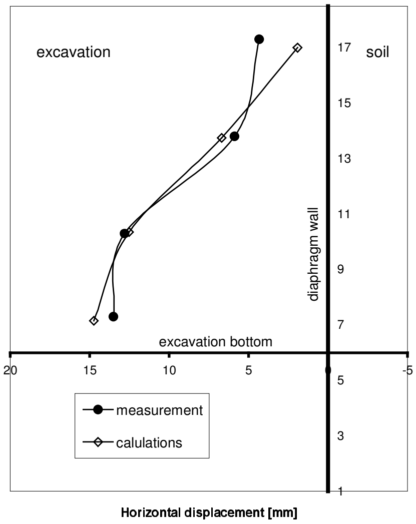

Fig. 9. Calculated and measured total horizontal movements of wall after “7” stage (excavation to “-3” level).

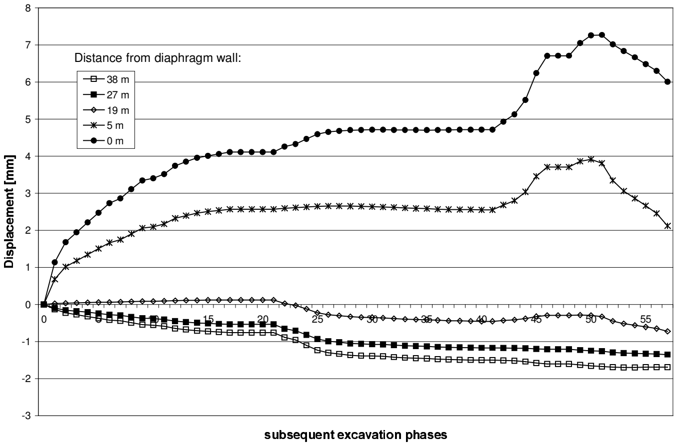

Fig. 10. Calculated vertical displacements of the soil for subsequent phases of excavation digging.

Calculated ground surface displacements behind the wall for points 0 m, 5 m, 19 m, 27 m, 38 m distant from the wall in subsequent phases of excavation digging are presented in Fig. 10.

DISCUSSION

Stress relief effect of a subsoil is consistent with theoretical distribution of immediate displacements (Burland et al, 1979). It effects in the uplift of walls and soil in the nearest vicinity together with small settlement of the subsoil in more distant areas from the excavation. Due to the application of secondary loads (from the construction) small settlements are observed around the excavation.

The calculated and measured horizontal displacements of diaphragm wall corresponding to the excavation stage to “–3”level for one excavation side are very similar, (Fig. 9). Due to the fact that horizontal displacements for next levels were measured after digging the excavation the these levels, the measured values were corrected based on the calculations by adding the displacements to the base measurements.

Calculated vertical displacements (2D back analysis) around the excavation are very close to the measured ones. Some differences in the range of soil uplift and settlements are related to the spatial effect and also to self-superimposing of subsequent work stages (deepening of the excavation and application of loads), which could not be included in the numerical analysis. It could be avoided when using 3D analysis, however some simplifications would have been necessary in this case, too, (Bolt et al, 2001). Measured large displacements of structures localised on the same side as main city route were probably caused by dynamic loading generated by the traffic (cars and trams). Non-symmetric range of the observed displacements can be related to the non-symmetric deepening of the excavation – Fig. 1. Similar effect of the influence of non-symmetric excavation digging on the range of the displacements around the excavation was observed also by Breymann at all. (1996).

In the case analysed assumed border of second zone corresponded approximately to the isoline describing zero vertical displacements, (Fig. 7).

It should be noted that observations and analyses presented refer in general to serviceability limit state, mostly elastic behaviour the subsoil and small displacements. Such displacements do not threat the safety of neighbouring structures. The predictions of surface displacements and influence zones borders are crucial for choosing the proper and optimal technology of earth works and type of supporting construction. It is also important to avoid the financial claims of the neighbourhood owners.

REFERENCES

Bolt A.F., Dembicki E., Horodecki G.A., Kryczałło A., Cudny M. [2001]. “An assessment of construction state and the subsoil conditions by loading tests and back analysis”, *Proc. of XV International Conference on Soil Mechanics and Geotechnical Engineering.* Istambuł 27-31.09.2001.

Breyman H., Fuchsberger M. and Schweiger H.F. [1996]. “Deep open excavation in soft plastic ground in Salzburg, Austria”, *Proc. Geotechnical Aspects of Underground Construction in Soft Ground,* Mair & Taylor (eds) Balkema, Rotterdam, pp. 69-74.

Burland, J.B., Simpson, B. and St John, H.D. [1979]. “Movements around excavations in London Clay”, *Proc. 7th European Conf. on Soil Mechanics and Foundation Engineering,* Brighton, Vol. 1, pp. 13-29.

Horodecki G.A., Bolt A.F., Dembicki E. [2003]. “Geotechnical problems of a design and performance of braced excavations in Gdańsk”. Croatian Geotechnical Journal. (in printed).

Kotlicki W., Wysokiński L. [2002]. “Buildings protection nearby deep excavations” (in Polish). Building Research Institute. Warszawa. Nr 376/2002.

Vermeer at al. [2000]. Plaxis. Finite Element Code for Soil and Rock Plasticity. V. 7.2. A.A. Balkema, Rotterdam, Brookfield.

Vermeer at al. [2002]. Plaxis. Finite Element Code for Soil and Rock Plasticity. V. 8.2. A.A. Balkema, Rotterdam, Brookfield.