Geotechnical Aspects of Underground Construction in Soft Ground – Viggiani (ed)

A 3D numerical study of deep excavations in clayey soils

E. Erbì & F.M. Soccodato
Department of GeoEngineering and Environmental Technologies, University of Cagliari, Italy

ABSTRACT: In this paper the results of a number of three-dimensional (3D) numerical analyses of deep excavations in a clayey soils are presented and discussed. The parametric study involved different geometries, with varying plan dimensions and depths of excavation. The analyses were carried out using a non linear, plasticity hardening soil model. The attention is focused on horizontal displacements (wall deflections) as well as on vertical displacements (settlement profiles behind wall). Two-dimensional (2D) plane strain numerical analyses were also carried out in order to compare results and to highlight 3D effects on predicted displacement fields. The results confirmed that, especially for excavations characterized by relatively low length to excavation depth or length to width ratios, 3D effects play a significant role in reducing the displacements obtained from 2D plane strain analyses.

1 INTRODUCTION

The evaluation of horizontal displacements of walls supporting deep excavations is a key-step in the subsequent estimation of ground movements. Frequently, deflections (and their maximum values) are evaluated following semi-empirical approaches (Clough & O'Rourke 1990, Mana & Clough 1981), or by carrying out one-dimensional (*Winkler* methods) or two-dimensional (2D) plane strain numerical analyses. In the first case, the effects of geometry, sequences of excavation, stiffness of wall and props are not explicitly taken into account, as the approach is based on a large collection of measured data. In the second case, representative numerical analyses must be based on adequate modelling of soil behaviour and, also, of the sequence of excavation and support installation. However, it is well known that three-dimensional (3D) effects related to the geometry (height, width and length) of the excavation can affect induced ground movements.

In fact, a number of studies based on the results of 3D numerical analyses allowed to highlight the effects of excavation geometry (e.g., Ou *et al.,* 1996, Zdravkovic *et al.,* 2005, Finno *et al.,* 2007). All these studies, although based on different soil and structural models, try to explain observed performances by comparing results obtained from 2D and 3D analyses.

With reference to the maximum horizontal deflections of the wall, Ou *et al.* (1996) suggested typical trends of a Plane Strain Ratio, defined as the movement computed by 3D analyses divided by that computed by a 2D plane strain simulation, as a function of the width to length (B/L) ratio and of the distance from the corner of excavation. Finno *et al.* (2007) carried out a large number of parametric 3D numerical analyses of supported excavations; they used a slightly different definition of the Plane Strain Ratio *PSR*, in this case defined as the maximum movement in the centre of the excavation wall computed by 3D analyses divided by that computed by a 2D plane strain simulation. Finno *et al.* (2007) showed that *PSR* in mainly influenced by the geometry, *i.e.*: length to excavation height (L/H_e) ratio and width to length (B/L) ratio, by the stiffness of supporting system and by the factor of safety against basal heave.

The results of these studies may be used to estimate the maximum deflection of the wall in order to evaluate, by following semi-empirical approaches such as that proposed by Ou & Hsieh (1998), settlement profiles behind wall. Distributions of ground movements parallel to the sides of the excavation may be finally estimated using semi-empirical procedures, such as that proposed by Fuentes & Devriendt (2010) or that based on the use of a complementary error function (Robosky & Finno, 2006).

This paper presents the main results of a number of 3D numerical analyses of supported deep excavations in clayey soils. The geometry of the parametric study, the soil profile and the modelling approaches are described in the following section; successively, the numerical results are presented and discussed.

2 LAYOUT OF NUMERICAL ANALYSES

Figure 1 shows a cross-section of the problem under consideration.

The soil profile is typical of the historical and central area of Rome. The top layer is made ground (*MG*), 10 m in thickness, mainly coarse grained; buried remains of ancient structures are also found in this layer. Soft silty clay (*Ag*) are found underneath, down to a depth of 60 m. Finally, a layer of sand and gravel (*GS*), a few meters thick, covers the stiff and overconsolidated clayey deposit (*Apl*) which represents the geological base formation of the Roman area. The pore water pressure distribution is almost hydrostatic, with a piezometric level located 10 m below ground level. Tip resistance q_C and small strain shear modulus G_0 profiles as obtained from cone penetration and cross-hole seismic tests are also shown in Figure 1.

The excavation height H_e ranges between 16 and 28 m (H_e = 16, 20, 24 and 28 m), with a fixed total height of the retaining structure H equal to 40 m. Thus, the ratio H / H_e is in the range 1.5 ÷ 2.5, which is typical of supported excavation in soft clayey soils. In plan, a total of 4 geometries were considered: 2 square (with $B = L$ = 20 and 40 m) and 2 rectangular (with B = 20 m and L = 40 and 80 m).

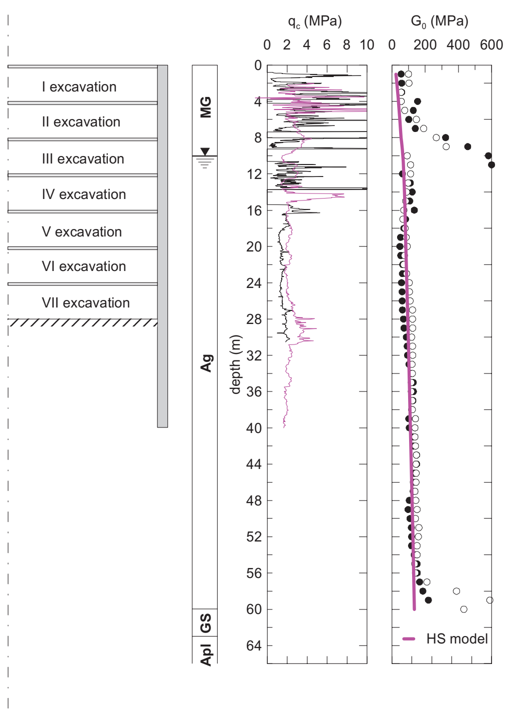


Figure 1. Cross-section of the problem and soil profile.

The numerical analyses were carried out using the commercial finite elements codes *PLAXIS 3D Foundation*. Figure 2 shows a view of a typical 3D mesh adopted. Symmetry allows to model only one quarter of the problem. Due to the high stiffness characterizing the *GS* and *Apl* layers, the bottom boundary of the mesh was located at a depth of 60 m; the mesh extends behind the wall at least 5 times the excavation height in order to minimize border effects (Robosky 2005, Ou & Shiau, 1998). Displacements were restrained in all directions along the bottom boundary, and in the out of plane direction on vertical lateral boundaries.

The mechanical behaviour of soils was modelled using the *Hardening Soil* model (Schanz *et al.*, 1999). This is an elasto-plastic soil model with stress-dependent stiffness, characterised by both deviatoric and volumetric hardening. Tables 1 and 2 report the constitutive soil parameters adopted in the analyses. Calibration of model parameters follows from the geotechnical characterization of the site (Rampello, 2011). In particular, the elastic stiffness E_{ur} was set equal to the small strain

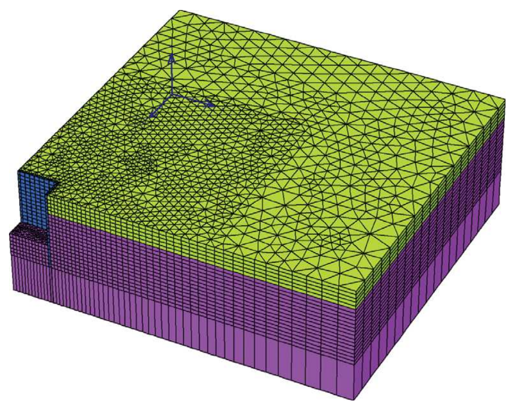


Figure 2. Typical 3D mesh.

Table 1. Hardening Soil model: general parameters.

Soil	γ (kN/m^3)	c' (kPa)	φ' (°)	OCR (–)	K_0 (–)	ν' (–)
MG	18.5	15	28	2.0	0.50	0.2
Ag	18.2	20	25	1.3	0.58	0.2

Table 2. Hardening Soil model: stiffness parameters.

Soil	E_{ur}^{ref} (MPa)	m (–)	$E_{ur}^{ref}/E_{50}^{ref}$ (–)	$E_{oed}^{ref}/E_{50}^{ref}$ (–)
MG	240	0.8	10.0	1.0
Ag	160	0.7	10.0	1.0

stiffness as obtained from seismic *in situ* tests and laboratory resonant column and bender elements tests. Thus, high values of E_{ur}/E_{50} and E_{ur}/E_{oed} ratios (E_{50} and E_{oed} being the stiffness in primary (virgin) deviatoric and isotropic loading, respectively) were adopted in order to match the marked non-linearity of soils observed in laboratory tests. Dilatancy was set equal to zero.

The analyses were carried out in terms of effective stresses, under undrained conditions for the clayey layer *Ag*. The retaining structure was considered to be a reinforced concrete (r.c.) diaphragm wall. Isotropic, elastic solid elements were used to model each single panel (sectional area: 2.4 × 1.0 m, Young modulus $E = 30$ GPa, Poisson ratio $\nu = 0.15$). In order to capture the influence of joints between panels, which give rise to an 'anisotropic' behaviour of walls, thin solid elements (10 cm in thickness) with reduced elasto-plastic mechanical properties were introduced to simulate the panel to panel contact.

The contact (interface) between wall and soils was also modelled with thin, solid elements (10 cm in thickness), characterised by the same constitutive model adopted for soils; stiffness and strength properties of these elements were set equal to 70% of the values corresponding to adjacent intact soils.

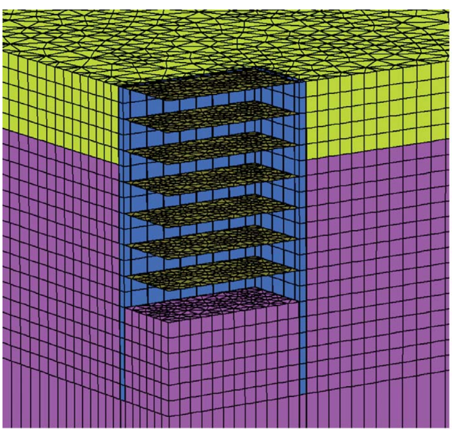
A *Top-Down* type construction sequence was assumed. Continuous r.c. floor slabs, 0.20 m thick and with a vertical spacing of 4.0 m, were modelled with anisotropic (membrane-like) shell elements.

Figure 3 shows two detailed views of the mesh with included wall and slab elements.

The steps of the numerical analyses were as follows:.

1. initial conditions (geostatic stresses);
2. activation of walls (wished in place);
3. excavation of 4 m;
4. installation of floor slab at previous elevation;
5. repetition of steps 3) and 4) until H_e is reached.

Two-dimensional, plane strain numerical analyses, with B = 20 and B = 40 m, were carried out with the same code, using the same soil and structural models adopted for the full 3D analyses.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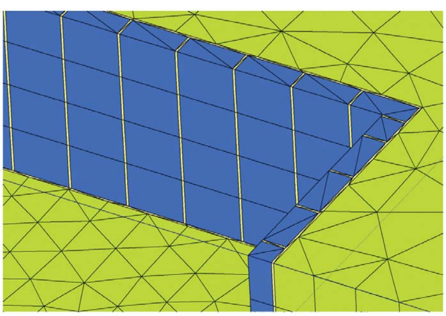
b)

Figure 3. Details of the 3D mesh: a) walls and slabs; b) panel interfaces.

3 ANALISYS OF RESULTS

Figure 4 shows a typical result obtained from the numerical analyses with B = 20 m, reporting wall deflections u and settlements behind wall at ground level w with reference to the symmetry section (at a distance from the corner $d = L/2$). For direct comparison, the results of the corresponding 2D plane strain analysis are also shown in the figure.

The shapes of wall deflections and of settlement profiles are very similar to those observed and reported in the literature for deep and multi-propped excavations in soft clayey soils.

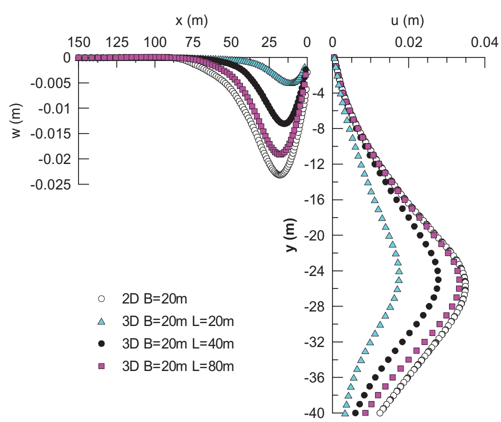

Figure 4. Settlement profiles and wall deflections obtained from numerical analyses (symmetry section; $H_e = 28$ m).

In fact, maximum deflections u_{max} occur at a depth close to excavation bottom and the maximum settlement w_{max} at some distance from wall position.

The different analyses, for a given excavation depth, showed deflection profiles very similar in shape, u_{max} occurring at the same depth; on the other hand, the distance from wall at which w_{max} appears decreases with increasing 3D geometry effects. With decreasing L, wall deflections are lower and settlement profiles are progressively narrower than those related to the corresponding 2D analysis.

Distributions of maximum wall deflection in the direction parallel to the wall are plotted in Figure 5 for two excavation depths. According to Ou *et al.* (1996), a distance of at least 20 m from corner seems to be necessary in order to reach an almost stationary value of u_{max}. Furthermore, the numerical results confirm that the 'anisotropic' modelling of the retaining structure (interfaces between adjacent panels) gives rise to non zero, even though relatively low, deflections at the corner of excavation (Zdravkovic *et al.,* 2005).

PSR is plotted as a function of L/H_e and of L/B in Figure 6. Open symbols refer to PSR associated to short sides (SS) of excavation. It may be noted that, according to Finno *et al.* (2007) and Ou *et al.* (1996), PSR increases with L/H_e and L/B, and decreases with increasing H_e. For all the 3D geometries considered, 2D maximum deflections were reached only for the excavation characterized by the maximum length (L = 80 m). The results obtained in this study fall in the range defined by the bounding curves suggested by Finno *et al.* (2007). For a given L/H_e ratio, average values of PSR decrease with decreasing side length. For $L/B \leq 1$ (and L = 20 m) 2D axisymmetric analyses with anisotropic wall properties should yield results closer to those obtained from 3D analyses (Zdravkovic *et al.,* 2005).

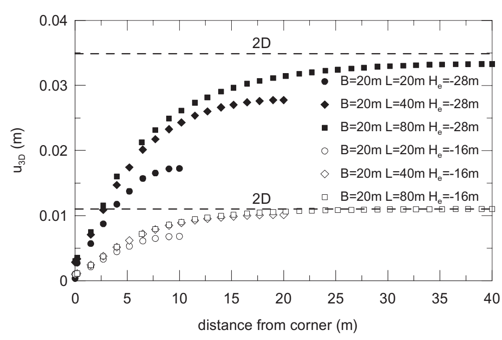

Figure 5. Distribution of maximum deflections along wall.

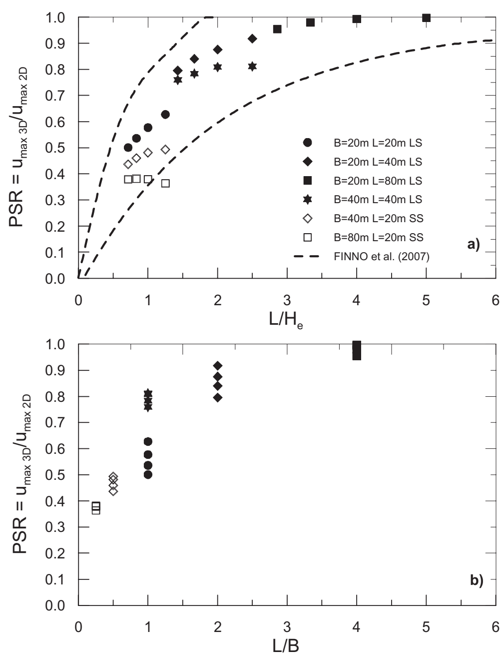

Figure 6. Plane strain ratios plotted as a function of: a) length to height of excavation ratio; b) length to width ratio.

Figure 7 shows the relation between maximum deflections and maximum settlements, normalised by excavation height, as obtained from numerical analyses. For L > 20 m, maximum normalised deflections range between 0.05 and 0.15%, values which are near the minimum values observed for deep excavations in clayey soil reported in literature (Long 2001, Moormann 2004). Figure 7 indicates that all the 2D analyses showed comparable and the highest normalized w_{max} to u_{max} ratios, ranging between 0.5 to 0.75: with increasing 3D effects, for a given H_e, the corresponding w_{max} to u_{max} ratios are progressively reducing.

These values are somewhat lower than the observed performances reported in literature, which suggests, for deep excavations in soft clayey soils, $w_{max} = u_{max}$ (Ou & Hsieh 1998, Long 2001). Finally, it may be also be noted the significant differences in w_{max} to u_{max} ratios between the 3D and the 2D analysis of the smallest squared excavation ($L = B = 20$ m).

The wall deflections already shown in Figure 4 are re-plotted in Figure 8, each profile being normalised by its maximum value. Shapes look similar, but it can be noted that, with increasing 3D effects, normalised deflections are significantly lower below excavation level and only just a little bit higher above.

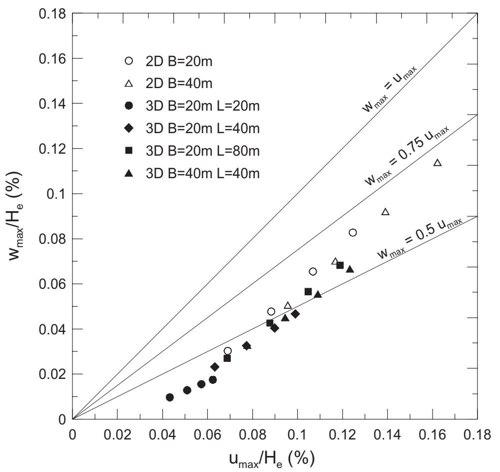

Figure 7. Normalised trends of maximum ground settlements versus maximum wall deflections.

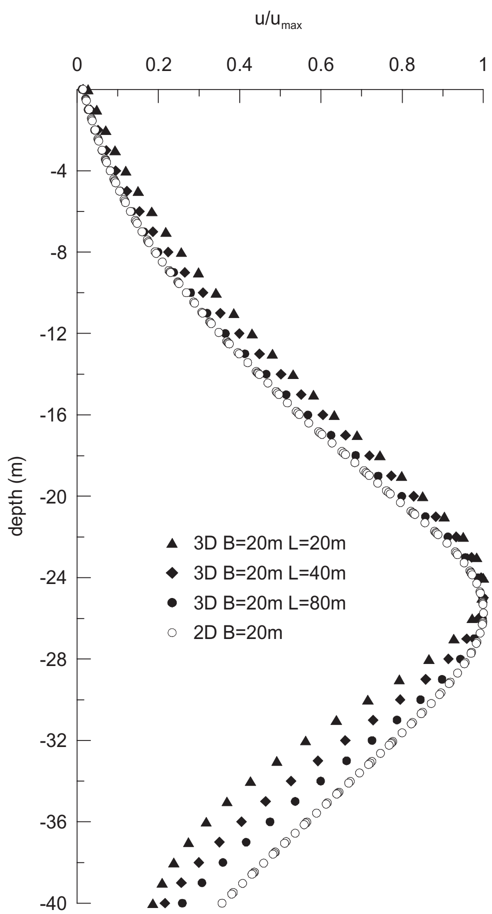

Figure 8. Normalized deflections ($H_e = 28$ m).

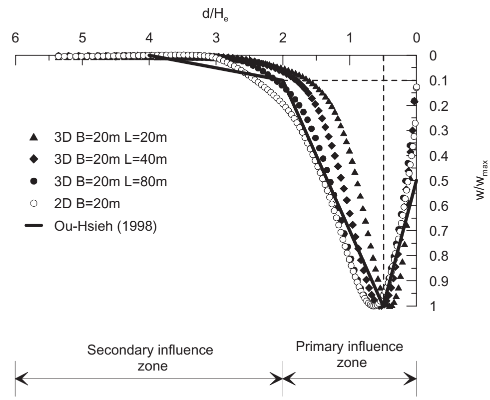

Figure 9. Normalized ground settlement profiles ($H_e = 28$ m).

Figure 9 reports normalised ground settlement profiles behind wall. Settlements and distances from wall position already shown in Figure 4 are normalised by excavation height and maximum settlement, respectively. It may be observed that, for all the analyses carried out, settlement profiles are in a nice agreement with the normalized shape proposed by Ou & Hsieh (1998) and based on a number of observed performances. This occurrence, which is not very common to be obtained from numerical analyses, can be attributed to the marked non-linearity that characterises the constitutive soil model used with the adopted selection of model parameters.

Figure 9 also shows that increasing 3D effects give rise to settlement profiles progressively narrower than those related to 2D conditions. This result is consistent with data shown in Figure 8: clearly, shapes of deflections and of ground settlement profiles are closely linked each other. Furthermore, the results indicate that the extension of ground settlement profiles is primarily related to deep (below excavation level) movements of the wall.

4 CONCLUSION

In this paper the results of a 3D numerical study of deep excavations in soft clayey soils have been presented and discussed.

The analyses were carried out using an advanced soil model capable to reproduce the significant non-linearity which characterises soil behaviour. The parametric study involved different lengths of excavation sides, considering square and rectangular excavations, and different excavation heights.

Thin solid elements with reduced mechanical properties were used to model the panel to panel and the panel to soil contacts.

2D plane strain numerical analyses were also carried out, with the same soil and structural models, in order to highlight 3D effects on wall and on ground movements.

The results obtained in this study appear to confirm some of the main findings reported in literature. More in detail:

– the ratio of maximum horizontal deflection obtained from the 3D and from the corresponding 2D plane strain analysis (*PSR*) tends to unity when L/H_e is greater than 4.0;
– for a given length to width ratio, *PSR* decreases with increasing excavation height;
– for squared excavations, or for $L/B < 1$, *PSR* are relatively low: 2D axisymmetric analyses with anisotropic wall properties should yield results closer to those obtained from 3D analyses;
– a distance from the corner greater than 20 m seems to be necessary to reach a stationary value of the maximum deflection;
– shapes of wall deflections are similar with varying L/B and L/H_e ratios. However, with increasing 3D effects the relative (normalized) amount of deep movements decreases;
– shapes of ground settlements profiles obtained behind the wall were in fair agreement with those obtained by well-established semi-empirical methods. This occurrence underlines the importance of the use of appropriate (highly nonlinear) constitutive soil models in this kind of analyses.

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