Settlement of single foundations due to diaphragm wall construction in soft clayey ground

R. Schäfer & Th. Triantafyllidis
Institute of Soil Mechanics and Foundation Engineering, Ruhr-University Bochum, Germany

ABSTRACT: The settlements of single foundations due to the excavation and pouring of an adjacent diaphragm wall panel in soft ground is a challenging and interesting question. Accordingly, for the simulation of the construction process of a single wall panel, a three-dimensional finite element model in conjunction with an advanced constitutive model for soft soils is presented. Numerous calculations have been performed analysing the influence of the panel geometry, the foundation loading as well as the distance between the foundation and the trench on the occurring settlements. Furthermore, the construction process of a diaphragm wall section consisting of three contiguous wall panels is simulated and recommendations have been drawn in respect of minimising the settlements of the foundation.

1 INTRODUCTION

Considering deep excavations in urban areas, diaphragm walls are frequently used as retaining structures in order to minimise the ground and wall movements due to the pit excavation and to ensure serviceability of nearby structures. However, the construction process of the retaining wall itself can already produce considerable surface ground settlements in case of soft soil conditions. The installation of a contiguous diaphragm wall follows a stepwise working plan comprising the excavation and the pouring of individual wall panels. In order to minimise the number of construction steps, the designer focuses on a great panel length, which is mainly controlled by the stability of the open trench supported by a bentonite slurry. Obviously, a great panel length will consequently produce greater surface settlements and possibly confines the serviceability of adjacent buildings.

Especially single footings have to be critically reviewed in this context. During the trench excavation process the loading cannot be transferred to areas adjacent to the open trench like in case of strip footings. Besides the panel length, the arising settlements depend on the foundation loading and the position of the footing in respect of the trench.

In case of a continuous diaphragm wall, the settlements of the nearby footing are not only controlled by the installation of the neighbouring trench, but furthermore by the excavation and pouring process of the adjacent panels.

2 NUMERICAL MODEL

2.1 *Simulation model*

In order to analyse the settlements of a single footing during the construction process of an adjacent single diaphragm wall panel, a three-dimensional finite-element model (Fig. 1) has been adopted. The model of a 35 m deep trench consists of approximately 17.000 tri-linear brick elements for a coupled consolidation analysis. To examine the influence of the panel geometry, three different panel lengths of $L = 3.6$, 5.4 and 7.2 m are considered. The single square footing (length $a = b = 1.8$ m) is located in the middle of the trench in a depth of 5.6 m below the surface ground level. The distance between the footing and the panel varies between 1.9 m and 7.2 m.

Starting from the initial geostatic stress condition, the footing is loaded in the first step of the calculation. The magnitude of the loading depends on the length of the trench and is calculated with respect to the factor of stability of the trench η. Table 1 presents the adopted effective footing bearing pressures p'. After an one-year consolidation period, the construction process of the wall panel starts. The excavation under slurry support is modeled by removing the finite elements inside the trench and applying loads on the open trench walls, which correspond to a hydrostatic slurry pressure with a bulk unit weight of 10.3 kN/m^3.

To simulate the pouring of the trench, the loads are subsequently increased to the fresh concrete pressure,

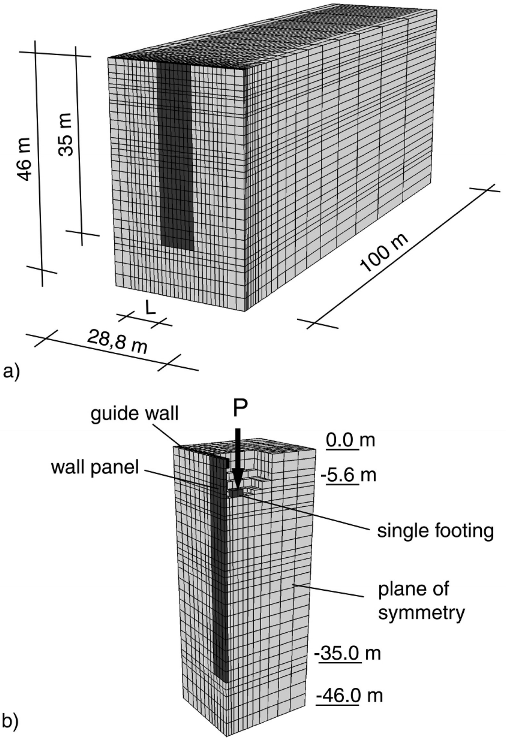

Figure 1. Three-dimensional FE-model of a single diaphragm wall panel (a) with an adjacent single footing (b).

Table 1. Adopted footing bearing pressures p′ and trench stabilities η in respect of the chosen length of the trench.

$L = 7.2\,m$					
η	1.05	1.14	1.25	1.38	1.52
p′ [kPa]	200	150	100	50	0
$L = 5.4\,m$					
η	1.06	1.15	1.29	1.41	1.54
p′ [kPa]	240	200	140	110	75
$L = 3.6\,m$					
η	1.04	1.1	1.21	1.32	1.56
p′ [kPa]	280	260	230	200	150

which is adopted by a bi-linear distribution over the panel depth according to Lings et al. (1994). Finally, the loads are removed and new finite elements representing the concrete are placed into the trench. The increasing stiffness due to aging is considered by a time-dependant evolution of the Young's modulus and the Poisson ratio (Schäfer & Triantafyllidis, 2004c). All in all, the excavation and the pouring of the trench last one day.

Moreover, the installation sequence of a diaphragm wall section consisting of three adjacent wall panels

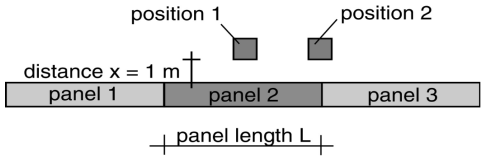

Figure 2. Schematic plan view of the diaphragm wall section and considered positions of the adjacent single footing.

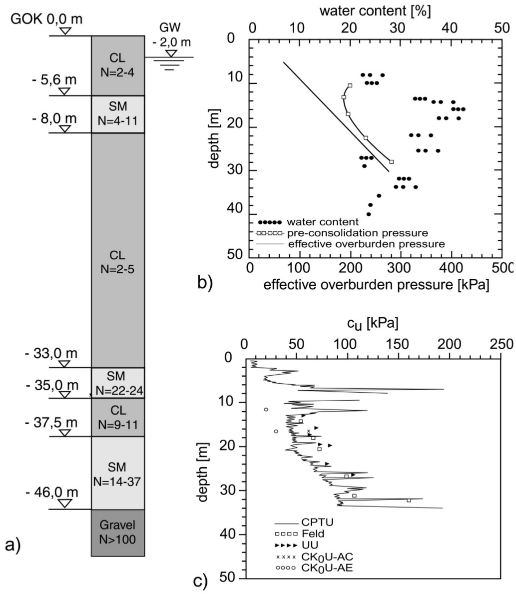

Figure 3. Subsurface soil conditions of Taipei [Ou et al., 1998].

is simulated. The construction process of each panel follows the description above. However, the position of the neighbouring single footing is varified as illustrated in Fig. 2.

2.2 *Subsurface soil conditions and constitutive equation*

Figure 3 illustrates the considered subsurface soil conditions, which correspond to the so-called Sungshan Formation of the Taipei-Basin.

In the course of the construction of the *Taipei National Enterprise Center* (TNEC), Ou et al. (1998, 2000) give a detailed description of the soil layers and the corresponding parameters. The ground is mainly characterised by a 25 m thick, slightly to normally consolidated clay layer of low to medium plasticity. The water content of the clay is close to the liquid limit ranging between 30 and 38% and the overconsolidation-ratio (OCR) decreases from 1.8 at the top to 1.05 at the bottom of the deposit. The coefficient of permeability was measured to be $k = 4 \cdot 10^{-8}$ m/s. This clay layer is overlain by a loose silty sand and an overconsolidated clay deposit. Below, medium dense silty sand and normally consolidated clay layers can be found. The natural bedrock stratum is located at a depth of 46 m below the surface ground level. In the following numerical calculations, the required soil parameters and state variables are directly taken from the literature (Ou et al., 2000, 1998), correlated with given quantities or estimated.

The stress-strain behaviour of the soft clay deposits is described by a visco-hypoplastic model (Niemunis, 2003), which takes into account the viscous properties and the small-strain-stiffness of the ground.

$$\dot{\mathbf{T}} = \mathcal{L}(\mathbf{T}, e) : (\mathbf{D} - \mathbf{D}_v) \tag{1}$$

$$\mathbf{D}_v = -\frac{\mathbf{B}}{\|\mathbf{B}\|} \cdot \dot{\gamma} \cdot \left(\frac{1}{OCR}\right)^{1/I_v} \tag{2}$$

The total strain increment $\mathbf{D}$ is decomposed into an elastic $\mathbf{D} - \mathbf{D}_v$ and a viscous part $\mathbf{D}_v$ (Eq. 1). The viscous strain increment controls creep, relaxation and rate dependence and is proportional to the over-consolidation-ratio OCR with a power function of $-(1/I_v)$ ($I_v =$ viscosity index) and a reference creep rate, see Eq. 2. The direction of $\mathbf{D}_v$ is given by the second-rank tensor $\mathbf{B}$, which results together with the stiffness tensor $\mathcal{L}$ from the theory of hypoplasticity. The sand layers are described by a non-linear elasto-plastic constitutive equation.

The high quality of finite-element predictions using the visco-hypolplastic model in case of normally to slightly overconsolidated clay has frequently been shown (Niemunis, 1996, Schäfer & Triantafyllidis 2004a/c). Especially the calculation of the TNEC-excavation project (Schäfer and Triantafyllidis, 2004a/b, Schäfer, 2004) and the comparison of the numerical results and in-situ measurements provides a very good verification for the considered soil conditions.

3 NUMERICAL RESULTS

3.1 *Single trench installation*

Figure 4 illustrates the averaged settlements of the single footing due to the excavation of a slurry-supported trench versus the effective loading p′.

The footing is located in the middle of the panel with a distance of x = 1.9 m. The settlements super proportionally increase with an increasing panel length and foundation loading. For the example of a panel length of L = 7.2 m, Fig. 5 presents the settlements due to the trench excavation and at the end of the subsequent pouring process versus the safety factor η of overall

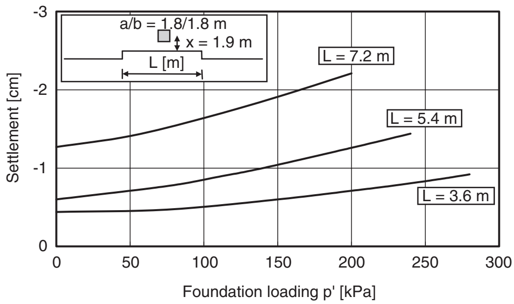

Figure 4. Settlements of the footing due to trench excavation versus effective loading p′.

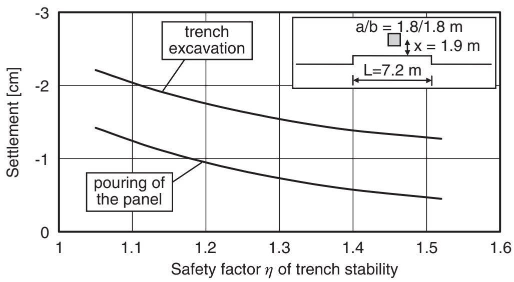

Figure 5. Settlements due to trench excavation (L = 7.2 m) and at the end of the pouring process versus safety factor η of the trench stability.

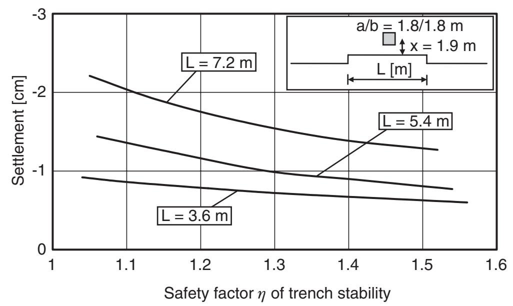

Figure 6. Settlements due to trench excavation for different panel geometries versus safety factor η of the trench stability.

stability of the open trench. A decreasing safety factor, which means an increasing loading of the foundation (see. Table 1), results in larger movements. However, the placement of fresh concrete obviously reduces the former settlements subsequently.

Figure 6 illustrates the settlements due to the excavation versus the safety factor η of the trench stability for all of the examined panel lengths. For a given safety factor η (e.g. $\eta = 1.3$), the settlements increase with a rising panel length, although the corresponding foundation loading decreases according to Table 1.

Considering the settlements normalised by the panel length L, however, the calculated movements of the footing can be summarised within a narrow range (Fig. 7) versus the safety factor η.

Actually, the settlements decrease with an increasing distance between the panel and the footing. Figure 8 presents (panel length L = 7.2 m) in this context the settlements due to the slurry-supported excavation versus the normalised distance x/L for a safety factor of $\eta = 1.3$ and 1.0 of the trench stability.

3.2 *Construction process of a wall section*

With respect to a contiguous diaphragm wall, the settlements of nearby footings are not only affected by the construction process of a single wall panel, but by the installation of adjacent wall panels as well.

In order to quantify the respective influence, a finite element model of a diaphragm wall section with three adjacent panels has been adopted. Figure 9 presents the settlements of neighbouring single footings during the stepwise construction of the wall section. For the installation process of the individual panels, an alternating construction sequence like 2-1-3 (for the panel number see Fig. 9) is adopted. Two different positions of the footing are considered: a) the single foundation is located in the centerline of panel 2 with a distance of x = 1.9 m (position 1, Fig. 9) and b) a location at the transition from panel 2 to 3 has been chosen (position 2, Fig. 9).

Considering position 1, the greatest settlements result from the excavation of the panel 2. In comparison, the installation procedures of the adjacent wall panels 1 and 3 only have a minor impact on the arising settlements. However, the greatest movements temporarily appear during the trench excavation of panel 3. Concerning the single foundation in position 2, smaller settlements can generally be observed.

Figure 10 presents a comparison of the maximum settlements versus the factor of safety of the trench stability, which can temporarily be expected during the construction process of the wall section. For the calculation of the safety factor η, a single wall panel with an adjacent single footing in the centerline of the trench has been assumed.

4 DISCUSSION

The results of the finite element calculations show, that the excavation process of a wall panel adjacent to the

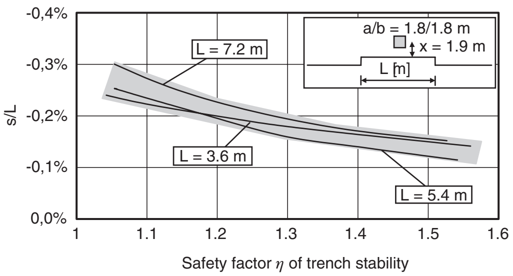

Figure 7. Normalised settlements s/L (L = panel length) due to trench excavation versus safety factor η of the trench stability.

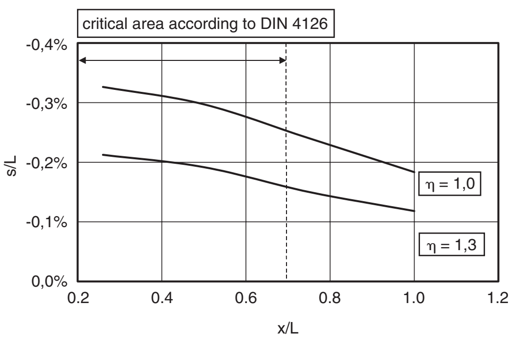

Figure 8. Normalised settlements (L = 7.2 m) due to trench excavation versus the normalised distance x/L between the panel and the footing.

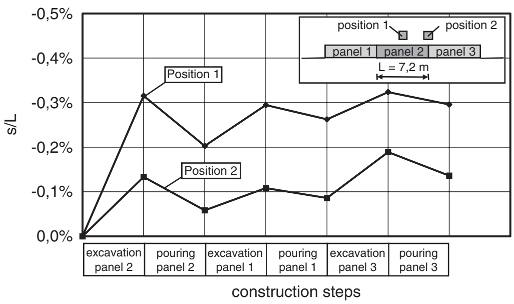

Figure 9. Settlements of the footing during the stepwise installation of a diaphragm wall section consisting of three adjacent wall panels.

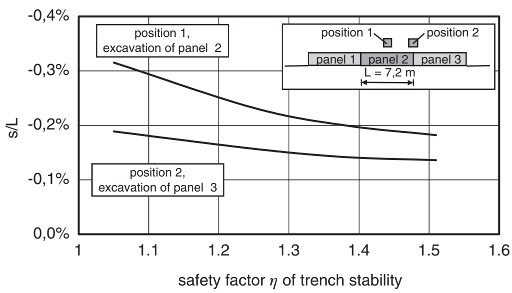

Figure 10. Comparison of the temporary maximum settlements of a single footing in position 1 and 2 during the installation process of the diaphragm wall section.

footing has the greatest impact on the development of settlements. For a given foundation loading, the settlements super proportionally increase with an increasing panel length (Fig. 4). Actually, an increasing length requires a higher mobilisation of the shear resistance of the ground, which goes along with greater deformations and settlements of the footing. However, the settlements still increase with the panel length, even if we consider the same factor of safety η (Fig. 6). Assuming a factor of $\eta = 1.3$ for example, a loading of $p' = 200$ kPa is valid in case of a trench length of $L = 3.6$ m whereas only $p' = 80$ kPa can be adopted for $L = 7.2$ m. Fig. 11 shows the corresponding stress paths in the clay layer 3 m below the footing. Starting from the initial K_0-line, the loading process (point $1 \rightarrow 2$) leads to an increasing mean effective stress p' and an deviatoric stress q. During the subsequent trench excavation under slurry support (point $2 \rightarrow 3$), p' drops and q remains more or less constant, independent of the panel length. However, the end points of the stress paths suggest a similar mobilisation of the shear resistance for both panel lengths.

This is actually confirmed by Fig. 12, which illustrates the mobilised friction angle φ_{mob} versus the deviatoric strain ε_q. At the end of the trench excavation process, the shear resistance below the single footing is mobilised to a comparable degree for a panel length of $L = 3.6$ and 7.2 m and the deviatoric strains are of the same magnitude. However, the greater the panel length is the larger is the volume of the soil mass, which has to sustain the shear deformation. Consequently, the settlements increase with a rising length, although φ_{mob} remains constant.

The calculated deformations of the footing in case of $\eta = 1.3$ vary from $s/L = 0.18$–0.22%. This magnitude corresponds to the results of numerical calculations, which have been run for sandy underground conditions (Mayer, 2000). However, the panel installation in sand is a drained process and the final settlements are expected to occur immediately. On the contrary, the trench excavation in clayey soil layers leads to excess pore water pressure, which slowly dissipates and causes a swelling of the ground. FE-calculations adopting a period of four weeks between the end of the trench excavation and the beginning of the pouring process actually show increasing settlements up to 40% due to this consolidation process.

The deformations of the footing do not only depend on the stability of the open trench, but furthermore on the distance between the footing and the panel. For a given safety factor η, the settlements decrease with an increasing normalised distance x/L; especially if x/L exceeds 0.5. This development can be attributed to an arching mechanism, which transfers the foundation load to areas besides the open trench which reduces the deformations. However, a minimum distance is necessary to mobilise the arching effect. In case of $x/L < 0.5$, most of the load is absorbed by the slurry pressure and accordingly higher settlements can be observed.

Figure 8 additionally highlights the critical area according to the German code of practice DIN 4126. If a single footing is located within this area ($x/L < 0.7$), a higher safety factor of stability ($\eta = 1.3$ instead of 1.1) is required in order to reduce the settlements during the construction process of the panel. From the curves in Fig. 8 it can be seen, that for a greater distance between the footing and the trench a lower safety factor η is actually acceptable with regard to the arising settlements.

Subsequent to the trench excavation, the pouring process leads to a heave of the footing. Considering a panel length of $L = 7.2$ m and a required safety factor of $\eta = 1.3$, this heave is about 50% of the former settlements due to the excavation process (Fig. 5). This can be ascribed to the high fresh concrete pressure, which exceeds the total earth pressure at rest in the upper half of the trench and causes convex deformations of the adjacent ground (Schäfer & Triantafyllidis, 2004c). These deformations are connected to a vertical movement of the ground in the vicinity of the trench resulting in decreasing settlements of the footing.

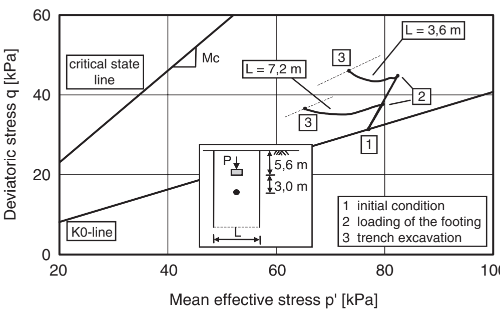

Figure 11. Stress path (Roscoe stress-invariants) below the single footing due to the loading and the trench excavation process.

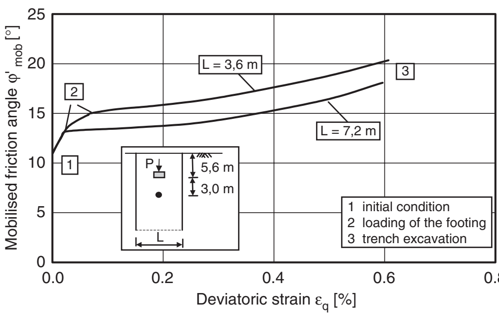

Figure 12. Mobilised friction angle φ_{mob} versus deviatoric strain ε_q below the single footing.

Considering the construction process of the diaphragm wall section, the installation procedures of the outer panels 1 and 3 only have a minor influence on the deformations of the single footing located in position 1 (see Fig. 9). Schäfer and Trianafyllidis (2004c) already showed, that the effective stress field and the pore water pressure behind a diaphragm wall panel are mainly affected by the excavation and the pouring of the panel itself and less by the construction process of the adjacent panels. Nevertheless, the greatest settlements temporarily occur during the trench excavation of panel 3 (Fig. 9, upper curve). This results from the fact, that the settlements induced by the installation of the panel 2 are superposed with those arising from the continuous installation procedure of the neighbouring panels.

More important is the position of the footing with respect to the panels. The greatest settlements can be expected if the footing is located in the middle of the panel (position 1). During the trench excavation, the footing is mainly retained by the slurry pressure. However, the closer the distance between the footing and the edge of the trench is, the lower is the impact of the trench excavation on the deformation behaviour. Therefore the division of the diaphragm wall into individual panels should be arranged in such a way, that the single footing of an existing nearby structure is located at the transition of two adjacent trenches (position 2, Figs 9 and 10). Accordingly, the settlements resulting from the installation procedure of the wall section can be reduced up to 40%.

5 CONCLUSIONS

The finite element calculations show, that the installation process of a diaphragm wall in soft clayey ground causes moderate settlements of neighbouring single footings. However, the basic requirement is a continuous and efficient installation procedure of the panel. A delay of the pouring process allows the ground to swell which results in increasing deformations.

The remaining deformations of the footing after the completion of the wall are of the same magnitude as those ones, which temporarily occur during the trench excavation under slurry support of an adjacent single wall panel. These deformations can easily be correlated with the safety factor η of trench stability. The greater the distance between the footing and the panel is, the lower are the settlements during the excavation process, even if we consider a constant factor η respectively. The reason for this is an arching effect in the ground, which transfers the footing load to areas besides the open trench.

However, the pouring of the trench provides a heave of the footing due to the high fresh concrete pressure especially close to the surface ground level. In case of a continuous diaphragm wall section, the arrangement of the individual panels should be chosen in consideration of the adjacent footings. The greatest induced settlements occur if the footing is located in the middle of a panel. The deformations can effectively be reduced, if the division of the diaphragm wall is provided in such a way, that the single footing is located at the transition of two adjacent panels.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the financial support from the German Research Council (DFG, AZ 218/4-1) and Federal Ministry of Research and Technology for the scientific program “preservation oriented geotechnics”. Furthermore they like to express their gratitude to Dr. D. König and Dr. A. Niemunis, who developed the FORTRAN-routine of the constitutive model.

REFERENCES

Lings, M.L., Ng, C.W.W., Nash, D.F.T. 1994. The lateral pressure of wet concrete in diaphragm wall panels cast under bentonite. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering 107: 163–172

Mayer, P.-M. 2000. Verformungen und Spannungsänderungen im Boden durch Schlitzwandhertsellung und Baugrubenaushub. Institute of Soil and Rock Mechanics, University Fridericana, Karslruhe, No. 151

Niemunis, A., 1996. A visco-hypoplastic model for clay and its FE-implementation. XI Colloque Franco-Polonais en Mécanique des Sols et des Roche Applique: 151–162

Niemunis, A. 2003. Extended hypoplastic models for soils. Habilitation, Institute of Soil Mechanics and Foundation Engineering, Ruhr-University Bochum, No. 34

Ou, C.-Y., Liao, J.-T., Lin, H.-D. 1998. Performance of diaphragm wall constructed using top-down method. Journal of Geotechnical and Geoenvironmental Engineering 124(9): 798–808

Ou, C.-Y., Shiau, B.-Y., Wang, I.-W. 2000. Three-dimensional deformation behavior of the Taipei National Enterprise Center (TNEC) excavation case history. Canadian Geotechnical Journal 37: 438–448

Schäfer, R. & Triantafyllidis, T. 2004a. Influence of the construction process on the deformation of diaphragm walls in soft clayey ground. Tunnelling and Underground Space Technology 19(4–5):475, Elsevier

Schäfer, R. & Triantafyllidis, T. 2004b. Auswirkung der Herstellungsmethode auf den Gebrauchszustand von Schlitzwänden in weichen bindigen Böden, Bautechnik, in print

Schäfer, R., Triantafyllidis, T. 2004c. Modelling of earth- and pore water pressure development during diaphragm wall construction in soft clay, International Journal for Numerical and Analytical Methods in Geomechanics 28:1305–1326

Schäfer, R. 2004. Einfluss der Herstellungsmethode auf das Verformungsverhalten von Schlitzwänden in weichen bindigen Böden, PhD-thesis, Institute of Soil Mechanics and Foundation Engineering, Ruhr-University Bochum, No. 36