Theme lecture: Retaining structures and excavated slopes

Exposé sur le thème: Soutènements et talus de déblais

M. Nussbaumer & P.-A. Von Wolffersdorff – *Ed. Züblin AG, Stuttgart, Germany*

ABSTRACT: The paper deals with retaining structures and excavated slopes in soil and rock. A number of examples from construction practice are presented to illustrate the current development trends of different types of structures. Methods for analyzing the stability of slopes and composite retaining structures are discussed. Also considered are aspects concerning the limit state analysis of structural systems with a marked soil-structure interaction and the behaviour of retaining structures under seismic loading. Examples are presented to explain the serviceability analysis of retaining structures and slopes. The focus here is on deformation prediction based on numerical methods and the application of the observational method.

RESUME: Cette exposé s'occupe des ouvrages pour retenir soit des terres en remblai, c'est-à- dire rapportées, soit le terrain en place et des talus, dans les sols meubles et rocheux. A l'aide de quelques types de constructions, un aperçu sera presenté sur les tendances de développement à l'heure actuelle, des divers constructions. Des méthodes de calcul de la stabilite des talus, des constructions composites flexibles seront discutés. On va expliquer quelques points de vue sur la stabilité des systeme portants sous action reciproque entre ouvrages et sol, ainsi le comportement à l' appui de ces ouvrages sous sollitation sismique. Des analyses d'utilité de quelques ouvrages de soutenèment et des talus seront presentées à l'aide des exemples concrets. Les points principaux sont: les pronostics de déformation avec la méthode de calcul numérique et l'utilsation de la méthode d'observation.

1 INTRODUCTION

This report gives an overview of the wide field of retaining structures and excavated slopes in soil and rock. The subject includes unsupported slopes; gravity walls; embedded, anchored or strutted retaining walls; different composite retaining structures – such as reinforced soil, crib retaining walls, geotextile walls, retaining structures of natural materials (gabions, soil reinforced with vegetation) – soil nailing and dowelling as well as combined structures. Not included are retaining structures as part of underpinning systems, uplift-resistant structures for deep excavations in groundwater, and special problems in connection with the renovation of historical retaining structures.

Chapter 2 presents different types of retaining structures, but without attempting a classification. Instead, examples from construction practice have been selected in order to provide a representative picture of the current developments of modem retaining structures and to point out the present requirements with respect to methods of design and analysis, construction and quality assurance.

Chapter 3 initially gives an overview of the most important methods for analyzing the stability of slopes. On this basis, the principles of the limit design of composite structures are explained. The limit state analysis and design methods for structures with a marked soil-structure interaction are discussed in relation to new safety definitions. A practical example of a three-dimensional stability analysis of a projecting comer of a retaining wall is presented. The calculation of a quay structure subsequent to destruction by an earthquake shows the framework for examining the dynamic structural behaviour by numerical methods.

The displacements and deformations of retaining structures and slopes are dealt with in the last chapter. Recommendations are made for the consideration of serviceability, which is related to displacements. It is demonstrated that deformation analyses for retaining structures are not yet sufficiently reliable, so that the use of monitoring equipment will often be necessary during construction. The example of an opencast mining slope and of an excavation pit with neighbouring buildings shows the capabilities of the observational method in conjunction with the application of modem numerical methods. Finally, the paper examines the problems of 3D deformation analysis of deep retaining walls.

2 TYPES OF STRUCTURES

2.1 *Variety of retaining structures*

There is a great variety of retaining structures involving almost all sectors of foundation engineering. The broad range of retaining structures – including excavated slopes – becomes apparent, e.g., when one tries to group these structures according to the following characteristics:

a) Material:
   Retaining walls of masonry, concrete, reinforced concrete; steel sheet piles; geotextile structures (reinforced earth); structures of natural materials (vegetation-reinforced earth), etc.
b) Structural system:
   Gravity walls; embedded, strutted or anchored retaining walls supported by passive earth pressure, composite structures; open retaining walls, etc.
c) Construction method:
   Backfilled structures; driven structures; structures assembled from precast elements; cast-in-place structures, etc.
d) Utilization:
   Slope stabilization; quay structures; excavations, etc.

Other classifications have been made by (Brandl, H. 1992, Lackner, E. & Müller, J. 1992, Smoltczyk, U. 1992, Stocker, M. & Walz, B. 1992, Weißenbach, A. 1992). In 1992 Gudehus published a summary of the various types of retaining structures. In particular, he showed the range of possibilities arising from the combination of materials, structural systems and structural elements.

Summarized presentations of quay structures have been provided by (Lackner, E. & Müller, J. 1992, Schenck, W. & Smoltczyk, U. & Lächler, W. 1992) and specifically for deep harbour basins by (Stehn, H.-J. 1992, Dücker, H.P. & Miller, C. 1996).

The retaining structures here are distinguished according to service life, i.e., a distinction is made between temporary and permanent structures (see Fig. 1). In this context, the following two sections present examples that are typical for the current development trends, and at the same time particularly noticeable because of their unusually large dimensions.

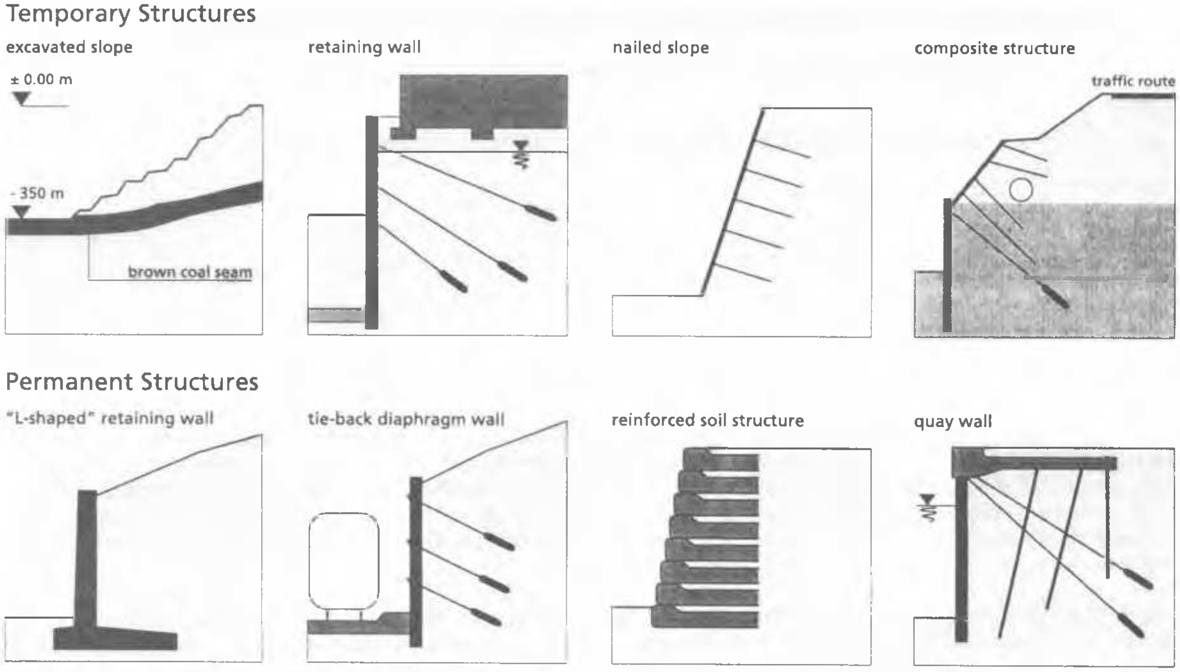


Figure 1. Examples of temporary and permanent retaining structures

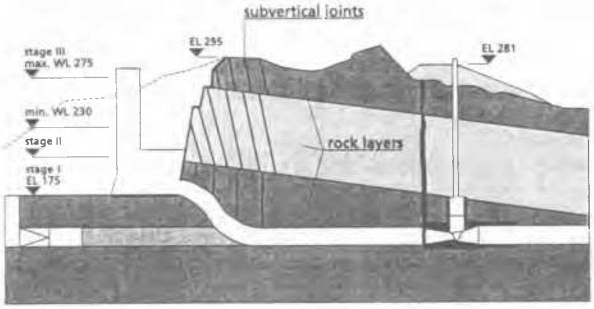


Figure 2. Rock cut at the intake tower

2.2 *Permanent structures*

– *Rock slope*

The Xiaolangdi multi-purpose dam in the Chinese Province of Henan is the key project for regulating the lower reaches of the Yellow River. The project also involved stabilization of a rock slope up to a height of about 120 m, which was completed in 1996. The about 300 m long slope section provides space for the intake structure.

The rock consists of alternating sedimentary brittle rock layers. The rock joints dip between 70° to 80° (see Fig. 2). The very steep cut, average inclination about 80°, had to be stabilized. Stabilization measures were:

- Prestressed rock anchors 25 to 40 m long with a service load of 1000 to 2000 kN
- Rock dowels 5 to 12 m long with a service load of 80 to 150 kN
- Rock bolts 4.5 to 10 m long with a service load of 100 to 150 kN,
- Shotcrete facing 10 cm thick.

The major stabilizing elements are the prestressed rock anchors, which were installed on a grid of 6 m x 6 m and at an upward angle of 15°, in view of the geological conditions. The unusually great depth of the rock cut and the high safety requirements for the operation of the dam and the hydroelectric power station, made great demands on design and construction work. It was hardly possible to draw on experiences with rock cuts of similar dimensions. The following stages were relevant for the stability analysis:

1. Construction stage after the 120 m deep cut
2. Construction stage after installation of the about 30 m thick base structure, with a remaining rock slope about 90 m high
3. Final stage after impounding of reservoir

These three stages do not only differ in depth of cut and loading assumptions, but also with respect to the water pressure in the joints. Furthermore, the rock is penetrated by numerous large tunnels. Since they were already driven into the rock before the final cut depth was reached, the resulting disturbances had to be taken into account in the stability analyses.

The stability analyses were carried out using non-linear 3D FEM models, in addition to 2D FEM models. Fig. 3 presents such a finite-element mesh with the maximum excavation depth as well as calculated displacements. The complexity of the stability analyses was increased by the need to consider seismic loadings.

Meanwhile the 30 m thick base structure has been completed, i.e., the second construction stage has now been implemented. The results of the monitoring, which was carried out as part of a comprehensive measuring program, indicated that the displacements of the rock slope were acceptable during the first construction stage involving the largest cut of 120 m.

– *Bored pile wall*

An impressive bored pile wall as a retaining structure for a slope was installed for the construction of an office building in Stuttgart (Seeger, H. 1992). The building had to be placed into a slope consisting largely of Keuper marl. Facing the hill, the building has 8 storeys, which required a cut of 30 m depth and 85 m width. It had to be taken into account that the building must not be permanently loaded with earth pressure from the hillside.

The permanent slope stabilization was carried out by means of an open bored pile wall with permanent tie-backs in up to 10 levels. The reinforced concrete piles with a diameter of 90 cm were installed with spacings of 1.50 m to 2.00 m. The interspaces were filled with reinforced shotcrete. At the top, the bored pile wall is capped by a reinforced concrete beam.

The length of the permanent tie-backs varies from 14 m in the lower levels to 32 m in the upper levels of the retaining structure (see Fig. 4).

There were no previous experiences with excavations of this size in unweathered Stuttgart Keuper marl. In particular, swelling pressures as a result of gypsification could not be excluded. In such a case one would have had to expect considerable earth-pressure increases with additional forces acting on the anchors. For this reason, the underground conditions were thoroughly investigated. The

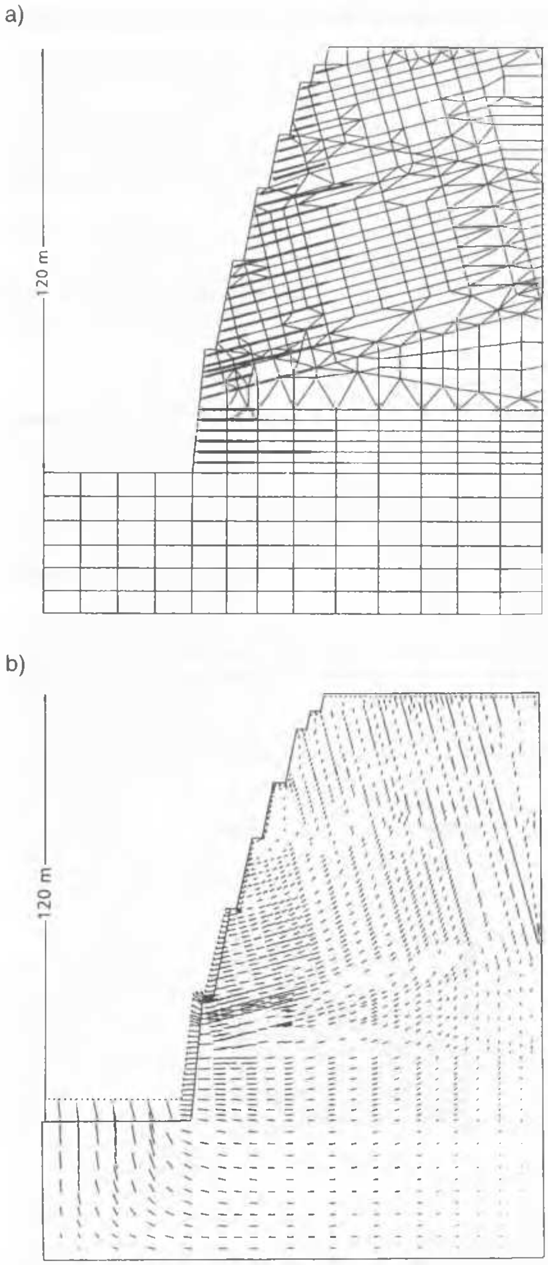


Figure 3. a) 2D finite-element mesh for the stability analysis of the rock at the intake tower with maximum excavation depth b) calculated displacements: settlements of slope crown (0.75 cm) and heave of slope toe (4 cm)

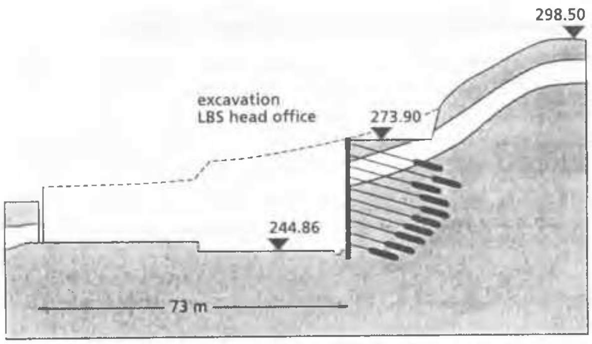


Figure 4. Cross-section of the 10-fold anchored bored pile wall

assumptions for the calculation of the permanent grouted anchors took account of this influence.

- Increased earth pressure $E = 0,75 \cdot E_a + 0,25 \cdot E_o$ for design of the anchors,
- Earth pressure with a coefficient of $K_a = 0,2$ as a lower limit substituted for the earth pressure reduced by cohesion,
- Stability analysis along a deep block sliding surface and additionally analysis of overall stability on a circular sliding surface with increased applied loads.

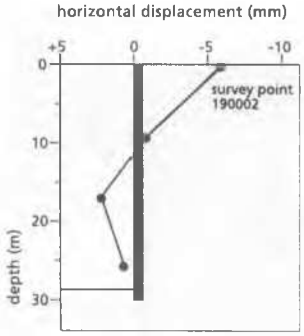


Figure 5. Horizontal deformations of the bored pile wall

Furthermore, the following stipulations were made for the dimensioning so as to provide additional safety:

- The tie-backs were only utilized for a maximum of 85% of the permissible load
- Each anchor was tested with 1.5 times the permissible load.
- The length of grout bodies was not less than 6 m.

A comprehensive measuring program – involving anchoring force and extensometer measurements and goedetic displacement measurements – was an essential part of the safety concept. The measured results showed that the suspected increase in earth pressure did not occur, the anchoring forces remained as expected and the horizontal deformations were less than had been estimated (see Fig. 5).

– *Quay structure*

The increasing depth of harbour basins requires new structural solutions. Although quays with large construction heights are still being built according to the conventional sheet pile system (Conrad, P. 1992, Reinke, U. & Vollstedt, H.-W. & Rahtge, M. 1995, Wilde, F.-K. 1995), there is a definite trend towards new types of construction. The use of diaphragm walls can have advantages, e.g., in the case of difficult soil conditions with layers of bolders and shingle (Dücker, H.P. & Wilde, F.K. & Scheele, J. & Hoppe, H.J. 1992, Dücker, H.P. & Rodatz, W. & Timm, G. & Vellguth, L. 1996, Scherrer, P. & Baril, J.-C. & Heber, P. 1996, Van Celst, E. & Reynaers, T. & Thibaut, W. & Vellemann, B.).

During the construction of the O'Swald Quay in Hamburg Harbour, which involved a 190 m long section as a diaphragm wall quay, it was possible to gather a large number of experiences about this type of construction and about the behaviour of the total system. Fig. 6 depicts a typical cross-section.

The completed quay structure has an overall height of 27 m. The depth of the reinforced concrete diaphragm wall amounts to 25 m. It has a thickness of 1.20 m and was installed in 4.35 m wide panels. The 18 m broad quay slab on top of the diaphragm wall is founded landwards on in-situ concrete driven piles. The tie-back of the diaphragm wall consists of about 30 m long prestressed grouted anchor piles.

The soil profile in Fig. 6 is typical of the ground conditions in the Hamburg Harbour. The layers of shingle in the transition zone from Pleistocene sand and gravel layers to Tertiary silt and clay layers pose special problems for the construction of quay structures.

As a result of comprehensive parameter studies and comparative calculations for this project, it became clear that conventional methods of analysis, which often employ empirical earth pressure values and empirical additions/deductions for determining the internal forces of the quay wall, cannot be applied in the usual way. This is generally valid for deep quay walls, i.e., both for sheet pile and diaphragm systems. The structural analysis has to consider the

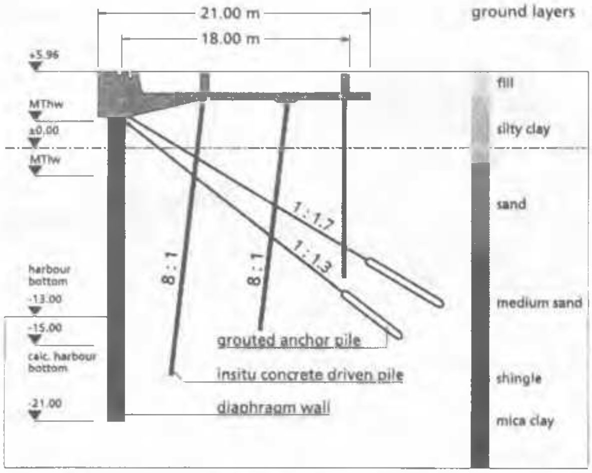


Figure 6. Cross-section of the O'Swald Quay with soil profile

total system "wall with anchoring + quay slab + pile foundation" and take account of the soil-structure interaction.

For investigating the diaphragm wall structure at O'Swald Quay, non-linear FE analyses were carried out in addition to the usual calculations. Here too, an extensive monitoring program (Rodatz, W. & Maybaum, G. 1995, Rodatz, W. & Maybaum, G. & Stahlhut, O. 1996) was carried out. The earth-pressure development on the diaphragm wall was the focus of monitoring.

Sheet pile walls are usually driven in the almost excavated harbour basin, and therefore the initial earth pressure does not occur over the full wall height. One-sided earth pressures only act on the sheet pile wall over the full height after soil-backfill.

With diaphragm walls, however, earth pressures – the size of which is determined by the fluid which supports the trench – already occur over the full height during construction. The further development of these pressures will initially be influenced by the installation of the diaphragm wall and subsequently by the construction of the anchor piles, the concrete piles and the quay slab.

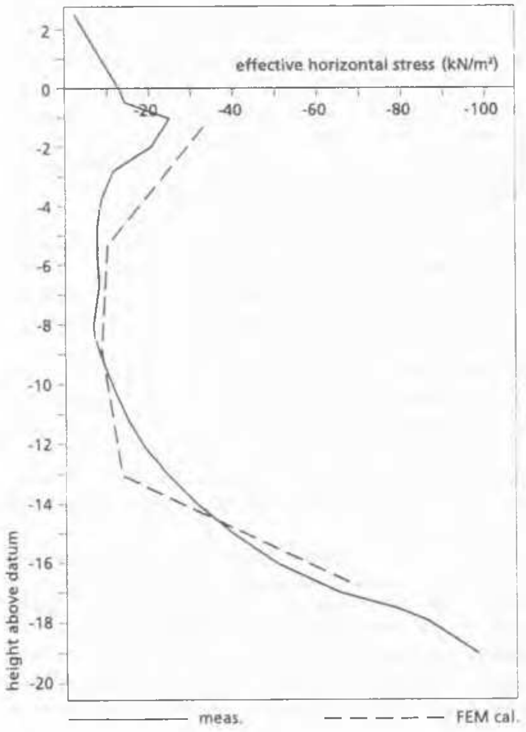


Figure 7. Measured and FEM calculated earth pressures behind the diaphragm wall for the loading case: completed excavation in conjunction with low tide

Fig. 7 shows the measured and calculated earth pressures for the relevant service condition.

The main findings for the design of quay structures with diaphragm walls can be summarized as follows:

- There is no restraint effect at the base, i.e., the required depth of embedment must be determined as for a freely supported wall.
- Despite the low degree of deformation of the rigid diaphragm wall, active earth pressure is mobilized.
- It is advisable to design the joint between the diaphragm wall and the quay slab as a rigid frame connection.

The investigations of the quay wall installed at O'Swald Quay did not indicate any overloading of individual elements of the structure.

– *Geosynthetic-reinforced soil structure*

The following example of a permanent retaining structure consists of two reinforced-soil walls stressed together with geosynthetic straps, designed and constructed for a section of the Dutton Park to port of Brisbane rail link in the Australian Province of Queensland (Lo, S.C.R. & Li, S.Q. & Gopalan, M. & Gao, Z. 1996). In view of the high surcharge due to railway traffic, the structure had to fulfill stringent requirements in terms of deformation and bearing capacity.

The width of the embankment amounts to only 6 m (see Fig. 8 a). For this reason, the two retaining walls were stressed together by geosynthetic reinforcements and formed a "tied back-to-back (TBB) wall". The reinforcing elements consist of 90 mm broad straps of high-strength polyester fibre bundles covered with polyethylene. They have a nominal strength up to 100 kN.

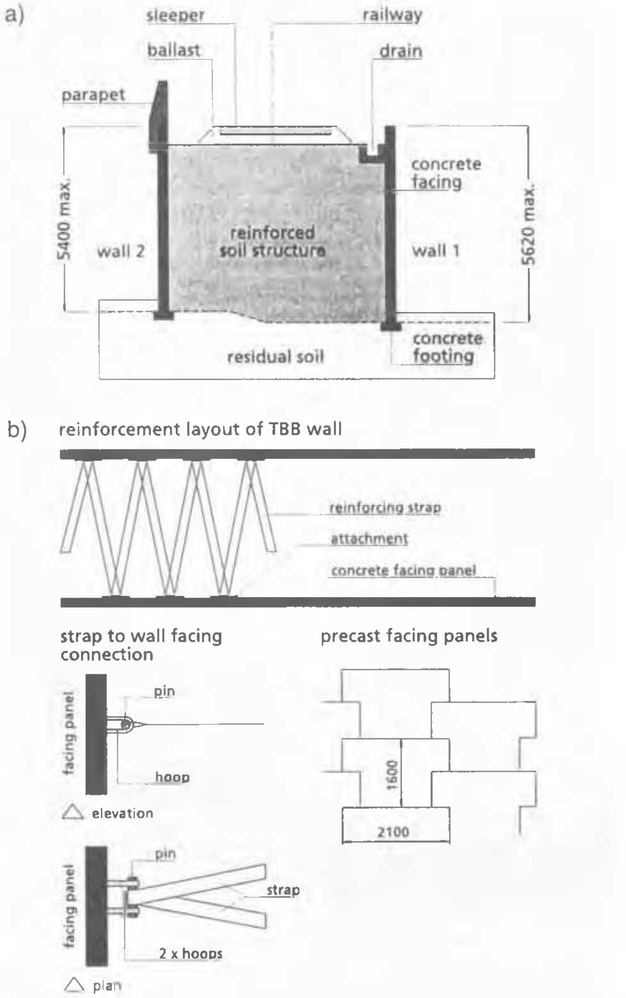


Figure 8. a) a) Cross-section of the railway embankment; b) arrangement of straps, connection of strap to concrete element (according to Lo, Li, Gopalan & Gao)

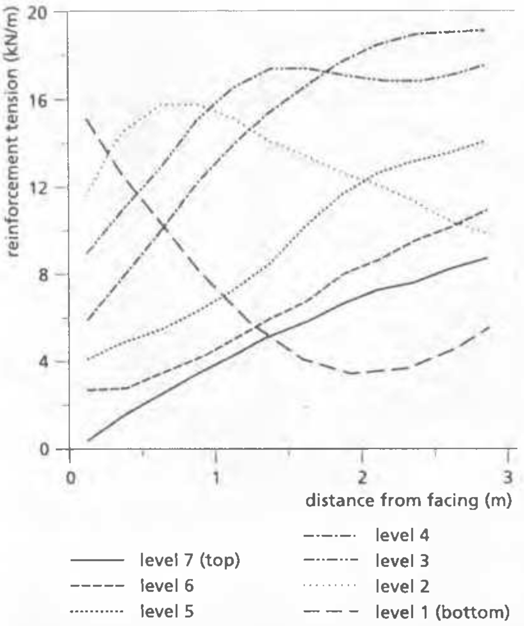


Figure 9. Distribution of longitudinal forces in the 7 reinforcement layers over the width of the embankment (according to Lo, Li, Gopalan & Gao)

The vertical distance between 2 reinforcement layers amounts to about 0.8 m. Horizontally, each strap runs in a zigzag line connecting the T-shaped, precast concrete wall elements. The arrangement of the reinforcement ties is presented in Fig. 8 b. The ties are pulled as loops through the eyes on the precast elements and tightened. This results in a more uniform tensile stress in the ties.

The bearing capacity of the ground was sufficient. Fully weathered granite was available as fill material.

The behaviour of a TBB wall differs considerably from that of a normal geosynthetic-reinforced wall. Conventional methods for checking the pullout of the reinforcement, e.g., the design method for the "Freysissol" system, cannot be used here. Initially an improved conventional design method – but with a reduction factor for the large railway load – was used for determining the maximum tensile force T_{max} in the reinforcement straps.

The results of the conventional design method were checked using non-linear FE calculations. The investigation concentrated particularly on the effects of the high surcharge. As shown in Fig. 9, the distribution of longitudinal forces along the geosynthetic straps with maximum values in the soil is typical for soil reinforcement, even though the straps are always firmly anchored to the opposite wall.

Since the normal design criteria could not be applied to the TBB structure, the input data for the FE calculations were varied as in a sensitivity analysis. The calculations were carried out with two different stiffness values for the reinforcement, two different sets of soil parameters and two models for the soil. The analysis proved that the values relevant for the design, such as the maximum tensile force T_{max}, were insensitive to the above variations. It was also shown that the design values for the straps were on the safe side with the method employed.

2.3 *Temporary structures*

– *Large opencast slope*

The duration of construction states, i.e., temporary situations, can vary considerably from a few hours to several years. Slopes in opencast mining can be in service for a fairly long time, but they still have a temporary character.

The opencast mines in the brown-coal area of North-Rhine Westphalia are among the largest in the world in terms of depth, groundwater lowering and volume. The stability of the slope systems, which reach up to 400 m, is analyzed with the most advanced methods (see section 3.1). Because of the great heights, even stable slopes will be subject to considerable deformations within the slope and the adjoining area.

Normally the deformations due to opencast mining do not cause any damage. For example, as a result of groundwater lowering, there may be widespread settlements of the ground surface up to 3 m without any damaging effect on the buildings in the vicinity (Pierschke, K. & Gudehus, G. & Hügel, H. & Niemunis, A. 1996). It can however not be excluded that the activation of tectonic faults may lead to surface damage in individual cases (see Fig. 10). It must therefore be part of safe opencast mining to keep deformations under control and avoid damaging ground movements. For this purpose, the known principles of the observational method are adhered to (see section 4.3).

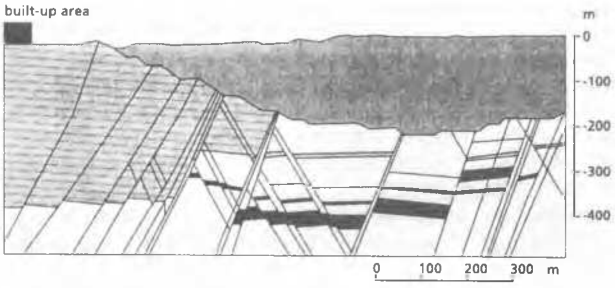


Figure 10. Geological section with an opencast rim slope

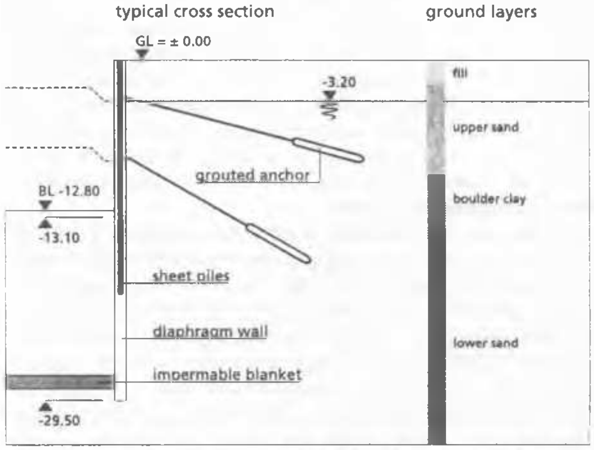


Figure 11. Typical cross-section of retaining wall with soil profile

– *Retaining wall in groundwater-saturated soil*

In the centre of large cities, one increasingly finds excavation pits with depths down to 20 m in the direct vicinity of existing buildings (Simpson, B. 1992, Nußbaumer, M. 1996). They often require watertight retaining structures with a minimum of deformation (Sänger, C. & Steinhagen, P.& Mayer, P.-M. 1994, Brem, G. & Triantafyllidis, T. 1996, Triantafyllidis, T. 1996). The retaining wall of the 13 m deep excavations (sector A) of the "Debis" site on Potsdamer Platz in Berlin is presented here as an example (Gücker, R. 1996).

Fig. 11 shows a typical cross-section of the retaining wall and the local geological situation. The ground consists largely of Tertiary and Pleistocene deposits. A particular feature in Berlin is the uniform fine sand locally embedded with marl. The groundwater level is 2 to 3 m below the ground surface.

A low level grout blanket was installed to ensure sealing and safety against uplift of the pit bottom. The surrounding watertight retaining walls consist of a slurry trench wall with inserted sheet piles (Stadler, G. & Kudella, P. 1996). The 60 cm thick single component slurry trench wall, which acts as a sealing element, was sunk down to a depth of 29.5 m, so that its foot rests below the grout blanket. Sheet piles of the required length were inserted into the cut-off wall before hardening of the slurry. During excavation, temporary grouted anchors were installed, one row also below the groundwater table. The two-fold anchoring of sheet piling proved to be an economically efficient solution (see Fig. 11).

A program to monitor wall movements and tie-back forces was an integral part of quality assurance. Even without any neighbouring buildings, monitoring of the retaining structures would have been necessary because of

- the large excavation area with sides up to 180 m, and
- the differential water pressure of up to 100 kPa acting on the retaining wall.

Inclinometer measurements were carried out to record wall movements in the specified cross-sections. The head displacements of the inclinometer tubes were measured geodetically. Load cells were used for measuring the anchor forces.

Fig. 12 shows the wall deformation for four different stages. At the height of the soft gel blanket it can clearly be seen that the base point of the cut-off wall has shifted by about 1.5 cm towards the pit. The horizontal displacements of the base points of the sheet piling are about twice as much. Excessively large deformations occurred during the final excavation. Due to an unsealed drill-hole in the retaining wall, sand was entrained into the pit. Consequently, the deflections increased to about 2 cm.

A number of other retaining structures of excavation pits in Berlin also show extensive displacements at and below the bottom of excavation (Gollub, P. & Klobe, B. 1995, Weißenbach, A. & Gollub, P. 1995). This is particularly significant in the case of tie-back structures where the wall members are relatively stiff (reinforced diaphragm walls, bored pile walls). With this type of structure, the horizontal displacements towards the pit will extend below the level of the grout blanket. The consequently widespread settlements and horizontal displacements on the ground surface have in some cases affected the serviceability of the surrounding buildings.

The causes for these widespread movements are assumed to be:

- High differential water pressure acts on the retaining structure down to the grout blanket.

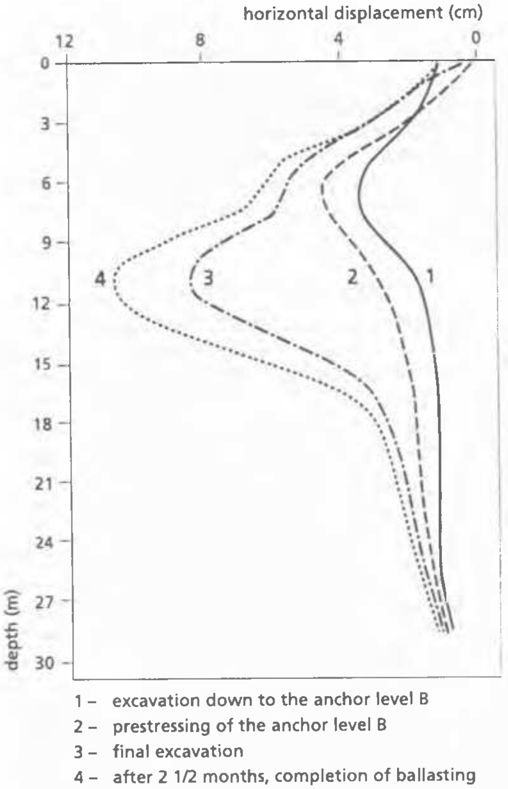


Figure 12. Horizontal wall displacements measured for a typical cross-section (according to Gücker)

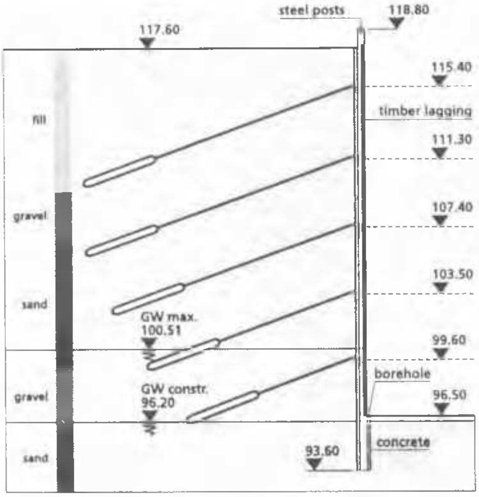


Figure 13. Cross-section of the retaining structure and geological profile

- The small remaining effective vertical stresses in the largely saturated excavation base overlying the grout blanket together with the reduced shear strength at the boundary surfaces below the grout blanket, resulting from uplift forces, and the relative softness of the gel, lead to a more flexible foot support.
- Due to the large size of the pit, appropriately large horizontal displacements occur below the excavation base

These recent observations have so far apparently not been reported elsewhere. The mechanisms for such widespread movements have not yet been sufficiently researched.

– *20 m high soldier pile wall*

The last example in this section demonstrates that 20 m high retaining walls can be constructed as anchored soldier pile walls (Sonder, H.-H. 1994). The high-speed railway link Cologne – Rhine/Main also involves a double-track railway tunnel underneath the Autobahn junction "Frankfurter Kreuz". The retaining structure consists of 20 m high soldier pile wall of 400 m length (see Fig. 13).

The following criteria must be met for using tie-back soldier pile walls as retaining structures in excavation pits of this depth:

- soil with sufficient shear strength for taking up the anchor forces,
- no special requirements on the restriction of wall deformations,
- low groundwater levels either below or only slightly above the pit bottom (lowering by open dewatering).

The ground south of Frankfurt consists of sand, coarse sand to sandy gravel (friction angle: $40° \leq \varphi \leq 45°$); the groundwater was below the pit bottom. There was only construction traffic in the vicinity of the excavation, so that increased surcharges did not have to be considered.

The retaining structure was designed as an elastically bedded beam, i.e., internal forces and moments, anchor forces and the necessary depth of embedment were calculated under the assumption of empirical earth-pressure redistributions and safety factors for the passive earth pressure at the base.

Fig. 14 shows the earth pressure distribution for the final excavation stage: trapezoidal redistribution of active earth pressure due to weight of soil and surcharge of 10 kN/m^2, rectangular additional earth pressure due to increased surcharge of 40 kN/m^2, elastic base bedding under consideration of a reduced passive earth pressure (safety factor $\eta = 3.0$). The calculation was carried out for each excavation. The required tie-back lengths were determined on the basis of the stability analysis with a deep sliding surface.

For the construction, 25 m long steel posts][320 were chosen, which were installed in 24 m deep boreholes of 90 cm dia. and embedded in concrete below the excavation base. The spacing of the posts was 2.75 m. Timber lagging of 14 cm thickness was used.

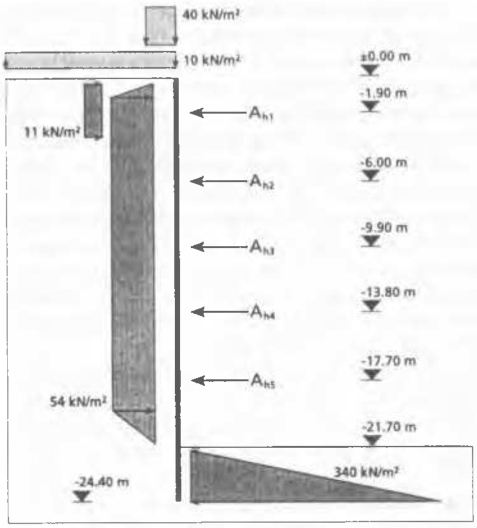


Figure 14. Earth pressure distribution for the final excavation stage

The temporary grouted anchors were prestressed with 80% of the calculated anchoring force applicable after realizing the subsequent excavation. The][profiles permitted anchoring directly into the steel posts, so that walings were not necessary.

The prestressing forces of the anchors were decisive for influencing and controlling the behaviour of this 5-fold anchored soldier pile wall. The horizontal earth resistance at the base only played a subordinate role here.

From a statical point of view, even deeper excavation pit structures with more than 5 anchor positions would certainly be possible as long as the vertical force components are supported by the concrete-embedded post foot.

3 STABILITY ANALYSES AND DESIGN METHODS

3.1 *Slope stability analysis*

The stability analysis of soil slopes is largely based on the method of limit equilibrium. The aim here is to find the relevant slip surface, i.e., failure mechanism, so as to determine the smallest factor of safety. Normally, the following safety definition (Fellenius rule) is employed:

$$F = \frac{\text{Shear strength of soil}}{\text{Shear stress required for equilibrium}}$$

Increasingly, computer-aided numerical methods are applied. The availability of high-capacity computers has motivated many scientists and engineers to develop more refined approaches and incorporate these into practicable methods of analysis, as reflected by numerous publications (Espinoza, R.D. & Bourdeau, P.L. & Muhunthan, B. 1994, Fredlund, D.G. & Zhang, Z.M. & Lam, L. 1992, Leshchinsky, D. & San, K.-C. 1994, Lo, S.-C.R. & Xu, D.-W. 1992).

There are two general categories of analysis:

Group I: Analysis with one rigid sliding body according to the method of slices

Group II: Analysis with several rigid sliding bodies on the basis of composite failure mechanisms.

The new developments of group I are generalizations of the ordinary method of slices according to Fellenius or Bishop and can be characterized as follow:

- Both the moment and the force equilibrium are fulfilled for each slice. Various suggestions have been made with regard to the necessary assumptions for the interslice side forces, e.g., (Espinoza, R.D. & Repetto, P.C. & Muhunthan, B. 1992).
- The above extended methods of slices are used for two- and three-dimensional geometries. The critical slip surface can have any form desired (monotone curvature with homogeneous soil) (Espinoza, R.D. & Bourdeau, P.L. & Muhunthan, B. 1994).
- The search for the critical slip surface, which must not necessarily be circular or spherical, represents a complex mathematical problem for which several methods have been developed, e.g., optimization methods, methods based on the calculus of variations (Leshchinsky, D. & Huang, C.-C. 1992).

In 1996 Duncan published a comprehensive state-of-the-art report on the analysis of slopes according to the limit equilibrium. He found that safety factors F, which were determined using different extended methods of slices, only differed by maximal 12%. If one considers the inaccuracies of the input data for the calculation with regard to slope geometry, soil structure, shear strengths and pore water pressures, computational accuracy with a scatter of ±6% is quite sufficient.

The safety factors of a 3D analysis are larger than those of a corresponding 2D analysis in the critical section, i.e., $F_{3D} > F_{2D}$. Fig. 15 presents an example of a critical 3D slip surface in a nonuniform slope. It was generated with the 3D-SLOPE program (Lam, L. & Fredlund, D.G. 1993).

The available computer programs are becoming increasingly more complex and user-friendly. However, the application of this type of software does not automatically result in "more accurate" slope analysis. The geotechnical knowledge of the engineer and his ability to assume realistic models are still of greatest importance.

The methods of group II are based upon kinematically possible failure mechanisms. The rigid sliding bodies normally have plane surfaces, thus only permitting translatory motions contrary to the methods of group I. Systems with more than three sliding bodies can only be dealt with numerically. Comparative calculations on slopes resulted in somewhat higher safety factors F_{2D} than those calculated according to the ordinary method of slices (Weber, K. & Gußmann, P. 1992, Navari, O. & Hartmann, R. & Lackner, R. 1997). An application example of a 3D multiple body mechanism is provided by the modelling of a 208 m high opencast rim slope at a mining "window" in the Rhineland opencast mining region (see Fig. 16).

The geological situation is characterized by a deep-seated clay bed. Since there are so far no generally valid optimization methods available for searching the critical 3D multiple body mechanism, it was necessary to stipulate, as a start model, a realistic failure mechanism for the above geological situation composed of altogether 7 sliding solids. For determining the smallest safety factor F, the failure mechanism was optimized without any difficulty (Goldscheider & Lizcano).

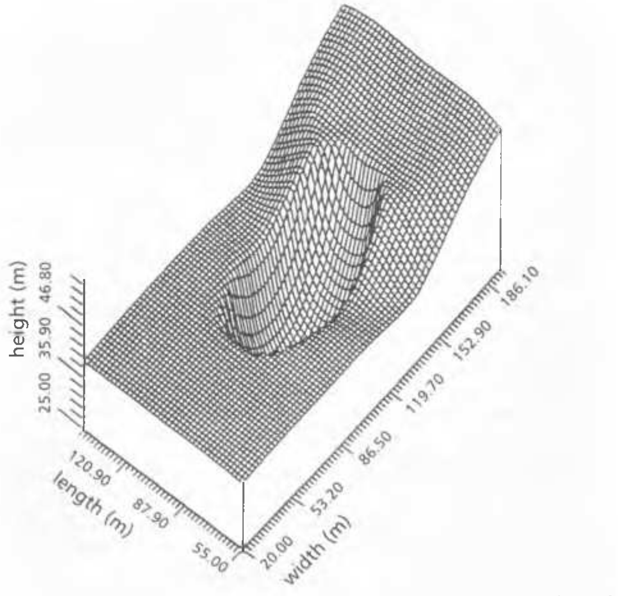


Figure 15. 3D slip surface for a nonuniform slope (according to Lam & Fredlund)

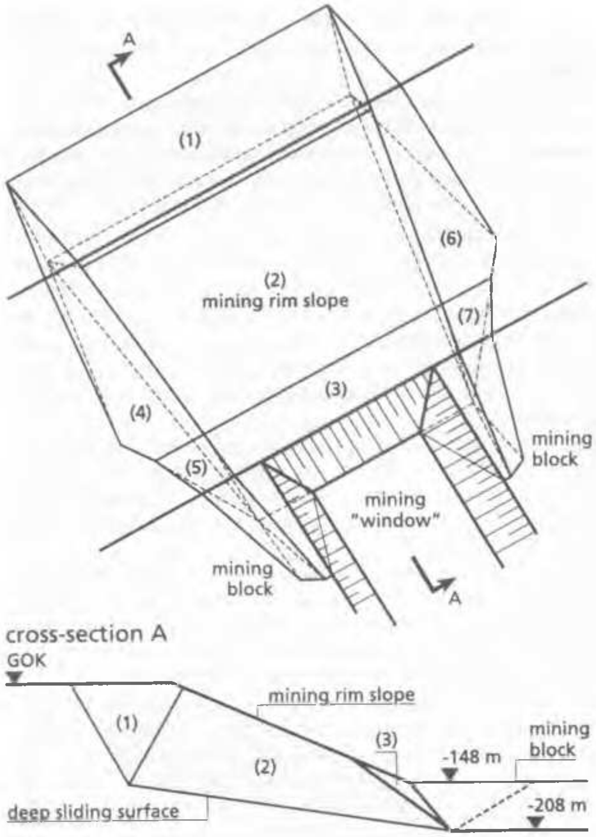


Figure 16. 3D multiple body mechanism near a mining "window" of an opencast mine (according to Goldscheider & Lizcano)

It should be pointed out that the stability analysis using finite elements also employs the Fellinius rule for the determination of F (Weber, K. & Gußmann, P. 1992). Here, F results from the stepwise reduction of the given shear parameters φ and c, which is continued in the course of the FE analysis up to the failure of the structure. It is advisable to apply this procedure only in the case of elastic ideal-plastic models for the soil, e.g., (Vermeer, P.A. & Brinkgreve, R.B.J. 1995).

3.2 *Stability analysis of composite structures*

There is a great variety of composite retaining structures, such as reinforced earth, and their behaviour is the subject of fundamental investigations both experimentally and by numerical methods (Ho, S.K. & Rowe, R.K. 1994, Wang, Y.H. & Wang, M.C. 1994, Fishman, K.L. & Desai, C.S. & Sogge, R.L. 1993, Porbaha, A. & Goodings, D.J. 1996, Huang, C.-C. & Tatsuoka, F. & Sato, Y. 1994, San, K.-C. & Matsui, T. 1995, Helwany, M.B. & Tatsuoka, F. & Tateyama, M. & Kojima, K.).

Generally, stability analyses are divided into:

- Analysis of the overall stability ("external" stability)
- Analysis of the load bearing capacity of components or structural elements ("internal" stability)

If the critical failure mechanism lies outside the reinforcement elements, the stability analysis should be regarded as a pure slope calculation according to section 3.1. It is recommended that sliding and bearing resistance also be analyzed according to classical methods in dependence on geometry and stiffness of the geotextile-reinforced or nailed soil body (Mannsbart, G. 1996). Instead of the Fellenius rule, one normally uses other safety definitions or coefficients here.

For stability analyses where the critical failure modes encompass the reinforcement elements, the methods of analysis for slopes are extended by including the forces of the structural elements into the analysis of the limit equilibrium, e.g., method of slices (Lo, S.-C.R. & Xu, D.-W. 1992), multiple body mechanisms (Michalowski, R.L. & Zhao, A. 1995, Edirisinghe, J. & Yagi, N. & Enoki, M. & Yatabe, R.). On the basis of the Fellenius rule, different partial safety factors are used for the shear parameters of the soil $\tan\varphi$ und c and for the pullout resistance of the structural elements.

The often complex pullout behaviour of structural elements for the analysis of the "internal" stability is being studied intensively, e.g., of geotextiles (Jewell, R.A. & Pedley, M.J. 1992, Ehrlich, M. & Mitchell, J.K. 1994), of polymeric geogrids (Bauer, G.E. & Shang, Q. 1993, Leshchinsky, D. & Kaliakin, V. & Bose, P. & Collin, J. 1994, Wilson-Fahmy, R.F. & Koerner, R.M. & Harpur, W.A. 1995). The investigations usually embrace the total system of the composite structure, so as to be able to consider the interaction between the structural element and the surrounding soil. For further studies of this problem see Plenary Session 5 "Soil Improvement and Reinforcement" of the XIVth ICSMFE.

On the basis of deformation-related failure criteria it is generally possible to take account of the strains in the structural elements, in the design of the necessary soil-reinforcement by using partial safety factors (San, K.-C. & Leshchinsky, D. & Matsui, T. 1994, Mannsbart, G. 1996).

In the case of rock slopes stabilized with nails or rock anchors, the stability is analyzed either with multiple body mechanisms or using the finite-element method (Wittke, W. & Erichsen, C. 1990). When applying rigid-body mechanisms, different safety definitions are employed depending on the failure mode. The FEM also makes it possible to model special rock-mechanical aspects, such as joints or seepage flow. With sophisticated models one must however expect a greater work input for creating the FE mesh and longer computing times.

3.3 *Design of retaining structures on the basis of ultimate limit states*

The structural systems of flexible excavation walls, e.g., tie-back sheet-pile walls, soldier pile walls, deep tie-back diaphragm walls, are nonlinear with a distinct interaction between soil and structure.

For such systems, it has proved useful in design practice to choose models that are not based purely on plastic ultimate limit states (Hartmann-Linden, R. & Sedlacek, G. & Schmitt, A. 1996). Generally, an elastic beam is assumed for modelling, and not the total system with the surrounding soil. For determining the necessary depths of embedment and the design of the required cross-sections, the approach uses redistributed active earth pressures and passive earth pressures, the latter being reduced by safety factors (see Fig. 14). Increased active earth pressures are employed when the design does not only aim to ensure a stable, but also a deformation-resistant structure.

With this semi-empirical method, the earth pressures and the corresponding safety definitions are so adapted to each other, that the associated soil displacements and beam deformations will be approximately compatible. The soil-structure interaction is therefore indirectly taken into account. By comparison, kinematically and statically plausible methods are theoretically better founded. They have the advantage that the redistribution of forces takes place as passive or active earth pressure depending on the direction of displacement, for which different safety definitions can be employed (Mortensen, K. 1995). They are, however, at present only valid for simple strutted or anchored systems.

The methods for verifying the overall stability or for designing the necessary anchor lengths basically correspond to the procedures described in sections 3.1 and 3.2.

The safety concept with partial safety factors, which was originally developed for steel and reinforced concrete structures, has largely been tested on linear statical systems. The attempt to apply this concept to the nonlinear statical systems of geotechnical structures and to develop a harmonized European safety concept on this basis, still produces certain inconsistencies. The subdivision of ultimate limit states (ULS) into cases A, B, and C with different associated partial safety factors, in order to overcome these contradictions, is normally not applicable to retaining structures. Especially where a marked soil-structure interaction occurs, it is not possible to distinguish between ULS case B "failure of structure or structural elements" and ULS case C "failure in the ground" (van Tol, A.F. 1994).

Experiences with transforming the harmonized safety concept into practicable standards have shown that it is necessary to introduce safety definitions that are not based upon the original partial safety concept. It has been suggested to estimate first the nominal internal forces and moments for designing the external and internal dimensions of structural elements, and subsequently apply corresponding partial safety coefficients to these internal forces and moments (Gudehus, G. & Weißenbach, A. 1996, Becker, D.E. 1996). Compared with the current EC 7 concept, this has the following advantages:

- The partial safety coefficients related to the internal forces and moments can also be derived from the previous global safety factors.
- Semi-empirical design methods can generally be adapted.
- In relation to previous design practice, over- or underdimensioning can be avoided, particularly with regard to temporary retaining structures.
- The excessive work input for calculation is reduced.

3.4 *Stability analysis of excavations with complex geometries*

In the case of projecting corners of retaining walls it is advisable to turn tie-backs horizontally parallel to the bisector of the corner, so as to avoid crossing. This type of turned tie-back was used for the "Hofgarten" excavation in Berlin for stabilizing a projecting corner (Sänger, C. & Hähnig, F. 1994).

The excavation required a watertight retaining structure subjected to only small deformations because of adjoining buildings and a final excavation depth of 17 m. A reinforced concrete diaphragm wall was chosen with 5-fold tie-backs in the area of the projecting corner. Analogous to the "Debis" excavation, a soft gel grout blanket was installed to ensure safety against uplift, as a horizontal sealing element of the pit. The reinforced concrete diaphragm wall was installed down to a depth of 32 m.

In the area of the projecting corner appropriate anchor lengths could only be determined under consideration of 3D effects in the context of which various failure modes were investigated. Eventually, a corresponding sliding body with the following assumptions was used to investigate the stability of the system for each of the 5 anchor rows (see Fig. 17):

- The lateral slip surfaces of the sliding body are vertical in the outer axes of the tie-backs
- The vertical slip surface at the back of the sliding body is plane and runs orthogonally to the tie-backs.
- Both the lower slip surfaces are determined by the intersection of the tie-back row and the slip surface at the back and run horizontally along the diaphragms walls.

With these assumptions, the range of possible sliding bodies is restricted to the variations of parameters H and L. The critical sliding body results from the conventional safety definition for the total of the anchors forces that can be taken up in the tie-back row analysed ($\eta_A = 1.5$).

This 3D stability analysis made it possible to implement a very economic and technically elegant solution for the projecting corner of the retaining wall.

Assuming three-dimensional failure modes, also leads to lower earth pressures at projecting corners. Hock-Berghaus 1995 proposes a simple calculation method for active three-dimensional earth pressure. He demonstrates that the load on a projecting corner is smaller than in the case of a comparable plane earth-pressure approach, which is of course favourable for the design of corner elements.

Walz 1994 provides practical recommendations for earth-pressure reduction in re-entering excavation corners on the basis of theories or methods of analysis for three-dimensional earth pressure.

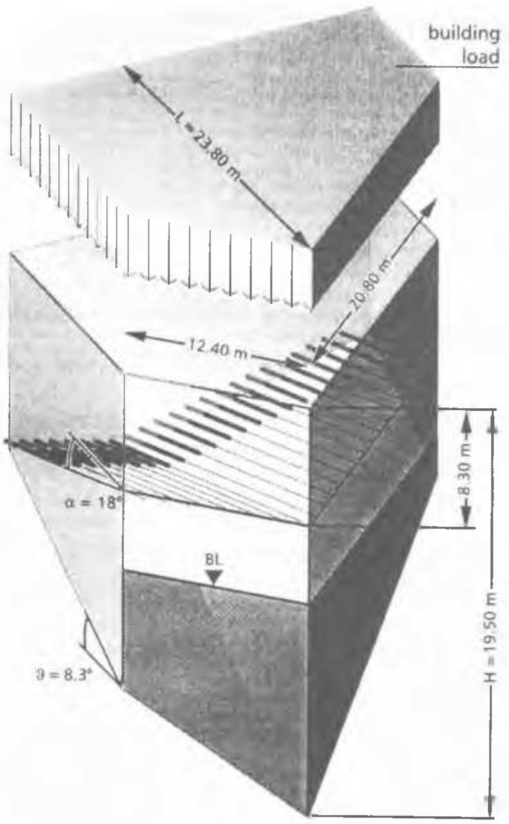


Figure 17. Critical 3D sliding body for the 4th anchor row depth 8.30 m ($H = 19.50$ m, $L = 23.80$ m)

3.5 *Dynamic analysis*

The selected FEM review of a sheet-pile quay structure destroyed by seismic action (Iai, S. & Kameoka, T. 1993) is a example demonstrating the efficiency of dynamic stability analyses of retaining structures using numerical methods, see also, e.g., (Siller, T.J. & Dolly, M.O. 1992, Yogendrakumar, M. & Bathurst, R.J. & Finn, W.D.L. 1992).

The Nihonkai Chubu earthquake in 1983 destroyed part of the quay walls in the harbour of the Japanese town of Akita. A section of the quay wall is shown in Fig. 18. The deformed and partly destroyed structure after the earthquake is drawn in dashed lines. The horizontal displacements at the top of the sheet-pile wall amounted to 1.1-1.8 m; behind the sheet-pile wall there were settlements up to 1.4 m.

Damage only occurred in the quay-wall sections with sand backfill. The made-up sand was less dense than the natural sandy soil

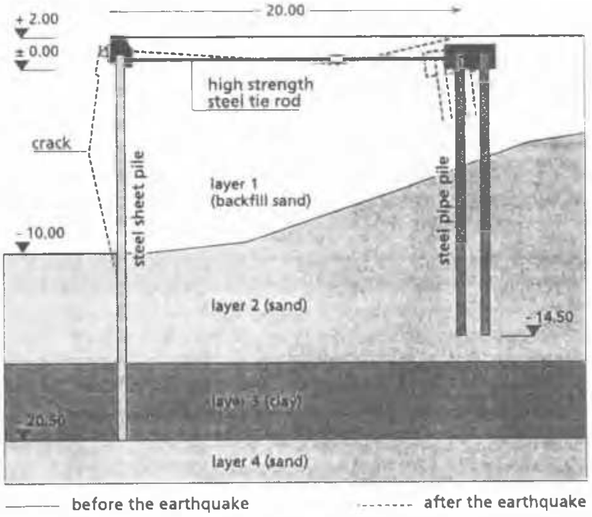


Figure 18. Quay cross-section before and after the earthquake (according to Iai & Kameoka)

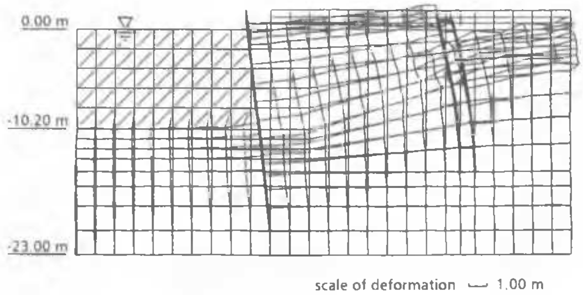


Figure 19. Calculated displacement of quay wall damaged by earthquake impact of 30 sec (according to Iai & Kameoka)

in the other quay-wall sections. The seismic impact caused liquefaction of the loose backfill, which finally destroyed the structure.

The elastoplastic model for sand, consisting of a multiple shear mechanism, used in the dynamic FE analysis was specifically developed for cyclic loading. The 10 model parameters were determined on the basis of SPT-values and the results of undrained cyclic triaxial tests for different degrees of density.

Before start of the dynamic FE analysis, a statical calculation was carried out in order to produce the initial state under drained conditions, but without considering the load history during installation of the structure. Fig. 19 shows the corresponding two-dimensional FE model. The dynamic analysis assumed that the supports at the edges and at the bottom are being viscously damped.

The dynamic analysis was carried out under undrained conditions with effective stresses. The seismograms recorded in the port of Akita served as a basis for the dynamic loading in the FE analysis.

Fig. 19 shows the true dimensions of the calculated deformations of the sheet-pile quay wall and the surrounding soil due to a 30 sec earthquake impact. The calculated horizontal displacement at the top of the sheet-pile wall amounted to about 1.3 m and thus remained within the observed range of values. However, the calculated settlement of 0.7 m behind the sheet-pile wall was less than the observed value.

The analysis assumed the sheet-pile wall to be linear-elastic without yield point, so that the development of a plastic hinge in the wall was not possible. However, the location of the calculated maximum bending in the wall lies more or less at the height of the observed failure and the stress exceeds the yield point of steel by 60% resulting in a failure of the wall. Liquefaction of the sand could be shown by means of stress paths of selected soil elements.

An analogous dynamic FE analysis was carried out for a typical cross-section of the undamaged quay-wall area in dense sandy soil. Here too, the results of the analysis are in agreement with the observations: much smaller displacements at the top of the sheet-pile wall, where the steel yield point was not exceeded.

4 SERVICEABILITY ANALYSIS

4.1 *Serviceability of retaining structures*

The term "serviceability" covers a wide range and therefore often leads to misunderstandings. In the case of retaining structures, serviceability analysis usually refers to tolerable deformation values. There are however also other serviceability requirements, e.g., the tightness requirements for "watertight" retaining structures of excavations.

Contrary to many other structures, the serviceability of retaining structures can usually not be verified solely analytically on the basis of specified limit state conditions (von Wolffersdorff, P.-A. 1996). The main reasons are:

- There are no generally applicable serviceability requirements for retaining structures. They are normally laid down individually for each project.
- Deformation calculations for retaining structures are still not reliable enough, especially in the case of flexible systems with a marked soil-structure interaction.

With massive retaining walls, it is usual to include the serviceability (e.g., limitations on tilt) in the safety factors of the stability analyses for the design of the structure (e.g., ground-failure verification). The deformations are not checked separately.

The deformation analysis of many flexible composite structures and retaining structures of deep excavations has to be based on the total system and must take account of the soil-structure interaction. In such cases, it is essential to use numerical methods of analysis, e.g., the finite-element method, which permits modelling of the retaining structure including surrounding areas within the framework of the continuum theory. Simpler models, e.g., the bedded linear-elastic beam, allow estimating the relative deflections of a sheet wall, but not the absolute displacements of the total system, including surrounding soils and adjacent buildings.

High-instrumented monitoring of the above mentioned structures with marked soil-structure interaction is indispensable during construction (Ulrichs, K.R. 1994). The serviceability of these structures is normally verified according to the principles of the observational method (von Wolffersdorff, P.-A. & Mayer, P.M. 1996).

4.2 *Deformation predictions for retaining structures using numerical methods*

FE analyses are nowadays a usual tool for determining deformations and internal forces and moments of retaining structures. The following are examples of their wide range of application:

- Earth pressure analysis of angular retaining walls: (Goh, A.T.C. 1993)
- Deformation analysis of quay structures: (Rodatz, W. & Maybaum, G. & Stahlhut, O. 1996, Van Celst, E. & Reynaers, T. & Thibaut, W. & Vellemann, B., Zadroga, B. & Dembicki, E. 1996)
- Deformation analysis of geosynthetic reinforced soil walls: (Ho, S.K. & Rowe, R.K. 1994, Ling, H.I. & Tatsuoka, F. & Tateyama, M. 1995, Helwany, M.B. & Tatsuoka, F. & Tateyama, M. & Kojima, K.)
- Deformation analysis of strutted and anchored retaining structures (Whittle, A.J. & Hashash, Y.M. & Whitman, R.V. 1993, Ng, C.W.W. & Lings, M.L. 1995, Young, D.K. & Ho, E.W.L. 1994)

Most of the above applications use commercial FE programs with the following nonlinear models for soils: elastic, ideal-plastic model with yield condition according to Mohr-Coulomb; hyperbolic pseudo-elastic model according to Duncan/Chang. FE analyses with more sophisticated constitutive models for soils are still the exception for project-specific deformation predictions or back-analyses, e.g, (Hsi, J.P. & Small, J.C. 1993). By comparison, numerical methods, developed from bedding, were more often employed in practice for the deformation analysis of retaining structures (Simpson, B. 1992, Vaziri, H.H. & Troughton, V.M. 1992, Vaziri, H.H. 1996, Triantafyllidis, T. 1996).

It is sometimes not realized that calculated deformation predictions also play a central role for the application of the observational method, since

- they are essential for understanding the deformation behaviour of the total system "retaining structure with soil and possible neighbouring buildings", and
- their results permit systematic and safer planning of measuring programs.

The application of modern numerical methods of analysis has been an important step forward permitting more reliable deformation predictions for retaining structures (Nova, R. & Boulon, M. & Gens, A. 1995). Nevertheless, there are certain limitations here as demonstrated by a German-Dutch project. Within the framework of a large-scale field test with a single-strutted, freely supported sheet-pile wall in sandy soil, various geotechnical experts had been asked to calculate and predict the earth pressures on the sheet-pile wall, the bending moments, deflections and displacements of the sheet-pile wall, the settlement of the adjoining ground surface as well as the strut forces (von Wolffersdorff, P.-A. 1994).

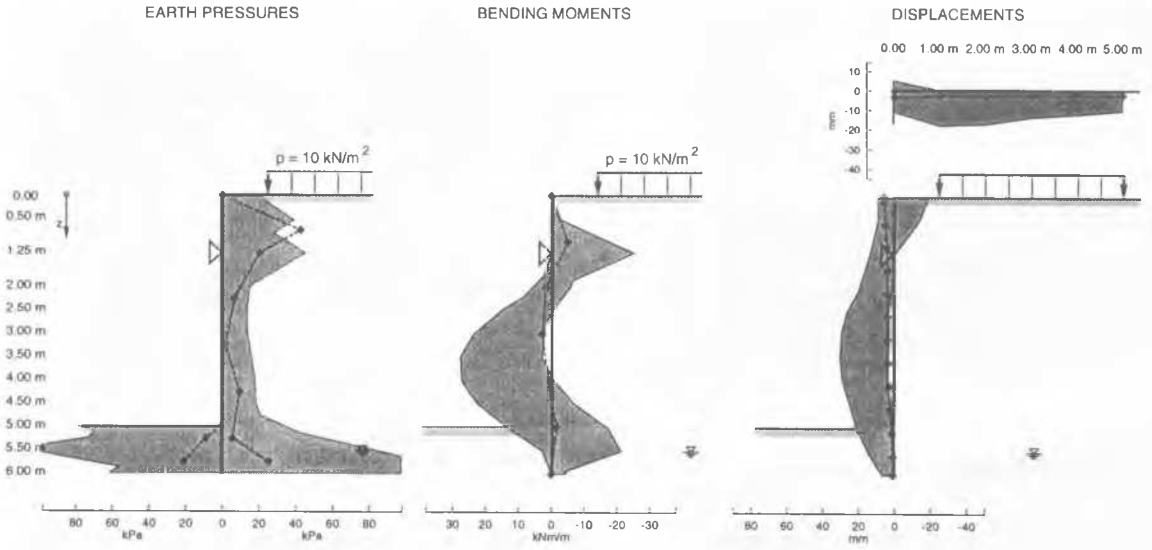


Figure 20. Range of 43 predictions (grey) for the sheet-pile wall field test compared with measured earth pressures, bending moments and deformations

Altogether 43 predictions were received from 14 countries, almost all of them in Western Europe. The predicting experts had been asked to select their own input data for their preferred method of analysis on the basis of extensive soil data and test results handed out to them. Generally, the following methods of analysis were applied:

- Finite-element method with different models for the soil and different interface models
- Bedding module method with different linear and nonlinear springs, in parts also coupled

The grey area in Fig. 20 indicates the range of earth pressures, bending moments and deformations predicted for the service state (completed excavation and water surcharge). Comparison with the test results reveals that the predictions were often too high for all three values, i.e. earth pressure, bending moment and displacement.

This project confirmed that it is currently often not possible to predict the behaviour of retaining structures with sufficient accuracy and reliability, not even with the aid of modern numerical methods.

4.3 *Application of the observational method for the control of soil deformations due to opencast mining*

To enable better definition of the extent of deformations in large slope systems and in the adjoining area of the opencast mines described under 2.3, a special method of prediction has been developed in addition to the usual monitoring (Pierschke, K. & Gudehus, G. & Hügel, H. & Niemunis, A. 1996).

The unusually large dimensions of cuts in opencast mining result in very complicated models, despite generalization of the geological situation (see Fig. 10). It is necessary here to separate the calculation of settlement due to groundwater lowering and the calculation of deformations due to excavation.

Settlement due to groundwater lowering was calculated on the basis of a one-dimensional model (soil strata with representative profile). Constitutive models implemented in commercial geotechnical FE programs are not sufficient for determining the time-dependent behaviour of the clay interstratifications. The new visco-hypoplastic model used here contains the void ratio as a second state variable in addition to granular stress (Hügel, H.M. 1995). Fig. 21 shows the corresponding calculation of time-settlement curves compared with the measured settlements of the ground surface.

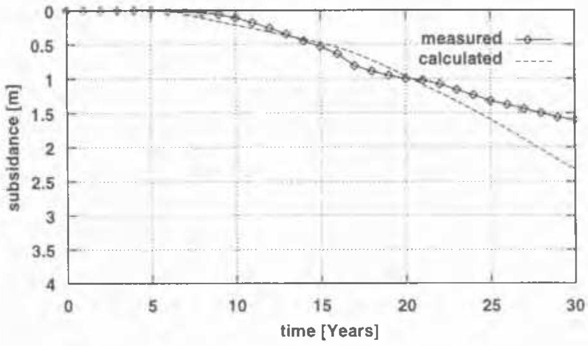


Figure 21. Measured and calculated time-settlement curves of the ground surface due to groundwater lowering

A realistic 2D strata model was defined for calculating deformations due to excavation. This model incorporates the relevant strata profiles and faults. Thinner layers and smaller fault zones are summarized in packages with mean state variables and material parameters (see Fig. 22 a). The FE mesh used for the numerical analysis is depicted in Fig. 22 b.

Fig. 22 c shows the calculated displacements at the top of the slope. In accordance with observations, the excavation bottom lifts upward and the slope edge is being displaced toward the toe of the slope. The adjoining area is subjected to gradually decreasing distortions and settlements, with particular concentrations at the faults (not included in Fig. 22 a).

The improved deformation predictions allow a more systematic control of ground deformations in opencast mining. Suitably differentiated monitoring has long been of great importance in the opencast brown-coal mines of North-Rhine Westphalia.

4.4 *Application of the observational method for complex excavation-building systems*

The example of the "GSW excavation" in Berlin – a pilot project with very extensive monitoring – serves to explain special aspects of the serviceability analysis in the case of complex excavation-building systems (Sänger, C. 1994, Sänger, C. & Hähnig, F. 1994).

The section shown in Fig. 23 gives an overview of the typical geological situation (see section 2.3) and the two excavation pits. Here, a maximal 20 m deep, triple-strutted excavation pit with a size of 57 m x 19 m was produced within a large shallow excavation measuring 119 m x 75 m. The watertight retaining structure consisted of a slurry cut-off wall with inserted sheet-piling as described in section 2.3. The 60 cm thick single component slurry cut-off wall is embedded at a depth of about 40 m into the sufficiently dense brown-coal horizon.

The existing 16-storey building surrounded by the shallow excavation, which was assumed to be on a rigid foundation, is located at a distance of about 18 m from the deep pit. The maximum

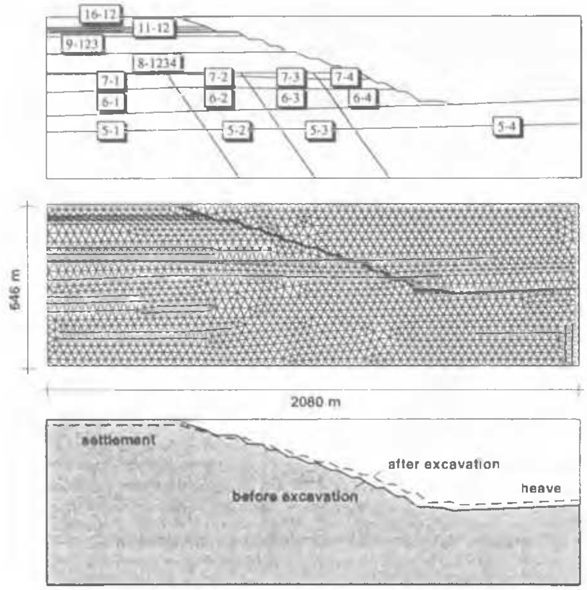


Figure 22. a) Geometrical strata model for calculating deformations with numbered layer packages b) FE mesh of the opencast mining cut c) calculated slope deformations in the opencast mine due to excavation (graphically magnified 15x)

permissible settlement during construction, when several different activities were being carried out, often at the same time, was laid down as 40 mm, while the tilt of the building was not allowed to exceed 1/1500. For construction of the deep excavation, more stringent serviceability requirements were made, because, due to the other activities, settlement of the high-rise building was also expected. In order to meet these serviceability requirements, a deformation-resistant retaining structure with three layers of bracing was installed. A special feature of this design was that parts of the temporary retaining structure were later integrated into the new building as permanent components. The struts of the two upper rows consist of precast reinforced concrete beams in conjunction with cast-in-place walings.

In order to be always aware of the impacts on the high-rise building, particularly due to the 20 m deep excavation, the high-instrumented program was designed to provide continuous monitoring of the

- horizontal and vertical displacements of the retaining structure of the deep excavation,
- horizontal and vertical ground movements in the area of the high-rise building and between the high-rise building and the deep excavation,
- heave within the excavation,

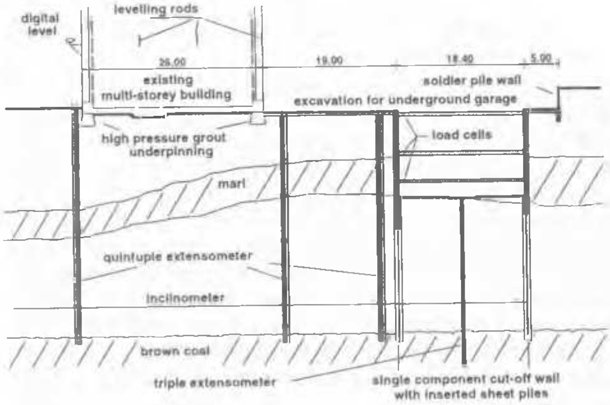
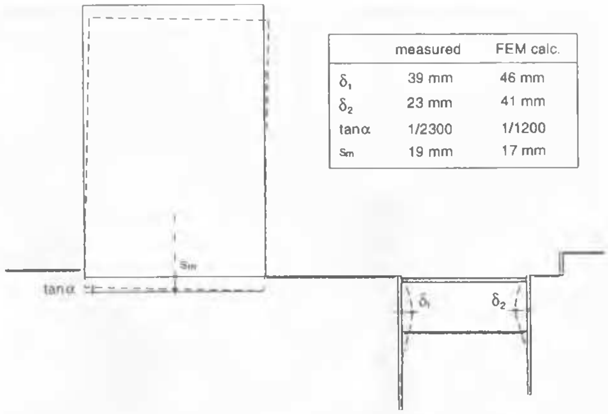


Figure 23. Cross-section of GSW excavation with geological situation and measuring installations



	measured	FEM calc.
δ_1	39 mm	46 mm
δ_2	23 mm	41 mm
tanα	1/2300	1/1200
s_m	19 mm	17 mm

Figure 24. Measured and FEM predicted settlement and tilt of the high-rise building during excavation before installation of the second level of struts

- settlements of the high-rise building,
- forces in the three levels of struts,
- ground water levels.

The movements of the high-rise building, which were particularly important, were recorded with a geodetic on-line measuring system. The measuring installations employed are presented in Fig. 23. Since all the measured data, apart from the inclinometer measurements, were recorded on-line, it was possible – with the aid of an electronic data acquisition and evaluation system – to carry out permanent monitoring of the major values in order to be able to respond immediately to a critical situation (Sänger, C. & Mayer, P.-M., 1996).

One of the main objectives of the deformation predictions was to determine the values due to the deep excavation with respect to settlement and tilt of the neighbouring high-rise building. For the linear-elastic, ideal-plastic FE analysis it was necessary to perform 2D modelling of the total system – encompassing shallow excavation, deep excavation and high-rise building – and of the geological situation down to the brown-coal horizon. The FE calculations also took account of the different stages of excavation in the deep pit.

Fig. 24 shows the FEM predicted settlement and tilt of the high-rise building due to the deep excavation before installation of the second level of struts as well as the corresponding measured data. The good agreement between the calculated and experimental results proves that it is possible to describe the deformation processes of complex excavation-structure systems. However, monitoring during construction and systematic application of the observational method must be regarded as indispensable here (von Wolffersdorff, P.-A. & Mayer, P.M. 1996).

4.5 *Three-dimensional deformation problems in the case of deep excavations*

The vibrating-in of steel sheet piles prior to excavating causes considerable stress changes and compactions in the adjoining ground. This can have an important influence on the behaviour of the wall (von Wolffersdorff, P.-A. 1994). During the installation of diaphragm walls, by comparison, stress redistributions will occur in connection with the unavoidable contraction of trench panels. The study of installation effects both for vibrated and excavated walls requires three-dimensional approaches.

At present, it is not yet possible to determine and predict stresses caused by vibrating-in of sheet-piles.

By contrast, several studies have been carried out about the installation effects of diaphragm walls, e.g., (De Moor, E.K. 1994, Ng, C.W.W. & Lings, M.L. & Simpson, B. & Nash, D.F.T. 1995, Mayer, P.M., & Kudella, P. 1996). The authors preferred the following simplified method with two separate nonlinear 2D finite-element analyses:

- Horizontal plan analysis (HPA):
  HPA modelling is used to calculate horizontal soil displace-

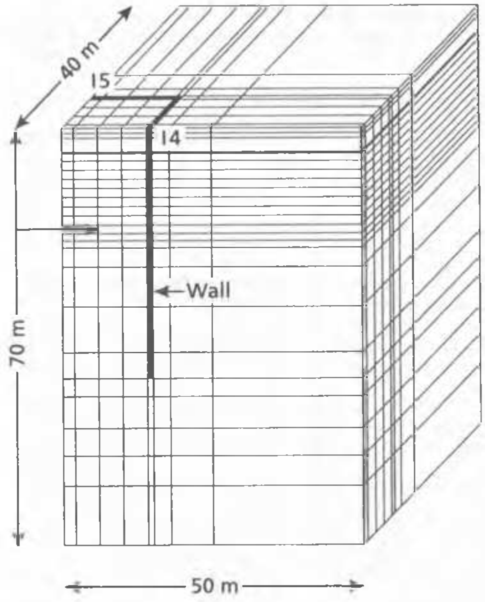


Figure 25. Finite-element mesh used for the excavation corner (according to Ou, Chiou & Wu)

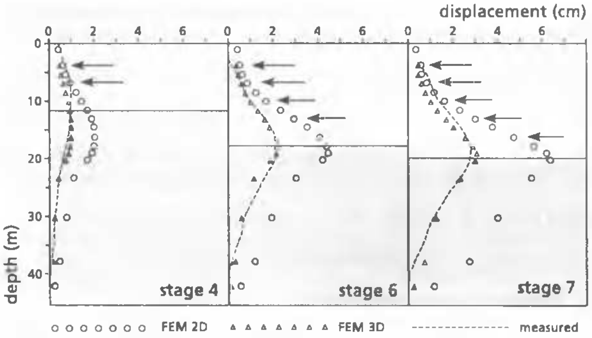


Figure 26. Measured and FEM predicted wall displacements in the section of inclinometer I4 close to the excavation corner (according to Ou, Chiou & Wu)

ments due to diaphragm wall installation at different levels. It is possible to model several neighbouring panels in succession and to increase the stiffnesses of the panels depending on the degree of hardening of the diaphragm wall material.

Vertical section analysis (VSA):

In the VSA model – which includes, the total retaining structure and the adjacent area for a typical cross-section – the mean displacements resulting from HPA are introduced as horizontal initial contractions of the wall panels.

The following interesting example of a 20 m deep diaphragm wall with a re-entering corner demonstrates the necessity for three-dimensional FE models to describe the deformation behaviour of retaining structures in corner regions (Ou, C.-Y. & Chiou, D.-C. & Wu, T.-S. 1996). The 1.10 m thick diaphragm wall of the 20 m deep excavation for the Hai-Hua Building in Taipei was installed to a depth of 42 m. The excavation was implemented according to the top-down method of construction with the following stages:

- Excavation down to 1.60 m below ground level
- Installation of the reinforced concrete slab for the ground floor and excavation down to 5.40 m
- Installation of the reinforced concrete slab for the 1st basement at a depth of 3.80 m and excavation down to 8.55 m
- Installation of the reinforced concrete slab for the 2nd basement at a depth of 6.95 m and excavation down to 11.70 m
- Installation of the reinforced concrete slab for the 3rd basement at a depth of 10.10 m and excavation down to 14.60 m
- Installation of the reinforced concrete slab for the 4th basement at a depth of 13.25 m and excavation down to 17.90 m
- Installation of the reinforced concrete slab for the 5th basement at a depth of 16.40 m and excavation down to 20.30 m

The ground largely consists of interstratified silty sand and silty clay. There is a thick gravel layer below a depth of 50 m.

The preparations included extensive studies to generate suitable 3D finite-element meshes with optimum reduction of the degrees of freedom. Fig. 25 shows the three-dimensional FE mesh used for the corner. The nonlinear pseudo-elastic model according to Duncan/Chang was employed for the soil. The concrete diaphragm wall was modelled with linear-elastic 3D elements. The concrete slabs were regarded as rigid supports.

The measured and FEM predicted wall displacements in the section of inclinometer I4 close to the excavation corner are represented in Fig. 26. The wall displacements of the 3D FE analysis are in good agreement with the results of the inclinometer measurements. By contrast, the wall displacements calculated by corresponding 2D FE modelling were about twice as high. These differences are solely due to inappropriate application of 2D modelling, because the wall displacements of the 2D model are in good agreement with the data of the inclinometers arranged in the middle of the about 80 m long excavation walls.

5 CONCLUSION

The above examples have demonstrated that measurements on large retaining structures and in adjacent areas are normally an integral part of design and implementation.

Even with the most sophisticated methods of numerical analysis, the deformation behaviour of retaining structures can only be predicted up to a certain point, especially where deformations due to the installation of the wall and the tie-backs cannot be neglected. Deformations as a result of cyclic or dynamic loading, e.g., the vibrating-in of steel sheet piles or turning movements during anchor boring, cannot be determined with the models available today

Despite great advances in computer technology and the development of efficient software, the work input for 3D deformation analysis is still too high for practical applications.

The tools for the realistic modelling of systems with a marked soil-structure interaction still need to be improved. This also applies to the modelling of certain elements, such as grouted anchor bodies, nails and geotextiles.

Pressure grouting which is increasingly used in deep excavations to stabilize the bottom or to act as sealing elements, cannot be adequately modelled with the current methods of analysis.

The application of modern numerical methods for stability and deformation analysis is becoming easier due to more complex and user-friendly computer programs. However, these aids will only be useful if the engineer dealing with retaining structures or excavated slopes possesses the necessary geotechnical knowledge, is familiar with the mechanical models on which the computer programs are based and is able to judge their capability.

ACKNOWLEDGEMENTS

The authors are grateful to Dipl.-Ing. C. Sänger for his valuable comments on this report and would also like to thank Ms. C. Hearne and Mr. M. Paul for their editorial support.

REFERENCES

Abramento, M. & Whittle, A.J. 1995. Analysis of pullout tests for planar reinforcements in soil. *J. Geotech. Engrg.*, ASCE, 121 (6): 476-485.

Abramento, M. & Whittle, A.J. 1995. Experimental evaluation of pullout analyses for planar reinforcements. *J. Geotech. Engrg.*, ASCE, 121 (6): 486-492.

Bauduin, C. & Maertens, L. 1996. Deterministic, semi-probabilistic and F.E.M. approach of gravity quay walls. *Proc. 11th Int. Harbour Congr.*, Antwerpen 1996: 161-172.

Bauer, G.E. & Shang, Q. 1993. Pullout resistance of large geogrid specimens in site specific soils. *Geotech. Engrg.*, J. S.E. Asian Geotech. Soc., 24 (1): 17-38.

Becker, D.E. 1996. Eighteens Canadian Geotechnical Colloquium: limit states design for foundations. part I. an overview of the foundation design process. part II. development for the national building code of Canada. *Can. Geotech. J.* 33 (7): 956-1007.

Belabed, L. 1995. Standsicherheitsuntersuchung zweifach verankerter Stützwände mit der kinematischen Starrkörpermethode. *Geotechnik* 18 (1): 171-174.

Bergado, D.T.& Lo, K.-H. & Chai, J.-C. & Shivashankar,R. & Alfaro, M.C. & Anderson, L.R. 1992. Pullout tests using steel grid reinforcements with low-quality backfill. *J. Geotech. Engrg.*, ASCE, 118 (7): 1047-1063.

Besler, D. 1994. Einfluß von Temperaturerhöhungen auf die Tragfähigkeit von Baugrubensteifen. *Bautechnik* 71 (9): 582-590.

Brandl, H. 1992. Konstruktive Hangsicherungen. *Grundbau-Taschenbuch*, Vierte Aufl. Teil 3, Herausg. Smoltczyk, Ernst & Sohn, Verl. Archit. u. techn. Wissensch., Berlin 1992: 533-652.

Brem, G. & Triantafyllidis, T. 1996. Tiefe Baugruben mit Schlitzwänden und Unterwasserbetonsohlen am Potsdamer Platz in Berlin. *Vortr. Baugrundtagung 1996 Berlin*: 55-76.

Budhu, M. & Gobin, R. 1995. Seepage-induced slope failures on sandbars in Grand Canyon. *J. Geotech. Engrg.*, ASCE, 121 (8): 601-609.

Christian, J.T. & Ladd, C.C. & Baecher, G.B. 1994. Reliability applied to slope stability analysis. *J. Geotech. Engrg.*, ASCE, 120 (12): 2180-2207.

Clayton, C.R.I. & Symons, I.F. 1992. The pressure of compacted fill on retaining walls. *Géotechnique* 42 (1): 127-130.

Conrad, P. 1992. Kaimauerkonstruktionen in Spundwandbauweise aus Norddeutschland. *SMM'92 Conf. SEA 2000 Hamburg*: 29-36.

Cools, J.P.A.E. & Decker, J. & Post, W.J. 1995. Hydraulic resistance of steel sheet pile joints. *J. Geotech. Engrg.*, ASCE, 121 (2): 105-110.

Day, P.W. 1994. Factors influencing the movement of retaining structures. *Proc. XIII. ICSMFE*, Vol 2, Neu Dehli 1994: 109-114.

De Man, M.& Thibaut, W. & Dedeyne, R. & De Poorter, P. & Bouquet, P. & Maertens, L. 1996. Some specific design aspects in quay wall renovation projects. *Proc. 11th Int. Harbour Congr*, Antwerpen 1996: 173-183.

De Moor, E.K. 1994. An analysis of bored pile/diaphragm wall installation effects. *Géotechnique* 44 (1): 341-347.

DeNatale, J.S. & Ibarra-Encinas, G.A. 1992. Total stress analysis of cantilever sheetpiling in layered clay. *J. Geotech. Engrg.*, ASCE, 118 (7): 1064-1082.

Dücker, H.P. & Miller, C. 1996. Comparison of technical and cost factors regarding different construction designs for the building of a container berth. *Proc. 11th Int. Harbour Congr.*, Antwerpen 1996: 185-189.

Dücker, H.P. & Rodatz, W. & Timm, G. & Vellguth, L. 1996. Tiefe Kaimauer in Schlitzbauweise. *Bautechnik* 73 (12): 811-820.

Dücker, H.P. & Wilde, F.K. & Scheele, J. & Hoppe, H.J. 1992. Neue Querschnittsgestaltung und Bauverfahren von Kaianlagen, erläutert am Beispiel des O'Swaldkai-Terminals. *Geotechnik* 15 (1): 183-189.

Duncan, J.M. 1996. State of the art: limit equilibrium and finite-element analysis of slopes. *J. Geotech. Engrg.*, ASCE, 122 (7): 577-596.

Edirisinghe, J. & Yagi, N. & Enoki, M. & Yatabe, R.& Ohashi, G. 1996. Reinforcing mechanisms and a stability analysis method for reinforced earth structures. *Soils and Foundations*, Jap. Geot. Soc., 36 (3): 21-30.

Ehrlich, M. & Mitchell, J.K. 1994. Working stress design method for reinforced soil walls. *J. Geotech. Engrg.*, ASCE, 120 (4): 625-645.

Elgamal, A-W. & Alampalli, S. & van Laak, P. 1996. Forced vibration of full-scale wall-backfill system. *J. Geotech. Engrg.*, ASCE, 122 (10): 849-858.

Espinoza, R.D. & Bourdeau, P.L. & Muhunthan, B. 1994. Unified formulation for analysis of slopes with general slip surface. *J. Geotech. Engrg.*, ASCE, 120 (7): 1185-1204.

Espinoza, R.D. & Repetto, P.C. & Muhunthan, B. 1992. General framework for stability analysis of slopes. *Géotechnique* 42 (1): 603-615.

Feddersen, I. 1994. Beitrag zum Aufbruch des Verankerungsbodens bei Verankerungen mit Ankerplatten und Ankerwänden. *Festschrift Prof. Weißenbach*, Univ. Dortmund, Heft 21: 53-75.

Fishman, K.L. & Desai, C.S. & Sogge, R.L. 1993. Field behavior of instrumented geogrid soil reinforced wall. *J. Geotech. Engrg.*, ASCE, 119 (8): 1293-1307.

Fredlund, D.G. & Zhang, Z.M. & Lam, L. 1992. Effect of the axis of moment equilibrium in slope stability analysis. *Can. Geotech. J.* 29 (6): 456-465.

Fröhlich, H. 1993. Beitrag zum Standsicherheitsnachweis von Stützkörpern vor Schmaldichtwänden. *Bauingenieur* 69: 463-469.

Fujita, K. 1994. Soft ground tunnelling and buried structures. *Proc. XIII. ICSMFE*, Vol 2, Neu Dehli 1994: 89-108.

Gaffran, P. & Sego, D.C. & Peterson, A.E. 1992. Performance of an anchored sheet pile wall on the CN Rail line, Boston Bar, British Columbia. *Can. Geotech. J.* 29 (1): 31-38.

Gattermann, J. & Horst, M. & Rodatz, W. 1996. Meßtechnische Überwachung eines verformungsarmen Verbaus. *Geotechnik* 19 (1): 9-17.

Goh, A.T.C. 1993. Prediction of earth pressures on concrete retaining walls from surface line loads. *Geotech. Engrg.*, J. S.E. Asian Geotech. Soc., 23 (1): 57-76.

Goh, A.T.C. 1993. Behavior of cantilever retaining walls. *J. Geotech. Engrg.*, ASCE 119 (11): 1751-1771.

Gollub, P. & Klobe, B. 1995. Tiefe Baugruben in Berlin: Bisherige Erfahrungen und geotechnische Probleme. *Geotechnik* 18 (1): 121-132.

Griffiths, D.V. & Li, C.O. 1993. Analysis of delayed failure in sloping excavations. *J. Geotech. Engrg.*, ASCE, 119 (9): 1360-1378.

Gudehus, G. 1991. General report: Displacements and soil-structure interaction of earth retaining structures. *Proc. X. ECSMFE*, Vol. 3, Firenze 1991: 1065-1071.

Gudehus, G. & Weißenbach, A. 1996. Limit state design of structural parts at and in the ground. *Ground Engineering* 29 (9): 42-45.

Gücker, R. 1996. Die Baugruben DEBIS Los A und D am Potsdamer Platz in Berlin. *Vortr. Baugrundtagung 1996 Berlin*: 41-54.

Gunn, M.J. & Clayton C.R.I 1992. Installation effects and their importance in the design of earth-retaining structures. *Géotechnique* 42 (1): 137-141.

Hanke, E. 1994. Beanspruchungen einer Trägerbohlwand in Sand. *Festschrift Prof. Weißenbach*, Univ. Dortmund, Heft 21: 77-102.

Hartmann-Linden, R. & Sedlacek, G. & Schmitt, A. 1996. Plastische Bemessung von Stahlspundwänden nach Eurocode 3, Teil 5. *Vortr. Baugrundtagung Berlin 1996*: 299- 309.

Heibaum, M. 1992. Anmerkungen zum probabilistischen Sicherheitskonzept für die Bemessung des Stützsystems Spundwand. *Geotechnik* 15 (1): 254-259.

Helwany, M.B. & Tatsuoka, F. & Tateyama, M. & Kojima, K.1996. Effects of facing rigidity on the performance of geosynthetic-reinforced soil retaining walls. *Soils and Foundations*, Jap. Geotech. Soc., 36 (1): 27-18.

Herzog, M. 1995. Zum Versagen einer verankerten Spundwand am Houston Ship Channel. *Bautechnik* 72 (1): 51-55.

Hettler, A. & Besler, D. 1996. FE-Simulation der aufgetretenen Spannungen und Verformungen an der Baugrube Gleiswechsel in Duisburg-Meiderich. *Beitr. 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 35-54.

Ho, S.K. & Rowe, R.K. 1994. Predicted behavior of two centrifugal model soil walls. *J. Geotech. Engrg.*, ASCE, 120 (10): 1845-1873.

Hock-Berghaus, K. 1995. Erddruckabminderung an vorspringenden Baugrubenecken. *Geotechnik* 18 (1): 1-6.

Hsi, J.P. & Small, J.C. 1993. Application of a fully coupled method to the analysis of an excavation. *Soils and Foundations*, Jap. Geotech. Soc., 33 (4): 36-48.

Huang, C.-C. & Tatsuoka, F. & Sato, Y. 1994. Failure mechanisms of reinforced sand slopes loaded with a footing. *Soils and Foundations*, Jap. Geotech. Soc., 34 (2): 27-40.

Hügel, H.M. 1995. Prognose von Bodenverformungen. *Veröff. Inst. Boden-u. Felsmech.*, Univ. Karlsruhe, Heft 136.

Iai, S. & Kameoka, T. 1993. Finite element analysis of earthquake induced damage to anchored sheet pile quay walls. *Soils and Foundations*, Jap. Geotech. Soc., 33 (1): 71-91.

Jewell, R.A. & Pedley, M.J. 1992. Analysis for soil reinforcement with bending stiffness. *J. Geotech. Engrg.*, ASCE, 118 (10): 1505-1528.

Juang, C.H. & Lee, D.H. & Sheu, C. 1992. Mapping slope failure potential using fuzzy sets. *J. Geotech. Engrg.*, ASCE, 118 (3): 475-494.

Kalle, U. & Zentgraf, J. 1992. Historische Entwicklung der Standsicherheitsnachweise von Stützwänden. *Geotechnik* 15 (1): 247-253.

Katzenbach, R. & Floss, R. 1992. Neues Baukonzept zur verformungsarmen Herstellung tiefer Baugruben in weichem Seeton. *Vortr. Baugrundtagung 1992 Dresden*: 13-31.

Kauther, R. & Auth, N. & Haarer, R. 1996. Die neue Staustufe Saarbrükken – Geotechnische Probleme bei der Sicherung der Felsböschungen für die tiefen Baugruben in und an der Saar. *Vortr. Baugrundtagung 1996 Berlin*: 317-339.

King, G.J.W. 1995. Analysis of cantilever sheet-pile walls in cohesionless soil. *J. Geotech. Engrg.*, ASCE, 121 (9): 629-635.

Kirchdörfer, V. 1992. Einsatz von Stahlspundwänden im Bereich der Binnenwasserstraßen. *Geotechnik* 15 (1): 190-197.

Kishnani, S.S. & Borja, R.I. 1993. Seepage and soil-structure interaction effects in braced excavations. *J. Geotech. Engrg.*, ASCE, 119 (5): 912-928.

Kirsten, H.A.D. 1992. Limiting equilibrium, closed-form elastic, and numerical plastic analyses of stability of earth wall reinforced with grouted rebar nails and cable anchors. *Can. Geotech. J.* 29 (2): 288-298.

Kirsten, H.A.D. & Dell, A.G. 1992. Application of empirical performance criteria and limiting equilibrium principles in design of an earth wall reinforced with grouted rebar nails and cable anchors. *Can. Geotech. J.* 29 (2): 299-308.

Köthe, E. & Müller, J. 1992. Spundwände in Binnenhäfen. *Geotechnik* 15 (1): 198-214.

Kropik, C. 1996. Einsatz von Polsterwänden im Zuge der Ostbahnhebung. *Beitr. 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 221-227.

Lackner, K. & Harer, G. 1996. Interplay geology/engineering shown on a deep excavation. *Proc. XI. ECSMFE*, Vol. 8, Copenhagen 1995: 8.85-8.94.

Lackner, E. & Müller, J. 1992. Spundwände für Häfen und Wasserstraßen. *Grundbau-Taschenbuch*, Vierte Aufl. Teil 3, Herausg. Smoltczyk, Ernst & Sohn, Verl. Archit. u. techn. Wissensch., Berlin 1992: 743-806.

Lade, P.V. 1992. Static instability and liquefaction of loose fine sandy slopes. *J. Geotech. Engrg.*, ASCE, 118 (1): 51-71.

Lam, L. & Fredlund, D.G. 1993. A general limit equilibrium model for three-dimensional slope stability analysis. *Can. Geotech. J.* 30 (6): 905-919.

Lawton, E.C. & Khire, M.V. & Fox, N.S. 1993. Reinforcement of soils by multioriented geosynthetic inclusions. *J. Geotech. Engrg.*, ASCE, 119 (2): 257-275.

Lee, K. & Jones, C.J.F.P. & Sullivan, W.R. & Trolinger, W. 1994. Failure and deformation of four reinforced soil walls in eastern Tennessee. *Géotechnique* 44 (1): 397-426.

Lenzi, M. & Rosetti, R. 1992. Evaluation of ground-structure interaction in the construction of a wharf inside the AGIP base north operating sector of the Port of Ravenna. *Proc. 10th Int. Harbour Congr.*, Antwerpen 1992: 3.33-3.43.

Leshchinsky, D. & Huang, C.-C. 1992. Generalized slope-stability analysis: interpretation, modification, and comparison. *J. Geotech. Engrg.*, ASCE, 118 (10): 1559-1576.

Leshchinsky, D. & Huang, C.-C. 1992. Generalized three-dimensional slope-stability analysis. *J. Geotech. Engrg.*, ASCE, 118 (11): 1748-1764.

Leshchinsky, D. & Kaliakin, V. & Bose, P. & Collin, J. 1994. Failure mechanism in geogrid-reinforced segmentall walls: experimental implications. *Soils and Foundations*, Jap. Geotech. Soc., 34 (4): 33-41.

Leshchinsky, D. & San, K.-C. 1994. Pseudostatic seismic stability of slopes: design charts. *J. Geotech. Engrg.*, ASCE, 120 (9): 1514-1532.

Ling, H.I. & Tatsuoka, F. 1994. Performance of anisotropic geosynthetic-reinforced cohesive soil mass. *J. Geotech. Engrg.*, ASCE, 120 (7): 1166-1184.

Ling, H.I. & Tatsuoka, F. & Tateyama, M. 1995. Simulating performance of GRS-RW by finite element procedure. *J. Geotech. Engrg.*, ASCE, 121 (4): 330-340.

Lo, S.C.R. & Li, S.Q. & Gopalan, M. & Gao, Z. 1996. Analysis and design of a tied back-to-back geosynthetic reinforced soil wall. *Geotech. Engrg.*, J. S.E. Asian Geotech. Soc., 26 (1): 37-51.

Lo, S.-C.R. & Xu, D.-W. 1992. A strain-based design method for the collapse limit state of reinforced soil walls or slopes. *Can. Geotech. J.* 29 (5): 832-842.

Mannsbart, G. 1996. Bemessungsansätze für geotextilverstärkte Erdstützkonstruktionen. *Beitr 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 205-220.

Martinek, K. 1994. Technische Lösungen beim S-Bahn-Tunnelbau in weichem Untergrund in Frankfurt/Main und Stuttgart. *Festschrift Prof. Weißenbach*, Univ. Dortmund, Heft 21: 139-158.

Mayer, P.M., & Kudella, P. 1996. Prognose von Bodenverformungen als Bestandteil der Beobachtungsmethode. *4. FEM/CAD-Tagung, Darmstadt 1996*, Fortschr.-Ber. VDI Reihe 20, Nr 214, VDI-Verlag, Düsseldorf: 45-57.

Michalowski, R.L. & Zhao, A. 1995. Continuum versus structural approach to stability of reinforced soil. *J. Geotech. Engrg.*, ASCE, 121 (2): 152-162.

Möckel, D. 1993. Zur Vordimensionierung von Stützmauern. *Bauingenieur* 68: 471-475.

Mönnich, D. & Kramer, J. 1996. Tiefe Baugruben mit verankerter Unterwasserbetonsohle für die Verkehrsanlagen im Zentralen Bereich Berlin. *Vortr Baugrundtagung 1996 Berlin*: 25-40.

Mortensen, K. 1995. Kinematically and statically plausible calculations for sheet walls. *Can. Geotech. J.* 32 (3): 408-419.

Moser, M. & Pregl, O. 1996. Bemessung von Stützwänden aufgrund verformungsabhängiger Belastungen. *Beitr. 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 117-138.

Muth, G. 1996. Tragverhalten und Bemessung von Bewehrte Erde Stützkonstruktionen auf weichem, setzungsempfindlichem Untergrund. *Beitr. 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 189-204.

Navari, O. & Hartmann, R. & Lackner, R. 1997. Standsicherheitsberechnung nach der Direkten Gleitblock-Methode. *Bautechnik* 74 (1): 21-24.

Neelakantan, G. & Budhu, M. & Richards Jr, R. 1992. Balanced seismic design of anchored retaining walls. *J. Geotech. Engrg.*, ASCE, 118 (6): 873-888.

Ng, C.W.W. & Lings, M.L. 1995. Effects of modeling soil nonlinearity and wall installation on back-analysis of deep excavation in stiff clay. *J. Geotech. Engrg.*, ASCE, 121 (10): 687-695.

Ng, C.W.W. & Lings, M.L. & Simpson, B. & Nash, D.F.T. 1995. An approximate analysis of the three-dimensional effects of diaphragm wall installation. *Géotechnique* 45 (1): 497-507.

Nova, R. & Boulon, M. & Gens, A. 1995. Theme Report 6. Numerical modelling and information technology. *Proc. XI. ECSMFE*, Vol. 9, Copenhagen 1995: 9.105-9.138.

Nußbaumer, M. 1996. Große Baugruben in Berlin, Entwicklungen von der Jahrhundertwende bis heute, derzeitige Problemstellungen. *Vortr. Baugrundtagung 1996 Berlin*: 5-23.

Ostermayer, H. 1995. Das Verhalten des Systems Bauwerk-Anker-Boden als Grundlage für den Entwurf verankerter Konstruktionen. *Bauingenieur* 70: 371-380.

Ostermayer, H. & Gollub, P. 1996. Baugrube Karstadt in Rosenheim. *Vortr. Baugrundtagung 1996 Berlin*: 341-360.

Ou, C.-Y. & Chiou, D.-C. & Wu, T.-S. 1996. Three-dimensional finite element analysis of deep excavations. *J. Geotech. Engrg.*, ASCE 122 (5): 337-345.

Pierschke, K. & Gudehus, G. & Hügel, H. & Niemunis, A. 1996 Vorhersage und Steuerung von Bodenverformungen infolge Tagebauaktivitäten. *Vortr. Baugrundtagung 1996 Berlin*: 361-370.

Porbaha, A. & Goodings, D.J. 1996. Centrifuge modeling of geotextile-reinforced cohesive soil retaining walls. *J. Geotech. Engrg.*, ASCE, 122 (10): 840-848.

Raabe, E.W. & Kalthoff, D. & Stewering, T. 1994. Sohlaussteifung einer tiefen Baugrube unter Anwendung des SOILCRETE-Verfahrens. *Vortr. Baugrundtagung 1994 Köln*: 259-270.

Reinke, U. & Vollstedt, H.-W. & Rahtge, M. 1995. Containerkaje Bremerhaven - Nördliche Erweiterung CT III. *Jahrb. Hafenbautech. Gesellsch.* 50: 130-135.

Rodatz, W. & Gattermann, J. & Horst, M. 1996. Anwendung der Beobachtungsmethode zur Sicherstellung geringer Verformungen einer zweilagig ausgesteiften Baugrube. *Bauingenieur* 71: 355-364.

Rodatz, W. & Maybaum, G. 1995. Erddruckentwicklung auf eine Kaimauer in Schlitzwandbauweise. *Jahrb. Hafenbautech. Gesellsch.* 50: 143-154.

Rodatz, W. & Maybaum, G. & Stahlhut, O. 1996. Geotechnische Messung und numerische Berechnung eines großen Geländesprungs im Hamburger Hafen. *Beitr 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 159-177.

Rodatz, W. & Meseck, H. & Meier, K. 1992. Baugruben in Spundwandbauweise. *Geotechnik* 15 (1): 227-236.

Rollberg, D. 1996. Böschungssicherungen durch Vernagelung. *Beitr. 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 229-245.

Roth, S. 1992. 90 Jahre Hoesch-Stahlspundwand – ein Rückblick. *Geotechnik* 15 (1): 178-182.

Roth, S. 1992. 90 Jahre Spundwand – Möglichkeiten und Grenzen. *Vortr. Baugrundtagung 1992 Dresden*: 227-264.

Sänger, C. 1994. Die Baugrube für die Friedrichstadt-Passagen in Berlin. *Festschrift Prof. Weißenbach*, Univ. Dortmund, Heft 21: 159-176.

Sänger, C. & Hähnig, F. 1994. Baugrube für den "Hofgarten am Gendarmenmarkt" in Berlin-Mitte. *Vortr. Baugrundtagung 1994 Köln*: 219-232.

Sänger, C. & Mayer, P.-M., 1996. Meßtechnische Überwachung der Interaktion zwischen einem bestehenden Hochhaus und einer benachbarten 20 m tiefen Baugrube in Berlin-Kreuzberg. *Vortr Baugrundtagung 1996 Berlin*: 385-413.

Sänger, C. & Steinhagen, P.& Mayer, P.-M. 1994. Friedrichstadt-Passagen Berlin – Planung und Ausführung der Baugrube. *Bauingenieur* 69: 267-272.

San, K.-C. & Leshchinsky, D. & Matsui, T. 1994. Geosynthetic reinforced slopes: limit equilibrium and finite element analyses. *Soils and Foundations*, Jap. Geotech. Soc., 34 (2): 79-85.

San, K.-C. & Matsui, T. 1995. Test wall failure prediction using generalized plasticity theory. *Soils and Foundations*, Jap. Geotech. Soc., 35 (3): 105-109.

Saran, S. & Garg, K.G. & Bhandari, R.K. 1992. Retaining wall with reinforced cohesionless backfill. *J. Geotech. Engrg.*, ASCE, 118 (12): 1869-1888.

Schäffner, H.-J. 1993. Verformungen und Standsicherheit von doppelten Spundwandbauwerken. *Geotechnik* 16 (1): 107-114.

Schäffner, H.-J. 1993. Verformungen und Standsicherheit von Spundwänden. *Bautechnik* 70 (10): 587-594.

Schenck, W. & Smoltczyk, U. & Lächler, W. 1992. Pfahlroste, Berechnung und Konstruktion. *Grundbau-Taschenbuch*, Vierte Aufl. Teil 3, Herausg. Smoltczyk, Ernst & Sohn, Verl. Archit. u. techn. Wissensch., Berlin 1992: 269-331.

Scherrer, P. & Baril, J.-C. & Heber, P. 1996. Construction of a diaphragm wall quay in tidel dock (Osaka berth). *Proc. 11th Int. Harbour Congr.*, Antwerpen 1996: 393-401.

von Schmettow, T. & Wittke, W. 1996. Vergleichende Untersuchungen zwischen Tunnelstatik mit Hilfe der Finite-Elemente- und der Distinct-Element-Methode. *Geotechnik* 19 (2): 110-114.

Schoitsch, G. & Schrank, M. 1996. Verankerung im Salzburger Seeton am Beispiel Umbau Bahnhof Salzburg. *Beitr. 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 55-66.

Schulz, H. 1992. Zum Ansatz von Zuggliedern beim Nachweis der Standsicherheit von Geländesprüngen. *Geotechnik* 15 (1): 260-264.

Schuppener, B. 1994. Die statische Berechnung der Bauweise Lebend Bewehrte Erde. *Geotechnik* 17 (4): 204-213.

Schuppener, B. 1996. Eine Proberammung vor einer Stützwand mit unzureichender Standsicherheit. *Vortr. Baugrundtagung 1996 Berlin*: 377-384.

Seeger, H. 1992. Tiefe Baugrube im Stuttgarter Gipskeuper. *Vortr. Baugrundtagung 1992 Dresden*: 49-60.

Shotton, P.G. 1992. Improved techniques for the construction of retaining walls and piles with particular reference to top-down construction. *Géotechnique* 42 (1): 143-150.

Siller, T.J. & Dolly, M.O. 1992. Design of tied-back walls for seismic loading. *J. Geotech. Engrg,*, ASCE, 118 (11): 1804-1821.

Siller, T.J. & Frawley D.D. 1992. Seismic response of multianchored retaining walls. *J. Geotech. Engrg.*, ASCE, 118 (11): 1787-1803.

Simpson, B. 1992. Partial factors of safety for the design of retaining walls. *Géotechnique* 42 (1): 131-136.

Simpson, B. 1992. Thirty-second Rankine Lecture: Retaining structures: displacement and design. *Géotechnique* 42 (1): 539-576.

Singh, D.N. & Basudhar, P.K. 1992. A note on the optimal lower bound pull-out capacity of inclined strip anchors in sand. *Can. Geotech. J.* 29 (5): 870-873.

Smoltczyk, U. 1992. Stützmauern. *Grundbau-Taschenbuch*, Vierte Aufl. Teil 3, Herausg. Smoltczyk, Ernst & Sohn, Verl. Archit. u. techn. Wissensch., Berlin 1992: 653-678.

Smoltczyk, U. 1996. Hat die europäische Normung in der deutschen Geotechnik eine Chance? *Bautechnik* 73 (3): 134-146.

Sonder, H.-H. 1994. Der Berliner Verbau. *Festschrift Prof. Weißenbach*, Univ. Dortmund, Heft 21: 229-245.

Stadler, G. & Kudella, P. 1996. Projekt GSW Berlin: Tiefe Baugrube in

Dichtwand/Spundwand-Bauweise. *Beitr. 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 1-13.

Stehn, H.-J. 1992. Einsatz von Beton-Schlitzwänden und Beton-Bohrpfahlwänden bei Kaimauern. *SMM'92 Conf. SEA 2000 Hamburg*: 48-51.

Stocker, M. & Walz, B. 1992. Pfahlwände, Schlitzwände, Dichtwände. *Grundbau-Taschenbuch*, Vierte Aufl. Teil 3, Herausg. Smoltczyk, Ernst & Sohn, Verl. Archit. u. techn. Wissensch., Berlin 1992: 493-532.

Symons, I.F. & Carder, D.R. 1992. Field measurements on embedded retaining walls. *Géotechnique* 42 (1): 117-126.

Tanaka, H. 1993. Behavior of braced excavations stabilized by deep mixing method. *Soils and Foundations*, Jap. Geotech. Soc., 33 (2): 105-115.

Tanaka, H. 1994. Behavior of a braced excavation in soft clay and the undrained shear strength for passive earth pressure. *Soils and Foundations*, Jap. Geotech. Soc., 34 (1): 53-64.

Timm, U. 1994. Tiefe Baugrube im Grundwasser, dargestellt am Beispiel des Fahrlachtunnels Mannheim. *Festschrift Prof. Weißenbach*, Univ. Dortmund, Heft 21: 247-265.

Triantafyllidis, T. 1996. Verformungsverhalten von Stützwänden bei tiefen Baugruben anhand von Ausführungsbeispielen. *Beitr. 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 15-34.

Ulrichs, K.R. 1994. Verformungen an unterschiedlich gestützten Schlitzwand-Baugruben im rheinischen Kies-Sand. *Festschrift Prof. Weißenbach*, Univ. Dortmund, Heft 21: 267-295.

Van Celst, E. & Reynaers, T. & Thibaut, W. & Vellemann, B.& Terryn, N. 1996. Design of the second container terminal, north of the "Lock of Zandvliet" in the Port of Antwerp. *Proc. 11th Int. Harbour Congr.*, Antwerpen 1996: 215-225.

van Tol, A.F. 1994. Der Entwurf von Spundwandkonstruktionen in den Niederlanden. *Vortr. Baugrundtagung Köln 1994*: 127-139.

Vaziri, H.H. 1996. Numerical study of parameters influencing the response of flexible retaining walls. *Can. Geotech. J.* 33 (2): 290-308.

Vaziri, H.H. & Troughton, V.M. 1992. An efficient three-dimensional soil-structure interaction model for analysis of earth retaining structures. *Can. Geotech. J.* 29 (8): 529-539.

Vermeer, P.A. & Brinkgreve, R.B.J. 1995. PLAXIS – Finite element code for soil and rock mechanics. Manual Version 6. A.A.Balkema, Rotterdam/Brookfield 1995

Vlavianos, G.J. 1995. Probabilistic analysis of seismic safety of retaining walls. *Proc. XI. ECSMFE*, Vol. 6, Copenhagen 1995: 6.215-6.220.

Vollstedt, H.-W. 1996. Ausbau des Containerterminals "Wilhelm Kaisen" in Bremerhaven. *Vortr. Baugrundtagung 1996 Berlin*: 371-375.

Walz, B. 1994. Erddruckabminderung an einspringenden Baugrubenecken. *Bautechnik* 71 (2): 90-95.

Wang, Y.H. & Wang, M.C. 1994. Performance of polypropylene-strip-reinforced soil retaining structure. *Geotech. Engrg.*, J. S.E. Asian Geotech. Soc., 25 (1): 39-54.

Weber, K. & Gußmann, P. 1992. Vergleichende Untersuchungen zu Bruchzuständen in Böden nach den Verfahren der FEM und KEM. *Bautechnik* 69 (11): 625-631.

Weißenbach, A. 1992. Baugrubensicherung. *Grundbau-Taschenbuch*, Vierte Aufl. Teil 3, Herausg. Smoltczyk, Ernst & Sohn, Verl. Archit. u. techn. Wissensch., Berlin 1992: 379-492.

Weißenbach, A. & Gollub, P. 1995. Neue Erkenntnisse über mehrfach verankerte Ortbetonwände bei Baugruben in Sandboden mit tiefliegender Injektionssohle, hohem Wasserüberdruck und großer Bauwerkslast. *Bautechnik* 72 (12): 780-799.

Wilde, F.-K. 1995. Neubau des 10. Liegeplatzes am Athabaskai. *Jahrb. Hafenbautech. Gesellsch.* 50: 125-129.

Wilson-Fahmy, R.F. & Koerner, R.M. & Harpur, W.A. 1995. Long-term pullout behavior of polymeric geogrids. *J. Geotech. Engrg.*, ASCE, 121 (10): 723-728.

Wilson-Fahmy, R.F. & Koerner, R.M. & Sansone, L.J. 1994. Experimental behavior of polymeric geogrids in pullout. *J. Geotech. Engrg.*, ASCE, 120 (4): 661-677.

Wittke, W. & Erichsen, C. 1990. Böschungsgleichgewicht im Fels. *Grundbau-Taschenbuch*, Vierte Aufl. Teil 1, Herausg. Smoltczyk, Ernst & Sohn, Verl. Archit. u. techn. Wissensch., Berlin 1990: 615-703.

Whittle, A.J. & Hashash, Y.M. & Whitman, R.V. 1993. Analysis of deep exceavation in Boston. *J. Geotech. Engrg.*, ASCE, 119 (1): 69-90.

von Wolffersdorff, P.-A. 1994. Feldversuch an einer Spundwand in Sandboden: Versuchsergebnisse und Prognosen. *Geotechnik* 17 (2): 73-83.

von Wolffersdorff, P.-A. 1996. Nachweise für eine Winkelstützwand nach dem neuen Sicherheitskonzept. *Bautechnik* 73 (3): 147-161.

von Wolffersdorff, P.-A. 1996. Berechnung von Stützkonstruktionen nach dem neuen Sicherheitskonzept im Grundbau. *Beitr. 11. Christian Veder Kolloquium*, Techn. Univ. Graz, Mitt. 14: 101-115.

von Wolffersdorff, P.-A. & Mayer, P.M. 1996. Gebrauchstauglichkeitsnachweise für Stützkonstruktionen. *Geotechnik* 19 (4): 291-300.

Wood, D.M. & Jendele, L. & Chan, A.H.C. & Cooper, R.C. 1995. Slope failure by pore pressure recharge: numerical analysis. *Proc. XI. ECSMFE*, Vol. 6., Copenhagen 1995: 6.1-6.8.

Yogendrakumar, M. & Bathurst, R.J. & Finn, W.D.L. 1992. Dynamic response analysis of reinforced-soil retaining wall. *J. Geotech. Engrg.*, ASCE, 118 (8): 1158-1167.

Young, D.K. & Ho, E.W.L. 1994. The observational approach to design of a sheet-piled retaining wall. *Géotechnique* 44 (1): 637-654.

Zadroga, B. & Dembicki, E. 1996. Stability analysis of existing and renovated hydroengineering structures in ports of Vistula Lagoon in Poland. *Proc. 11th Int. Harbour Congr.*, Antwerpen 1996: 141-152.

Zanzinger, H. & Gartung, E. 1992. Modellversuch zum Tragverhalten einer Polsterwand aus gewebten Geogittern. *Geotechnik* 15 (1): 113-123.