Prague Geotechnical Days 2017

Seminar – invited lectures

PILES AS RETAINING STRUCTURES

25th Prague Geotechnical Lecture

Design Issues for Steel Pipe Piles for Bridge Foundations, Coastal Structures and Offshore Applications

Resume

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PRAGUE GEOTECHNICAL DAYS 2017

Resume

Piles as Retaining Structures

25th PRAGUE GEOTECHNICAL LECTURE

PILES AS RETAINING STRUCTURES IN SLOPES – CASE HISTORIES

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Abstract

Piles, structures made of piles, and pile-like structures are useful structural elements to support deep excavations and cuts in slopes, and to retain creeping or sliding slopes, not uncommonly in seismic areas. Depending on the static system and the dimensions the structural elements transfer forces mainly by shear ("dowel") and/or mainly by bending ("beam") to the ground. In numerous cases they are particularly effective in combination with other structural measures like (pre-stressed) anchors and/or drainage systems. The paper presents case histories including piles and pile-like structures, which are applied for retaining structures in slopes. The main focus is on infrastructure projects in creeping slopes. Two case histories from Austria and Slovenia are presented in detail. Miscellaneous projects from European countries concentrating on various aspects complement the contribution.

1. Introduction

For more than five decades piles, structures made of piles (pile wall, pile box, etc.) and pile-like structures (sockets, shafts, slurry trench walls, etc.) have been applied for retaining structures in (creeping) slopes. In the late sixties of the 20th century the boom of motorway and highway construction began in the Alpine region (e.g. A10 Motorway "Tauernautobahn", Austria). Since that time design tools and execution of structural measures have been sophisticated, novel approaches to measure, prove and control construction processes have been enhanced. E.g., the observational method has been established as a valuable philosophy and tool to produce safer and more efficient retaining structures by minimizing risk of unforeseen failure and to maintain serviceability in the long-term by continuous monitoring and setting up contingencies plans. A fundamental theoretical background of geotechnical knowledge combined with experience gained in numerous challenging projects and engineering judgement in all decisive design and construction steps creates the environment for the realization of successfully accomplished complex projects.

2. City Tunnel Waidhofen an der Ybbs, Austria

2.1 Project Overview and Geology

The city tunnel Waidhofen an der Ybbs is the core of the city bypass of a small town of the same name in the south-west of Lower Austria. The road tunnel comprising a total length of 1,485 m is situated along the mountain ridge called Buchenberg and the maximum overburden of the tunnel is about 50 m. Tunnel construction started in November 2007 whereby different construction techniques were applied. The tunnel was opened to traffic in the end of 2011.

The tunnel is situated in an intricate geological-geotechnical complex consisting of soil, hard soil, weak rock and solid rock. Marls and marly limestone (Waidhofener formation) predominate followed by tectonic breccia of the Alpine cliff zone, which is a tectonic melange zone. Materials of this kind universally have isolated interior blocks (shaded areas) embedded in a matrix (blank area) called "block-in-matrix" requiring a particular characterization of the mechanical rock mass properties. The complex geological and morphological situation required the application of various tunnelling methods including open cuts, cut-and-cover sections and the New Austrian Tunnelling Method (NATM). During construction in creeping slopes increased movements were triggered, which required the installation of a sophisticated monitoring system. On the basis of monitoring data and additional ground investigations structural measures including piles, anchors, drainage by stone columns, etc. were assessed that enabled safe tunnelling in the creeping slope. (Adam et al., 2012)

2.2 Tunnelling

Depending on the geological-geotechnical ground properties and the overburden the tunnel was constructed with different tunnel construction methods (see Fig. 1). In shallow tunnel sections with an overburden of a few meters only open cuts and (arched) cut-and-cover tunnels made of intermittent bored pile walls were designed taking into account the specific geological conditions. In sections with overburdens up to about 50 m the New Austrian Tunnelling Method (NATM) was applied. In the Waidhofener formation where no swelling potential was expected in the rock mass no ring closure was necessary. In all other ground conditions a closed ring was realized. The steel reinforcement of the closed ring was adapted in dependence of the results of the swelling tests from samples taken during tunnelling. NATM was combined with blasting within the Waidhofener formation whereby the defined vibration limit values had not to be exceeded due to the vicinity of the inhabited area. In the other rock masses excavation took place with hydraulic excavation equipment with light blasting if necessary. In the central area where the tunnel was situated above the ground surface a rectangular open tunnel was designed. Between the open cut and the NATM section the cut-and-cover method with an arched ceiling ("Kärntner Deckel") was applied. In the portal sections the tunnel was constructed in open cuts. (Adam et al., 2012)

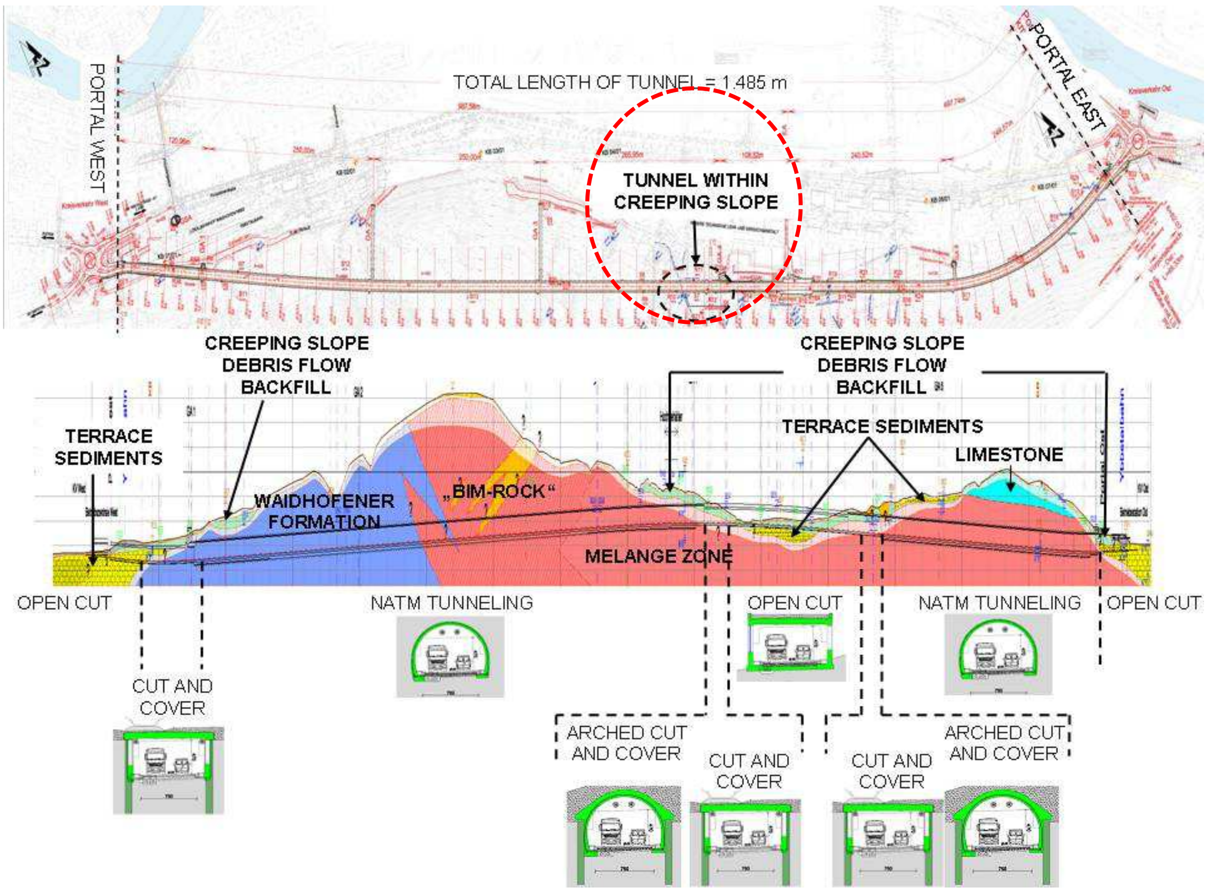


Fig. 1. City Tunnel Waidhofen an der Ybbs. Tunnel alignment in the project area and geological longitudinal section with different tunnelling methods. Zones with creeping slopes are shown.

2.3 Retaining Structure in Creeping Slope

Already in the design stage inclinometer measurements indicated creep behaviour in the central section of the tunnel (see Fig. 1) with a (natural) annual creeping rate of about 14 mm/a in average. Moreover, residual shear angles determined on samples taken from this section, swelling clay minerals and relatively high natural water substantiated the instable slope. Consequently, for drainage of the slope gravel piles were designed, thus, increasing the long-term slope stability.

The design of the open cut and the affected cut and cut-and-cover section of the tunnel made of intermittent bored pile walls (diameter 90 cm, axial spacing 1.39 m, pile length about 20 m) was performed taking into account a creep earth pressure E_{cr}. This creep earth pressure was determined according to the formulation of Brandl & Dalmatiner (1988) for the particular case that the slope inclination *ß* equated to the friction angle *j*. In equation (1) *h* represents the thickness of creep mass affecting the tunnel wall and *m(j)* is a factor depending on the stiffness of the retaining structure (see Fig. 2):

$$E_{cr,h} = m(\varphi)\,\gamma\,\frac{h^2}{2}\cos^2\varphi = K_{cr,h}\,\gamma\,\frac{h^2}{2} \tag{1}$$

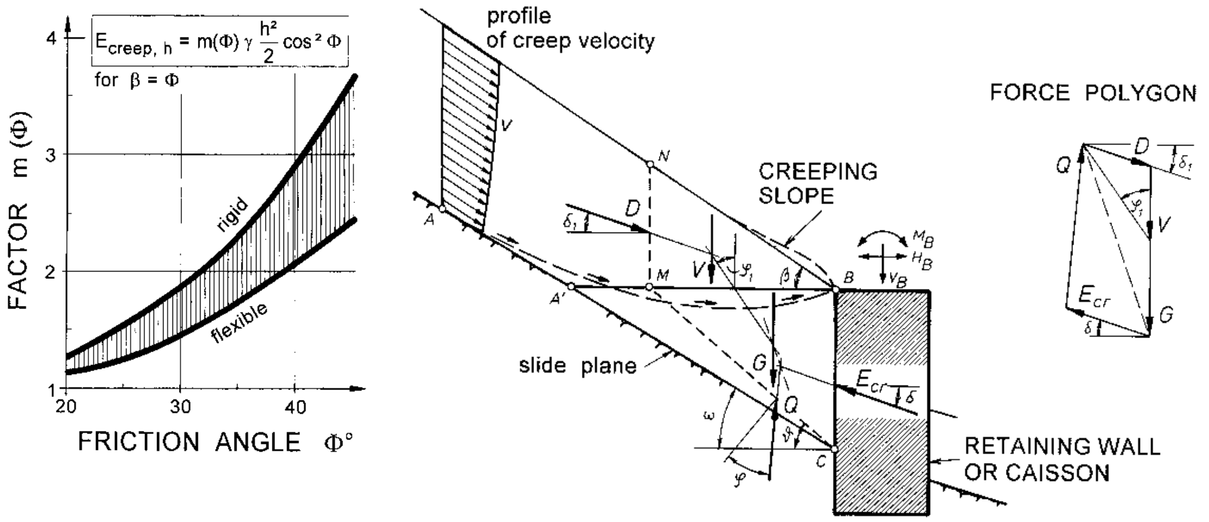


Fig. 2. Determination of creep earth pressure E_{cr} according to Brandl & Dalmatiner (1988).

Inclinometer measurements detected the influence of precipitation in particular after forest clearance by a significant increase of the creeping rate. Finally, in August 2008 the gravel piles were bored. During installation significant additional slope movements were triggered and cracks were observed in a water supply tank situated above the gravel piles (see Fig. 3). The results of the near inclinometer B21 identified a sliding horizon in a depth of about 10 m below surface (see 0). By means of the morphology it was suggested that a set of local creep mass bodies with various sliding horizons exist. Consequently, a comprehensive monitoring and data acquisition system was installed consisting of inclinometers, geodetic measurements and water discharge recordings (in the gravel piles).

From October 2008 additional slope deformations occurred during construction of the tunnel sections No. 71 and No. 72 (see Fig. 3), the water supply tank was affected again. These slope movements resulted not only from the construction activities but originated from an increased wetting of the slope by intense precipitation and beginning of snow melting. After the installation of temporary anchorage of the retaining wall made of an anchored bored pile wall (see 0) the slope creep rate could be reduced again (surface deformation in Fig. 3).

Again in the period March to April 2009 short-term slope movements were observed during excavation works, which got under control after construction of the tunnel ceiling.

Inclinometer B21 showed total surface deformations of about 100 mm from the reference measurement in 2005 to April 2009. It had to be considered that the natural creeping rate was included into the total deformations which were superimposed by the movements triggered by the construction activities. Taking into account the natural creeping rate additional deformations of about 55 mm occurred due to construction works (see Fig. 3).

Measurements of anchor forces showed that the maximum design working load was exceeded up to 25%, however, no additional increase could be observed after the end of the construction works.

(Adam et al., 2012)

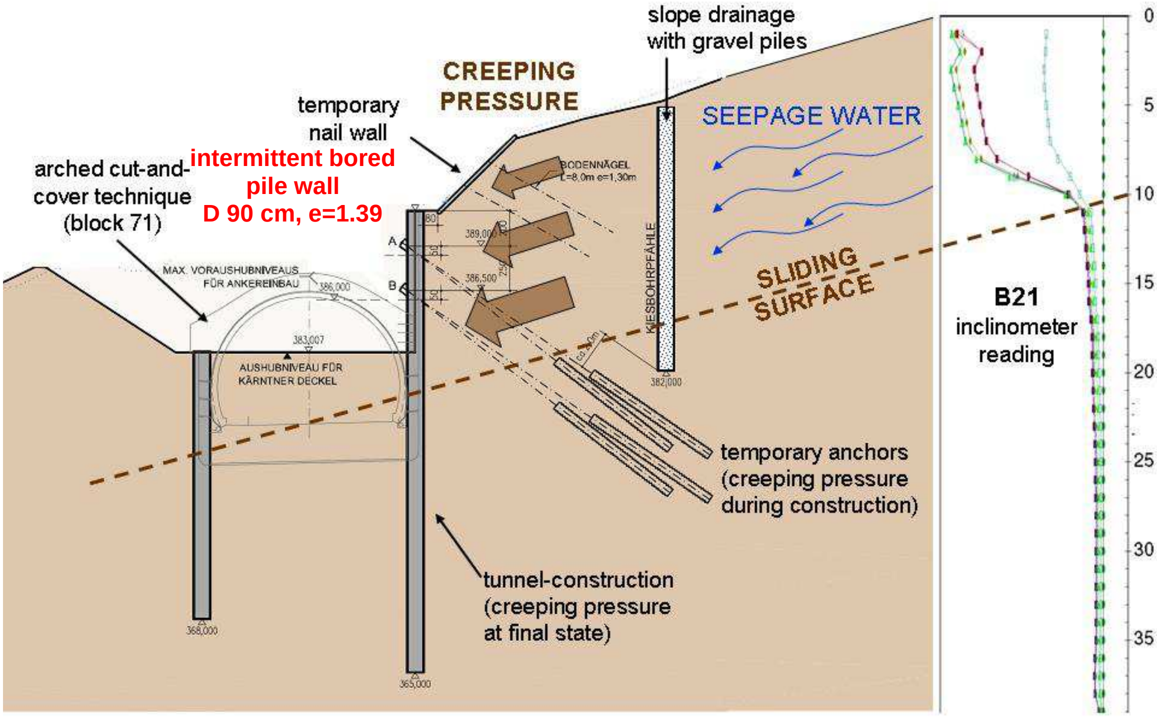


Fig. 3. City Tunnel Waidhofen an der Ybbs. Tunnel cross section in the area of the cut-and-cover tunnel with an arched ceiling (section 71) with identified main sliding surface within the creeping mass. Retaining structure made of an intermittent anchored bored pile wall, pre-stressed anchors and temporary nail wall. The position of the gravel piles and the results of inclinometer measurements are shown.

In the period April 2009 to May 2010 the monitored movements were in the range of the natural creep rates. Gravel piles successfully drained the slope, the water discharge was determined to an annual average of about 780 litres per day.

Although the slope deformation rate decreased after completion of the tunnel additional inclinometers were installed in the creeping slope and geodetic reading points at the tunnel lining as well. Thus, the monitoring of the slope and deformation measurement of the tunnel has been continued for at least 5 years after setting the tunnel in operation.

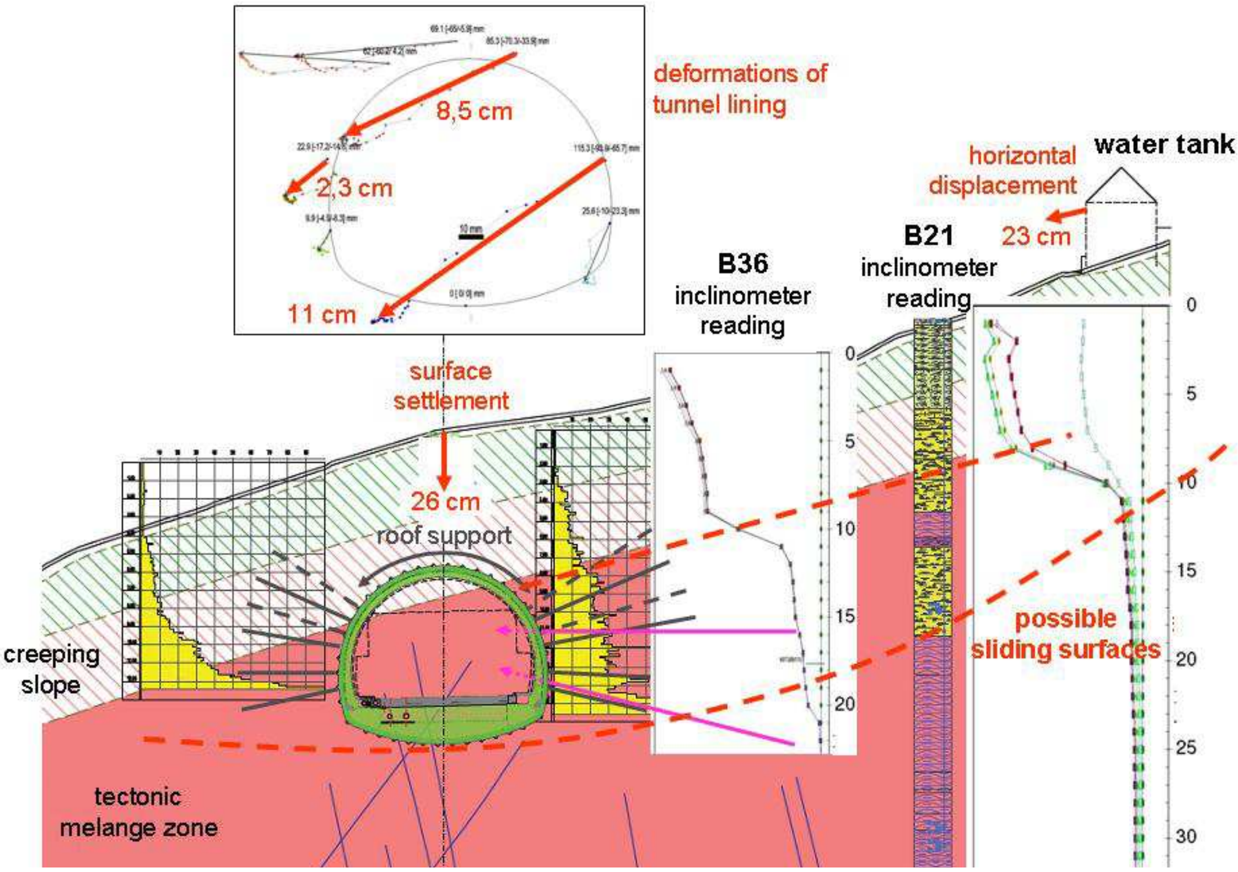


Fig. 4. City Tunnel Waidhofen an der Ybbs. NATM tunnel cross section in the area of the creeping mass with identified sliding surfaces. The location of the water tank and the inclinometers are shown as well as the results of inclinometer and deformation measurements at the tunnel lining.

However, in May 2010 an increase of slope deformations was observed again caused by the NATM tunnelling in the creep mass. In advance parametric studies were performed in order to investigate the influence of the creep behaviour on the tunnel. The results of the analyses disclosed that the creep earth pressure (creep horizon close to the tunnel head) did not affect the bearing capacity but the defined serviceability limit state of the tunnel lining was exceeded so that large deformations and even cracks and damage had to be expected. The attention had to be turned to the outer lining since the reinforced inner lining influenced the deformation only to a minor degree but served for a better distribution of the stresses and allowable fissured cracks. These findings resulted in the decision to increase the thickness of the outer and the inner lining from 30 to 40 cm and to reinforce the inner lining.

Moreover, a contingency plan was prepared to be able to carry out stabilization measures quickly if necessary. The stabilization measures included extensive slope dowelling by vertical and/or inclined piles. (Adam et al., 2012)

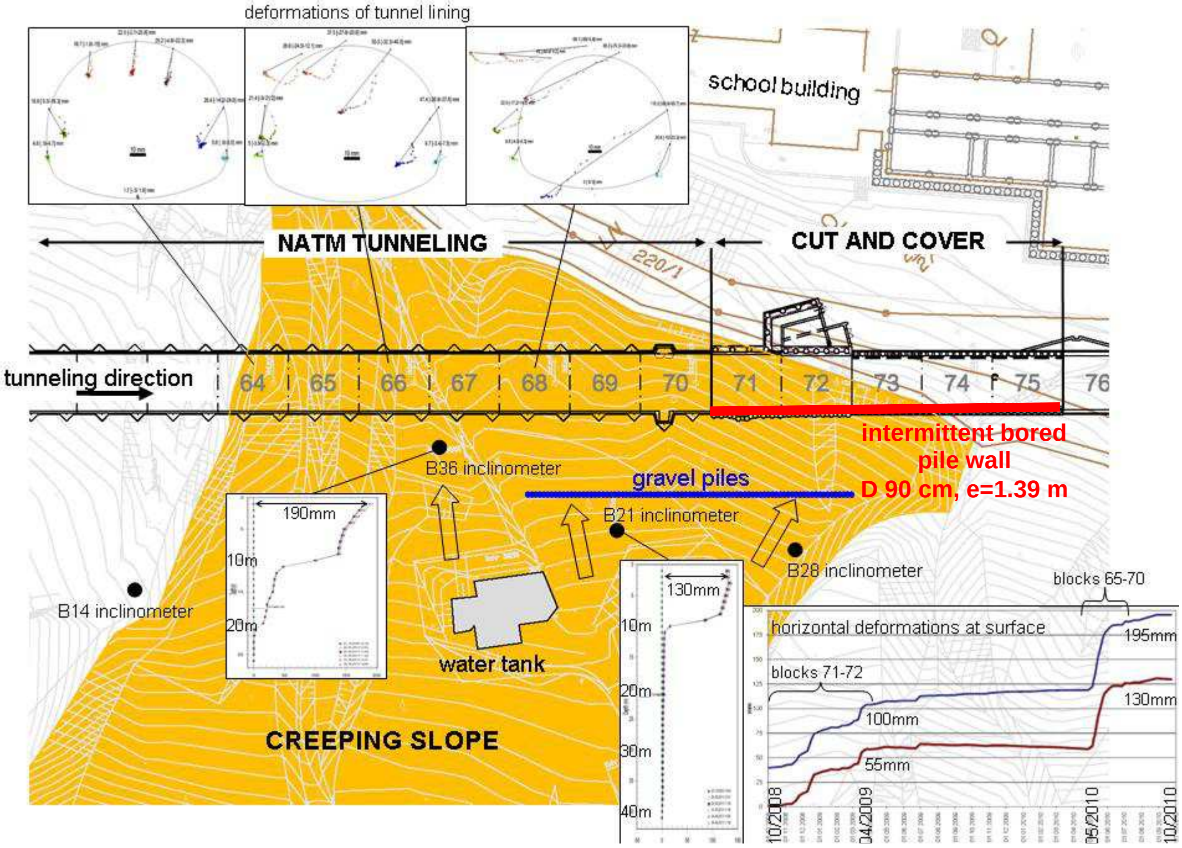


Fig. 5. City Tunnel Waidhofen an der Ybbs. Ground plan of the tunnel within the creeping slope. The location of the gravel piles, the water supply tank and the inclinometers are shown as well as the results of inclinometer measurements and deformation measurements at the tunnel lining.

According to 0 and 0 the deformations in the tunnel roof amounted up to 8.5 cm and at the surface up to about 26 cm. The maximum deformations of the tunnel lining were about 11 cm, whereby asymmetric deformation of the tunnel was observed as expected due to the lateral creeping behaviour of the slope.

With respect to the total deformations of the tunnel lining the static calculations yielded that the ultimate bearing capacity of the shotcrete shell will not be reached and deformations at an early stage are not critical since the tunnel lining is relatively soft and the stresses are redistributed by creep. Only deformations occurring to a later state were detected to be problematic.

The tunnel excavation was executed in segments (top heading, bench, and invert) with short-term ring closure. A pipe roof ensured protection by means of pipes of a length up to 18 m that were driven into the ground preparative and then secured by the use of shotcrete and, if applicable, supported by steel arches. Moreover self-bore anchors were installed (schematically in 0). All anchors oriented to the creeping slope showed a loss of the applied pre-stress force and anchor heads of anchors less than 10 m long displaced in direction of the cavity punching the girder. Thus, it could be assumed that the bond length of the anchors ended in the creep mass, which was confirmed by inclinometer measurements. Inclinometer B36 (see 0) indicated a significant sliding plane. However, the total deformations originated from deeper regions as well, which were obviously linked with the tunnelling. Presumably, in a depth of about 20 m another sliding plane was generated. (Adam et al., 2012)

Fig. 6. City Tunnel Waidhofen an der Ybbs. Construction phase. Open cut and cut-and-cover section in creeping slope. Retaining structure made of intermittent bored pile wall (diameter 90 cm, axial spacing 1.39 m, pile length about 20 m) anchored with pre-stressed anchors (Photo: Dietmar Adam/Roman Markiewicz).

3. H4 Motorway, Rebernice Landslide, Slovenia

3.1 Project Overview and Geology

Along the alignment of the motorway between Razdrto and the Italian border (Slovenia), at the section Razdrto-Vipava, a slope failure occurred in the Rebernice area during the excavation of a 26 m deep cut in November 2001. The bottom of the motorway cut was approximately 6 m above the final design level when the failure cracks were observed on the right slope far beyond the upper cut edge just after heavy precipitation. The designed slope inclination was 1:2 (H:L). The landslide clearly marked by wide and deep failure cracks reached a width of 360 m, while its length varied from 180 m up to 310 m. Observations, monitoring and interpretation of the collected data disclosed a deep-seated translational slide. The inclination measurements in the central part of the landslide showed the depth of the failure plane between 11 and 1 m. (Pulko et al., 2005)

The location is geologically dominated by the overthrust of Cretaceous limestone formation over the Eocene flysch ($E_{1,2}$), which comprises alternating, variously thick beds of marly claystone, siltstone, sandstone and marly calcarenite. Due to tectonic effects and water seepage, a rock base is weathered up to several meters in depth. Stratification is still noticed in the weathered rock formation ($E_{1,2}$), which is overlain by the layer of flysch debris Q_{del}. The deposit above consists of up to 12 m thick limestone debris (scree) Q_{pg}. Between limestone debris Q_{pg} and flysch debris Q_{del} there is a 2 to 3 m thick intermediate layer Q_{del}-Q_{pg}. The layer is heterogeneous with more than 50% of sand and gravel particles in plastic clayey and silty matrix. The clay of high plasticity as soil matrix is the weakest material from the geotechnical point of view. In all boreholes drilled in the landslide area intensive water inflows were recorded in this layer and the layer above, thus, revealing up to 20 kPa of excess pore pressure as a consequence of intensive precipitation (see 0 (top)). (Pulko et al., 2005)

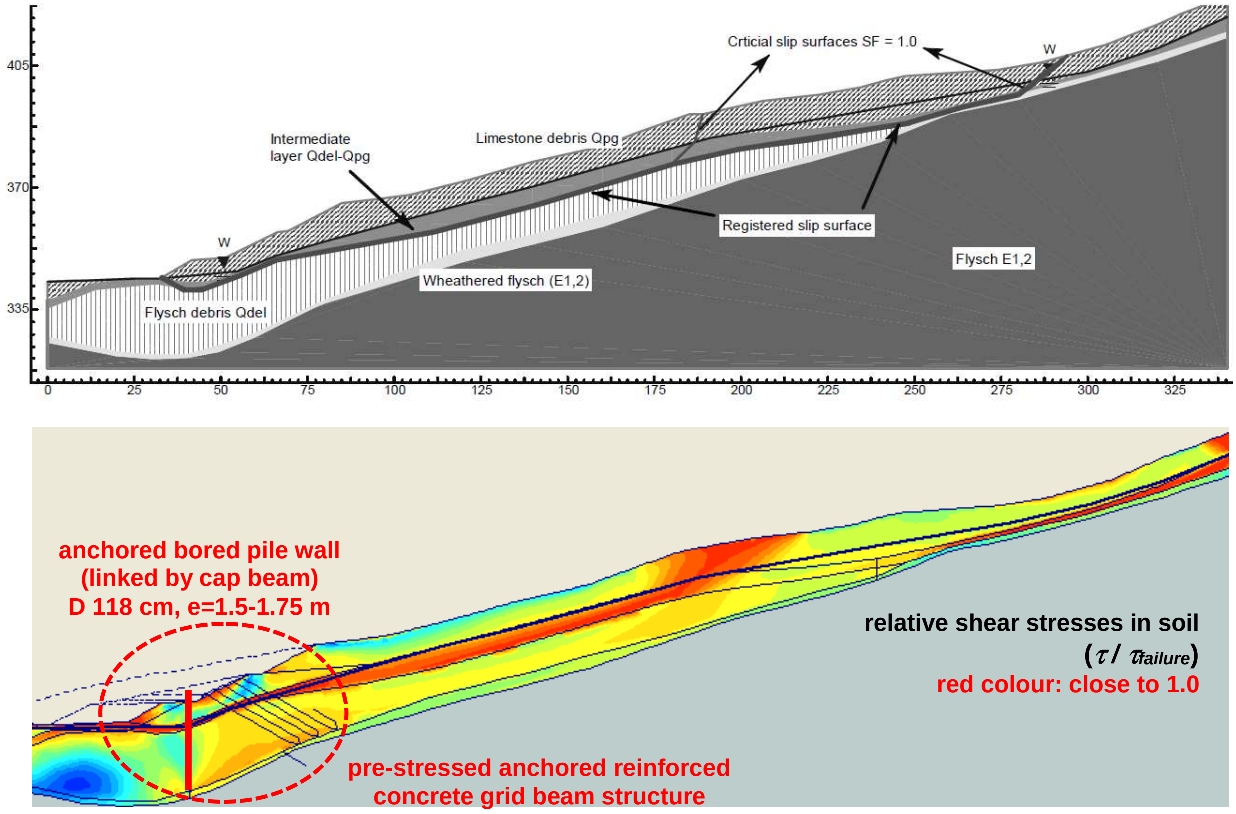


Fig. 7. Rebernice Landslide. Typical cross section (P 257) of the landslide with intermediate soil layer formed of clayey gravel and flysch debris (Pulko et al., 2005) (top) and numerical stability analysis (FEM) for rehabilitated situation (anchored bored pile wall linked by cap beam and pre-stressed anchored reinforced concrete grid beam structure), colours represent relative shear stress in the soil (Pulko et al., 2004) (bottom).

3.2 Retaining Structure in Creeping Slope

Calculations were performed for representative cross sections taking into account different states (before excavation, after excavation and after installing the backfill) in order to verify the developed geological-geotechnical model. Numerical simulations by Pulko et al. (2005) (finite element analyses; for soils constitutive model of Hardening Soil was applied) clearly indicated a stable situation before excavation (safety factor SF = 1.10 to 1.52) and an unstable situation after excavation (SF ~ 1.0, i.e. limit equilibrium). Critical shear stresses were found mainly along intermediate layer Q_{del}-Q_{pg} (see 0 (bottom)). The stage after backfilling yielded safety factors of about SF = 1.13 to 1.41.

Several possibilities were considered for the stabilisation of the landslide. A cut-and-cover tunnel, reinforced concrete dowels of large diameter and anchored pile walls were initially considered for the final rehabilitation of the landslide. Finally, an anchored pile wall with additional anchored grid-beam structure and two deep trench drains were chosen and accepted by the state road authorities for the final stabilisation of the landslide.

For the design of the supporting structures finite element analysis by Pulko et al. (2005) was employed. All the necessary construction steps were considered in the analysis. The stability of the final state was checked by "phi-c" reduction procedure to achieve the required safety factor SF = 1.25. The design of structural elements was accomplished according to Eurocode standards.

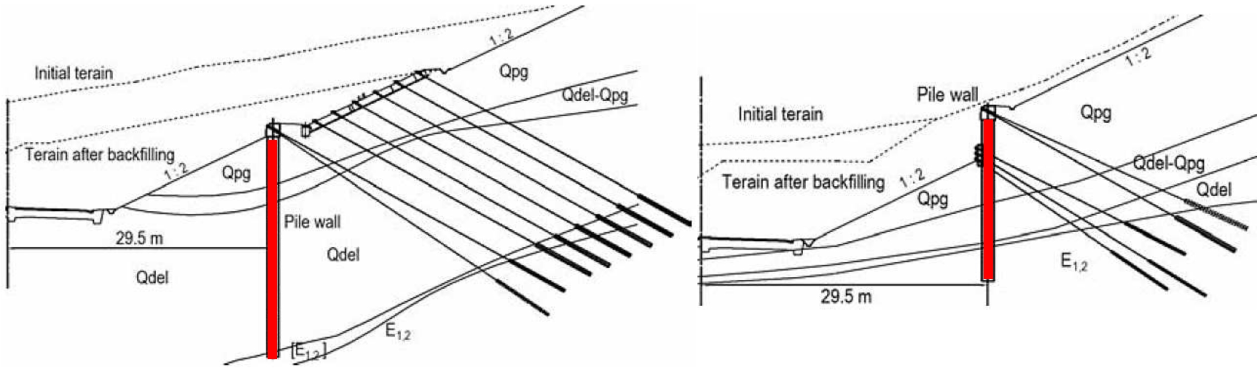


Fig. 8. Rebernice Landslide. Typical cross section P 257 (left) and P 261 (right) for rehabilitated situation: anchored bored pile wall linked by cap beam (left and right) and additional pre-stressed anchored reinforced concrete grid beam structure (right) (Pulko et al., 2005).

The pile wall comprising a length of 347 m was located at a distance of 29.5 m from the road axis. The slope between the road and pile wall was adopted at an inclination of 1:2. Bored piles with a diameter of 118 cm and lengths from 10 to 26 m at spacing of 1.50 and 1.75 m, respectively, were installed. At the top, a cap beam links the piles. Most of the structure was anchored in one level with anchored grid beam structure above it (l = 203 m; 0 (left) and Fig. 9 (right)). Next section of the pile wall was anchored in two levels (l = 110 m; Fig. 8 (right) and Fig. 9 (left). The very last section (l=34 m) was only anchored in one level.

Long-term monitoring (e.g. by inclinometer measurements) performed and documented by Popović (2017) revealed that creep deformations decreased with time and that both retaining structure and drainage has been working satisfactorily over the years.

Fig. 9. Rebernice Landslide. Construction phase. Anchored bored pile wall linked by cap beam (left) and pre-stressed anchored reinforced concrete grid beam structure (right) (Photos: Boštjan Pulko).

4. Miscellaneous Case Histories, Europe

4.1 A2 Motorway, Degendamm, Austria

In the scope of the construction of motorway A2 in Lower Austria massive slope movements were initiated by the embankment fill on a steep slope consisting of weathered mylonitic rock and phyllitic mica-schist more than three decades ago. Sliding planes at depth were revealed by inclinometer measurements. In a first step drainage of the ground beneath the embankment was executed and pre-stressed anchors were installed in 1985, thus, causing even more and deeper sliding planes. In the next rehabilitation phase socket structures comprising a diameter of 5.5 m were installed with a length of more than 50 m. Moreover, 160 triple SBMA-anchors (single borehole multiple anchors) with anchor forces of 3,000 kPa and lengths of 85 m were installed near the existing anchors in 2008/09. The contingency plan contains additional anchors to support the socket elements on top if needed. Observational method including overall slope monitoring (anchor forces, inclinometer and geodetic measurements, discharge of drainage water, etc.) has been continuously applied since the beginning of the construction works more than three decades ago.

4.2 Egnatia Odos Motorway, Bridge T, Greece

In the scope of the construction of the motorway "Egnatia Odos" in the north of Greece in the year 2000 a slope failure occurred endangering the already built bridge foundation. Soon after the sliding rock mass was entirely removed and the steep slope was stabilized with pre-stressed anchors in combination with vertical anchor ribs. The unfavourable orientation of discontinuities had to be taken into account for the assessment of inclination of the anchors. The endangered pier foundations were protected by an arched contiguous bored pile wall, thus, taking effect as a shell-like structure. The bridge is located in a major seismic risk zone (II) of magnitude 6.2 (in 80 years), thus, a horizontal design acceleration of $x_g = 0{,}239$ g had to be taken into account for the design of the retaining structures.

4.3 S6 Motorway, Tunnel Niklasdorf, Austria

The Tunnel Niklasdorf in Styria (Austria) is a near-surface tunnel along a steep creeping slope consisting of quartz schists and phyllite schists. Already during tunnel construction from 1982 to 1986 massive slope movements were observed, which did not significantly decrease or stop over the years. First of all elliptical sockets were installed in 1984/85, moreover, in 2003/04 stone columns were installed behind the shaft elements to provide a better drainage. In 2007 massive damages (deep open cracks) in the sockets were detected. In a next step, sockets were pre-stressed by vertical anchors in 2007 and 2008. Nevertheless, the monitored slope movements (inclinometer measurements since 1987) could not be stopped and caused significant deformations and deep cracks in the tunnel lining so that the tunnel had to be temporarily closed. In 2011/12 two alternative approaches for slope stabilization were investigated: installation of additional deeper sockets including drainage wells or removal of the sliding rock mass comprising a thickness of 11 m to some hundred meters up the slope. Numerical analyses disclosed that additional sockets take an unfavourable effect to the tunnel since the mobilization of the socket could cause unallowable additional deformations of the whole tunnel. Consequently, the creeping slope mass of more than 500,000 m³ was recently removed.

4.4 D1 Motorway, Fricovce – Svinia, Slovakia

In the scope of the tender phase (2011) prior to the construction of the motorway D1 in Slovakia alternative approaches for stabilization of two active and potential landslide areas in Flysch ground were considered. In addition to intended drainage drillings in parallel to the slope, temporary soil nailing and permanent retaining walls it was recommended to carry out surface drainage and toe counterweight fillings, and, moreover, to install anchored pile walls (dowel effect), anchor grids, anchor beams and anchor ribs. Proposed pile walls consisted of bored piles (diameter 150 cm, axial spacing 2.5 m) with an embedding depth at least 10 m beneath the deepest sliding surface.

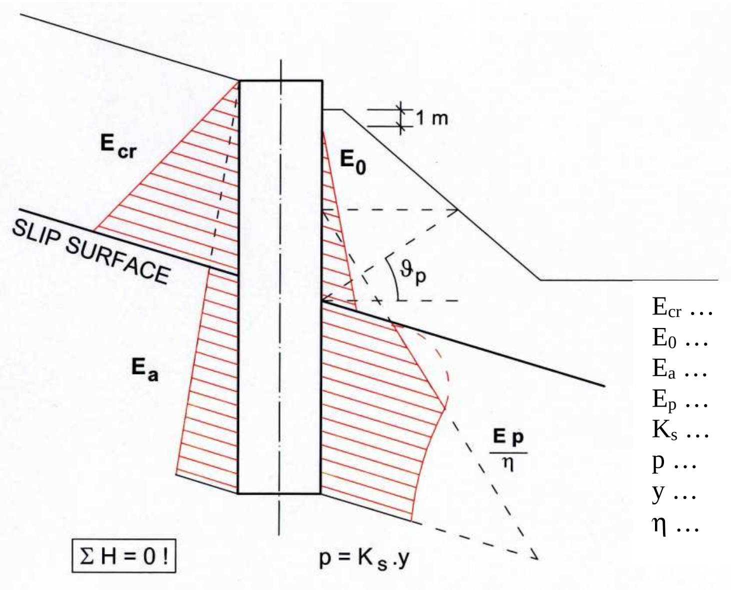
4.5 Sebes-Turda Motorway, Lot 4, Romania

The motorway Sebes-Turda (Romania) crosses Stejeris Lake, an artificial lake arranged in 1984 for fishery. In tender design a bridge with a length of more than 400 m was considered for crossing the lake. The ramps of the bridge involved the execution of certain embankment fillings on soft soil identified to have a weak bearing capacity. Settlements of the bridge ramps of about 100 cm were expected. Therefore, an alternative approach to tender design was proposed by lowering the motorway level as well as filling up the lake on the lakeside of the motorway. The works for partial filling of the lake was started in February 2015. During earthworks cracks at the top of the filling were observed. Furthermore, large settlements of up to more than 100 cm were monitored. Stability calculations revealed that failure occurred in undrained mud and soft clayey material ($\varphi_u = 0$; $c_u = 15$ kPa) due to the embankment filling procedure. Moreover, soft soil was squeezed out beneath the embankment toe. Consequently, six alternative approaches were investigated ((1) overload ballast, (2) vertical drains in combination with overload ballast, (2a) vertical drains combined with overload ballast and pile wall at the side of the embankment slope, (3) pile foundation, (4) ground improvement, i.e. by Deep Soil Mixing (DSM), (5) installation of stone columns [vibro replacement technique], (6) bridge structure on horizontally loaded pile foundation instead of embankment). Finally, only the bridge structure resting on horizontally loaded piles proved to be feasible. However, the stabilization measures have not been executed yet.

5. Conclusions

In the paper case histories including piles and pile-like structures were presented to be applied for retaining structures in slopes. The main focus was on infrastructure projects in creeping slopes. Two case histories from Austria and Slovenia were presented in detail. Miscellaneous projects from European countries concentrating on various geotechnical aspects complemented the contribution.

Piles, structures made of piles, and pile-like structures are useful and manifold applicable structural elements to support deep excavations and cuts in slopes, and to retain creeping or sliding slopes, not uncommonly in seismic areas. Depending on the static system and the dimensions the structural elements transfer forces mainly by shear ("dowel") and/or mainly by bending ("beam") to the ground. In numerous cases they are particularly effective in combination with other structural measures like (pre-stressed) anchors and/or drainage systems.



E_{cr} … creep earth pressure
E_0 … earth pressure at rest
E_a … active earth pressure
E_p … passive earth pressure
K_s … (horizontal) modulus of subgrade reaction
p … mobilized (horizontal) earth pressure
y … (horizontal) displacement/deformation
η … safety factor

Fig. 10. Exemplary distribution of earth pressure and earth resistance on a socket due to creeping pressure. Idealised calculation model (Rebernice Landslide) (Adam & Brandl, 2003).

It is strongly emphasized that design has to be done strictly based on comprehensive ground investigation and soil testing. Calculation methods have to be selected carefully. If limit equilibrium design calculation procedures are applied, it has to be taken into account that there are limitations with respect to deformation calculations. Static system of retaining structure, and in particular magnitude and (re)distribution of earth pressure (active, passive, at rest, creep) always requires plausibility check and engineering judgement. 0 shows exemplary the distribution of earth pressure and earth resistance on a socket due to creeping pressure (idealised calculation model). 0 depicts a novel approach for modelling creeping landslide constrained by a retaining structure.

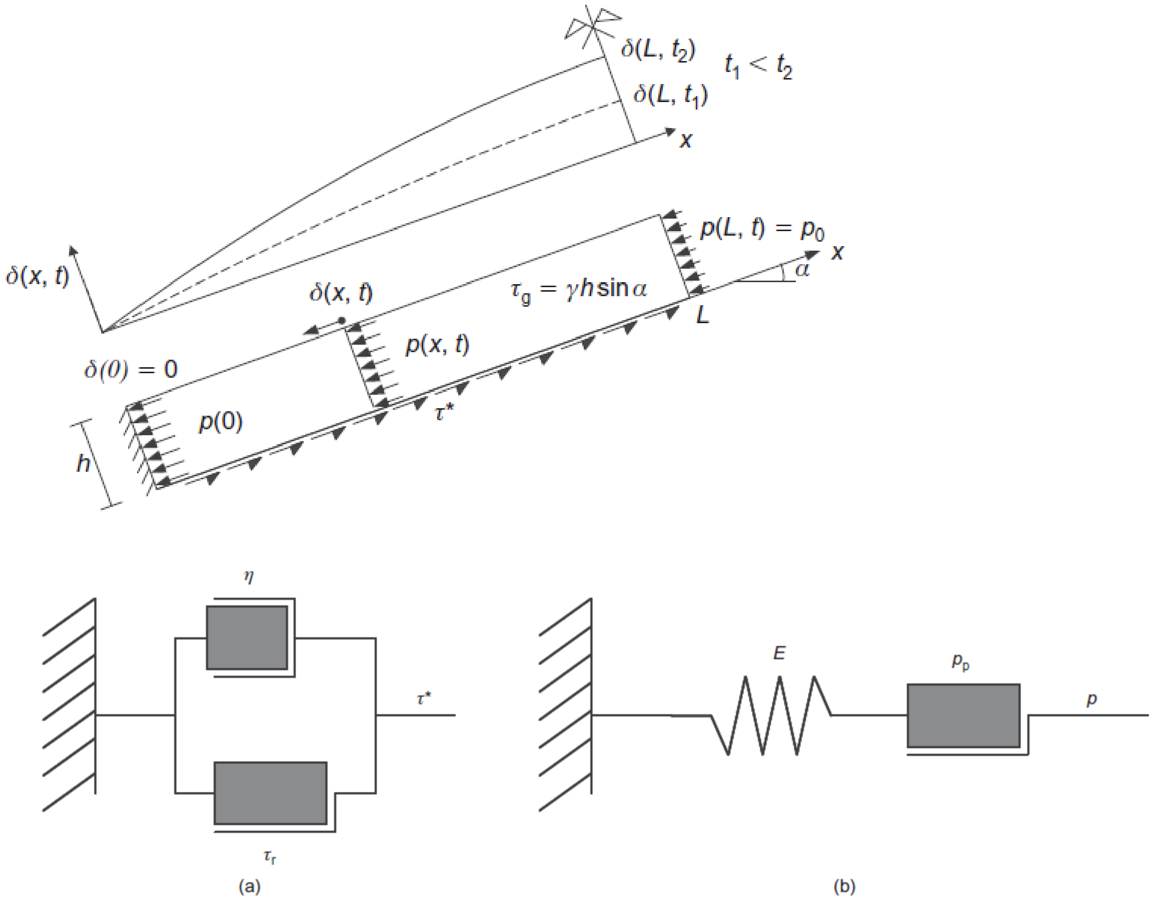


Fig. 11. Innovative approach for modelling creeping landslides. Schematic layout of landslide constrained by retaining structure (top). Schematic constitutive behaviour of soil: (a) on the sliding surface; (b) in the sliding layer (bottom) (Puzrin & Schmid, 2012).

If numerical analyses are applied, complex constitutive modelling helps to describe the behaviour of the soil-structure interaction more precisely and to perform deformation-based calculations. However, there is less experience compared to conventional calculations and the selection of input parameters needs sophisticated consideration and knowledge.

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LATERAL SOIL SUPPORT ENHANCED BY PILES

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Abstract

Retaining walls are inherently structures retaining soil, i.e. providing lateral support with sufficient moment capacity to resist the action of lateral load from soil stresses. In contrast, piles are traditionally single structures with the ability to resist axial loads. The latter is particularly true for the vast majority of piles, i.e. precast concrete piles with a relatively small cross sectional area (typically up to 0.45 by 0.45 m²). In practice, this black or white approach is not feasible and piles are often relied on to resist lateral loads and walls to absorb vertical loads.

Traditional steel sheet pile walls are very efficient as retaining walls but questionable in relation to vertical point loads, where it is more cost effective to add dedicated piles with high axial capacity to carry the vertical load.

However, using large diameter tubular steel piles or bored cast in place concrete piles it is possible to offer both lateral and axial resistance at the same time. This is important where inclined piles are not possible due to geometrical restrictions or non-desirable in seismic areas where these will attract too much load (due to the axial stiffness). In the lecture, lateral soil support enhanced by vertical piles is demonstrated by three diverse case histories:

- Reinforcement of an existing steel sheet pile wall quay structure by H-piles installed in front of the wall to substitute the passive pressure reduction after dredging in front of the wall
- Vertical tubular steel piles driven by innovative plugging procedure to resist mooring loads for rock loading facility
- Tubular steel piles used as inclusion piles to reinforce soft soil strata for suspension bridge tower foundations subjected to very onerous ship impact and seismic loads

1. Reinforcement of Existing Quay Structure by H-piles

Before Global Warming was on the agenda, the reliance on oil was a big issue and a majority of the Danish power plants were converted to coal burning after the oil crisis in 1973! In Stigsnaes, south-west Sealand (see Figure 1), this conversion was just completed in 1979 when the next energy crisis occurred.

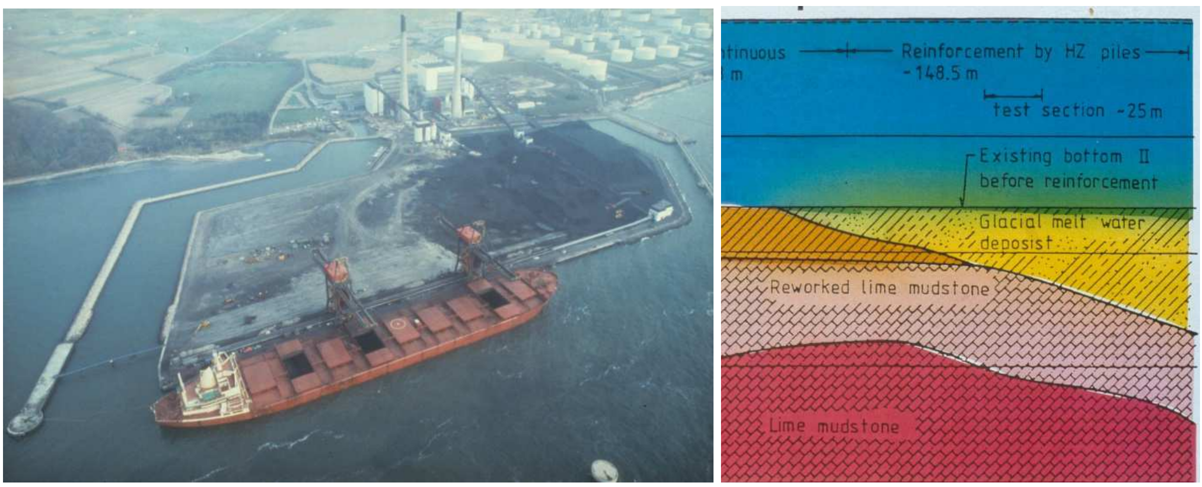

Figure 1. Overview of Stigsnæs Coal Terminal, Denmark after extension; (b) Schematic geological profile at H-pile section

In order to allow vessels up to 180.000 tons to berth at the quay, implying huge cost savings, an additional draught of 2 m was required. The project accepted involved a “Münchhausen” type of reinforcement project where a new wall was established with a clearance of 1.35 m from the original quay wall as seen in Figure 2. Belval H 1000L steel piles at 1.35 m centres were driven to level -30.5 m over a length of 150 m. To transfer the passive resistance afforded by the H-piles to the wall a concrete beam was cast in situ between wall and H-piles.

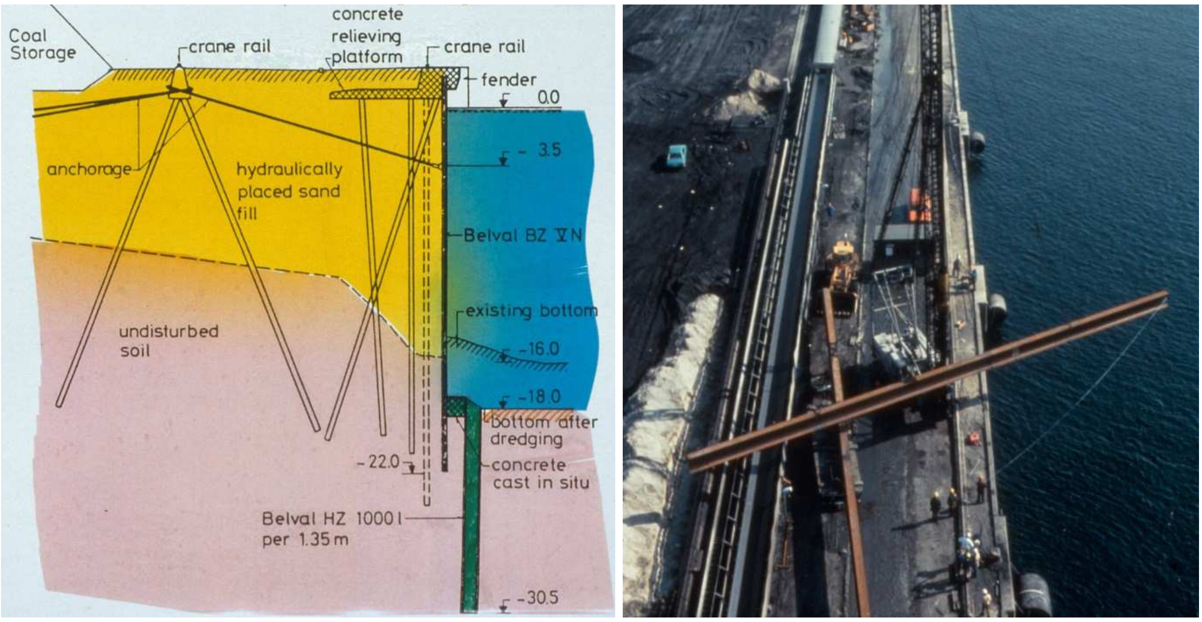

Figure 2. (a) Cross section of quay wall; (b) Driving of H-steel piles

The monitoring of the twist and rake of the H-piles and the performance of an automated control system along a test section of the wall are described together with an evaluation of the displacement of the wall due to coal storage after implementation of the reinforcement.

2. Musandam Rock Loading Facility

The Musandam rock loading facility is located close to the Strait of Hormuz, in Aisha Bay, Oman, see Figure 3.

Ø1422 mm tubular steel piles with 25 mm wall thickness were driven into a sequence of marine sand, Calcerenite (cap rock), Limestone gravel in clay matrix, Calcerenite/Calcisiltie and Limestone. To reduce pile length an innovative plugging method was developed, primarily to provide sufficient axial capacity.

However, the fender piles for the Berth were solely subjected to horizontal loads. Due to severe seismicity, these piles were also installed as vertical piles. As the Caprock close to the seabed invoked very high dynamic capacities followed by a drop to near zero, it was decided to carry out horizontal load test of the piles in order to validate the design using L-pile calculations and based on empirical p-y curves.

In the lecture the plugging method (see Figure 4) and the horizontal load test set-up (see Figure 5) are described together with the validation of the design principles.

Figure 3. (a) Location and overall layout of facility; (b) Driving of piles in progress

Figure 4. (a) Pile with gravel surface after final driving, (b) gravel excavation and preparation for casting of concrete on top of gravel

Figure 5. (a) Horizontal load set-up (testing two fender piles simultaneously)

3. Inclusion piles for the Izmit Bay Bridge tower foundations

The fourth longest suspension bridge in the word (main span 1550 m), the Izmit Bay Bridge in Turkey, was inaugurated in 2016 (see Figure 6).

It is placed in an area with extreme seismicity with horizontal loads typically of the order 400 MN and subjected to very high ship impact loads (of the order 246 MN) on the tower foundations. The water depth at the towers is of the order 40 m and the top 20 m of the subsoil exhibited low shear strength ruling out a direct foundation. A piled foundation would be very costly and difficult to verify for the very high seismic loads. The solution was a hybrid foundation solution with reinforcement of the inferior subsoil by 195 Ø2 m tubular steel piles and a 3 m thick gravel bed acting as a horizontal fuse between the concrete caisson and the freestanding piles.

Figure 6. (a) Aerial view of completed bridge; (b) Foundation principle for tower foundations with inclusion piles

In the lecture the project is briefly described with emphasis on the verification of the tower foundation and the horizontal load transfer afforded by the vertical inclusion piles.

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LATERALLY LOADED PILES – FAILURES AND DESIGN RECOMMENDATIONS

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Abstract

Case histories of different countries with slope failures are presented where rigid inclusion have been designed as ground improvement method to increase slope stability. However, they should have been designed as piles including reinforcement. Back analyses with FEM including deformations and bending moments of the rigid inclusions show the necessity of reinforcement at least near the toe of the slope.

Design risks increase with the application of non-ductile methods with small column diameter, like rigid inclusions, because the ultimate limit state is controlling the behaviour. In order to mobilize high skin friction, the lower and upper end of the column have to fail during punching in the soil below and the load transfer platform above. Even a small variation in material parameters, system geometry or loads may lead to a complete loss of slope stability and progressive failure, resulting in expensive damages and time consuming repair works.

International Standards and Recommendations

Unfortunately there are no international standards on the design of rigid inclusion. However, in some countries there are quite detailed recommendations. The first recommendation on this subject has been published in France under the name ASIRI in 2012. In Germany and Poland similar recommendations are under preparation.

Two design cases (domains) are distinguished in the first design step.

1. If rigid inclusions are necessary to fulfil bearing capacity and stability they have to be designed as piles (domain 1).
2. If rigid inclusions not are necessary to fulfil bearing capacity and stability they can be designed as ground improvement just to reduce settlements (domain 2). Here all ultimate limit state calculations can be fulfilled without any columns.

An overview is provided by Bohn (2016) and Katzenbach et al (2013) including all relevant partial safety factors in different countries.

Case History from Singapore

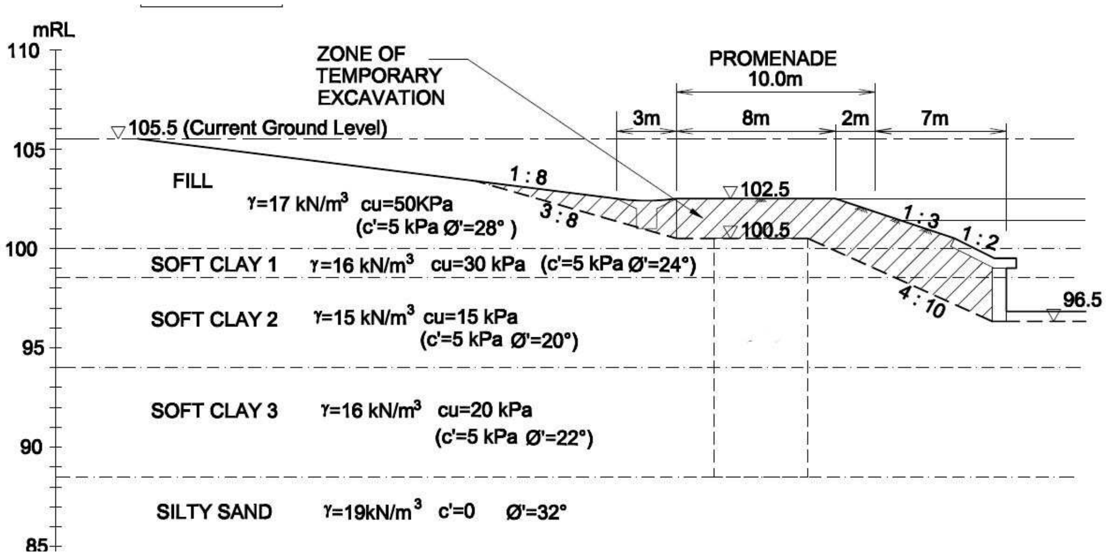

Figure 1. Excavation with rigid inclusions underneath the promenade inside the dotted lines

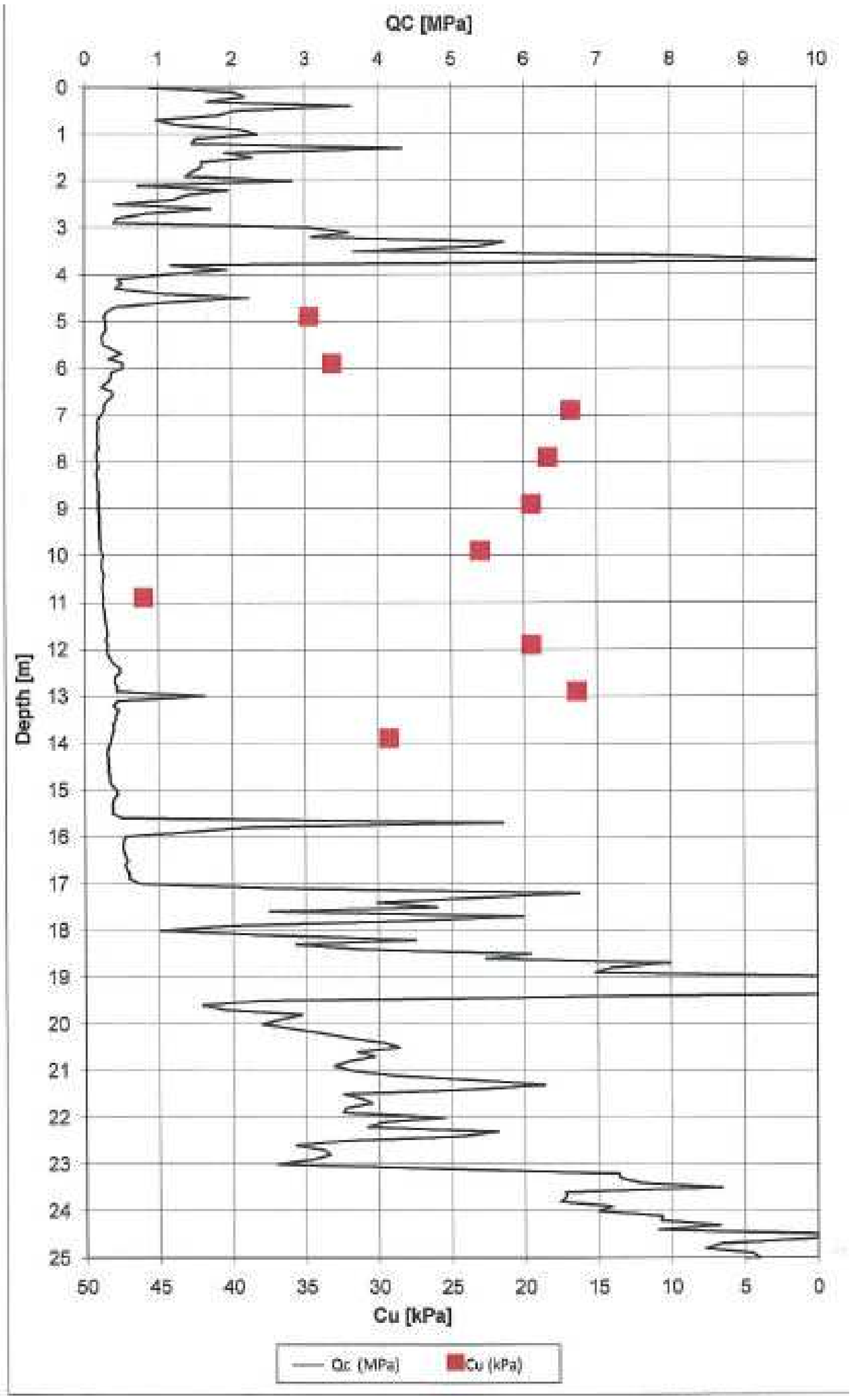

Figure 2. Cone penetration tests and vane tests

If rigid inclusions are necessary for slope stability it is proposed to design them as piles and to check the amount of reinforcement needed (Wehr 2011). This has been demonstrated by checking a 4.5m deep excavation in soft clay in Singapore, see fig.1 which was initially stabilized with rigid inclusions. Cone penetration tests in fig. 2 demonstrate the softness of the soil with cone resistances lower than 0.5 MPa and undrained cohesion as low as 15kPa. Shortly after the excavation was completed the slope failed in several sections. A back calculation with FEM-Plaxis yielded that the rigid inclusions broke due to excessive bending moments, see fig.3. Reinforcement bars in each rigid inclusion would have been necessary to stabilize the slope with a sufficient factor of safety.

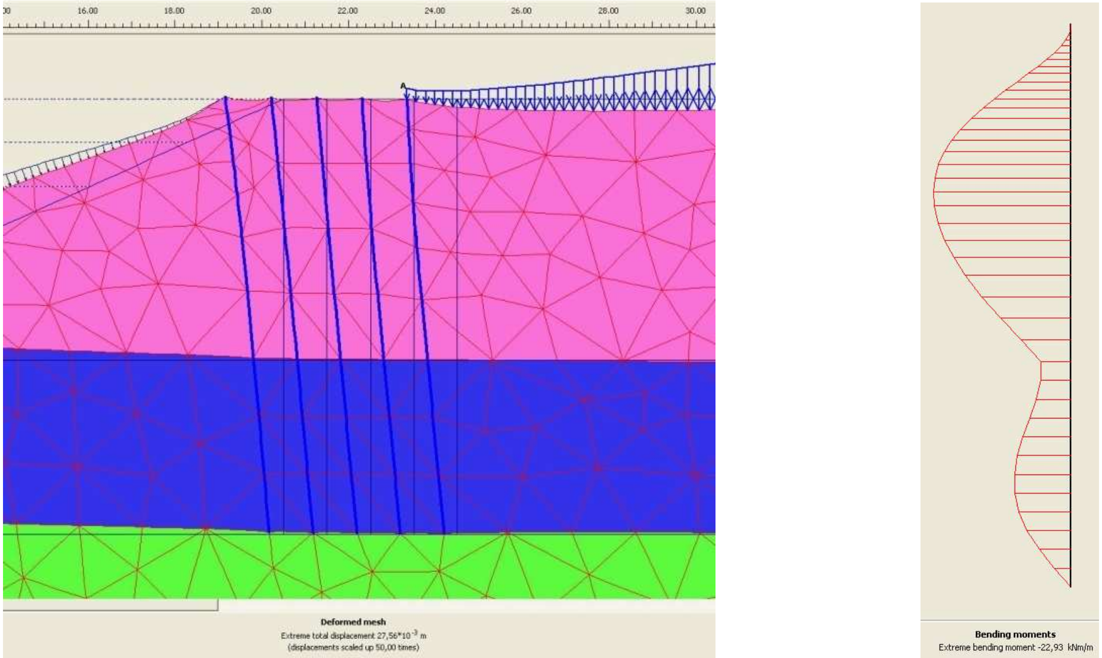

Figure 3. Back calculation with FEM: 3cm displacement of the column top and 23 kNm/m bending moment

Other Case Histories

were presented recently. A very impressive slope failure including repair works has been highlighted by Wieser et al. (2016). In this case from Poland a 5.6m high road embankment was constructed on 11m thick cohesive and organic soil. The soil was improved with unreinforced rigid inclusions and the embankment above was reinforced with geotextiles. In the first 7 months settlements of up to 75cm occurred. An FEM analyses yielded buckling and cracking of the columns due to lateral spreading of the embankment. In order to repair this 35m long section of the embankment reinforced CFA (continuous flight auger) piles were chosen with a reinforced concrete slab on top. Until now only 1cm of settlement occurred displaying the effectiveness of the piling system.

There are more case histories from Spain, Germany and Austria where similar experience have been made.

Mass movements of slopes including creep should not be designed with rigid inclusion because an embedment into the underlying competent soil or rock layer is necessary. Only large diameter reinforced piles are suitable, i.e. (Lenzi 2009).

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RETAINING STRUCTURES FOR HYDRO POWER PLANTS ON ALPINE RIVERS

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The GKI („Gemeinschaftskraftwerk Inn“) is a unique, run-of-the river power plant in the Swiss-Austrian border region. The largely invisible, since mainly underground built, power plant extends from the village Martina in the Swiss municipality Valsot over the area of seven municipalities in the Oberes Gericht region in Tyrol. In essence, the GKI power station consists of three elements: the upper storage space with an Ovella dam structure, penstocks and the powerhouse. In the border area between Martina and Nauders, the dam structure of 15 m height has a backwater length of 2.6 km in the Austrian-Swiss border area. From the accumulated water up to 75 m^3 / s will be diverted in 23.2 km long penstocks leading to the turbines Prutz / Ried.

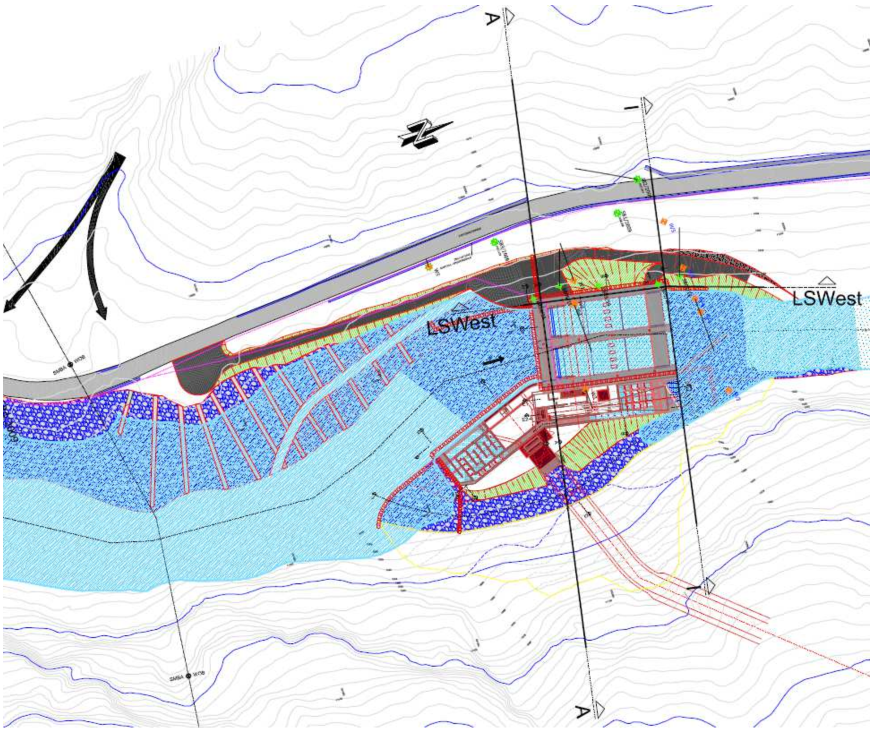

Fig. 1. Ground view of an Ovella dam structure with a backwater (on left) and penstocks exiting towards lower right corner

The site for the power station of the GKI was selected in a narrow “canyon-like” valley cross section of the river Inn. The rocky slopes (Kalkglimmerschiefer, Kalkmarmor) rising steeply several hundred meters over block-dominated river bed. Between the river bed and rocky flanks, the deposits of rockfall often several m^3 are noticeable. The rocky bottom in the canyon runs more than 70 m below Inn as shown on Fig. 2.

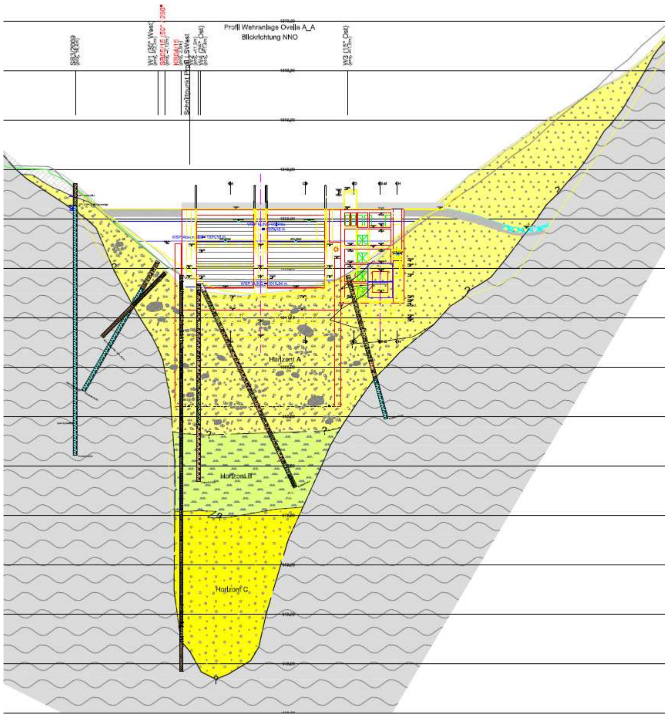

Fig. 2. Cross section through the Inn valley. Up to 70 m of blocky sediments bellow the river bed

For an Ovella Dam an excavation pit of 16 m depth was to be constructed in most demanding geological conditions (blocks). A secant pile wall and a contiguous pile wall (pile diameter of 1,2 m) with jet grouting sealing between the piles were selected as the most optimal solution. The depth of these walls reached up to 45 m. Drilling deviations proved to be the most important issue. In order to reduce deviations during jetting works, a predrilling with water powered Wassara drilling technology was used. All retaining walls were tied back with up to 4 levels of strand anchors which were all drilled below ground water level. The deepest anchor level was drilled 14 m below GWL.

Fig. 3. Overall view of the site

Fig. 4. Exposed blocks during excavation works

Very difficult soil conditions, combined with demanding depth of the pit and very congested site conditions resulted into an extraordinary project in which many novel solutions were adopted.

Fig. 5. Approaching final excavation level

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PILES AS RETAINING STRUCTURES – COMMON PRACTICE IN THE CZECH BUILDING INDUSTRY

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1. Introduction

1.1. Types of Commonly Used Pile Walls

Pile as a structural element was originaly developed and regularly proposed as a part of building foundations resisting primarily vertical load, eventually followed by bending moments resulting from fixing the pile head into the supported pilecap. Use of piles as an element resisting horizontal forces and bending moments, i.e. as a retaining structure, was initiated later.

On the Czech building market pile walls of reinforced concrete large diameter drilled piles are specifically applied, either simple structures as contiguous pile wall (where individual piles are spaced at 1.1 to 2 pile diameters apart installed in a phased sequence), or tangent and secant pile walls (where individual piles are spaced at 0.7 to 0.8 pile diameters apart also drilled in a phased sequence).

From the technical point of view there are of-course some rival technologies within the scope of building industry, most commonly diaphragm walls and soldier pile walls. Thinking about details on structural functioning (e.g. the structure rigidity), use of bentonite slurry, ability to penetrate obstacles or hard soil strata, rate of production progress, etc., the advantages and disadvantages of the particular techniques are obvious and each mentioned one has its own favourable employment in different conditions.

In the following paper we are going to present some case histories of technically convenient and budget-wise use of piles as retaining structure.

1.2. Positives of Piles in the Retaining Structures

The pile wall structures are advantageous from many reasons:

- Utilization of material (concrete, steel) can be tuned up by proposing diversed pile diameters and/or piles can be spaced at optional c/c distances the single piles to be fully exposed.
- Retaining structure of contiguous piles with arching action of the soil in between them allows free flow (leakage) of underground water - in low permeable soils piles can retain earth pressure only and dewatering of excavation pit can be employed reducing substantially the structure loading.
- Considerating the minimal restriction of water flow through the soil in between single piles in case of contiguous pile wall such structure fits exactly to the basic call of landslide nature and represents the ideal kind of stabilizing structure .
- Secant piles form a watertight retaining structure – so the pile wall structure is perfectly variable.
- The proper drilled borehole usually cased with steel tube casing is very stable and consequently causes negligible deformations of the adjacent soil mass.
- From the environment protection policy there are no substantial difficulties within the production process.
- Speaking about moderate dimensions of pits the production procedure is in principle rather simple comparing to other techniques, so that there are many competitive companies – thus, this results in reasonable low/acceptable price of the structure.
- For special constructions placing of piles can be very flexible to fit complex site boundaries or in-built construction perimeter so that we often use secant pile walls for circular or elliptical shafts where there an arching effect of the whole structure brings the compression in horizontal across the wall and no struts or anchorage are incorporated.

- Comparing to driving soldier piles or excavation of slurry wall trench by a grab the penetration power of the rotary drilling through soil comprising rigid debris or through rock layers using appropriate drilling bits is of higher performace (excluding hydrophraise). Pile drilling has the ability to go through underground obstacles such as steel, reinforced concrete, rocks and masonry while minimizing risk of construction induced settlements to neighbouring structures.
- Piling offers comparatively lower level of vibration and noise.
- Piles as massive reinforced concrete element serve naturally long life, they can be implemented as s permanent structure. Especially permanent retaining structures along road or railway lines in excavations are very popular. Also some examples of outer basement walls of a parking houses can be effectively formed by the tangent or secant pile walls.

1.2. Drawbacks of Piles in the Retaining Structures

- As we are using solely large diameter drilled piles we must admit that from the structural view a circle does not represent an optimal section shape to resist bending moments. Despite extensive skills of the personnel on sites we do not habitually propose orientated reinforcement cages and the bars are placed regularly along the cage perimeter - such placing of single bars is not perfect and does not allow economical utilization of steel.
- Installation of pile wall goes usually under little bit lower rate of progress on site comparing to diaphragm walls or soldier pile walls (especially when driven).

2. Examples of Pile Wall use in the Czech Republic

Massive Collapse of Railway Tunnel Structure under Construction near Březno Village

Information on the Project

During the tunnel excavation works by the technology of prevault method a large collapse occured and the tunneling machine was burried deep below terrain.

Consequently remedial works were scheduled in two phases – first several diaphragms were erected to divide the collapsed filled-in tunnel tube into sectors to simplify works, secondly a large diameter deep shaft was built to brace excavations in order to heave the burried prevault tunneling machine.

Tunnel Diaphragm Structure

Description of the structure

Seven rigid diaphragms across the tunnel tube at distances of 9.0m were installed in dimensions 16*16m in the vertical plain by the technique of piling secant pile wall. The lower edge of the diaphragm was approx.40m below terrain, the upper part of boreholes was filled with lean low strength concrete (primary piles) and soil respectively in case of secondary piles.

Geology

- sandy-clayey mixture to the depth of 4.5m
- clayeye gravel with water down to 7.0m
- clay of high plasticity to 28.0m, in some levels moulded with the tunnel lining debris
- claystone to 29m with pieces of steel
- stiff clay with concrete of tunnel invert to the depth of 32.5m
- lower stiff claystone of the quality R6

Reasons for pile wall employment

Substantial part of the pile drilling penetrated the collapsed tunnel lining debris, i.e. concrete mass and reinforcement bars.

Structural details

Seven vertical diaphragms of 17 drilled piles dia 1180mm, drilling to the depth 35.15-37.20m, lenght of proper piles 16m (9 primary of plain concrete C16/20, 8 secondary od reinforced concrete C16/20), c/c distances 1.0m.

The verticality of pile drilling was monitored by the laser alignement technology.

Large Diameter Shaft

Description of the structure

The shaft was of circular layout internal dia 19.80m, designed as secant pile wall with 4 compression rings inside the pit, total depth of the shaft 40m below the terrain.

Geology

- sandy-clayey mixture to the depth of 1.2m
- clayeye gravel with water down to 5.2m
- very stiff clay of high plasticity to 17.7m, in some levels moulded with the tunnel lining debris
- very stiff clay to claystone to 23.2m
- lower claystone weathered R6 to sound R4

Fig. 1. View on the site

Reasons for pile wall employment

Secant piles form an ideal circle bringing adequate bearing capacity of the deep shaft walls without any need of struts or anchorage implementation.

Structural details

Secant pile wall was performed of drilled piles dia 1180mm, length 39.6m each. Total No of piles 70 (half of them primary- plain concrete and secondary – reinforced concrete, both concrete C25/30).

The verticality of pile drilling was monitored by the laser alignement technique.

Fig. 2. View into the shaft

Fig. 3. View from the bottom

Metro Station Anděl, Prague 5, Line B – Wheelchair Accessible Entrance

Information on the Project

In the frame of upgrading the accessibility of Prague Metro stations a new entrance for handicapped/disabled to the Station Anděl was planned and in 2014 realized. In order to incorporate a new special lift into the entry system of the already existing station and transport the passengers directly on the metro platform shaft of a depth of 30m below street pavement was excavated.

Description of the structure

The shaft of elliptical layout had been designed, dimensions of the main elipse axes 8.9*6.9m.

The vertical structure of the shaft was formed by secant concrete pile wall down to approx. 2/3 of the total shaft depth, lower part of the pit was excavated step by step in the way of classic excavations in stable formations of sound rock with primary lining of reinforced shotcrete on the wall surface.

To eliminate any leakage through the rock layer just below pile tips grouting works were performed with cement-bentonite mixture.

Reinforced shotcrete was installed on the pile wall surface as a final lining of the retaining structure.

Geology:

- man-made fill to the depth of 5.2m
- sandy to gravelly deposits to 14.4m
- lower shale – surface decayed, 2.5 weathered and slightly weathered and sound shale lower
- GWL at the level 8.0m

Reasons for pile wall employment

The elliptical shape of the pile wall brings enough bearing capacity - the earth and water pressure around the pit result in compressive pressure along the center line of the pile wall in horizontal. Supporting ring beam inside the pit was erected only at the level of 2.0m above the pile tips and from that level the mentioned grouting works were perfomed.

Structural details

Retaining structure – shaft of 26 piles (reinforced concrete C 25/30XA2) dia 1180mm c/c 953mm, length of primary piles (not reinforced) 20.75m, length of secondary piles (reinforced) 19.95m, embedded into rock layer quality R4 min.2.5m.

High level of alignement (±20 mm at the working platform), and drilling verticality accuracy (max. 0,5 %) was applied to ensure a wall watertighness.

The verticality of pile drilling was monitored by the laser alignement technique.

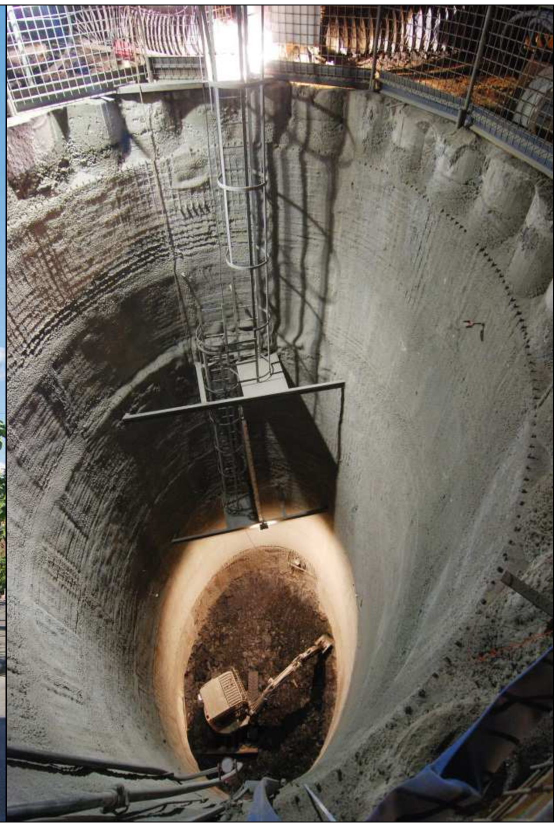


Fig. 4. Site in the built-up area

Fig. 5. View into the shaft

Metro Station Florenc, Prague, Line B – Wheelchair Accessible Entrance

Information on the Project

The mentioned Project was implemented within the same aim of Prague City authorities to facilitate entrance to the station for handicapped/disabled people. The lift shaft is located close to the backbone of the Prague road network and in the dense built-up area of the City center.

Secant pile wall was performed to retain the earth and water pressure to the depth of sound bedrock, lower part of the pit was excavated in the way of classic excavations in stable formations of sound rock with primary lining of reinforced shotcrete on the wall surface.

Description of the structure

The shaft of elliptical layout had been designed (dimensions of the main elipse axes approx.8*6m) with the secant pile wall to the depth of 27.8m, lower to the depth of 38.8m the shaft was excavated in circular shape by the classical mining method with shotcrete primary lining.

Down to the depth of the secant pile wall no internal compression rings were installed but the pile wall surface was covered and smoothened with reinforced shotcrete.

Geology:

- man-made fill and clay-sand mixture to the depth of 9.0m
- Quarternary sediments (watery dense to very dense gravels) to 18.0m
- ordovic shale weathered to 20.0m
- lower – sound shale
- GWL at the level 9.0m

Reasons for pile wall employment

The elliptical shape of the upper part of the shaft corresponds to the site space conditions and simoultaneously brings enough bearing capacity to the retaining structure.

Structural details

Retaining structure – shaft of 32 piles (reinforced/plain concrete C 25/30) dia 880mm c/c 670mm, length of piles 27.8m, pile tips embedded into rock layer quality R4.

High level of alignement accuracy (±20 mm at the working platform) and drilling verticality accuracy (max. 0,8 %) was applied to ensure the wall watertighness and to keep certain width of the adjacent piles contact.

The verticality of pile drilling was monitored by the laser alignement technique.

Fig. 6. View on the site   Fig. 7. Pile wall in the shale bedrock

Assembly Shaft Vypich, Prague Metro Line A Extension - Secant Pile Wall

Information on the Project

The whole Extension tunnels (pair) in the length of 6km were proposed as driven and more then one half of the tunnel length was driven by TBM.

Just for the assembly of the TBM machine the mentioned shaft was erected, during the TBM work in progress supply to the TBM machine was organized through the shaft, anyway the use of the shaft was temporary only withing the building period.

Description of the structure

The shaft is of circular layout 23.6m in inner diameter, retaining structures of secant piles dia 1180mm, c/c distance 885mm, concrete C30/37, depth od the shaft excavation 33.9m below terrain, length of piles 35.0m, total No of piles 88.

Four closed compression rings were installed as the excavation proceeded to the depth of the metro tunnel vault, lower, at the level of TBM operations side anchorage wales at two levels were concreted and anchored by strand anchors.

Geology

- 0,0 - 6,0 m – man-made fill (sand/clayey mixtures, sands, building debris)
- 6,0 – 26,0 m – fine- to middle-grained sandstone
- 26,0 – 31,0 m – claystone, siltstone
- 31,0 – 32,0 m – quartzite sandstone
- 32,0 – lower – silty shale

GWL at a depth of 20.4m in the sandstone layers.

Reasons for pile wall employment

Penetrating hard rock strata.

Utilization of circular shaft shape.

Structural details

Piles above the tunnel driving were shortened not to coincide with TMB cutting bits.

Laser technique were used to master the strict requirements on drilling verticality precision.

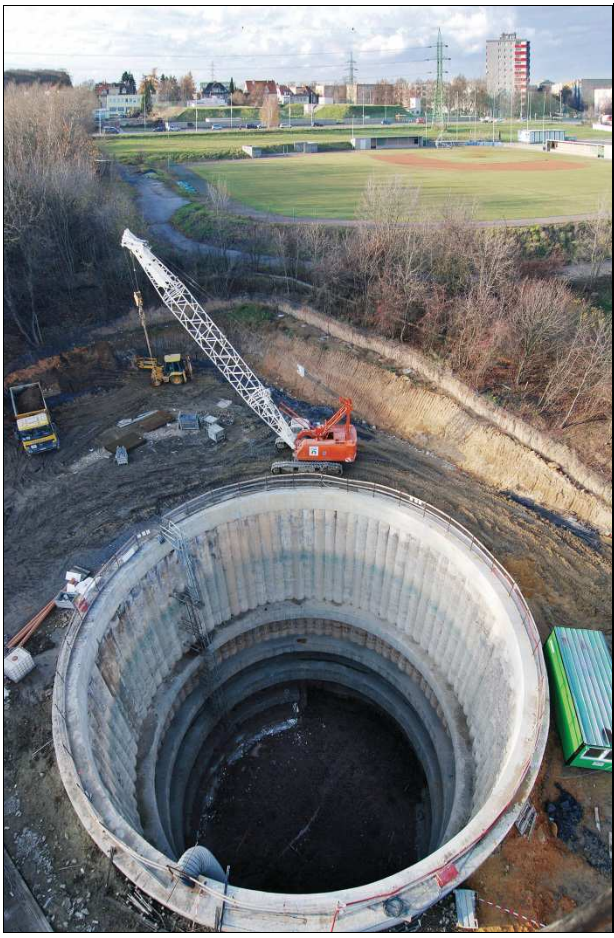


Fig. 8. View into the shaft

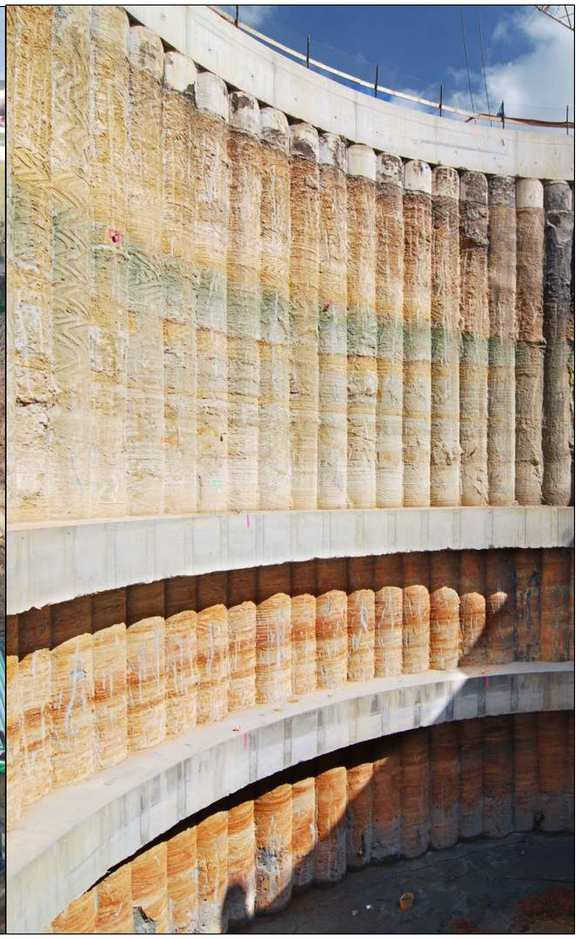


Fig. 9. Inside the shaft

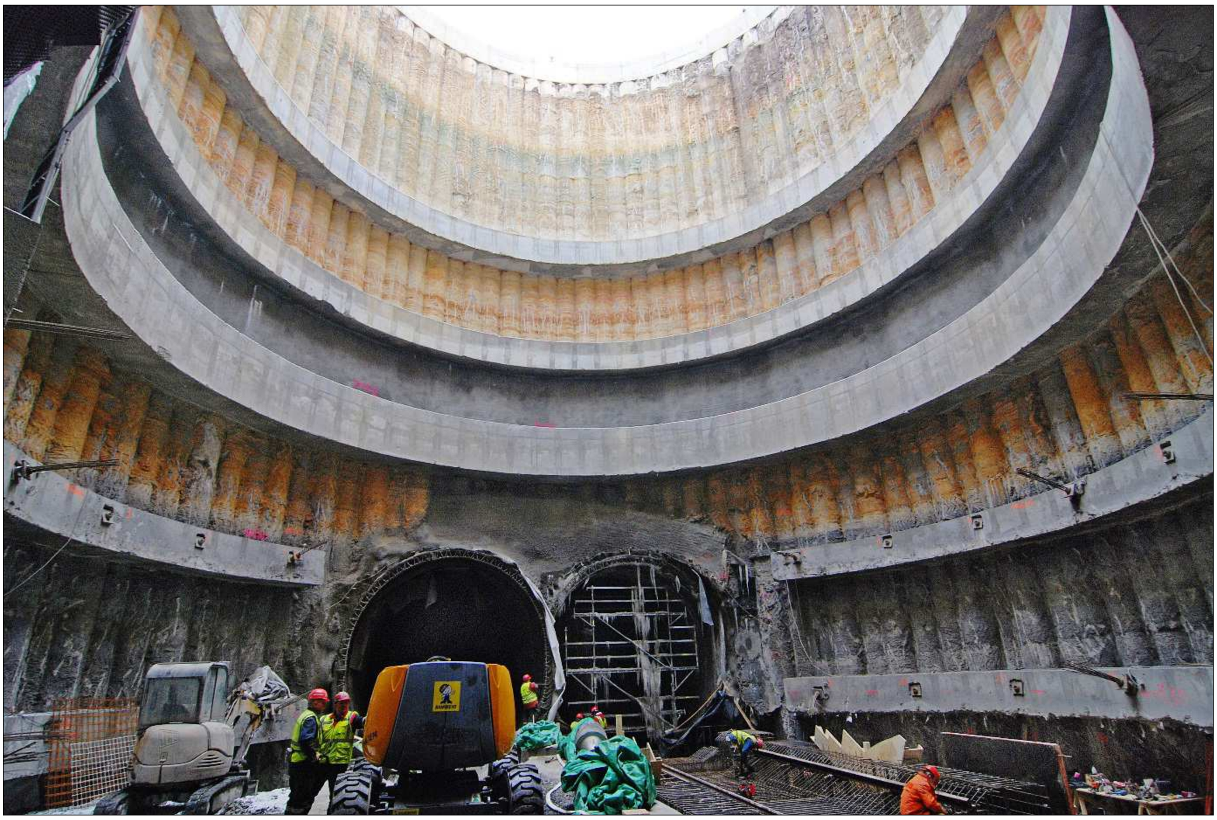


Fig. 10, 11.   Views from the bottom

Foundation Pit Motol, Prague Metro Line A Extension V.A – Contiguous pile wall

Information on the Project

The Extension tunnels terminate currently in the Prague district Motol and behind the terminal station of that name 460m long turning tracks for trains have been built. As the last installation on the tracks a foundation pit was erected for placing an air-handling engine house and a part of the pit retaining structure has been produced as a permanent structure (pile walls stabilized with permanent anchors) and will be used as a starting block in case of further extension of the respective metro line to the Prague Airport in the future.

Description of the structure

The shaft is of irregular layout generally approx.30*38.2m, maximum depth of excavation 24.9m.

Along the whole perimeter the excavation was stabilized with anchored pile wall of piles dia 900mm c/c distance 2.0m (concrete C25/30) with shotcrete in between them, prestressed strand anchors in up to 6 levels were installed in the course of the excavation works. Length of piles from 22.4 to 28.7m.

An integral part of the excavation stabilization is the adequate dewatering system – behind the shotcrete layer in between piles vertical drainage PVC tubes of dia 100mm (separated by geotextile membrane from the shotcrete) were placed and a collecting drainage system in the excavation bottom was organized.

Geology

Across the site a tectonic failure was found, types of soils and rocks are frequently varying.

The upper Quarternary part was represented mostly by sand and clay mixtures of changing strength properties, generally the lower 1/3 of the pile length was drilled in rock formations of shale type of different age.

GWL at a depth of 5.7 below the average terrain surface level.

Reasons for pile wall employment

Penetrating hard rock strata.

Possibility of reduction of the water pressure on the retaining structure due to dewatering of the pit.

Fig. 12, 13. Views into the pit

Fig. 14. Portal of the turning tracks tunnel

Modernization of the railway Line Plzeň – Rokycany, Ejpovice Tunnel

Information on the Project

The excavation works on the longest tunnel in the Czech raiway network near Ejpovice Village are in progress nowadays.

Approx. 720m long excavation in front of the tunnel portal „Homolka“ brings the raiway tracks deep into the proper tunnel and within the building procedure the pit serves also as an assembly room for completion of TBM machine prior to driving.

Description of the structure

The retaining structure was proposed as contiguous pile wall of U-shape where there both long sides are of permanent lifespan (permanent strand anchors) while the cross portal wall works temporarily only.

The depth od excavation reaches a value of 17m near the portal, width of the excavation goes from 30 to 50m.

Most of the perimeter of the pit was stabilized with pile wall od piles dia 880mm, c/c distance 2.0m., length from 5.0 to 23.9m (corresponding to the slope of the track in excavation). No of piles 481 with the total length of 6 045m.

Anchors in up to 3 levels prestressed on the massive reinforced concrete wales.

A dewatering system was implemented – behind the shotcrete layer in between piles vertical drainage tubes were installed and collecting drainage system in the excavation bottom was organized.

Geology

Quaternary sediments of sand-clay mixtures and gravelly, sandy and clayey soils and their mixtures of Tertiary age were found - the thicknes of these layers changes substantially along the excavation from 2 to 15m below terrain.

The bedrock is built with fine mica–shale with subtle upper weathered layer and with a rapid change to the slightly weathered shale. In that areas where the bedrock comes near the surface the thickness of weathering increases.

The area suffers from several tectonic failures.

GWL is not continuous and changes substantially from 0 to 11.5m below terrain surface.

Reasons for pile wall employment

Penetrating hard rock strata.

Possibility of reduction of the water pressure on the retaining structure due to dewatering of the pit.


Fig. 15. Pile wall


Fig. 16. TBM assembly

Fig. 17. Portal part of the pit

River Park Bratislava

Information on the Project

A new multifunctional complex of dwellings, offices, shops, hotels and more additional amenities was erected on the left quay of the Donau River in Bratislava, SK.

To enable excavation of 3 basements it was necessary to brace the foundation pit of the layout 274*54m, depth of excavation 12m below the original terrain, i.e. 6.5m below the water level.

Description of the structure

As to the water inflow the retaining structure ensures watertighness temporary only during the building period, it resists the earth pressure permanently.

Geology

- the Quarternary sediments are represented by sandy-gravelly and gravelly soils, highly permeable
- the bedrock of the granodiorite at the depth from 16.0m to 12m
- GWL at the depth of 6.5m

Reasons for pile wall employment

The retaining structure should have been watertight and simoultaneously should have penetrated into the bedrock layer even in parts of the foundation pit where the bedrock surface rises close to the excavation level.

Additionally a technology test proved that the gravels below the underground water are of of very scattered grain size, eventually the sand grains are missing. This phenomenon brings difficulties with excavating trenches of diapragm wall under bentonite slurry.

Structural details

Secant piles, total No 768, of dia 1180 and 880mm were performed on the site; drilled under steel tube protection, concrete C25/30 in both primary and secondary piles.

The wall was stabilized with one level of prestressed strand anchors.

Fig. 17, 18. View into the foundation pit

ISTANBUL NEW AIRPORT – REMEDY OF LARGE LANDSLIDE BY MEANS OF RETAINING PILES

Ing. Petr Kučera
SG Geotechnika a.s., Prague

Construction of new Istanbul airport started in 2015. The new airport is situated on the Black Sea shore in the area of former coal mine. Earthworks at the construction site with the estimated volume of 300 million cubic meters comprise construction of embankments with height up to 70 meters and cuts up to 75 meters deep, all in highly unfavorable geological conditions which can be partly compared to the North-West Bohemian coal districts as to their complexity. Almost the entire area of the future airport can be designated as potential landslide zone.

The reason for this conclusion can be seen firstly in the occurrence of soils with low shear strength and mainly with low residual shear strength and secondly in underground water flow through residual lignite strata. These conditions together with inadequate construction procedures resulted in numerous landslides of various depths and extents. One of the major landslides, which directly endangered the construction of the main airport terminal, was remediated with a set of measures including triple pile wall construction.

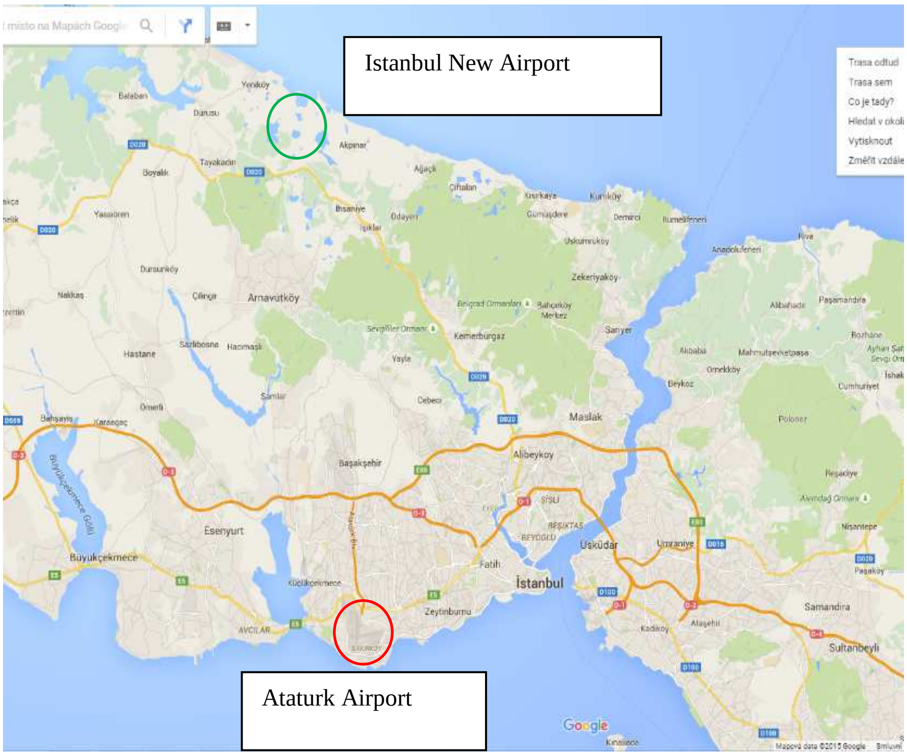


Figure 1. Geological conditions and the way of excavation

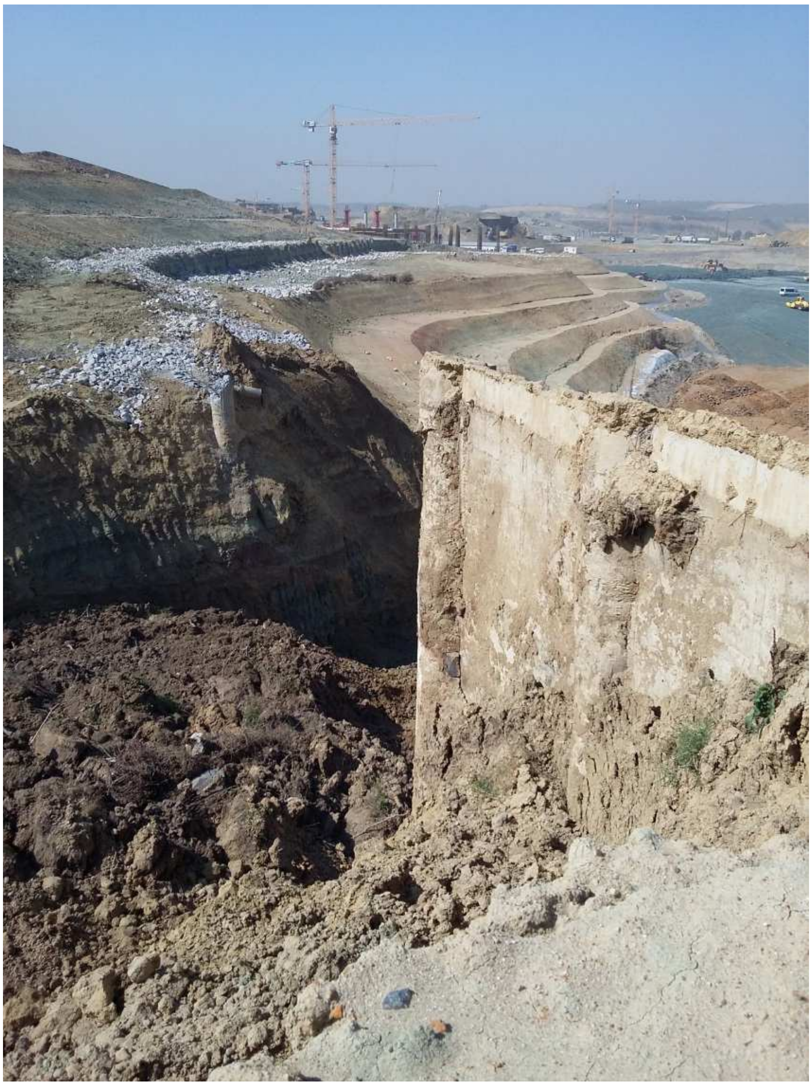


Figure 2. The landslide of pile wall protecting the former wather supply of Istanbul

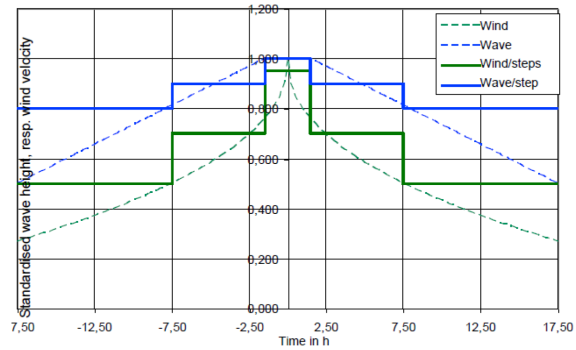
FOUNDATION OF A FLOATING OFFSHORE WINDMILL - INSIGHTS IN THE REALISATION OF A PILOT UNIT

Thomas Meier and Peter-Andreas von Wolffersdorff,
BAUGRUND Dresden

1. Introduction

A prototype of a floating wind mill has been planned to be constructed in the German Baltic Sea. An advanced principle of tension leg platforms has been designed using additional diagonal steel ropes to stiffen the submerged retaining structure consisting of steel ropes, connectors and open-end steel pipe piles (fig. 1). This system is meant to be installed in water depths ≥ 35 to 40 m.

Figure 1. GICON “Floating Foundation (SOF®)” and anchoring system (Adam, 2016)



DLC 6.1: Parking – idling, EWM & H_s50 yaw error = +/-8°	
Wind model	Turbulent wind model (EWM)
Wind speed (m/s)	V50, 1h
Intensity of turbulence	12%
Wind direction	0 degrees
Yaw angle	-8, 0. 8 at the beginning of the computation
Wave direction	0 degrees

Figure 2. Design event (U 2)

According to relevant standards and design rules valid for German sea areas a 35h design storm event (fig. 2), which is also part of the new German standard “Structures for wind turbines and platforms” (U 1) has to be taken into account for the required proofs of safety for ultimate limit states (ULS) and serviceability limit states (SLS). In addition to conventional proofs (ULS), the assessment of a foundation system shall also take into account: influence of installation process, soil-structure-interaction, influence of cyclic or dynamic loading, displacement predictions, changes in its bearing behavior due to possible pore water pressure buildup.

This is a very challenging task, especially with regard to the very complex and non-linear soil-structure interaction under alternating loading conditions.

The following requirements have to be met to be able to fulfill this task:

- Thorough field investigation (borings in conjunction with soil sampling, CPTU cone penetration tests) and laboratory investigations (index tests, cyclic oedometric compression tests, cyclic triaxial shear tests).
- Availability of advanced and validated constitutive models for soils with the capacity of reproducing the main physical characteristics of granular materials (barotropy, pyknotropy and path-dependent behaviour)
- Availability of a validated numerical model most commonly a finite element (FE) model

These points and their realization in the course of the pilot project are described in short in the following sections.

2. Field and Laboratory Investigations

On the basis of a preceding desk study based on data from an adjacent existing wind park, the field investigation program according to figure 3 was performed using onshore equipment placed on a platform.

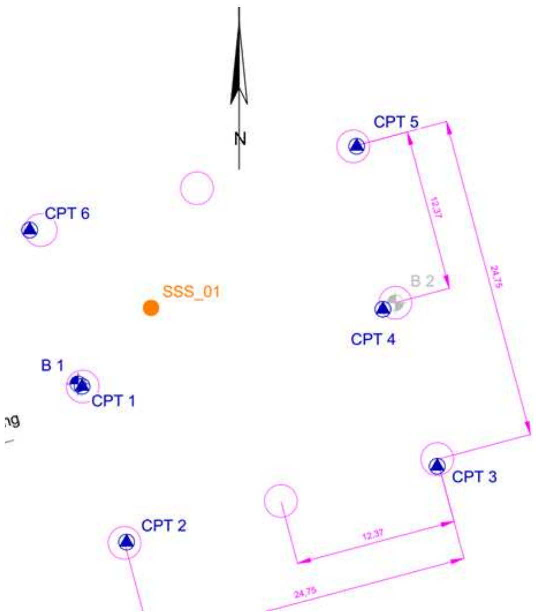


Figure 3. Layout of field tests

Figure 4. Platform with onshore CPT equipment

The soil samples (liner) were sealed, taken to the laboratory and the tests as mentioned above were performed.

3. Constitutive Modelling

Hypoplastic constitutive models for both fine and coarse grained soils (cf. e.g. U 6, U 8, U 9 and U 10) are the most suitable for realistic numerical simulations since they fulfill the above mentioned criteria <u>and</u> are publicly available (U 5). Comprehensive calibration instructions are also published (U 4). Figure 5 demonstrates the performance of the extended model for sands with respect to cyclic loading showing the results of a back analysis of an oedometer test. If volumetric changes in case of drained conditions can be realistically reproduced, the same holds true for pore water pressure development under undrained conditions. Nevertheless, the hypoplastic constitutive models with small strain extension (the so-called intergranular strain concept, U 9) tends to slightly overpredict densification and therewith excess pore pressures and more than a few hundred cycles in a boundary value problem can hardly be considered until today. Although there is a very interesting and promising approach for highly cyclic problems (U 7), from a practitioner's point of view it remains meaningless at present, since the constitutive model is not public domain and special test devices only available in a few universities worldwide are necessary for the calibration of the material parameters.

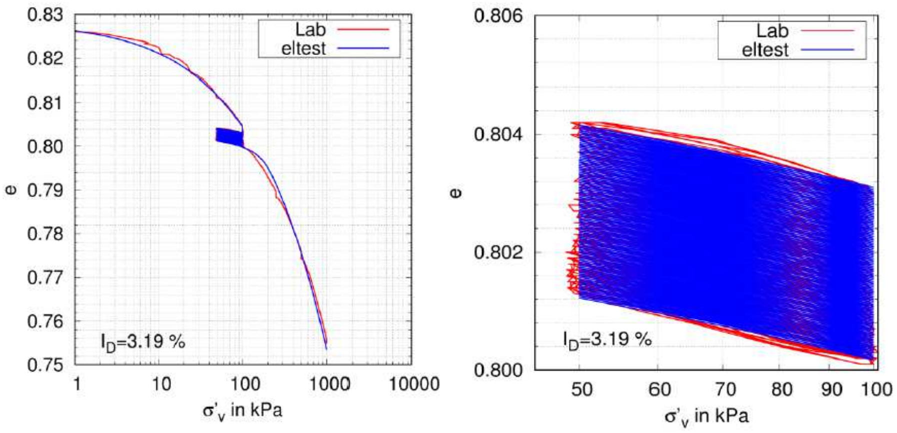


Figure 5. Back analysis of oedometric compression test with 100 unloading/reloading cycles (Rehman, 2017)

4. Finite Element Modeling

The finite element method in combination with hypoplastic constitutive models has been successfully applied to many problems of geotechnical engineering over the last two decades. Nevertheless, relatively little experience with tension piles under additional alternating lateral loading exists. For this reason a large model test stand, 2.5 m in height and 3.0 m in diameter, was designed and built in the course of an additional and ongoing research and development project (fig. 6). These tests were demanded by the certifier to validate the numerical models.

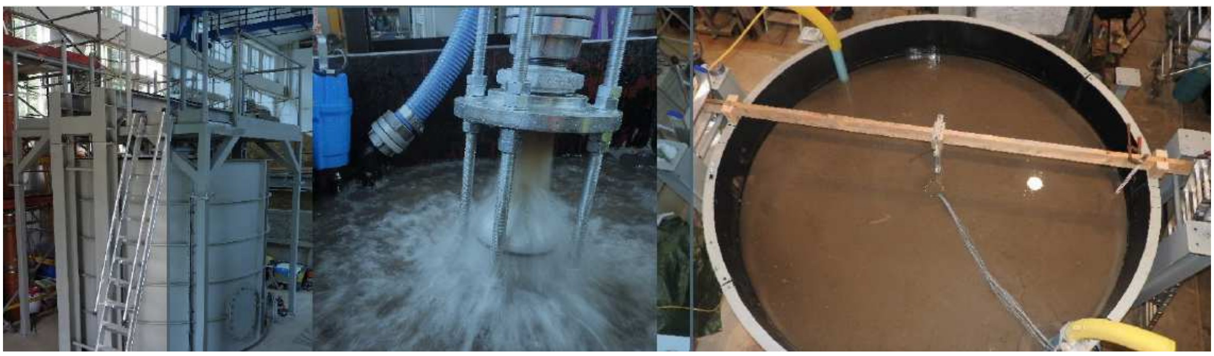


Figure 6. Pile test stand (Weichhold, 2016)

Figures 7 and 8 depict the finite element models used for the back analysis of the model tests and the actual offshore predictions. Since until today it is not possible to consider a complete design event as shown in figure 2 by means of even advanced finite element analyses as described here, a main task performing stress-deformation analyses of such alternatingly loaded offshore structures is the determination of a representative load regime. In this context the rainflow method is often applied to synthetic time histories of relevant loading in accordance with the criteria summarized in figure 2. During this operation "cycles" of equal amplitudes are counted resulting in load packages, which are then applied in parts one after the other to the FE model. This implies that Miner's rule holds true for soils, which contradicts experimental findings, since it is a well-known fact that the mechanical behavior of soil is path-dependent (U 3). In practice damage equivalent load ranges are often used anyway, also in the project described here. The load amplitude was simply derived from the minimum and maximum value of the relevant time history of loading and the dominant frequency was derived from a Fourier transformation of the signal. This approach is considered to be conservative with respect to a potential pore water pressure buildup in the soil surrounding the piles and accumulated displacements of the pile heads.

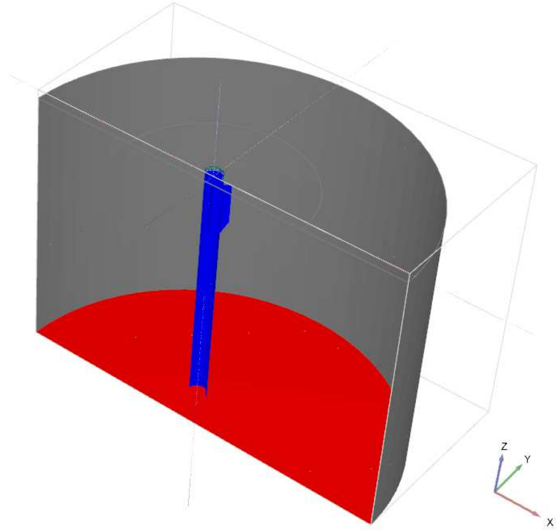


Figure 8. 3D-FE model of the pile test stand

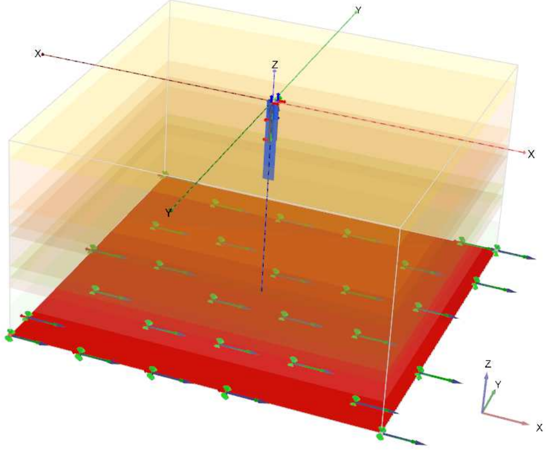


Figure 9. 3D-FE model of single pile "in-situ"

As already stated above, more than approximately 200 cycles are hardly feasible due to calculation time and summation of calculation errors. This makes necessary the application of a suitable extrapolation formula. In this context, logarithmic approaches have been suggested and used in practice for a long time. Figure 10 depicts an example, where different published equations (EAP - Recommendations on Piling of the German geotechnical Society, Ril 836 – German railway standard) and one setup by the authors (BGD) are calibrated on the basis of the FE results (first 200 cycles, which are also shown in the diagram) are used to extrapolate displacements for the number of cycles according to the design event (fig. 2).

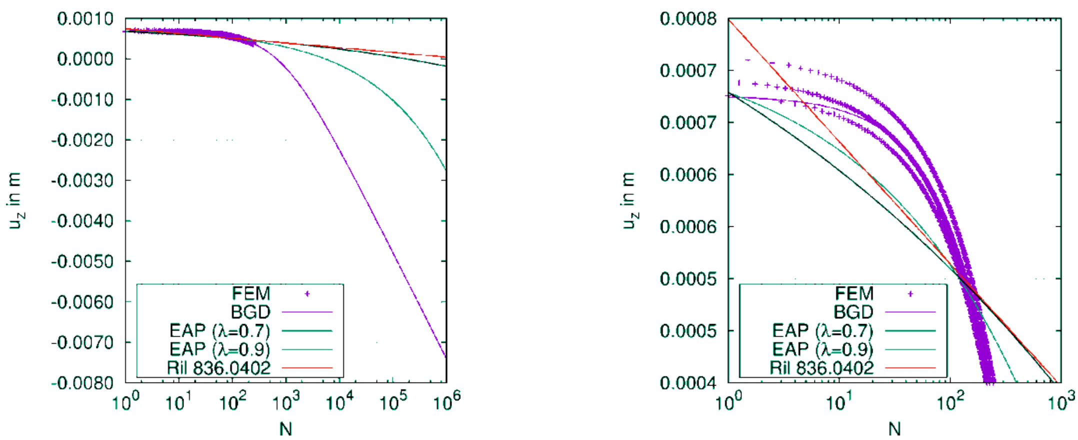


Figure 10. Vertical displacement vs. number of cycles – extrapolation of FE results (left) and blowup for first 200 cycles (right)

An essential advantage of the applied constitutive model is the realistic, yet conservative, prognosis of excess pore water pressure development especially in fully coupled (quasi-static) flow-deformation analyses, when inertia effects can be neglected. This renders possible the judgment of a potential degradation of the bearing behavior with respect to bearing capacity and “bedding”. It also enables the investigation of possible group effects. Rules of thumb as widely used in foundation engineering practice, which assume that group effects can be neglected for distances between the piles greater than e.g. 2.5 to 5 times the diameter are known to be questionable even under common onshore conditions. For large diameter piles (d > 2 m) as often used in offshore applications there is a lack of experience up to now and potential group effects can only be examined by means of suitable numerical tools.

5. Summary

It has been shown how advanced numerical tools in terms of the finite element method together with a physically sound constitutive model can be used for solving offshore soil/structure interaction problems under alternating or dynamic horizontal and vertical loading. The current limitations of the models with respect to the restricted number of cycles that can be reliably prognosed has been pointed out. This shortcoming can be overcome by the application of suitable extrapolation functions calibrated on the basis of calculation results of the first few hundred cycles. The basis for such advanced numerical approaches are thorough field and laboratory investigations including cyclic laboratory tests under similar conditions as in situ.

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25th Prague Geotechnical Lecture

Design Issues for Steel Pipe Piles for Bridge Foundations, Coastal Structures and Offshore Applications

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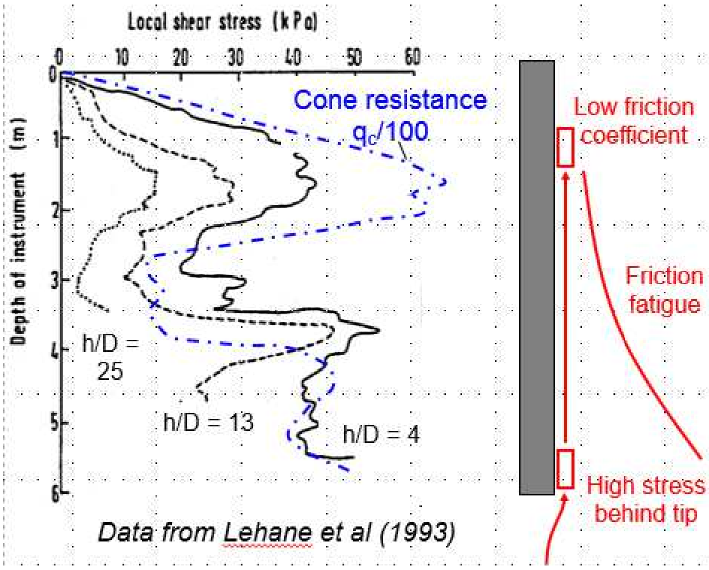
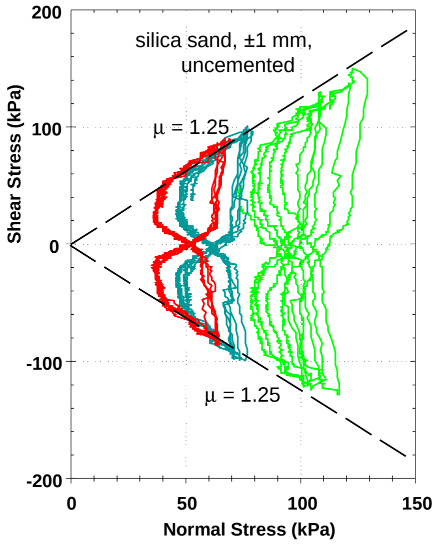
Introduction

Steel pipe piles are the workhorse of foundations for bridges, coastal structures and offshore applications because of their good bending response and ease of installation. They are generally driven open-ended although internal plates or even complete end-closures may be used to improve their bearing capacity.

Axial Capacity

Advances in estimation of axial capacity of driven piles have come from much improved understanding of the mechanisms during pile installation, in particular the effects of the so-called ‘area ratio’ – the ratio of outward soil displacement to the gross volume of the pile – and also the significance of the friction degradation that arises from the cyclic shearing associated with driving the pile. Although empirical correlations are still the basis of design, approaches for sands and other free-draining material that are linked directly to profiles of cone resistance have removed much of the bias associated with earth-pressure based approaches. The CPT-based approaches capture in much better detail differences between the axial capacity of closed-ended, partially plugged, and fully open-ended piles.





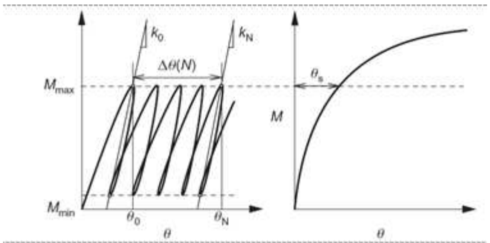
Lateral pile response

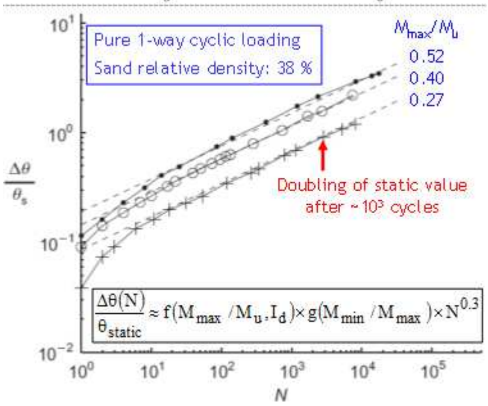

Ground Engineering, August 2014

A very topical application of steel pipe piles is in the offshore wind industry, where extremely large diameter 'monopiles' have provided the primary foundation choice for developments in European coastal waters of the North Sea.

The large diameters, now approaching 8 m, and low slenderness ratios (typically L/D ratios of 2.5 to 4) have necessitated review of traditional load transfer methods of analysis, which need to be supplemented by the addition of base shear and rotational springs and, increasingly, rotational springs distributed along the shaft of the pile.

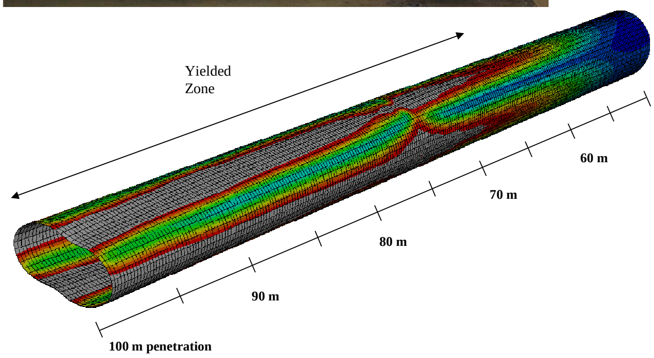
A further difference with respect to conventional lateral pile design is the focus on serviceability, with rotations of the monopile generally restricted to 0.5 degrees at the seabed surface. The effects of cyclic loading, which may double the monotonic rotation magnitude, become critical to the overall design.

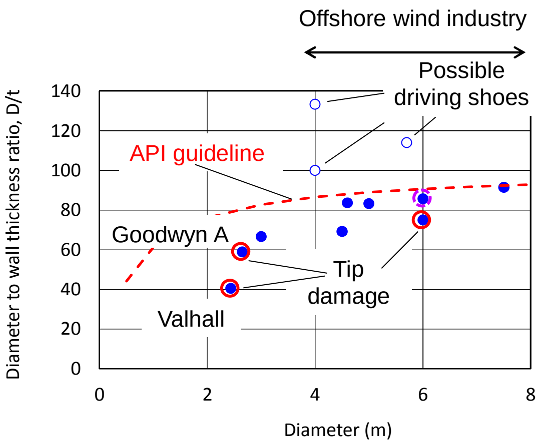


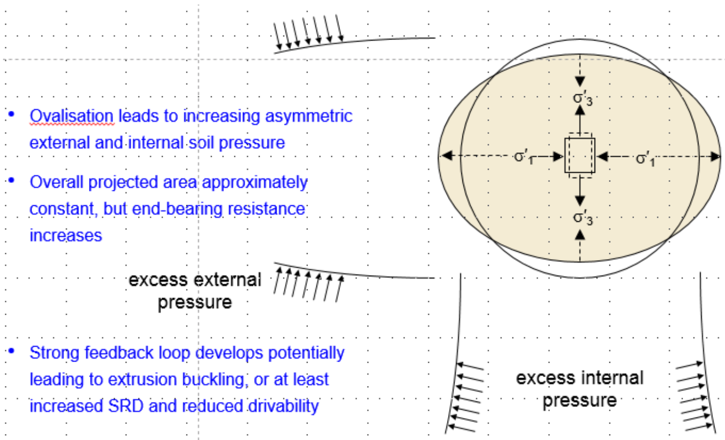


Damage During Installation

Pipe piles are vulnerable to tip damage during installation, particularly (but certainly not exclusively) for the very large diameter, high D/t ratios, that have become routine in the offshore wind industry. Although localised tip damage may occur if a hard layer is encountered unexpectedly, the most common mechanism of damage is progressive distortion of the pile tip, referred to as extrusion buckling. Case histories of pile damage, including numerical analysis techniques to assess the vulnerability of a pile to extrusion buckling, will be discussed.







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Bio Notes

Mark Randolph holds the Fugro Chair in Geotechnics in the Centre for Offshore Foundation Systems at the University of Western Australia. His two main research interests are piled foundations and offshore geotechnics, co-authoring books in each area: Piling Engineering, now in its third edition, and Offshore Geotechnical Engineering. He has published over 250 journal articles, providing novel solutions to practical problems. Although his research has embraced centrifuge model testing, numerical analysis and plasticity solutions, his primary focus has always been on developing simplified models of analysis that are suitable for application. These have included various pieces of software for analysis and design of piles and pile groups.

Professor Randolph interacts closely with industry, both in research and through his role as Technical Advisor within Fugro AG. He is a Fellow of several learned academies, including the Royal Society, and in 2013 was elected WA Scientist of the Year. In 2015 he received an honorary doctorate from ETH Zurich.