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 (pale 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 overburden 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.

2.2 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 $\beta$ equated to the friction angle $\phi$. In equation (1) $h$ represents the thickness of creep mass affecting the tunnel wall and $m(\phi)$ is a factor depending on the stiffness of the retaining structure (see Fig. 2):

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

(1)

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

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 Fig. 4). 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 Fig. 4) 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.

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.
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.

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)

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**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 Fig. 4 and Fig. 5 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 Fig. 4). 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 Fig. 5) 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 (E1,2), 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 (E1,2), 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_pgg. Between limestone debris Q_pgg and flysch debris Q_del there is a 2 to 3 m thick intermediate layer Q_del-Q_pgg. 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 Fig. 7 (top)). (Pulko et al., 2005)

4.2. A2 Motorway, Degendamm, Austria

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_pgg (see Fig. 7 (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; Fig. 8 (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 \( x_\text{g} = 0.239 \text{ g} \) 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 ($\phi_u = 0; c_u = 15$ 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.

![Exemplary distribution of earth pressure and earth resistance on a socket due to creeping pressure. Idealised calculation model (Rebernice Landslide) (Adam & Brandl, 2003).](image-url)
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. Fig. 10 shows exemplary the distribution of earth pressure and earth resistance on a socket due to creeping pressure (idealised calculation model). Fig. 11 depicts a novel approach for modelling creeping landslide constrained by a retaining structure.

Fig. 10.

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.

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).
References


