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Deep Pit Protection – Case Study

Construction of Three Underground Levels, "Çerçiz Topulli" Square, Gjirokastra, Albania

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***Abstract*— This paper presents a design concept and acceptance test application procedure for a deep pit protection structure. The structure is intended for use in the construction of three underground levels (for parking) in "Çerçiz Topulli" Square, Gjirokastra, Albania.**

Appropriate construction methods should be fully considered at the design stage, since different construction methods may require different detailed design approaches. An excavation cannot be made without causing ground movement. The chosen retaining structure should ensure that these movements remain within pre-defined limits. It is inappropriate to adopt advanced technological solutions that minimize the dimensions and material costs of the retaining structure while prolonging the design and construction periods, resulting in increased overall costs. Balance is required.

Limit state design practice should be used in the verification of safety and serviceability requirements of a retaining structure. For non-gravity cantilevered retaining walls, structural support is achieved primarily through the shear and flexural stiffness of the vertical wall elements, supplemented by the passive resistance mobilized in the soil mass located beneath the final excavation grade. Anchored wall support relies on these components, as well as the lateral resistance provided by the ground anchors, to resist horizontal pressures (e.g., earth, water, and external loads) acting on the wall. The anchored wall analyzed in this paper will be recommended for use as a temporary supporting structure necessary for the excavation and erection of the underground structure. The design life of the temporary ground anchors is two years. Dynamic loads are not considered in this analysis.

Keywords—Design, pit, non-gravity, pile, ground, anchor, slurry, excavation.

I. Introduction

The concept of an anchored wall system is to create an internally stable mass of soil that will resist external failure modes at an adequate level of safety. The design should consider the mobilization of resistance by both anchors and wall elements in response to loads applied to the wall system.

The magnitude of the total anchor force required to maintain the wall in equilibrium is based on the forces caused by soil, water, and external loads. Anchors can provide the required stabilizing forces, which, in turn, are transmitted back into the soil at a suitable distance behind the active zone. This requirement generally defines the minimum distance behind the wall at which the anchor bond length is formed. The anchor bond length must extend into the ground to intersect any potentially critical failure surfaces that might pass behind the anchors and below the base of the wall. The required depth to which anchors must be installed in the soil should be determined based on the location of the deepest potential failure surfaces that have an insufficient factor of safety without any anchor force.

The following items are necessary to provide pit protection by means of an excavation supported by an anchored wall:

- **Ultimate Limit States (ULS)**

Ultimate limit states associated with failure are concerned with the safety of people and structures. According to ultimate limit state design, it must be considered:

- Loss of equilibrium of the structure or any part of it, considered as a rigid body
- Failure by rotation or translation of the wall or parts
- Failure due to a lack of vertical equilibrium of the wall
- Failure of a structural element, such as a wall, anchor, strut or failure of the connection between elements
- Combined failure in the ground and in the structural element
- Movements of the retaining structure that may cause collapse of the structure and nearby structures upon it.

The most common cases of failure in embedded walls are:

- Inadequate support to the wall from the ground due to insufficient embedment
- Buckling of struts providing lateral supports.
- Structural inadequacy of the connection between the strut and the wall
- Inadequate foundations of raking struts
- Over-excavation of the soil berm or premature removal prior to installation of the struts.

Serviceability Limit States (SLS)

Serviceability Limit States associated with service performance requirements are insufficiently fulfilled.

According to the serviceability limit state, design must be considered:

- Unacceptable wall deflections associated with ground movements
- Unacceptable leakage through or beneath the wall
- Unacceptable transport of soil grains through or beneath the wall
- Unacceptable change to the flow of groundwater.

2. Construction Consideration and Wall Selection

2.1. Construction Methods for Soil Support

Different factors guide the designer in the choice of soil supporting methods in deep pit excavation works. The main factors are:

2.1.1 Construction sequences appropriate for temporary and permanent works

Once site preparation works have been completed, construction usually starts with the installation of the embedded retaining wall. Any excavation before wall installation, while reducing the depth of the wall, may involve additional temporary works and extra ground movement. The use of the temporary retaining wall as a permanent has an economic advantage in that it is necessary to install only one wall. When making this decision, should be considered the form of permanent internal face and watertightness requirements. Also, the structural capacity of the wall can be used to support the soil loads and any vertical loads from permanent works, while a secondary wall (e.g RC wall connected to the inside face of a pile wall) provides watertightness.

For an efficient design, the wall capacity required for the permanent condition should also satisfy the requirements of the temporary condition, avoiding the need to provide a stronger or stiffer wall only for temporary conditions.

Deep excavation is not suitable for using the simplest option of a cantilever wall because of unacceptable deflection during the temporary excavation stages. Table 1 lists the advantages and limitations of using a cantilever wall.

TABLE 1 CANTILEVER WALL

Advantages	Limitations
• A simple construction sequence with no temporary propping to the wall	• uneconomic for deeper excavation
• The permanent works are constructed in an open excavation free from restrictions of working around temporary props	• The deflections generated by the unpropped excavation are unacceptable
	• The depth and strength of the wall to ensure stability against overturning are considerable

Different alternatives to a cantilever solution can be the propping of the wall during excavation sequences that can be categorized as top-down or bottom-up construction sequences.

“Top-down” is defined by the use of the permanent internal structure as the temporary propping to the retaining wall, cast in top-down sequence. The higher-level slabs are cast before the lower-level slabs to act as horizontal frames for wall supports as the excavation progresses. A top-down solution requires:

- Support for the vertical load of the permanent slabs in the temporary conditions
- Access for the removal of soil and the supply of materials, through slab openings.
- Ventilation for the work below ground beneath permanent slabs
- A method for the excavation and construction of the substructure that is compatible with limited access. This can become a critical issue for small confined sites.

The main advantage of the top-down approach is that allows the superstructure to be constructed at the same time as the substructure.

Table 2 lists the advantages and limitations of using top-down construction sequence.

TABLE 2 TOP-DOWN CONSTRUCTION

Advantages	Limitations
• The superstructure construction can proceed at the same time as the substructure, provided the necessary vertical supports, generally piles, are in place	• The excavation works and substructure construction are slower and costly due to the restrictions on the size of the plant and the limited access
• Temporary propping is replaced by the use of permanent slabs	• Holes may have to be left in the slabs to provide access for the subsequent excavation
•provides a stiff support system for the wall, minimising movement	• vertical support for the permanent slabs is required in the temporary condition
	• The stiffer construction during the intermediate construction stages attracts higher loads into the permanent structure

“Bottom-up” construction sequence is defined by the construction of the permanent works from the lowest level upwards, casting the foundation slab before the internal walls and slabs above.

Table 3 lists the advantages and limitations of using a bottom-up construction sequence.

TABLE 3 BOTTOM-UP CONSTRUCTION

Advantages	Limitations
• Deflections are controlled by the use of propping to the wall	• Compared to a cantilever wall, there are cost and time penalties with the use of temporary props
• Compared to a cantilever solution, the wall strength, stiffness, and depth are reduced	• The propping impedes the final excavation and the construction of the permanent works
•sequential construction of the substructure and the superstructure	

2.1.2 Temporary and permanent support systems, props, ground anchorages

Temporary propping in retaining wall should be removed as the permanent works are able to replace the temporary props. Temporary props are made of steel or concrete, particularly as corner braces across the ends of excavations. For narrow excavations, props can be supported by the perimeter retaining walls. Table 4 lists the advantages and limitations of ground anchorages.
Ground anchorages in retaining walls provide the horizontal support when cantilever wall is not suitable solution. The main advantage of the use of ground anchorages is that excavation remains unobstructed by propping.

TABLE 4 GROUND ANCHORAGES

Advantages	Limitations
• Once installed, the excavation is free of any obstructions allowing for efficient construction of the permanent works	• The time to install and stress the ground anchorage increases the excavation time
• The ground anchorage prestress may reduce wall deflection and settlement behind the wall, depending on the magnitude of prestress	• The ground anchorage often extends outside the site boundaries, and the necessary permissions are required
	• The ground anchorage requires de-stressing and occasionally removing at the end of construction

Ground anchorages use result in savings over propping schemes where time and space is available to locate them. In their installation should be considered:

- The ground anchorages will be prestressed to a percentage of their working load upon installation
- The ground anchorages will be installed at an angle to the horizontal, imposing a vertical component of load to be resisted by the retaining wall. Depending on the fixing detail, a moment may also be induced in the wall.
- A condition of the permission to install the ground anchorages beneath an adjacent property may be removing at the end of construction.
- Space is necessary outside the wall to install ground anchorages.

Permanent slabs as props is a common way of providing the permanent support to a wall where the wall is part of permanent works. In addition to the prop load, the connection also supports vertical loads from the slab. In the case of the base slab, the connection will support any heave and groundwater uplift load as well.
The connection between the wall and the slabs may be costly and time-consuming, negating the advantages of using the temporary wall as part of the permanent works.

2.2 Types of embedded retaining wall

Wall types can be categorized by their material (concrete, steel) and by their installation method. In general, steel walls are driven or pushed into the ground without any material being removed. Reinforced concrete walls are created by first removing the ground and where necessary providing some form of temporary ground support in advance of placing the concrete and the reinforcement. The wall types used in deep pit protection are:

- Sheet piles

Different sections common for temporary and permanent works can be used. Some typical sections are shown in the Fig.1 below

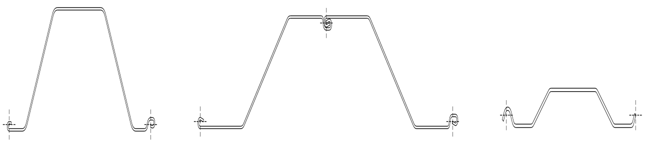


Fig. 1 Sheet pile typical sections

- Contiguous piles

A contiguous pile consists of bored cast-in-place piles along the line of the wall as shown in the Fig. 2. Construction is bored piling with or without temporary casings. The dimension of the gap between the piles can be varied to suit site dimensions and the specific ground conditions within a range 5-15cm.

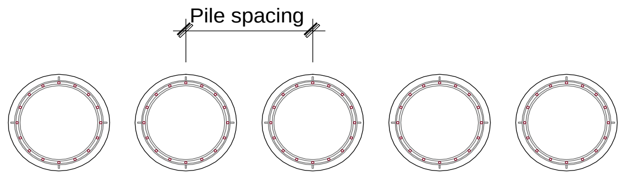


Fig. 2: Contiguous pile wall

The maximum achievable wall depths is limited by the drilling rigs, typically 30-55m. Vertically tolerances should be considered to ensure either that the potential gap between piles does not increase unacceptably with depth and the piles not overlap. Contiguous pile walls are not suitable for retaining-water-bearing coarse-grained soils and are usually only specified as temporary walls. A permanent wall can be created with a structural facing applied to the piles to fill in the gaps. This may either take the form of a structural concrete facing wall, tied to the contiguous piles, or sprayed concrete, which fill the gaps to form a key between the piles and the spray concrete. Typical diameters and pile spacing are shown in Table 5, although the use of diameters greater than 120cm is unusual.

TABLE 5 CONTIGUOUS PILE WALL-TYPICAL DIAMETERS AND SPACING

Diameter	Spacing	Diameter	Spacing	Diameter	Spacing
mm	mm	mm	mm	mm	mm
300	400	900	1000	1800	1900
450	550	1050	1150	2100	2200
600	700	1200	1300	2400	2500
750	850	1500	1600		

- Hard/Soft Secant

A hard/soft secant pile wall consist of overlapping piles, as shown in Fig.3. The primary (female) piles are cast first and consist of a soft pile, typically cement, bentonite and sand with compressive strength of 1-3N/mm2. They are unreinforced.

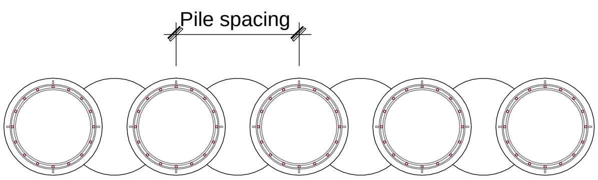


Fig. 3 Hard/Soft secant pile wall

The secondary (male) piles are subsequently installed to intersect the primary(female) piles as shown. As for contiguous pile walls, the reinforcement depth may be limited by the installation technique. The depth of the hard/soft secant wall is limited by the ability to control the verticality tolerances in order to maintain the secanting. The male and female piles may have different diameters and may extend to different depths. For small projects, it is preferable to retain the same diameter of both male and female piles to avoid either duplicating piling rigs or the time taken to change the drilling diameter for a single rig operation. Soft piles can retain up to 8m below the groundwater level, penetrating fine-grained non-water-bearing strata. Typical diameters and pile spacing are shown in Table 6.

TABLE 6 HARD/SOFT SECANT PILE WALL-TYPICAL DIAMETERS AND SPACING

Diameter mm		Spacing	Diameter mm		Spacing
Male	Female	mm	Male	Female	mm
450	450	600	900	600	1100
600	600	800	1200	600	1400
750	750	1000	1200	750	1450

Note: The gap between the male piles should not exceed 40 % of the diameter of the soft piles

The soft pile is not a permanent solution to retain water, due to shrinkage and cracking characteristics of the mix when it is dried out. Alternatively, a structural wall can be applied to the face of the hard/soft secant piles to reinforce the soft piles in the long term.

- Hard/Firm Secant

A hard/firm secant pile wall consist also in overlapping piles, as shown in Fig.4. The female pile has a characteristic compressive strength of 10-20N/mm2, which is retarded to reduce the strength of the mix while the male piles are drilled between the female piles. Typically, the characteristic strength of the female mix is specified as a 56-day strength rather than the more usual 28-day strength. This enables such walls to be installed using less energy to drill the softer female piles than for a hard/hard secant. The female pile may be installed to a lesser depth as hard/soft secant. The female piles are typically unreinforced.

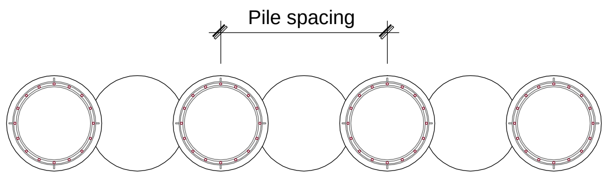


Fig. 4 Hard/Firm secant pile wall

The spacing of the male piles is calculated by ensuring that a minimum overlap of 25mm is provided at the maximum depth required to retain water. Typical pile spacings are shown in the Table 7.

TABLE 7 HARD/FIRM SECANT PILE WALL-TYPICAL DIAMETERS AND SPACING

Diameter mm	Spacing
Male and Female	mm
600	900
750	1150

- Hard/Hard Secant

The male and female piles of a hard/hard secant pile wall are both cast with full-strength concrete and both reinforced. The female piles are cast first, followed by the male piles, which are formed by drilling into the female piles using casing rotated by a hydraulic rig. A thick-walled casing, typically 40mm thick, is used to resist the high torque generated during the cutting process. As shown in Fig. 5, the female pile reinforcement should be detailed and placed to avoid being cut during the installation of the male piles.

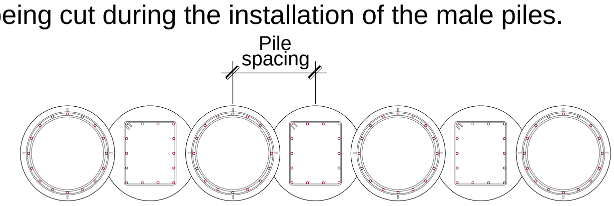


Fig. 5 Hard/Hard secant pile wall

The limiting depth for hard/hard secant pile walls is about 25m and is limited by the piling rig's ability to rotate casing. The verticality tolerances ensure that secanting is maintaining over the depth of the wall to avoid the possibility of cutting into female pile cage. Typical diameter and pile spacings are shown in the Table 8.

TABLE 8 HARD/HARD SECANT PILE WALL-TYPICAL DIAMETERS AND SPACING

Diameter mm	Spacing
Male and Female	mm
750	650
880	760
1180	1025

Hard/Hard pile walls using hoop compression resist the ground and groundwater forces.

- Diaphragm Walls

Diaphragm walls are formed by sequenced excavation under a support fluid (slurry), lowering the reinforcement cage and concreting using a tremie pipe to displace the fluid. Excavation is carried out by a grab to break soft materials or mills(cutter) to cut through harder materials.

The reinforced concrete for diaphragm walls may be post-tensioned or precast panels. The precast panels are lowered into the fluid-filled trench and sealed into the ground with in-situ concrete. Precast panels provide a high-quality surface finish. The huge weight of the precast panels makes this solution impractical. Fig.6. shows a typical diaphragm wall panel. Panel widths are a function of the grab and cutter widths available and are typically 600,800,1000,1200, or 1500mm wide. The number of bites to excavate the panel and the grab or cutter length determine the panel length. A typical grab length is 2.8m, but may reduce to 2.2m.

The maximum depth of the diaphragm wall is limited by the rope or mill length. Diaphragm walls have been constructed to depths of 120m. There are also difficulties with installing deep reinforcement cages, splices, cage length, and weight.

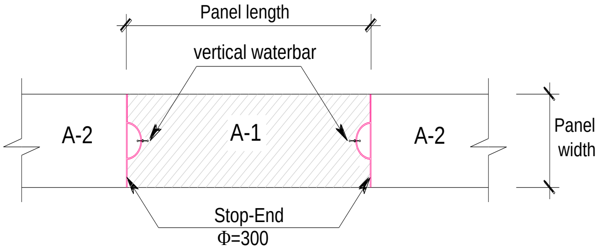


Fig. 6 Diaphragm wall panel and joint

The incorporation of a vertical waterbar (waterstop) ensures the maximum watertightness between adjoining panels. The principle is that a temporary steel stop end is supporting the vertical waterbar.

The typical dimensions of the vertical waterbar (PVC, polyvinylchloride) are shown in Fig. 7

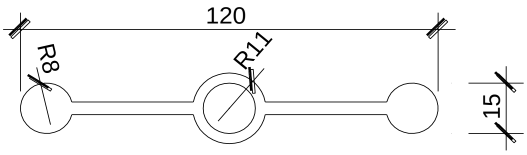


Fig. 7 Vertical waterbar (PVC, polyvinylchloride) typical dimensions

2.2.3 Wall selection

The form of the wall has a fundamental effect on the design. Anyway, the final choice is a compromise between the following criteria:

- Cost
- Ground conditions, particularly the need to retain groundwater and the presence of the obstructions, including any remains of historical interests, in the path of the wall.
- The need to restrict ground movements to within acceptable limits
- Extent of the site to accommodate construction plant. This is particularly important for the use of diaphragm walling techniques, with need to accommodate support fluid mixing, reinforcement cage and storage facilities. - Environmental issues
- Speed of construction

3. Retaining structure

According to upper recommendations is selected Anchored Temporary Secant Pile Wall as appropriate option to protect deep pit excavation under square 'Cerciz Topulli", Gjirokaster. Under the square three underground parking levels should be constructed going up to -12.5m below the terrain

3.1 Anchored Temporary Hard/firm Secant Pile Wall

The general solution considers a separation of 10 cm between the new building structure and the secant pile wall necessary to protect the excavation; therefore, it has only a temporary character, defining the inner dimensions of the pit.

The secant pile wall is a specific structure that is used to ensure the stability of the soil, installing primary(female) and secondary(male) piles with diameter d=80cm, height h=20m; h=18m (h=18m; h=16m for primary pile respectively) and pile spacing s=70cm. The female pile (concrete C16/20) is retarded to reduce the strength of the mix, while the male piles (concrete C 30/37) are drilled between female piles.

From a static aspect, the structure represents a multi anchored secant pile wall. The anchoring is made by using specific pre-stress anchors, introducing additional stabilizing force into the system. By doing so, a free space is left in the excavation pit, enabling all the construction jobs to be undisturbed.

The disposition of the anchors installed in two levels is set with anchor spacing L_a=2,8m. The solution is set for precisely chosen anchor types T15, and the number in front of the type, with the number denoting 5 (5T15), 6 (6T15), stands for the number of cables (strands, treads). Every cable is comprised of 7 wires with a diameter of φ5mm with a total diameter of the cable of 15.2mm, therefore, the label is T15. The anchorage is made in layers below the ground level of -2.53÷-4.32m (First Row S-1) and -5.68÷-7.85m (Second Row S-2) with anchor total length L (20÷22)m.

To install the secant pile protection wall, a special type of mechanization (for excavation) is used, and the process starts before the pit excavation by setting the line of the wall using concrete guidelines with a thickness 17.5cm.

3.2 Technical description and construction procedure

The construction works connected to the installation of the secant pile wall and anchors, as well as for the soil (pit) excavation, are made according to the following procedure:

-Preliminary excavation
Preliminary Excavation works should be made in advance, at the building site up to level of -0.70m.(relative level).

-Guidelines for the construction of the secant pile wall
The reinforced-concrete guidelines are first set with respect to the exact survey as two parallel beams making an opening of 82cm for the excavation of the wall.

-Excavation of the secant pile wall
The excavation of the secant pile wall is made with special machines. To ensure a precise excavation during the process, a slurry suspension is purged in the excavated segment. The female unreinforced piles should be installed first to a depth of h=16m, h=18m (specified in drawings). An overlap of 10cm is to be provided between primary(female) and secondary(male) piles. The male pile is reinforced.

-Reinforcing and concreting of the secant pile wall
The secant pile wall is reinforced through a circular cage installed in secondary full-strength piles (specified in drawings) after excavation.
The concrete used for the secant pile is made by using a fixed pipe (tremie), which is continuously poured with the concrete mass from the bottom to the top of the wall, and in the water presence.

-Excavation of the soil material from the pit
The excavation is executed in several, namely 3 phases or stages progressing continuously as:
- Preliminary phase, excavation until -0.70m and construction of guide walls.
- Phase-1, excavation until -2.97÷-5.66m (Installation, First Row Anchor S-1).
- Phase-2, excavation until -6.12÷-8.81m (Installation, Second Row Anchor S-2).
- Phase-3, excavation until -10.25÷-12.96m.

Parallel with the excavation, a dewatering of the pit must be made using water pumps with sufficient capacity and number in the construction area. The aquifer layer is represented with weathered sandstone at intervals of 7.50 – 8.0 m. During the excavation beneath this level, water can be kept under control by lowering the level by pumps with a capacity Q = 3-4 l/s.

Lowering the level is possible by deepening of wells in the source area up to 12.96m depth and by regulated dewatering, towards the existing slope channels.

-Anchorage
The boring of the anchors is made by rotational drilling and piping with a diameter of 150 mm.
The anchors are comprised of two lengths, namely a free length of the anchor, which is variable in different sections and depths and a bond or grouted length of the anchor, which varies from Lb=(10÷12)m.
The grouting is made using a cement injection mass with 3% bentonite as an additive.
After reaching the condition of hardening of the injection mass the next phase is to pre-stress the anchors. When this process for all the anchors from a certain level is finished, the excavation from the next phase is to follow.
To ensure that the design values for the anchor force in tensions are properly taken into the calculation its necessary that 5-10% of the total anchor number to be tested according to some relevant standard. It is suggested that a special report is to be prepared for the anchor pull-out test describing in details the whole procedure. The maximum stressing force or lock-off force P_0 must be limited to 0.6 Ptk. Strand technical characteristics and temporary anchors are shown in Table 9 and Fig.8 respectively.

TABLE 9 STRAND TECHNICAL CHARACTERISTICS

Diameter	Standard	Strand type	Nominal diameter	Nominal area	Mass	ftk	Breaking strain (Ptk)	Elastic limit at 0.1% (Pt0,1k)	Relaxation after 1000 h.0,7 - 0,8	
			mm	mm2	gr/m	N/mm2	KN	KN	%	%
(T15)	pr EN 10138	standard	15.2	140	1095	1860	260	224	2.5	4.5
		super	15.7	150	1170	1860	279	240	2.5	4.5

P_0=1.2DL
Temporary anchor
Grout Protected Bond Length
GREASED & PE EXTRUDED UNBONDED LENGTH

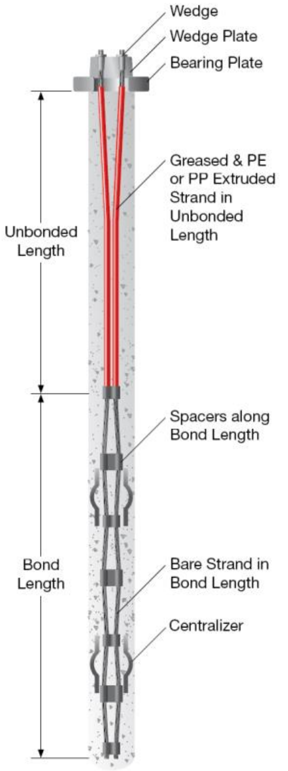


Figure 8 Typical uses: Temporary application with a service life of ≤24 months

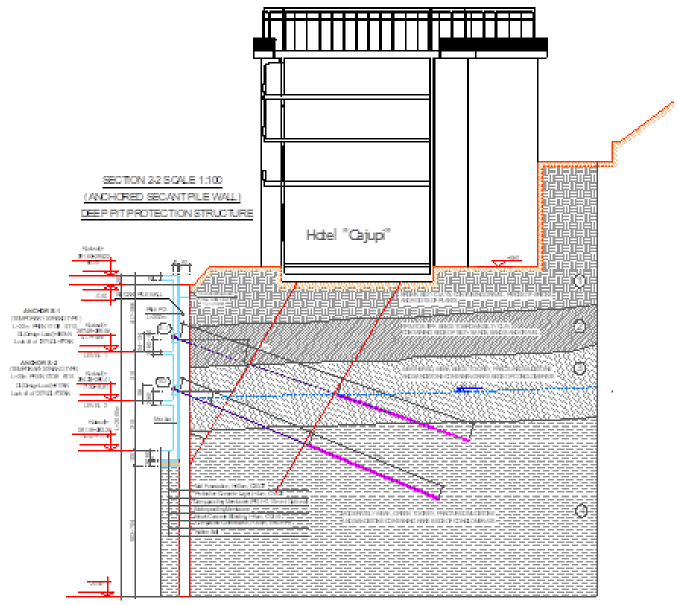


Fig. 9 Anchored Secant Pile wall, Temporary application

3.3 Anchor capacity (force)

3.3.1 Anchor Bond Length

The length of the pressure-grouted body, known as the bonded length, varies depending on the type of soil, the diameter of the pressure-grouted body and the amount of tension.
Two conditions are applied as follows:

3.3.1.1 Bond length is calculated to guarantee bond strength capacity at **soil/grouting body interface** according to expression.

$$N = \pi D L_f W_d H K_n tg\delta + \pi D L_f c_a$$

$$L_f = \frac{N}{\pi D W_d H K_n tg\delta + \pi D c_a} \qquad (1)$$

N-ultimate anchor load capacity
D-bulb diameter
L_f-Anchor bond length
w_d- soil volumetric weight in natural condition
K_n – coefficient of neutral lateral stress
H – overburden bulb cover length.
δ-friction angle soil/grout body interface; δ=(0.5÷0.8)φ;

It is accepted δ=0.6φ
c_a-adherence soil/grout body interface; $c_a=(2/3)c_u$ c_u-undrained cohesion; $c_u=[\sigma]/2$

3.3.1.2 Bond length must be calculated to guarantee bond strength capacity at **strand/grouting body interface** according to expresion.

$$N = \pi d L_f f_{bd} \omega$$

$$L_f = \frac{N}{\pi d f_{bd} \omega} \qquad (2)$$

d- sum of strand diameters comprises inside anchor
f_{bd}- adherence strand/grout body interface.
ω-reduction coefficient according to number n of separated strands comprised inside anchor.

$$\omega = 1 - 0.075(n-1)$$

Bond length consists of maximum value derived from two expressions (1) and (2), maxL_f.

3.4 Soil Properties

The soil properties used in the analysis of deep pit protection structure:

TABLE 12: THE SOIL PROPERTIES ACCORDING TO THE GEOTECHNICAL REPORT

Nr. (Geol.Rep.)	Width of layer	W	Wo	Wd	Ws	n	φ	c	Ka	Kp	Kn	e
1	2.0	19.20	25.00	14.71	18.82	0.41	16	10	0.57	1.76	0.72	0.70
2	2.0	19.50	26.30	15.65	19.70	0.40	20.55	30.15	0.48	2.08	0.65	0.68
3	0.5	22.20	26.30	16.23	20.06	0.38	29.4	42.8	0.34	2.93	0.51	0.62
3	1.0	22.20	26.30	16.23	20.06	0.38	29.4	42.8	0.34	2.93	0.51	0.62
3	1.5	22.20	26.30	16.23	20.06	0.38	29.4	42.8	0.34	2.93	0.51	0.62
3	1.0	22.20	26.30	16.23	20.06	0.38	29.4	42.8	0.34	2.93	0.51	0.62
3	1.0	22.20	26.30	16.23	20.06	0.38	29.4	42.8	0.34	2.93	0.51	0.62
4	3.5	23.20	26.40	19.41	22.06	0.26	30.2	54.8	0.33	3.02	0.50	0.36
4	5.5	23.20	26.40	19.41	22.06	0.26	30.2	54.8	0.33	3.02	0.50	0.36

Soil layers parameter symbols used:

w-soil layer volumetric weight in natural condition, in kN/m^3
w_0-soil layer specific volumetric weight, in kN/m^3.
w_d-soil layer volumetric weight in dry condition, in kN/m^3.
W_s-soil layer volumetric weight in saturated condition, in kN/m^3
n-soil porosity
φ- angle of internal friction of soil layer
c- cohesion, in kN/m^2
c_u-undrained cohesion, in kN/m^2
K_a- coefficient of active lateral stress.
K_p- coefficient of passive lateral stress.
K_n- coefficient of neutral lateral stress
Z_w-depth of the water level below the natural terrain, in m.
c_{ap}-thickness of capillary zone, above the water level, in m.
q-surcharge on this layer, and on all layers below it, in kN/m^2
D_w-stroke, lateral displacement difference between the generation of active and passive
Lateral stress, in m.

$$D_w=(\sigma_p-\sigma_a)/K_h$$

TABLE 10 THE SOIL PROPERTIES – INPUT DATA

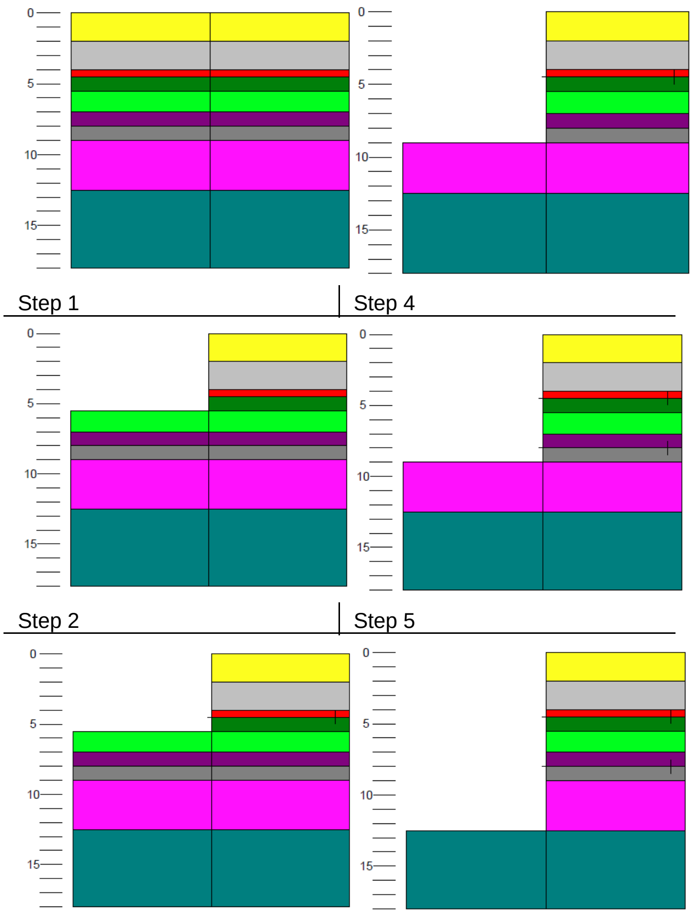
No	φ	c	W	K_a	K_p	K_n	H	σ_a	σ_p	K_h	Dw-(stroke)
	(grade)	(daN/cm^2)	(kN/m^3)				(m)	(daN/cm^2)	(daN/cm^2)	(kN/m^3)	(m)
1	16	0.10	19.20	0.5678	1.7610	0.7244	2.00	0.0673	0.9417	12000	0.01
2	20.55	0.30	19.50	0.4804	2.0818	0.6490	4.00	0.0673	2.6192	30000	0.01
3	29.4	0.43	22.20	0.3415	2.9285	0.5091	4.50	0.0673	4.4160	40000	0.01
3	29.4	0.43	22.20	0.3415	2.9285	0.5091	5.50	0.0673	5.0662	40000	0.01
3	29.4	0.43	22.20	0.3415	2.9285	0.5091	7.00	0.0673	7.5131	40000	0.02
3	29.4	0.43	22.20	0.3415	2.9285	0.5091	8.00	0.0673	9.6349	40000	0.02
3	29.4	0.43	22.20	0.3415	2.9285	0.5091	9.00	0.0673	11.7568	40000	0.03
4	30.2	0.55	23.20	0.3307	3.0243	0.4970	12.50	0.0673	16.1255	45000	0.04
4	30.2	0.55	23.20	0.3307	3.0243	0.4970	18.00	0.1252	19.9845	45000	0.04

TABLE 11. CATALOGUE OF PROFILES

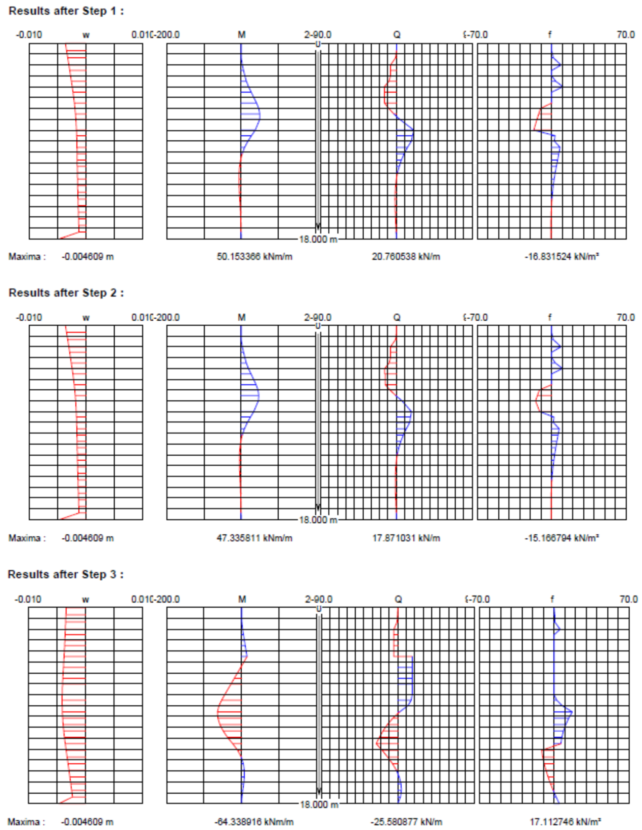
No Profile Name	Area	Weight	E	h	i	EI
13	(m^2/m)	(kN/m^3)	(GN/m^2)	(m)	(m)	(kNm^2/m)
D=80cm / 70cm	0.680714	24.000	31.000	0.80000	0.196001	810664.1

4. Analysis Results

Pit Depth : H= 12.5m
Pile Diameter (cm) : d=80cm
Length (m) : 18.00m
Number of Elements : 30
Number of Layers : 9
Number of Loading Steps : 5
Profile from Catalogue : 13



Step 0 | Step 3



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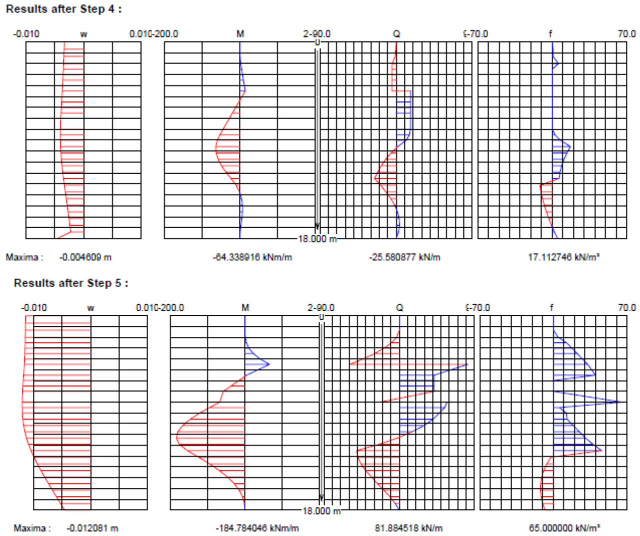

Fig. 10. Analysis results

Fig. 11 On site, "Cerciz Topulli" Square, Gjirokaster

6. Conclusions and Recommendations

This paper describes a technical solution of excavation pit protection.

The secant pile wall is analyzed as a specific structure that is used to ensure stability of the soil and, by doing so protects the pit and enables the further excavation. Their dimensions are D=0.8m as the width of the wall and L=18.0m as its height or depth. A reinforced concrete material will be used, and the construction will be in sections composed of unreinforced and reinforced piles at a distance 0.7m.

In terms of structural behavior, the system functions as a multi-anchored pile wall. Stabilization is achieved through the use of prestressed anchors, which introduce additional resisting forces into the support system. The anchor layout is defined by a horizontal spacing of (La = 2.8m) and a vertical spacing of approximately ha≈3.2m.

To ensure that the design tensile forces for the anchors are accurately incorporated into the calculations, it is necessary to test approximately 5–10% of the total number of anchors in accordance with the relevant standards. It is recommended that a dedicated report be prepared for the anchor pull-out tests, providing a detailed description of the full testing procedure.

Furthermore, to verify that the calculated displacements remain within the permissible design limits, it is recommended to install four to five inclinometers to monitor subsurface displacements below the natural ground surface.

For monitoring displacements above the ground surface, detailed topographic surveying of installed reference points on all adjacent structures, as well as on the ground surface itself, is required and should be conducted in accordance with a dedicated monitoring program.

7. References

Braja, M. D. (2011). *Principles of foundation engineering*. PWS-KENT Publishing Company.
British Standards Institution. (1986). BS 8004: *Code of practice for foundations*. BSI
Ente Nazionale Italiano di Unificazione. (2002). *Esecuzione di lavori geotecnici speciali. Tiranti d'ancoraggio* (in Italian). Ente Nazionale Italiano di Unificazione.
Petros, P. X. (1991). *Ground anchors and anchored structures*. John Wiley & Sons.
Sabatini, P. J., Pass, D. G., & Bachus, R. C. (1999). *Geotechnical engineering circular No. 4: Ground anchors and anchored systems* (Report No. FHWA-IF-99-015). Federal Highway Administration.
Technical Committee CEN/TC 250. (2004). *Eurocode 7: Geotechnical design – Part 1: General rules (prEN 1997-1)*. CEN.
Technical Committee CEN/TC 288. (2009). *Execution of special geotechnical work – Ground anchors (prEN 1537:1999)*. CEN.
Technical Committee CEN/TC 341 & ISO/TC 182. (2009). *Geotechnical investigation and testing – Testing of geotechnical structures. Part 5: Testing of anchorages (EN ISO 22477-5)*. CEN.