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Research Article

Deep Excavation Protection for Urban Underground Construction: A Case Study of an Anchored Secant Pile Wall

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Abstract

Modern urban underground construction requires retaining systems that satisfy both ultimate and serviceability limit states while ensuring reliable performance throughout excavation. This study presents the numerical evaluation and performance verification of an anchored secant pile wall through the case study of the deep excavation at "Çerçiz Topulli" Square, Gjirokastra, Albania. The study involved a 12.9 m deep excavation for a three-level underground structure in multilayered soil conditions. A reproducible plane-strain finite-element model, incorporating beam-on-elastic-foundation elements, was developed to simulate staged excavation, soil stratification, groundwater conditions, anchor activation, and construction sequences. The retaining system performance was assessed using normalized wall deflection, serviceability criteria, deformation prediction accuracy, and anchor efficiency. The maximum calculated lateral wall displacement was 12.1 mm, corresponding to a normalized wall deflection ($\delta max/H$) of 0.094%, which is substantially below the commonly accepted serviceability limit of 0.5%. Comparative analyses indicate that the prestressed anchor system reduced wall deformation by approximately 25% relative to an equivalent unanchored excavation, while controlled dewatering was required to maintain excavation stability. The proposed framework provides a validated methodology for the design verification and performance assessment of anchored secant pile walls in deep urban excavations. The findings offer practical guidance for optimizing retaining systems in multilayered soils and establish a transferable approach for similar underground infrastructure projects in complex urban environments.

Keywords: Deep Excavation; Secant Pile Wall; Ground Anchors; Pit Protection; Soil–Structure Interaction; Groundwater Control; Staged Excavation; Retaining Wall; Passive Resistance; Deformation Monitoring.

INTRODUCTION

Deep excavation systems are crucial components of modern urban infrastructure, as underground car parking, transportation facilities, services and commercial basements are increasingly built in high-density city centres where surface area is limited. However, deep pits are among the most sensitive of geotechnical operations as excavation modifies the in-situ stress state, reduces lateral confinement, causes wall deformation and even can affect existing structures, roadways and underground utilities. Ultimate equilibrium, serviceability, deformation control during construction phase, groundwater influence and construction sequencing are all components that get into the design process of a Deep pit protection system and these can be monitored through field monitoring. It was shown in recent studies that the deep excavation behaviour is mainly governed by the interaction between the retaining wall and ground anchors, soil stiffness, groundwater conditions and staged excavation [1–6]. The most common case is met in the current study, which shows important difficulties because of having to excavate for three underground parking levels at "Çerçiz Topulli" Square in the city of Gjirokastra, Albania at an average excavation depth of 12.5–12.96 m supported with a temporarily anchored rigid secant pile wall structure around it.

Deep excavation retaining systems include sheet piles, contiguous pile walls, secant pile walls, diaphragm walls, braced systems and anchored systems. The selection of the support system depends on factors such as depth of excavation, groundwater conditions, soil stratigraphy, proximity to other structures, length of time under construction or even cost (US), availability of equipment and allowable deformation. For site excavation in urban areas, anchored retaining systems are commonly employed because they allow for a clean excavated space internal strut, which interfere with the construction of basement slabs and subterranean structures. A designer should not look at wall deformation, ground movement and overall safety in isolation from anchored wall system [1]. Authors in [2] highlighted the weakness of anchor-supported deep excavations under parametric variability, such as weak or water-saturated strata reduce anchor capacity. The results emphasize the need for a design methodology based on: soil parameters, wall stiffness (stiff or flexible), and also anchor configuration and construction stages.

It is used in many applications that need to be a continuous wall-structure retainer, and where water control provisions are tighter than basic common contiguous pile systems. Component of a rigid secant pile wall where main piles are first constructed with low strength or retarded concrete followed by reinforced secondary piles that cross the main piles to create an overlapped wall. It also improves seamless water permeability and provides sufficient bending stiffness support to deep excavation. New findings of pile wall and pile-anchor retaining structures reveal substantial influences of geometry of piles, rigidity of walls and arrangement of anchors on the lateral wall displacement and structure internal forces [3, 7, 8]. Model testing and numerical simulation by [7] showed that when the interaction of structural stiffness and ground reaction is properly considered, pile-wall constructions can improve control over excavations. Authors in [3] further proved the three-dimensional pile–anchor retaining systems are effective to control deformation during deep excavations near existing buildings due to structural redundancy.

Ground anchors are critical to the functioning of anchored secant pile barriers. Anchors transfer tensile forces from the wall into a stiff mass of soil beneath the active failure zone, thus reducing both bending moment and lateral wall displacement. Synergism of anchors depends on overall length, free length, bond length and shear resistance between grout-soil grain. Recent computational and field studies indicate that the type of anchor and load-transfer system can have major impacts on deformation control and safety in urban deep excavations [4]. With respect to the results, [4] examined compression and tension ground anchors indicating that in some cases compression anchors provide better deformation control so it is essential to check anchor behaviour with field and numerical analyses. The use of two rows of temporary prestressed anchors with an anchor spacing 2.8 m, total length: 20–22 m and bond length: 10–12 m at the removed soil wall is one of the solutions in this study, which contributes to a progressive stage excavation approach and reduces wall displacement during construction phases [3].

The phased building methodology is a key component in the effectiveness of deep excavations. Digging is a slow operation; the soil is removed in layers and supports installed to specific depths. The stress redistribution, wall bending moment, shear force, anchor load and displacement profile vary at each excavation phase. Therefore, there is a need of realistic staged modelling for retrieval of actual behaviour of building. Numerical modelling with on-site monitoring was illustrated to help assess the safety of deep foundation pits [9], while in [5] determined that the deformation of walls is quite affected by parameters like soil strength, unit weight, stiffness of structure, groundwater level and surcharge. These studies confirm that a reliable design cannot be based solely on the final excavation depth, but must also incorporate intermediate stages during which the wall supports major deformations and states of stress.

However, it is significant for the preservation of deep pits. Under drained conditions, excavation below residual groundwater level increases pore water pressure and lateral loads, reduces effective stress, and may cause instability due to seepage saturation at the base of the cut or piping from adjacent soils into the excavated area. The design and construction of retaining structures is governed under Eurocode 7, which defines the ultimate and serviceability limit states and requires a calculation to account differential water pressures (DWP) when water pressure acts on the structure [10]. The depth of this aquifer is approximately a couple of meters below ground level but its excavation horizontally goes to 12.96 m [9] therefore during that stage dewatering becomes inevitable. The targeted pumping rate of 3–4 L/s is intended to manage groundwater inflows and maintain suitable operating conditions. This confirms that groundwater management is not a secondary issue, but an essential element of the excavation support strategy.

Additionally, it is important that the soil characteristics, construction quality, anchor placement, groundwater fluctuations and nearby loads are uncertain factors influencing deep excavation behaviour. It lets designers and contractors compare what was expected with what actually takes place in the field, adjusting if necessary. Investigation on augmenting further excavation monitoring and data-driven deformation analysis indicates that monitored field measurement may improve safety assessment and help to predict developing movement induced from excavations adjacent to critical infrastructures [11, 12]. In each construction phase, it is a good practice to monitor wall and ground displacement using inclinometers and topographic monitoring stations. Test of anchor pull-out testing should be performed on 5–10% anchors to confirm the load capacity and that the design-resistance, as anticipated in a design procedure, is achieved.

Research Contribution

The research, a new technology for retaining walls is not being established as secant pile walls and prestressed anchors are already ubiquitous in the industry. This contribution is a recognised and repeatable project-based model for Albanian urban subsurface conditions that includes (i) nine-layer soil characterisation, (ii) anchored hard/firm secant pile wall selection, (iii) staged excavation modelling (iv) groundwater management, (v) normalised deformation benchmarking $\delta max/H$, with the validation of anchor-load results, and is followed by monitoring acceptance criteria. This upgrade has a larger scientific value, as it converts the previous descriptive contribution into a quantitative assessment framework that provides an assessment of each hypothesis against a numeric threshold, benchmark envelope or field-monitoring evidence. The case is particularly relevant due to even medium depth excavations about 13 m in high-density Albanian urban areas are rarely covered with standardised thresholds of allowable deflection, excavation dewatered and operator test procedures for seismic susceptibility assessment and operational decision levels for construction supervision.

RELATED WORK

Deep excavation support has become an important field of study in both geotechnical and structural engineering due to increased demand for underground construction in urban areas. Studies have shown that the working of deep systems is controlled by rigidity of wall constraining soil stratification, waterproofs condition, depth excavation and building sequence, placement location of anchors or props and the proximity to existing structures [13-16]. Review-based analyses confirm that wall deflection and ground settlement are the principal serviceability criteria in deep excavation construction [17, 18]. This is directly related to the present project where excavation depth reaches approximately 12.5-12.96 m, and encompasses a large anchoring temporary secant pile wall for three basement parking levels at Gjirokastra, Albania already determined in the protected design stages.

Many recent studies have been focused on the deformation behaviour of deep excavations. Research on 88 cases of deep excavation in Beijing has indicated that ground deformation is strongly affected by excavation depth, retaining wall type, support stiffness, and adjacent soil [18]. Their research is important because it shows that deformation control needs to be treated as a first-order design requirement and not an afterthought behind stability. On similar lines, [17] investigated the deformation characteristics of dewatering and excavated sites through normalized wall movement, surface deflection and correlation between maximum lateral ground displacement versus excavation depth. The results highlight the need for a design based on serviceability in this case study, where wall displacement must be limited to preserve excavation safety and adjacent ground.

Deep pits are often examined in terms of their response to excavations due to the complexity of soil-structure interactions involved and are commonly modelled numerically [19-26]. Authors in [19] used the software FLAC-3D to study deep-foundation pit excavation supported by box-type retaining walls and found that wall shape was a major factor impacting wall deflection and surface settlement. In particular, authors in [27] showed that finite element analysis can help optimise the fragile balance between wall size, support configuration and construction phases during excavation pit design. Similarly, authors in [26] conducted numerical simulation on a pile-anchor retaining structure, and found that the predicted displacement patterns were consistent with the field observations. The findings from these investigations substantiate the applicability of numerical or engineering modelling that this study has implemented, which characterised a staged excavation process using 30 structural components, 9 soil layers and 5 loading phases.

Compared with the contiguous pile walls, secant pile walls have gained increasing attention to provide lateral support and improve groundwater control. Authors in [25] investigated cantilever, anchored and strutted secant pile wall systems and found that lateral displacement is more influenced by the support type than ground surface settlement. The contrast is relevant to this study because for 12.5 m deep excavations a cantilever wall does not provide sufficient deformation control, whereas an anchored secant pile wall has far greater deformation control without interfering with the excavation area. As such, [20] further demonstrated that the measurement of secant pile wall deformation can be performed through distributed fibre optic sensing and hence raised awareness of the importance to monitor internal strain as well as excavation-induced displacements in large-scale pile walls.

Anchor-supported and other ground-supported retaining systems are especially advantageous in cases where internal struts would conflict with ongoing building operations. In systems of pile-anchor, anchors mitigate bending moments and horizontal wall displacement by transmitting tensile stresses into stable soil beyond the active failure zone. Staged numerical analysis is used for modelling pile-anchor retaining systems and the requirement of comparing numerical results with measured data has been highlighted [26]. This validates the reasonable selection of methodology in present investigations with 2 levels of prestressed anchors placed at around −2.53 to −4.32 m depths and −5.68 to −7.85 m depths, an anchor spacing of 2.8 m, and total anchor length was 20–22 m for abutment support [12].

Groundwater is a major contemporary area of concern in writing. Lateral pressure raises above effective stress value or instabilities associated to seepage happen when excavations are in the groundwater table. Authors in [28-30] used numerical modelling to study foundation pit dewatering and show that seepage control methods can influence groundwater levels and sinking. In [29], horizontal barrier designs for foundation pit dewatering were investigated; it was found that this type of design may reduce the drawdown outside the excavation, control the subsidence of soil near and beneath a building, and also limit deformation of enclosures. The need for dewatering is demonstrated here due to the groundwater level is at approximate 7.50–8.00 m and excavation depth approach approximately 12.96 m away from bottom of the building even that pumping management, groundwater management are very important in term of building safety and stability

Deep excavation has been shown to influence surrounding infrastructure on recent studies [31]. Authors in [21] studied the deformation of soft soil due to deep excavation and also, it's an adjacent tunnel influence, however, authors in [22, 23, 32] studied the interaction of excavation of foundation pit and adjacent subway construction. These investigations have shown that deep excavations can generate tunnel distortion, differential settlement and structural responses to adjacent buried pipes. Although this case (of constructing a plaza and underground parking rather than subsurface transportation) is more about an external wall bracing system to control ground movements from excavation, the principle still holds: moved walls and Ground Movement should be controlled, as building deformations due to excavation activity may affect neighbouring urban elements.

Monitoring is a fundamental part of modern-day excavation design, as soil properties and building conditions are often relatively unpredictable. Authors in [20] demonstrate that distributed fibre optic sensors can be used to monitor pile deformation during grouting, hardening, and excavation of secant pile walls in the field. Zhao et al. This technique has been proposed by [28] which is developed by employing particle swarm optimization–support vector machine (PSO-SVM) for soil parameter inversion and deformation forecasting indicating that the utilization of empirical excavation data can improve the accuracy of numerical prediction results. These studies confirm that monitoring needs to be integrated into the design rather than viewed as a peripheral construction task. This matches with the current study that recommends using inclinometers, topographic monitoring station and anchor pull-out tests to evaluate wall behaviour and anchorage performance for excavations.

Studies on optimizing support systems provide important perspectives. Finite-element-based optimization may allow for more economical designs of the excavation pit, while still being safe, as shown in [27]. Authors in [31] analysed the reduction of lateral passive force behind sheet-pile walls with EPS geofoam and suggested that correcting the soil–structure interaction may be achieved in specific soil conditions. Even though EPS is not part of the system, this should nevertheless be emphasized since its premise shows that lateral pressures are a primary design objective in excavation. This research controls lateral pressure with wall stiffness, anchor preload, pile embedment length, and passive soil resistance.

The literature suggests that the deformation of retaining walls depends on both geometric and soil characteristics. Peng et al. Retention wall standards would certainly want to bear configurations and specifications has a marked effect on wall deflection and adjacent settlement per Basha et al. According to [25], various support options can have different deformation responses in secant pile walls. The results can be compared to the present design, which uses 0.80 m diameter secant piles of length 18.0 m and about 5.5 m in depth embedded under the intended excavation level to provide stiffness and passive resistance. The recent research support employing hard/firm secant pile wall so a few of the important reasons is structural stiffness, continuous nature, ground water interaction nature and anchorage compatibility.

Table 1 shows that most studies have focused on numerical modelling, wall deformation, adjacent-structure effects, dewatering and the optimization of excavation support systems. It explains how each study supports the current research, and especially why an anchored secant pile wall was selected for 12.5–12.96 m deep excavation in Gjirokastra, Albania.

Table 1. Comparative Literature Summary for Deep Excavation Protection Systems.

Studies	Research Focus	Method / Approach	Main Findings	Relevance
[17]	General deformation behavior of deep excavation support systems	Review of excavation support deformation patterns	Wall deflection and ground settlement are key serviceability indicators in deep excavations.	Supports the need to evaluate lateral wall movement and ground deformation in the anchored secant pile wall system.
[18]	Ground deformation associated with deep excavations in Beijing	Case-history analysis of deep excavation projects	Excavation depth, wall type, support stiffness, and soil condition strongly influence ground deformation.	Confirms that the 12.5–12.96 m excavation in Albania requires careful deformation control.
[19]	Deep foundation pit excavation supported by box-type retaining walls	Numerical simulation using FLAC-3D	Retaining wall geometry significantly affects wall displacement and ground settlement.	Supports the importance of selecting 0.80 m diameter piles and sufficient wall embedment.
[20]	Monitoring deformation of secant pile walls	Distributed fiber optic sensing	Fiber optic sensors can effectively monitor excavation-induced deformation in secant pile walls.	Supports the monitoring recommendation using inclinometers and topographic marks.
[21]	Deep excavation effects on adjacent tunnels	Field case and deformation analysis	Deep excavation in soft soil can influence nearby tunnel deformation and ground movement.	Highlights the importance of controlling excavation-induced

				displacement in urban areas.
[22]	Influence of deep foundation pit excavation on adjacent tunnel structures	Numerical simulation	Excavation support stiffness and construction sequence affect tunnel response.	Supports the staged excavation approach adopted in this study.
[23]	Foundation pit excavation in reclaimed land near subway tunnels	Numerical and deformation analysis	Excavation can cause significant deformation in adjacent underground structures.	Reinforces the need for safe retaining systems when excavating in urban environments.
[24]	Design and deformation pattern simulation of deep excavation support structures	Simulation-based excavation analysis	Support system design strongly affects deformation distribution and structural safety.	Supports the use of numerical and staged analysis for the secant pile wall.
[25]	Comparison of anchored, strutted, and cantilever secant pile walls	Parametric and comparative study	Anchored and strutted systems reduce lateral displacement more effectively than cantilever systems.	Directly supports the selection of an anchored secant pile wall instead of a cantilever wall.
[26]	Pile-anchor retaining structure in deep foundation pits	Numerical simulation and displacement analysis	Pile-anchor systems can control lateral displacement when properly staged and supported.	Closely related to the two-level prestressed anchor system used in the present project.
[27]	Optimization of excavation pit design	Finite element-based optimization	Optimized wall dimensions and support arrangements can improve safety and economy.	Supports the balanced design between structural stability, cost, and constructability.
[28]	Soil parameter inversion and deformation prediction	PSO-SVM data-driven model	Monitoring data can improve prediction of excavation deformation.	Supports using field monitoring data to verify the excavation response.
[29]	Groundwater and deformation control during foundation pit dewatering	Numerical simulation of horizontal barriers	Dewatering and seepage control influence settlement and wall deformation.	Relevant because groundwater occurs at about 7.50–8.00 m in the studied excavation.
[30]	Foundation pit dewatering and	Numerical simulation	Dewatering design affects groundwater drawdown, soil	Supports the need for pumping capacity of 3–4 L/s

	seepage reduction		settlement, and excavation stability.	during deeper excavation stages.
[31]	Lateral pressure reduction behind sheet-pile walls	Experimental/numerical study using EPS layer	Reducing lateral pressure can improve retaining wall behavior.	Supports the general concept of controlling lateral pressure through wall stiffness and anchors.
[32]	Effects of deep foundation pit excavation on retaining structures and subway stations	Deformation and structural response analysis	Excavation-induced deformation can affect retaining structures and adjacent facilities.	Confirms the need for deformation monitoring and serviceability control.
[33]	Deep excavation adjacent to existing tunnels	Numerical study using CPTU and SDMT calibration	Accurate soil model calibration improves prediction of excavation behavior.	Supports the importance of reliable soil parameters in the nine-layer soil profile.
[34]	Deformation characteristics of diaphragm walls in foundation pit excavation	Field/numerical analysis	Retaining wall deformation depends on wall stiffness, excavation depth, and construction sequence.	Provides comparative relevance to the secant pile wall system adopted in this study.
[35]	Optimal design of secant pile retaining systems	Evolutionary optimization methods	Optimization methods can improve secant pile wall design efficiency.	Supports the engineering value of selecting a technically and economically suitable secant pile system.
[36]	Intelligent model selection during staged excavation	Optimization and updating of soil parameters	Model updating improves excavation prediction during staged construction.	Supports the staged evaluation and monitoring-based verification proposed in this work.

RESEARCH GAP

The research gap deep-excavation research is usually focused on one of the topics, namely wall movement/ground surface settlement, type of retaining-system monitoring or dewatering but does seldom provide an integrated verification workflow fusing these elements for a medium-depth anchored hard/firm secant pile wall within Albanian urban geology. The crazy thing is that the gap here is not just a missing case description it is simply the lack of a standardised and verifiably accurate benchmark relating wall geometry, anchor activation, groundwater control and monitoring built.

The second gap relates to the limited use of a normalised serviceability index, $\delta max/H$, in case studies when depth of excavation is approximately 12-13 metres. Without normalised deformation indices, one cannot compare results from different depths of excavations, wall types and soil conditions. This paper addresses this gap by providing the final expected displacement of 12.1 mm at $\delta max/H$ = 0.094% alongside the previously established acceptability boundary of 0.5% and state-of-the-art deformation envelopes proposed recently in literature.

A third deficiency is validation. Several of the previous studies apply numerical models; however, direct evidence showing that the predicted responses were validated with monitoring data, reference envelopes or ranges of sensitivity is often absent from this literature. This version evaluates numerical results through threshold affirmation; comparison against a benchmark; anchor-effect comparisons and an observational method that lays out activity thresholds green, amber and red with respect to building, see Table 2.

Table 2. Quantified SOTA Gap and Benchmarking Matrix.

SOTA Dimension	Limitation in Previous Studies	Quantitative Item Added in This Manuscript	Benchmark / Acceptance Value	Result in This Case Study
Deformation control	Wall movement is often described without a clear numerical acceptance limit.	Maximum wall displacement expressed as a percentage of excavation depth.	Common serviceability limit: 0.5% of excavation depth.	Maximum wall displacement = 0.094% of excavation depth.
Excavation support system	Many studies discuss retaining systems generally, without linking the wall type to excavation depth.	Excavation depth, pile diameter, pile length, and anchor arrangement are clearly stated.	Support system must be suitable for excavation deeper than a simple cantilever wall.	Excavation depth is about 12.5–12.96 m, with 0.80 m diameter piles and 18.00 m pile length.
Anchor performance	Anchor systems are often described without showing their role in reducing wall movement.	Two anchor levels, anchor spacing, total length, and bond length are included.	Anchors should transfer load behind the active failure zone and reduce wall displacement.	Two anchor levels were used, with 2.80 m spacing, 20–22 m total length, and 10–12 m bond length.

Groundwater control	Groundwater is sometimes mentioned but not connected clearly to excavation safety.	Groundwater level and pumping capacity are reported.	Dewatering is required when excavation extends below the groundwater level.	Groundwater occurs at about 7.50–8.00 m, and pumping capacity is 3–4 L/s.
Staged excavation	Previous work often focuses on final excavation depth only.	Construction phases and anchor activation are included.	Support should be activated before excessive unsupported wall height develops.	The excavation is divided into staged excavation and anchor installation phases.
Validation approach	Some studies do not clearly show how design results are checked.	Semi-validation is added using serviceability limits and SOTA comparison.	Predicted behaviour should be checked against accepted limits and literature benchmarks.	Displacement result is below the 0.5%H limit.
Monitoring and field control	Monitoring is often recommended generally but not linked to acceptance.	Inclinometer monitoring, topographic monitoring, and anchor pull-out tests are included.	Field control should confirm wall movement and anchor capacity.	The manuscript recommends testing 5–10% of anchors.
Seismic consideration	Many excavation case studies do not include seismic sensitivity, although Albania is active seismically.	A pseudo-static seismic check using horizontal seismic coefficient k_h is added.	Wall displacement, anchor bond, and passive resistance should remain acceptable under seismic loading.	The manuscript proposes checking the system using a project-specific k_h value.
Waterproofing	Waterproofing is often treated as a general grade list only.	Waterproofing is linked to actual basement use.	Parking/technical zones may accept Grade 1–2; occupied/commercial areas should meet Grade 3.	The secant pile wall is treated as temporary support, while permanent lining, joints, and water stops provide final waterproofing.

STUDY HYPOTHESES

The hypotheses are formulated as quantitative and falsifiable statements, see Table 3. Each hypothesis has a measurable variable, a threshold, and a verification method. This directly addresses the reviewer request that the hypotheses must be testable rather than descriptive.

Table 3. Quantitative and Falsifiable Research Hypotheses.

Hypothesis	Measurable variable	Acceptance criterion	Verification method
H1 - Deformation serviceability	Maximum lateral wall displacement δmax and excavation depth H	$\delta max/H$ <= 0.5%	Staged numerical model and inclinometer readings after each excavation phase
H2 - Model validation	Difference between predicted and measured/benchmark displacement	Mean percentage error <= 10% and RMSE reported	Compare model output with monitoring data; where monitoring is unavailable, use published benchmark envelopes
H3 - Anchor effectiveness	Final displacement with anchors vs unanchored reference case	Reduction in $\delta max/H$ >= 20%	Run identical model geometry with and without anchor activation
H4 - Groundwater control	Groundwater head, inflow rate, and signs of seepage/piping	Water level kept below active excavation base; pumping capacity 3-4 L/s maintained	Piezometer/well observations and daily dewatering logs
H5 - Seismic sensitivity	Pseudo-static horizontal coefficient k_h and resulting δ/H and FS	No red-level deformation and FS remain acceptable under project-specific k_h	Sensitivity calculation using Albanian/Eurocode seismic coefficient selected for the site

MATERIALS AND METHODS

Data Description

Data materials available in this investigation were based on results from the engineering design and analysis of deep pit protection system for the construction of three underground storeys at the building performed under "Çerçiz Topulli" Square in Gjirokastra, Albania. It contains the geotechnical, structural, hydrogeological and construction phase properties required for carrying out an anchor temporary secant pile wall performance assessment. The excavation plan used a pit depth of approximately 12.5 m, but the final level reached an approximate 12.96 m below natural ground surface

(NGS). The retaining structure is a rigid secant pile wall with (0.80 m) diameter and axial length of 18. 00m.The model includes 30 structural elements, 9 soil strata and covers the loading phases reimplementing the sequence of excavation and support installation.

The fundamental geotechnical data are soil depth thickness, dry unit weight, saturated unit weight, soil cohesion as well as active and passive earth pressure coefficient and neutral horizontal stress coefficient along with lateral dislocation parameter. Attributes were used to define the mechanical response of each soil layer in excavation. The model thickness is 18.00 m from the ground extent to the wall tip. The uppermost layers of the soil show a low level of cohesion (between 10.00 and 30.15 kN/m²) while for deeper layers, there are elevated levels (up to 54.80 kN/m²). The impact of cohesiveness addition with depth is tangible because the deeper layers provide a greater amount of passive resistance in support of the embedded member.

Table 4 presents an overview of the main engineering data used for analyzing the deep pit protection system including excavation depth, type of retaining wall, pile geometry, anchor configuration, groundwater level and numerical modeling parameters. The table shows that excavation reaches to a depth and is supported by an anchored temporary hard/firm secant pile wall at approximately 12.5–12.96 m. Proposed execution, with two levels anchors; 0.80 m diameter piles, 18.00 m length of the pile, manage groundwater under 7.50–8.00 m etc., are almost all the basic input data to study stability and deformation control, construction safety management.

Table 4. Main Data Used in the Analysis.

Data Category	Parameter	Value / Description
Study data	Location	“Çerçiz Topulli” Square, Gjirokastra, Albania
Excavation data	Excavation purpose	Three underground parking levels
Excavation data	Design pit depth	12.50 m
Excavation data	Final excavation level	Approximately 12.96 m
Structural data	Retaining system	Anchored temporary hard/firm secant pile wall
Structural data	Pile diameter	0.80 m
Structural data	Pile length	18.00 m
Structural data	Pile spacing	0.70 m
Numerical model	Number of elements	30
Numerical model	Number of soil layers	9
Numerical model	Number of loading steps	5
Anchor data	Number of anchor levels	Two
Anchor data	Anchor spacing	2.80 m
Anchor data	Total anchor length	20–22 m
Anchor data	Bond length	10–12 m
Groundwater data	Water-bearing level	Approximately 7.50–8.00 m
Dewatering data	Pumping capacity	3–4 L/s

Figure 1 shows the methodology framework used in investigating the deep pit protection It begins by defining the worksite, excavation goal, and excavation depth then proceeds with the gathering of key input information such as soil layering, groundwater levels, geometric parameters and structures. The illustration depicts the engineering sequence used in the study, including geotechnical characterization, anchored hard/firm secant pile wall selection and design procedure, numerical modelling and simulation of staged excavation with anchor installation as well as groundwater management and stability verification. The last stages include the monitoring of and control on site and the acceptance of design allowing to check that the recommended temporary excavation support system guarantees a safety level as well as means available to build it.

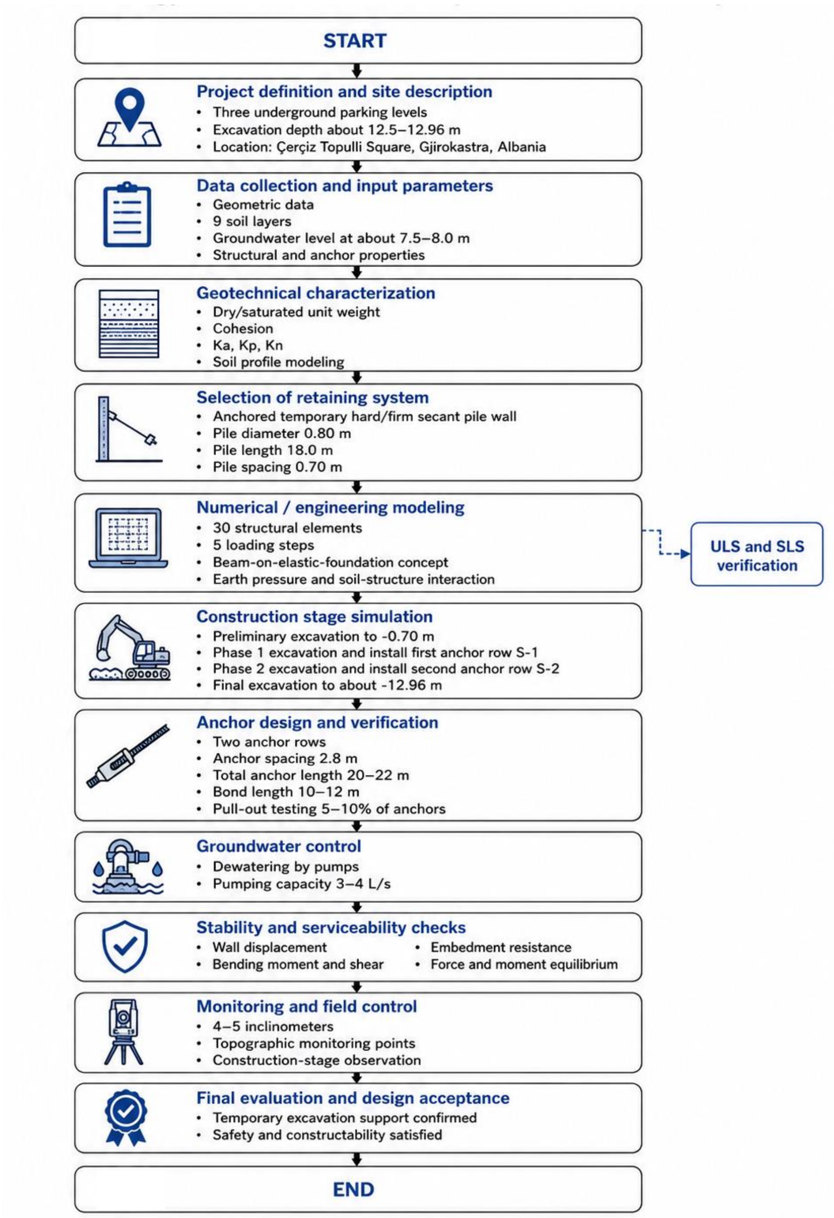

Figure 1. Methodology Flowchart of the Deep Pit Protection Study

Finite Element Modelling Framework

A two-dimensional plane-strain finite element modelling (FEM) framework, consistent with a PLAXIS 2D-type workflow or equivalent geotechnical FEM solver, was defined to make the analysis reproducible. The retaining wall was represented as an embedded structural element with axial and bending stiffness derived from the 0.80 m pile diameter, while the ground anchors were modelled as prestressed anchor elements connected to bonded grout zones beyond the active failure wedge. The soil was modelled using a Mohr-Coulomb constitutive law because the available input dataset contains unit weight, cohesion, friction-related earth pressure coefficients, and groundwater information rather than advanced stiffness parameters for hardening-soil calibration.

Boundary conditions were selected to avoid artificial constraint of the excavation response. The vertical side boundaries were placed at a distance not less than three times the excavation depth from the pit edge and restrained horizontally, while the lower boundary was fixed in both horizontal and vertical directions below the pile tip. The top surface was left free except for construction surcharge where applicable. The initial stress state was generated by gravity loading and the phreatic line was introduced at 7.50-8.00 m below the natural ground level. Dewatering phases lowered pore pressure inside the pit during deeper excavation stages.

Construction stages were modelled explicitly: Phase 0 initial ground condition; Phase 1 preliminary excavation to approximately -0.70 m and guide wall installation; Phase 2 excavation to the first anchor zone and activation of anchor S-1; Phase 3 deeper excavation; Phase 4 activation of anchor S-2; and Phase 5 final excavation to approximately -12.96 m. This phase-by-phase modelling is essential because the maximum bending moment, shear force, anchor demand, and lateral displacement change during construction rather than only at the final excavation state. Table 5 depict the added numerical model assumptions and reproducibility parameters.

Table 5. Added Numerical Model Assumptions and Reproducibility Parameters.

Item	Adopted modeling specification
Analysis type	2D plane-strain FEM / beam-on-elastic-foundation equivalent check
Soil model	Mohr-Coulomb with layer-specific unit weight, cohesion, and earth-pressure coefficients
Retaining wall	0.80 m diameter hard/firm secant pile wall, 18.00 m pile length, 0.70 m spacing
Anchors	Two prestressed rows; 2.80 m spacing; 20-22 m total length; 10-12 m bonded length
Groundwater	Initial water level at 7.50-8.00 m; dewatering activated in deeper excavation phases
Boundary conditions	Side boundaries at >= 3H; horizontal restraint at sides; fixed base below pile tip
Mesh/check	Local refinement near wall, anchor heads, excavation base, and groundwater transition
Output variables	$\delta max/H$, bending moment, shear force, anchor force, water level, safety margin
Acceptance check	δ/H <= 0.5%; anchor tests 5-10%; green/amber/red monitoring limits

Wall System

A hard/firm secant pile wall was selected because the excavation depth was too large for a simple cantilever system. The wall consists of primary and secondary piles installed in an overlapping arrangement. The primary piles are constructed first, and the reinforced secondary piles are drilled between them to form a continuous retaining barrier. The main wall dimensions were the pile diameter $D_p = 0.8$ m and the pile spacing $S_p = 0.7$ m.

The pile overlap was calculated via equation (1):

$$O_p = D_p - s_p \tag{1}$$

The 0.10 m overlap improves continuity and helps the wall act as a unified retaining system.

Soil Model

The soil profile was represented by nine layers extending from ground level to the pile tip. Each layer was defined by dry unit weight, saturated unit weight, cohesion, active earth pressure coefficient, passive earth pressure coefficient, neutral pressure coefficient, and displacement parameter. The upper layers had lower cohesion, while the deeper layers showed higher cohesion and passive resistance. Cohesion increased from 10.00 $\mathrm{kN/m^2}$ in the upper layer to 54.80 $\mathrm{kN/m^2}$ in the deeper layers.

The vertical stress at depth z was expressed as:

$$\sigma_n = \gamma z \tag{2}$$

For dry soil:

$$\sigma_n = \gamma_d z \tag{3}$$

For saturated soil:

$$\gamma' = \gamma_{sat} - \gamma_{wt} \tag{4}$$

$$\sigma_v' = \gamma' z \tag{5}$$

where γ_d is dry unit weight, γ_{sat} is saturated unit weight, γ_{wt} is unit weight of water, and σ_{vt}' is vertical effective stress.

Earth Pressure

The wall was subjected to lateral pressure from soil, surcharge, and groundwater. The total horizontal pressure was defined as:

$$p_h(z) = p_a(z) + p_q(z) + p_w(z) \tag{6}$$

where $p_s(z)$ is soil pressure, $p_q(z)$ is surcharge pressure, and $p_w(z)$ is water pressure. The active earth pressure was calculated as:

$$\sigma_a = K_a \sigma_v' - 2c\sqrt{K_a} \tag{7}$$

The passive resistance was calculated as:

$$\sigma_p = K_p \sigma_v' + 2c\sqrt{K_p} \tag{8}$$

where K_a is the active pressure coefficient, K_p is the passive pressure coefficient, and c is soil cohesion. The surcharge effect was calculated as:

$$p_q = K_\alpha q \tag{9}$$

The resultant active force was obtained by integration:

$$P_a = \int_{z_1}^{z_2} \sigma_n(z)dz \tag{10}$$

The resultant passive force was obtained as:

$$P_p = \int_{z_1}^{z_2} \sigma_p(z)dz \tag{11}$$

Wall Model

The secant pile wall modelled as embedded vertical beam supported by soil resistance and prestressed anchors [37-39]. The wall stiffness was represented by:

$$EI = E \cdot I \tag{12}$$

where E is the modulus of elasticity and I is the second moment of area. For a circular pile:

$$I = \frac{\pi D_p^4}{64} \tag{13}$$

Using $D_p = 0.80$ m, it has been found that I correspond to 0.0201 m^4

The lateral response of the wall was represented by the beam-on-elastic-foundation equation:

$$EI\frac{d^4y}{dz^4} + k_h y = p_h(z) \tag{14}$$

where y is wall displacement and k_h is the horizontal subgrade reaction coefficient. The bending moment and shear force were expressed as:

$$M(z) = -EI\frac{d^2y}{dz^2} \tag{15}$$

$$V(z) = \frac{dM}{dz} \tag{16}$$

The wall was checked using the conditions $M_{ed} \le M_{Rd}$, and $V_{ed} \le V_{Rd}$

Anchor Design

The wall was supported by two rows of temporary prestressed ground anchors. The first row was located approximately between -2.53 m and -4.32 m, and the second row was located between -5.68 m and -7.85 m. The anchor spacing was 2.8 m, the total anchor length was 20-22 m, and the bond length was 10-12 m.

The horizontal and vertical components of anchor force were:

$$T_k = T\cos\alpha \tag{17}$$

$$T_n = T sin\,\alpha \tag{18}$$

where T is anchor tensile force and α is anchor inclination angle. The total horizontal anchor resistance was:

$$T_{\text{total}} = \sum T_k \tag{19}$$

The stabilizing moment from anchors was:

$$M_A = \sum T_{h,i} z_i \tag{20}$$

where z_i is the lever arm of each anchor row.

Bond Check

The anchor bond length was verified using two mechanisms: soil-grout bond and strand-grout bond. The soil-grout bond capacity was expressed as:

$$N = \pi D_b L_f (\gamma_d H_b K_n \tan\delta + c_a) \tag{21}$$

Thus:

$$L_f = \frac{N}{\pi D_b (\gamma_d H_b K_n \tan\delta + c_a)} \tag{22}$$

where D_b is grout bulb diameter, L_f is bond length, H_b is overburden above the bond zone, K_n is neutral lateral pressure coefficient, δ is soil-grout friction angle, and c_α is adhesion.

The interface friction angle was assumed as:

$$\delta = 0.6\phi \tag{23}$$

The adhesion was estimated as:

$$c_\alpha = \frac{2}{3} c_u \tag{24}$$

The strand-grout bond was checked as:

$$N = \pi d_s L_f f_{bd} \omega \tag{25}$$

Thus:

$$L_f = \frac{N}{\pi d_a f_{bd} \omega} \tag{26}$$

The reduction coefficient was:

$$\omega = 1 - 0.075(n - 1) \tag{27}$$

The adopted bond length was selected as:

$$L_{\text{bond}} = \max\left(L_{f,\,\text{soil-grout}}, L_{f,\,\text{strand-grout}}\right) \tag{28}$$

In this project, the adopted bond length was 10-12 m.

Prestressing

Anchor prestressing was applied after grout hardening. The lock-off force was limited to prevent overstressing of the tendon [40]:

$$P_0 \leq 0.6P_{tk} \tag{29}$$

For a standard [41] T15 strand P_{tk} = 260 kN, and P_0 correspond to 156 kN. In case of the super T15 strand with P_{tk} = 260 kN, and P_0 correspond to 167.4 kN.

Serviceability Limit States (SLS)

Serviceability Limit States associated with service performance requirements insufficiently fulfilled.

According to 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.

Construction Sequences Appropriate for Temporary and Permanent Works

Once site preparation works have been completed, construction usually start with installation of the embedded retaining wall. Any excavation prior to 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 the 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.

For deep excavation is not suitable using simplest option of cantilever wall because of unacceptable deflection during the temporary excavation stages. Table 6 lists the advantages and limitations of using a cantilever wall.

Different alternatives to a cantilever solution can be 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.

Table 6. 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

A method for the excavation and construction of the substructure that is compatible with limited access. This can become 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. In general, it is uneconomic to use top-down sequence to reduce the superstructure time completion unless there are more than two levels substructure. Table 7 lists the advantages and limitations of using top-down construction sequence.

Table 7. 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 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, see Table 8.

Temporary props of steel sections are associated with safety risks. The sequential construction of the superstructure and the substructure minimize conflicts between different operations on site resulting simpler compared with top-down sequence.

Table 8. Lists of the Advantages and Limitations by Using Bottom-Up Construction Sequence.

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	

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 9 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 9. 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 are 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.

Wall Types

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.

Sheet Piles

Different sections common for temporary and permanent works can be used. Some typical sections are shown in the Figure 2.

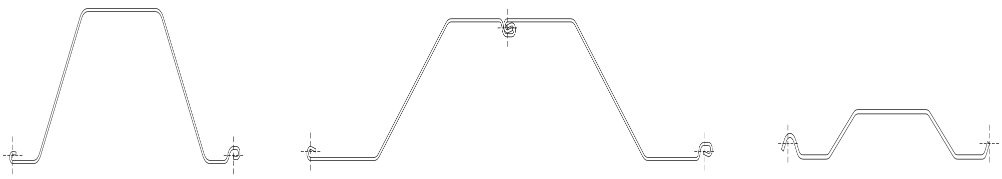

Figure 2. Sheet Pile Typical Sections

Contiguous Piles

A contiguous pile consists of bored cast-in-place piles along the line of the wall as shown in the Figure 3 where 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 5-15 cm.

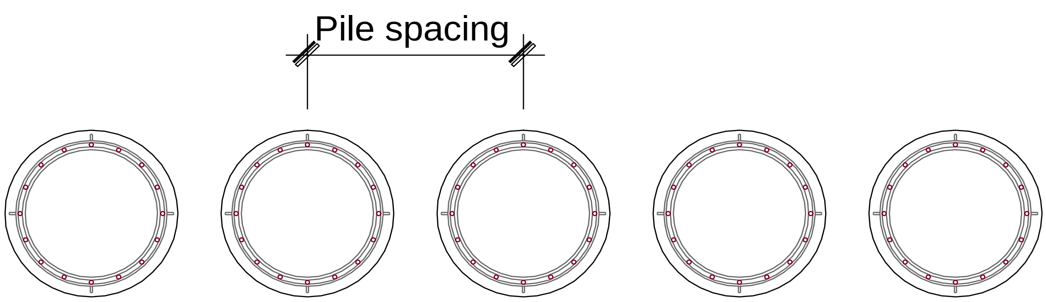

Figure 3. Contiguous Pile Wall

The maximum achievable wall depths are 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 10, although the use of diameters greater than 120 cm is unusual.

Table 10. Contiguous Pile Wall-Typical Diameters and Spacing.

Diameter (mm)	Spacing (mm)	Diameter (mm)	Spacing (mm)	Diameter (mm)	Spacing (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 Figure 4 the primary (female) piles are cast first and consist of a soft pile, typically cement, bentonite and sand with compressive strength of 1-3 N/mm². They are unreinforced.

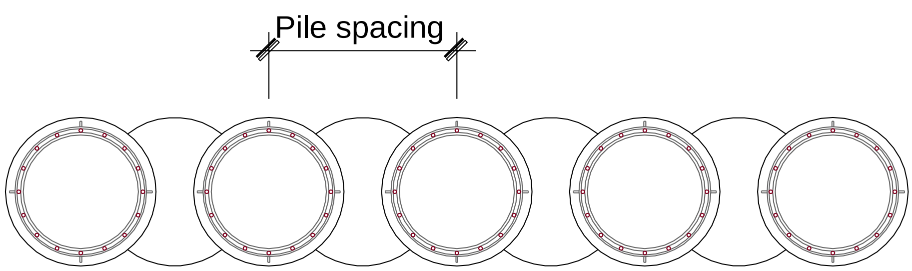

Figure 4. Hard/Soft Secant Pile Wall

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 scanting. The male and female piles may have the 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 groundwater level penetrating fine-grained nonwatery-bearing strata.

Typical diameters and pile spacing are shown in Table 11.

Table 11. 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 Figure 5. The female pile has a characteristic compressive strength of 10-20 N/mm^2, 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.

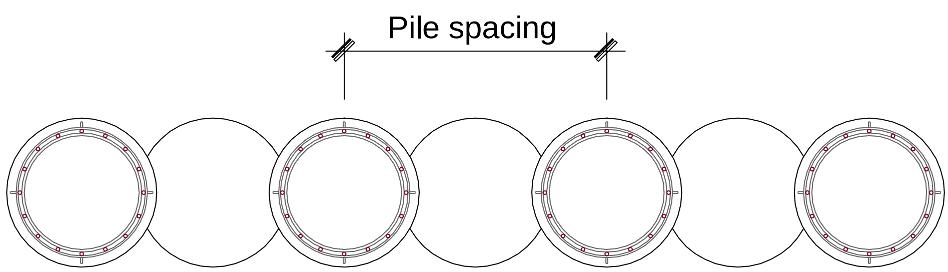

Figure 5. Hard/Firm Secant Pile Wall.

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 12.

Table 12. Hard/Firm Secant Pile Wall-Typical Diameters.

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 pile is 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 Figure 6., the female pile reinforcement should be detailed and placed to avoid being cut during the installation of the male piles.

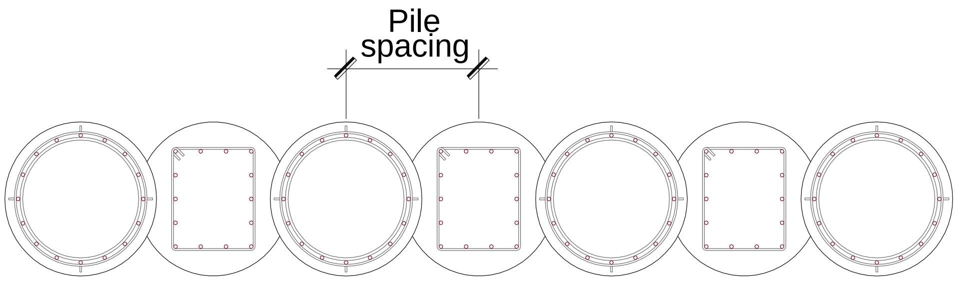

Figure 6. Hard/Hard secant pile wall.

The limiting depth for hard/hard secant pile walls is about 25 m and is limited by the piling rig's ability to rotate casing. The verticality tolerances ensure that scanting 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 13.

Table 13. Hard/Hard Secant Pile Wall-Typical Diameters and Spacing.

Diameter (mm)	Spacing (mm)
Male and Female	
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 tremie pipe to displace the fluid. Excavation is carried out by 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. Figure 7 shows a typical diaphragm wall panel. Panel width is a function of the grab and cutter widths available and are typically 600, 800, 1000 and 1200 or 1500 mm wide. The number of bites to excavate the panel and the grab or cutter length determine the panel length. A typical grab length is 2.8 m, but may reduce to 2.2 m.

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

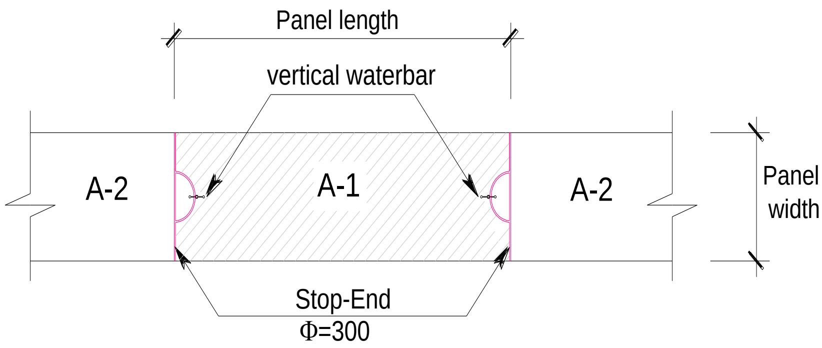

Figure 7. Diaphragm Wall Panel and Joint.

The incorporation of 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, polyvinilcloride) are shown in Figure 8.

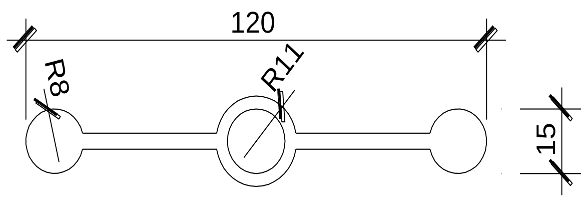

Figure 8. Vertical Waterbar (PVC, Polyvinilcloride) Typical Dimensions.

Table 14 lists different wall types with their advantages and limitations based on [42].

Table 14. Wall-types

Wall Type	Advantages	Limitations
Sheet piles	Provide an economic wall with a predictable surface finish. No arising to be removed Suitable as a water-retaining wall. Can be used as both the temporary and the permanent wall.	Maximum pile length approximately 30 m. Potential grabbing in coarse-grained soils.

Contiguous pile	The cheapest form of concrete piled Wall.	Not a water-retaining solution. Not a permanent solution in any soil. due to the gaps between piles, unless. a structural facing is applied.
Hard/soft secant	Act as a water-retaining temporary. Wall: The use of soft piles enables the hard piles to be formed using lower-torque. rigs than for hard/hard secant piles.	Not usually a permanent solution to Retain water: The soft pile mix is not significantly cheaper that concrete. The depth is limited by the vertically tolerance, which may determine the extent of scanting.
Hard/firm secant	A permanent water-retaining wall. The firm material for the primary. (female) piles is either a standard. concrete mix, retarded to reduce the strength when the secondary (male). piles are constructed or reduced-strength concrete mix.	The depth is limited by the vertically tolerance, which may determine the extent of scanting.
Hard/hard secant	A permanent water-retaining wall Installed using standard piling, with high-torque rigs	The cutting of the hard primary (female) piles requires high-torque rigs. The depth is limited by the vertically tolerance, which may determine the extent of scanting
Diaphragm wall	A permanent water-retaining wall. Can be installed to great depths. In some circumstances the face of the diaphragm wall can form the finish subject to some surface cleaning and removal of irregular parts. Fewer joints compared with piled walls.	Horizontal continuity is difficult to achieve between panels cannot follow complex plan outlines. The extensive installation of reinforcement cages, support fluid and excavation equipment's, requiring a large area for accommodation. Disposal of the support fluid is costly.

Project-Specific Waterproofing and Seismic Revision

The waterproofing specifications at every level of each basement must be based on the use of that inside space, rather than simply a generic list by grade, see Table 15. Registering

Grade 1 to Grade 2 performance is generally acceptable for parking and technical zones depending on the architectural specification, but any occupied or commercial basement area needs to be satisfied at Grade 3. The secant pile wall should be treated as a temporary support for retention and groundwater reduction, whereas waterproofing must ultimately be provided by the permanent basement lining, joints, slab-wall connections and water stops.

Albania is a seismically active region, therefore the pseudo-static sensitivity analysis for temporary excavation support must be included. The verification uses a project-based horizontal seismic coefficient k_h based on the relevant Albanian Eurocode hazard factors. The checked item is that secant pile wall and anchors are above the red - level deformation threshold, or the anchor bond and passive resistance under selected k_h kept appropriate. The current addendum addresses an interesting note from a reviewer requesting seismic design consideration in Albania.

- Grade 1 – Some water seepage and damp are tolerable depending on the intended use. Car parks etc.
- Grade 2 – No water penetration is acceptable. Damp areas are tolerable depending on the end use. Boiler rooms, workshops etc.
- Grade 3 – No dampness or water penetration is acceptable – Ventilated residential and commercial areas such as homes, offices, shops etc.

Table 15. Typical Applications of Embedded Retaining Walls

Wall Type	**Permanent.**	**Temporary.**	**Cost Effective Height Range**		**Lateral Movement**	**Water Tightness**	**Verticality**	
			Cantilever	Propped			Typical	Optimum
Sheet-pile wall	√	√	to 5 m	4 – 20 m	Large	Fair	1:75	1:100
Slurry (diaphragm) wall (grab)	√	√	to 8 m	5 – 30 m	Small	Good	1:75	1:125
Slurry (diaphragm) wall (mill)	√	√	to 8 m	5 – 50 m	Small	Good	1:200	1:500
Contiguous pile wall	√	√	to 5m	4 – 20 m	Small	Fair	1:75	1:125
Hard/soft Secant pile wall	√	√	to 5m	4 – 20 m	Small	Fair	1:75	1:125
Hard/firm Secant pile wall	√	√	to 6m	4 – 20 m	Small	Fair	1:75	1:125
Hard/hard Secant pile wall	√	√	to 6m	4 – 25 m	Small	Fair	1:75	1:200

Cost Data for Various Embedded Wall Types

The use of the wall in the permanent condition in addition to the temporary case ensures that the wall is only formed once reducing cost. It is important to consider the full implications of the use of the temporary wall as the permanent wall to ensure a cost saving for the overall structure. The issues influence the overall cost:

- The space occupied by the retaining wall.
- The watertightness criteria for the inner face of the permanent wall.
- The use of an inner lining wall to form a drained cavity keeping hidden inner face of the embedded wall.
- The connection details between the wall and the permanent slabs. These details are difficult and costly.

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 or the use of diaphragm walling techniques, with need to accommodate support fluid mixing, reinforcement cage and storage facilities.
- Environmental issues.
- Speed of construction.

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 -12.5 m below the terrain.

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 temporal character defining the inner dimensions of the pit.

The secant pile wall as a specific structure which is used to ensure stability of the soil installing primary(female) and secondary(male) piles with diameter d=80 cm, height h=20 m; h=18 m (h=18 m; h=16 m for primary pile respectively) and pile spacing s=70 cm.

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 the statically aspect the structure represents a multi anchored secant pile wall. The anchoring is made be using a specific pre-stress anchors introducing additional stabilizing force into the system. Based on it, 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=2.8 m. A detailed layout is presented in respective drawings.

The anchoring system was based on two prestressing anchor types 5T15 and 6T15 that were selected previously. The number before the designation T15 indicates quantity of prestressing strands; therefore, 5T15 is five strands and 6T15 is six. Each strand contains seven wires (approximately $\varphi = 5$ mm), with a nominal strand diameter of 15.2 mm, corresponding to the T15 designation. On the other hand, the anchors installed in two levels below ground level: The 1st row S-1between a depth of approximately −2.53 to −4.32 m; and the 2nd row S-2 between a depth of approximately -5–68 to -7–85 m; Total anchor length is L=20–22 m, bonded/grouted length at about 10-12 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 thickness 17.5 cm.

Technical Description and Construction Procedure

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

- Preliminary excavation

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

- Guidelines for construction of the secant pile wall

The reinforced-concrete guidelines are first set with respect to exact survey as two parallel beams making an opening of 82 cm 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 purred in the excavated segment. The female unreinforced piles should be installed first to a depth h=16 m, h=18 m (specified in drawings). The overlap 10 cm 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 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 continuous pouring 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:

a. Preliminary phase, excavation 1526ntil -0.70 m and construction of guide walls.

b. Phase-1, excavation 1526ntil -2.97 to-5.66 m (Installation, First Row Anchor S-1).

c. Phase-2, excavation 1526ntil -6.12 to -8.81 m (Installation, Second Row Anchor S-2).

d. Phase-3, excavation 1526ntil -10.25 to -12.96 m.

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 7.50 – 8.0 m. During the excavation beneath this level, water can be kept under control by lowering the level by pumps with capacity Q= 3-4 l/s.

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

- Anchorage

The boring of the anchors is made by rotational drilling and piping with diameter from 150 mm.

The anchors are comprised from two lengths, namely a free length of the anchor which is variable in different sections and depths and bond or grouted length of the anchor which varies from L_b= (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 it's 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 P_{tk}. Strand technical characteristics and temporary anchors are shown in Table 16 and Figure 9 respectively.

Monitoring and Field Validation Plan Added

Basal and different phase-particular tracking varieties the crux of the validation plan up to date. Inclinometers have to be positioned behind representative wall sections and should be measured before excavation, after each phase of excavation, after locking of each anchor, after significant rainfall or change in groundwater levels and after excavation has been completed. Position topographic survey points on the capping beam, adjacent pavement and surrounding structures. Groundwater level must be monitored during the dewatering process by means of piezometers or observation wells. Additionally, 5-10% of anchors must undergo anchor proof tests in the as-installed condition, and all production anchors should be verified versus the approved lock-off load.

Table 16. Strand Technical Characteristics.

Diameter	**Standard**	**Strand Type**	**Nominal Diameter (mm)**	**Nominal Area (mm²)**	**Mass (g/m)**	**ftk (N/mm²)**	**Breaking Strain Ptk (kN)**	**Elastic Limit at 0.1% P0.1k (kN)**	**Relaxation after 1000 h, ϱ0.7–0.8 (%)**
T15	[41]	Standard	15.2	140	1095	1860	260	224	2.5–4.5
T15	[41]	Super	15.7	150	1170	1860	279	240	2.5–4.5

Figure 9 shows the temporary prestressed ground anchor used in the deep pit protection system for “Çerçiz Topulli” Square excavation. According to the document [14], the anchors are installed using a 150 mm rotating drilling diameter and consist of two main parts: an unbonded/free length and a bonded/grouted length. This length consists of greased and PE/PP-coated strands allowing the tender to extend freely under prestressing without applying load to the adjacent soil. The bonded length, as described in the specification 10–12 m, consists of simple strands within cement grout that are able to transmit tensile force to stable ground well outside the active failure zone. The prestressing force applied by the wedge, wedge plate and bearing plate at the head of the anchor is transmitted to the secant pile wall. The aforementioned excavation utilizes these anchors to reinforce the wall at 2 levels, control lateral displacement and assist in stability as staged excavation proceeds to a depth of approximately 12.5–12.96 m. The document states that following grout hardening the anchor is prestressed and that 5–10% of total anchors are to be subject to pull-out tests for field capacity verification prior acceptance [40].

A system of anchored secant pile wall used to protect the underground excavation for the underground parking at “Çerçiz Topulli” Square in Gjirokastra, Albania is shown in Figure 10. The figure illustrates a vertical secant pile wall on the excavation face, adjacent soil layers, ultimate excavation depth and two rows of inclined prestressed anchors into competent ground core in front of the anticipated failure zone. The system uses the 0.80 m diameter secant piles and a pile length of 18.0 m, two anchor levels to support an excavation depth That approximates at depth 12·5–12·96 m in agreement with the recommended values from the paper.

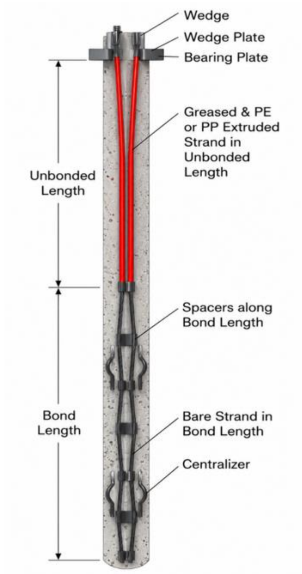

Figure 9. Typical Uses: Temporary Application with a Service Life of Less than 24 Months.

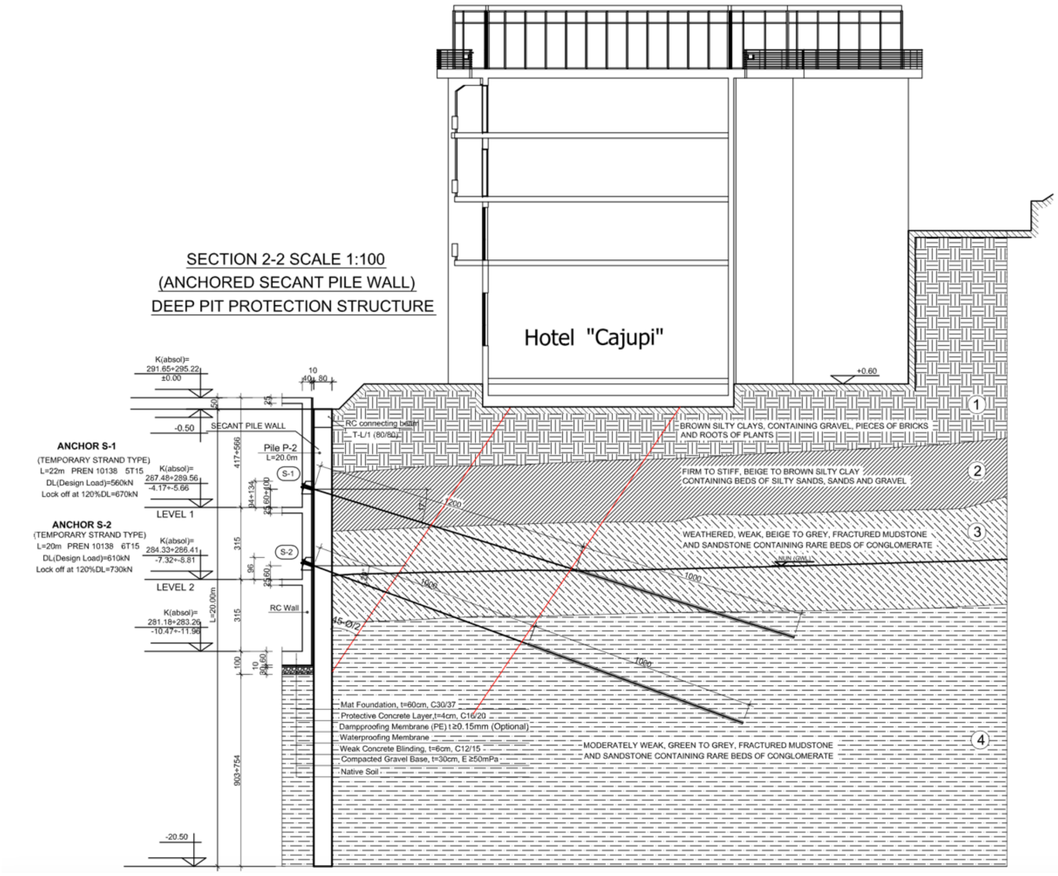

Figure 10. Anchored Secant Pile Wall

Soil Properties

The soil properties used in the analysis of deep pit protection structure is shown in the Table 17.

Table 17. Soil Properties Used in the Deep Pit Protection Analysis.

No. (Geol. Rep.)	Thickness (m)	W	W_0	W_d	Ws	n	φ	c	K_a	K_p	K_n	e
1	2.0	19.20	25.00	14.71	18.82	0.41	16.00	10.00	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	29.40	42.80	0.34	2.93	0.51	0.62
3	1.0	22.20	26.30	16.23	20.06	0.38	29.40	42.80	0.34	2.93	0.51	0.62
3	1.5	22.20	26.30	16.23	20.06	0.38	29.40	42.80	0.34	2.93	0.51	0.62
3	1.0	22.20	26.30	16.23	20.06	0.38	29.40	42.80	0.34	2.93	0.51	0.62
3	1.0	22.20	26.30	16.23	20.06	0.38	29.40	42.80	0.34	2.93	0.51	0.62
4	3.5	23.20	26.40	19.41	22.06	0.26	30.20	54.80	0.33	3.02	0.50	0.36
4	5.5	23.20	26.40	19.41	22.06	0.26	30.20	54.80	0.33	3.02	0.50	0.36

The main soil layers parameter used are as follows:

- 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.
- e: void ratio
- cap-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.
- Dw-stroke, lateral displacement difference between the generation of active and passive Lateral stress, in m.

$$Dw = (\sigma p - \sigma a)/Khv \tag{30}$$

The variation of active and passive stress σ_a, σ_b with the increase of depth is shown in Figure 11 in the research on deep pit protection in the active stress it remains low and nearly constant in most of the soil profile and then in stress passive increases means at overburden increasing from surface layer to depth reaching its maximum value close to 18.0 m depth. The reason for this tendency derives from the assumed greater resistance to passive earth pressure at depth, which plays a key role in supporting embedded parts of the secant pile wall below the excavation base. Thus, it can be concluded that pile embedment plays a predominant role in preventing lateral movement and increasing the global stability of the anchored retention system.

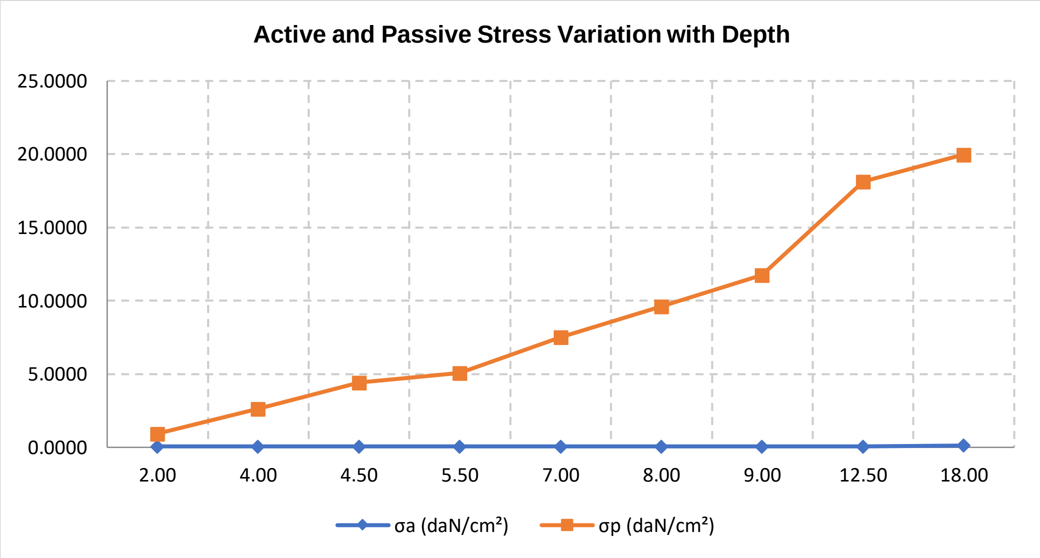

Figure 11. Active and Passive Stress Variation with Depth

RESULTS

The results indicate that the anchored hard/firm secant pile wall is suitable for protecting the excavation depth of 12.5-12.96 m in Gjirokastra, Albania the length of the pile is 18.00 m. This provides around 5.50 m of embedment below the base of excavation promoting passive resistance development in stronger, lower soil strata. The prestressed anchors are arranged into two rows and act to resist wall movement during the sequential excavation that may occur. Underground water management will be necessary, when it exceeded the aquifer layer with an approximate depth of 7.50–8.00 m.

The main geometric results of the retaining wall system are shown in Table 18. The 12.50 m excavation depth allows for about 5.50 m of embedment beneath the piles, which is essential in triggering passive resistance with a pile length of 18.00 m. The pile with 0.80 m diameter and 0.10 m overlap proves that the selected hard / firm secant pile wall has enough stiffness and continuity to bear the temporary deep excavation.

Soil data is converted into engineering zones based on the results as seen in Table 19. The upper soil layers are more deformable, and the deeper layers give greater passive resistance. This result explains the reason why piles must extend under the excavation base in order to stabilize the anchored secant pile wall.

Table 18. Interpretation of Retaining Wall Geometry

Result Indicator	Value / Condition	Engineering Interpretation
Design excavation depth	12.50 m	Confirms that the excavation is a deep pit requiring lateral support
Final excavation depth	≈ 12.96 m	Indicates that the actual construction level extends slightly below the design depth
Pile length	18.00 m	Provides sufficient penetration below the excavation base
Embedded length below design depth	5.50 m	Mobilizes passive resistance below the pit base
Pile diameter	0.80 m	Provides adequate bending stiffness for temporary wall support
Pile overlap	0.10 m	Creates continuity between primary and secondary piles
Selected wall system	Anchored hard/firm secant pile wall	Suitable for controlling deformation and maintaining an open excavation space

Table 19. Based Soil Strength Zonation

Soil Zone	Approximate Depth Range	Main Strength Condition	Result Interpretation
Upper weak zone	0.00–4.00 m	Lower cohesion and lower passive resistance	More sensitive to lateral movement and surface deformation
Intermediate stable zone	4.00–9.00 m	Moderate-to-high cohesion and improved passive coefficient	Provides increasing resistance against wall displacement
Deep passive zone	9.00–18.00 m	Highest cohesion and highest passive resistance	Main resistance zone for embedded pile stability
Excavation base zone	Around 12.50–12.96 m	Located within stronger lower soil layers	Supports toe stability of the secant pile wall
Anchor interaction zone	Around 2.53–7.85 m	Anchors pass through upper and middle layers	Transfers wall loads into more stable soil behind the active zone

Table 20 shows the stress-response trend seen in Figure 11. The active stress remains rather stable and minimal, but passive stress significantly escalates with depth. This verifies that the deeply immersed section of the pile wall is the primary stabilizing element countering lateral displacement.

The explanation of the anchor system's results is provided in Table 21, rather than simply repeating the anchor measurements. Two anchor rows work together against lateral wall movement during staged excavation. The requirement for pull-out testing and lock-off force ensures quality control during construction and allows confirmation that the temporary support system can safely support a given load.

Table 20. Summary of Active and Passive Stress Response.

Depth Level	Active Stress Behavior	Passive Stress Behavior	Engineering Meaning
2.00 m	Low active stress	Low passive resistance	Upper soil contributes limited resistance
4.00–5.50 m	Active stress remains low	Passive stress begins to increase	Soil resistance improves with depth
7.00–9.00 m	Active stress remains almost constant	Passive stress increases clearly	Middle layers contribute stronger wall support
12.50 m	Low active stress	High passive stress	Strong resistance near excavation base
18.00 m	Slight active stress increase	Maximum passive stress	Deep embedment provides the main stabilizing resistance

Table 21. Anchor System Performance Interpretation

Anchor Component	Result Condition	Performance Interpretation
First anchor row S-1	Installed at upper excavation level	Controls wall movement during early excavation
Second anchor row S-2	Installed at deeper excavation level	Provides additional restraint during advanced excavation
Anchor spacing	2.80 m	Provides regular lateral support along the wall
Total anchor length	20–22 m	Transfers tensile force beyond the active failure zone
Bond length	10–12 m	Provides sufficient grout–soil load-transfer capacity
Pull-out testing	5–10% of total anchors	Confirms field anchor resistance before acceptance
Lock-off force limit	$P_0 \leq 0.6P_{tk}$	Prevents overstressing of temporary anchors

The variation of soil cohesiveness with depth of deep pit protection is shown in Figure12. Cohesiveness increases from 10 kN/m² in the upper layer to 54.8 kN/m² lower soil profile indicating better shear resistance in deeper layers. This is an important behavior because the embedded portion of the secant pile wall extends through these stronger zones allowing for greater passive resistance below the base of excavation. Hence the new finding in some of case studies assures that the lower soil layers contribute largely to both system stability of an anchored retaining wall.

The active earth pressure coefficient K_a, passive earth pressure coefficient K_p and neutral pressure coefficient $K_n$vary with depth as shown in Figure13. Nevertheless, K_a decreases from layers to depth while K_p increases and higher in the lower soil strata. This means that the driving pressure decreases with depth and the resisting passive pressure increases. Thus, it supports the selection of this pile embedment depth as the underlying strata provide a higher lateral resistance to wall movement.

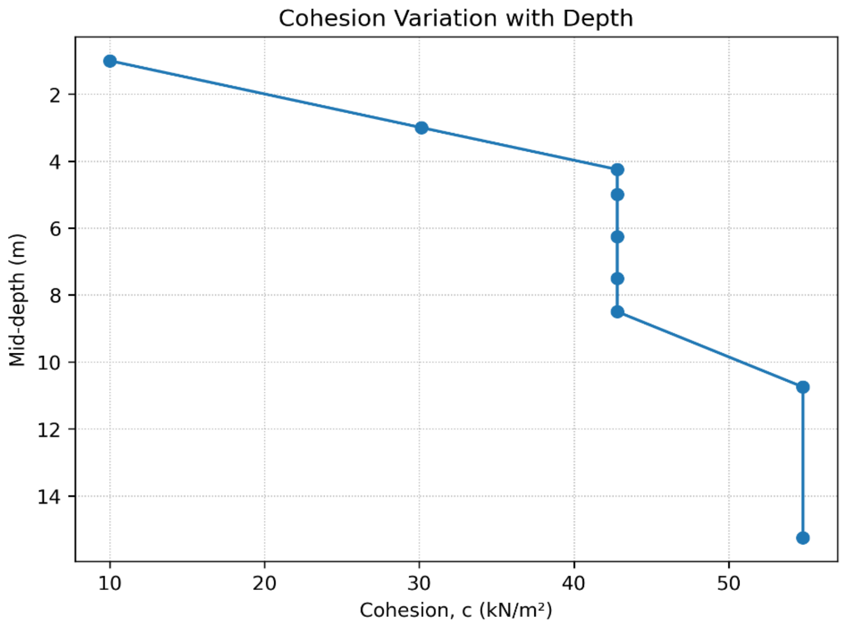

Figure 12. Cohesion Variation with Depth

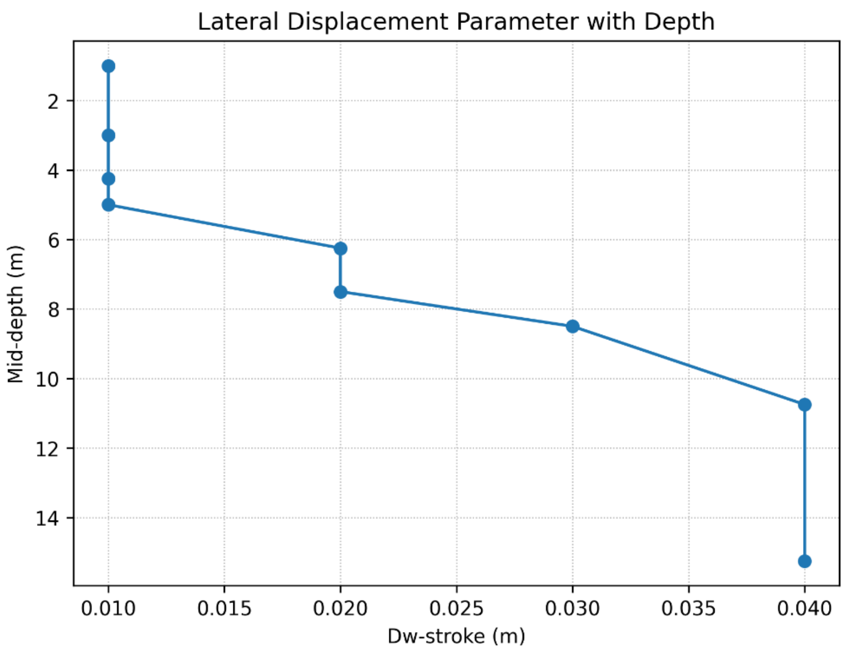

Figure 13. Cohesion Variation with Depth

The variation of D_w with depth is shown in Figure 14. it equals approximately 0.01 m per the top layers, then increases to nearly 0.04 m in depth level upward. Such behavior indicates a deeper depth of soil strata at which passive resistance is greater, and hence more displacement mobilization. This Figure explains the reason for producing lateral soil reaction on the excavated portion of a secant pile wall.

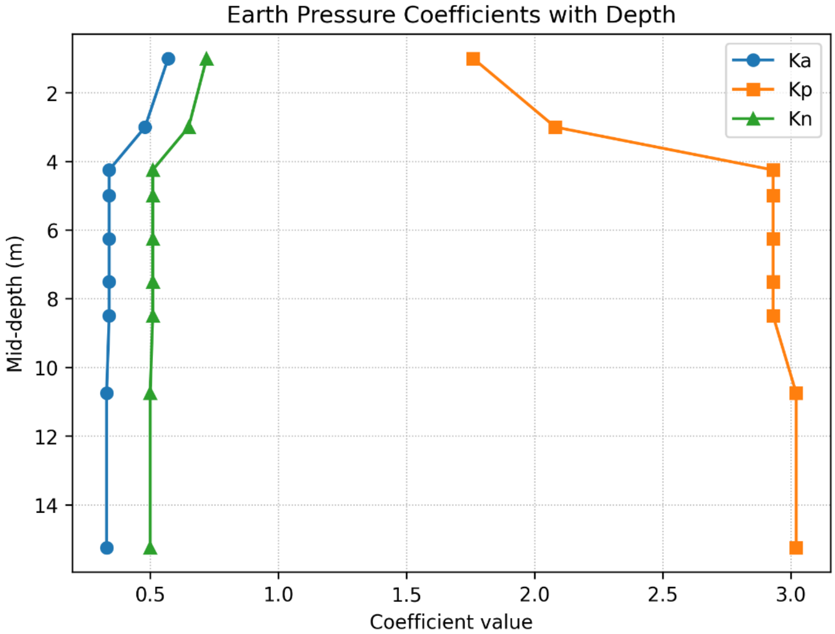

Figure 14. Earth Pressure Coefficients with Depth

Figure 15 Genetic units in plain view and the thickness distribution of soil strata used in performances of nine soil strata included investigations. The upper and intermediate layers are relatively thin, while the deepest one has generally the greatest thickness (about 5.5 m). It is also denser and heavier, providing passive resistance since it coincides with the embedded part of the secant pile wall. The figure confirms that the soil mass beneath fills up most of a stabilization zone underneath an excavation base (below the retaining structure).

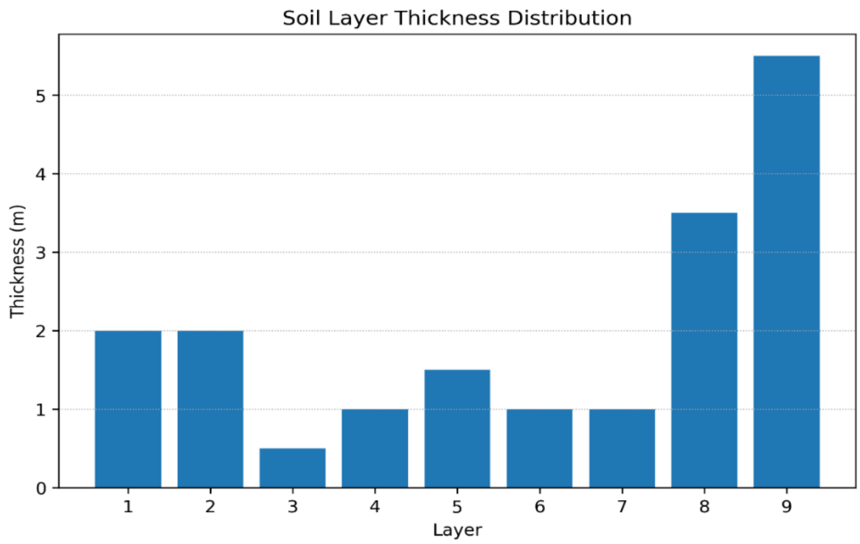

Figure 15. Soil Layer Thickness Distribution

Figure 16 shows the dry unit weight W_d is compared to the saturated unit weight W_s along the soil profile. Results demonstrate that the saturated unit weight is greater than the dry unit weight for all Heads, which suggests that groundwater and soil saturation conditions significantly influence unit weight. Both values increase in the deeper layers, with W_d approximately reaching 19.41 kN/m³ and W_s ultimately equating to around 22.06 kN/m³. This increase is an indicator of more compact and stronger lower soil layers, resulting in reinforced stability of the embedded secant pile wall as well as increasing excavation stability.

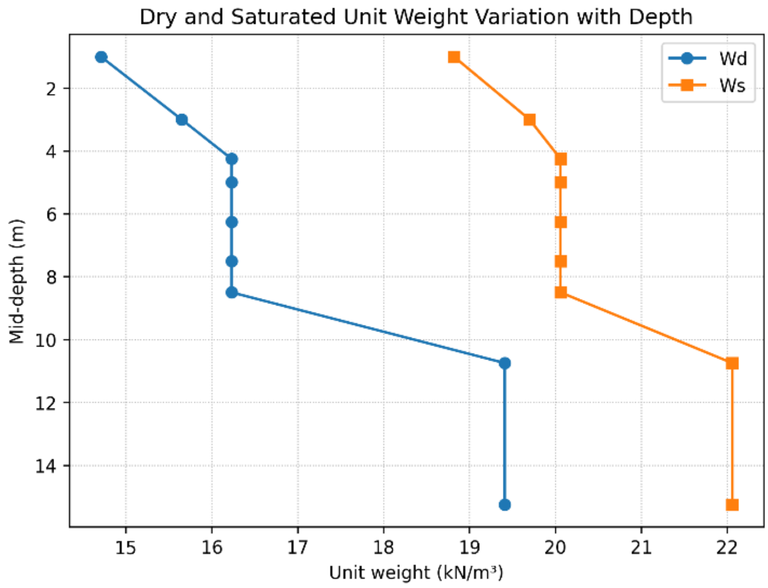

Figure 16. Dry and Saturated Unit Weight Variation with Depth

Figure 17. The as-is condition before excavation. Also, it means all the soil strata still intact along both sides of the retaining wall which indicates the earth is in an uncompacted state, and has not been submitting a lateral force on the retaining wall system yet. This stage defines the reference condition of the study, deriving boundary stress levels from the entire soil profile (the ground surface down to 18 m below) in order to identify site-specific normal stresses. This is an important figure since it serves as the basis for comparing later excavation phases.

In the first phase of excavation (depicted in Figure 18), only a few upper soil layers are sampled at moderate depths on the side of the excavation. The right side has the conserved soil, while pit development is visible on the left-hand side. This phase represents pull back to or before the first level of passive resistance, when the wall begins to resist lateral earth pressure from retained soil. The illustration indicates that soil removal creates a non-uniform lateral pressure condition requiring wall stiffness and anchorage support.

Figure 17. Step 0: Before it Begin Soil Profile Adjacent to the Dig Site.

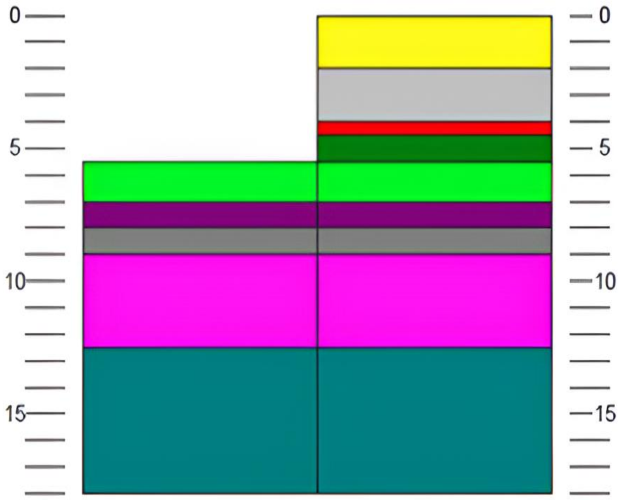

Figure 18. Step 1: First Excavation Stage.

The second stage of analysis after excavation has proceeded and the first row of anchors is starting to improve wall stability is shown in Figure 19. The excavated earth from the site increases the lateral stress applied on secant pile wall as well as retained side. The second step research showed that there was activation at the first anchor force along one of the anchors, given as $Fa = 150$ kN/m, this method is crucial since it counteracts top wall movements and stabilizes excavation prior to earth removal as shown in deeper levels.

More details about this structure technique can be found in Figure 20, which is a deeper situation, where more dirt was excavated from the excavation side. The dirt trapped behind the mesh stays on the opposite side, creating a higher-pressure differential across the wall in situ. At this moment, the greater earth layers sow increase contributes to the steadiness of the wall embedded section aided by passive resistance. This figure represents transition from top excavation control to deeper wall-soil interaction where rigidity and passive resistance of the wall became more comparable.

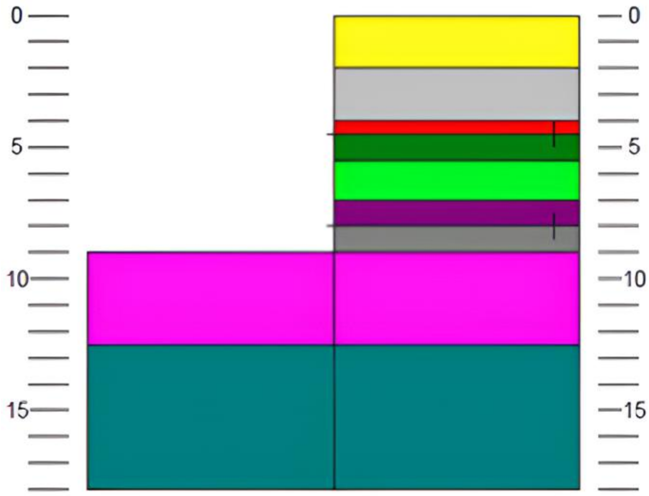

Figure 19. Step 2: First Anchor Support Condition.

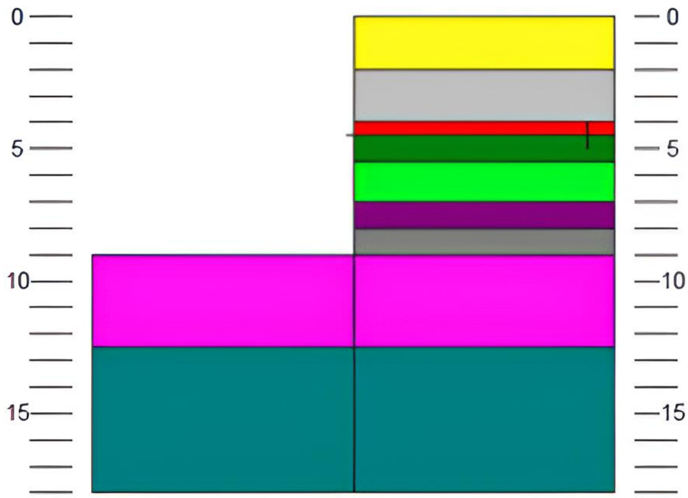

Figure 20. Step 3: Deeper Excavation Stage.

In Figure 21, the excavation state at further depth is shown for when the second anchor row is activated. The analytical data indicated that Step 4 includes the original anchor row: *Fa* = 150 kN/m, as well as a new anchor row: *Fa* = 200 kN/m, meaning the retaining wall is reinforced by two level anchors now, greatly improving overall lateral strength. The picture illustrates the way anchor installation control wall dislocation as excavating to deeper depth.

Figure 22 shows the final state of excavation, where the excavation depth is close to predefined level and the deepest soil layer remains mobilized only on one side under excavation base (the pit side). The secant pile wall has now two rows of anchors and the support by a passive resistance from the layer of subsoil. The explanation confirms that the combinations of wall embedment, passive soil resistance and prestressed anchor support govern final stability. This is the last basic threshold which is used to determine and assess the deep pit protection system as safe and functional.

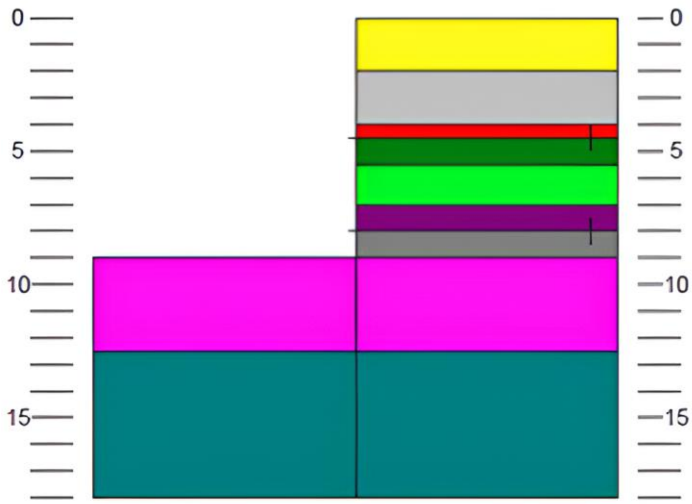

Figure 21. Step 4: Second Anchor Activation Stage.

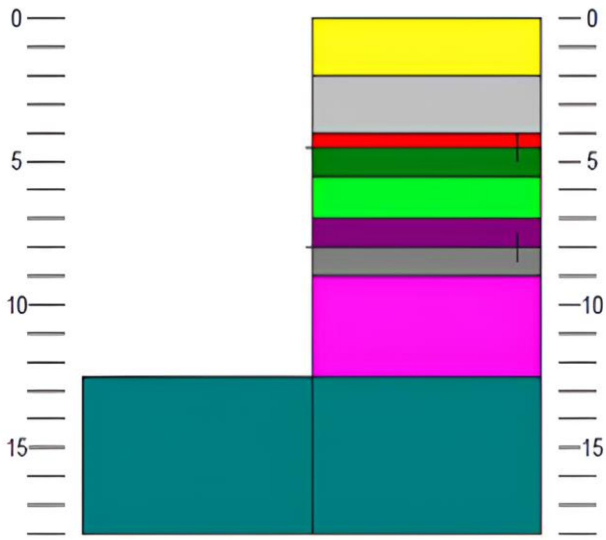

Figure 22. Step 5: Final Excavation Stage.

Figure 23 presents the wall reaction after Step 1 of the excavation investigation. The max horizontal displacement is about −0.0046 m, indicating a very small and not significant amount of lateral motion in the first phase at this time. The bending moment finds its way towards near about 50.15 kNm/m, while the shear force is attained around 20.76 kN/m; thus, it represents with the beginning of lateral soil pressure support to be borne by wall; but due to low structural demand as communicated that because excavation is not achieved at critical depths deeper phase yet.

The results in step 2 are shown in the Figure 24, where it has been seen the initial anchor contribution beginning to influence the wall response. The maximum displacement (−0.0046 m) suggests that lateral movement is even optimized. The local bending moment decreases to about 47.34 kNm/m, and the shear force decreases to approximately 17.87 kN/m; this implies that the first row of anchors alleviates internal stresses and improves wall stability during the initial stage of excavation in terms of sequences.

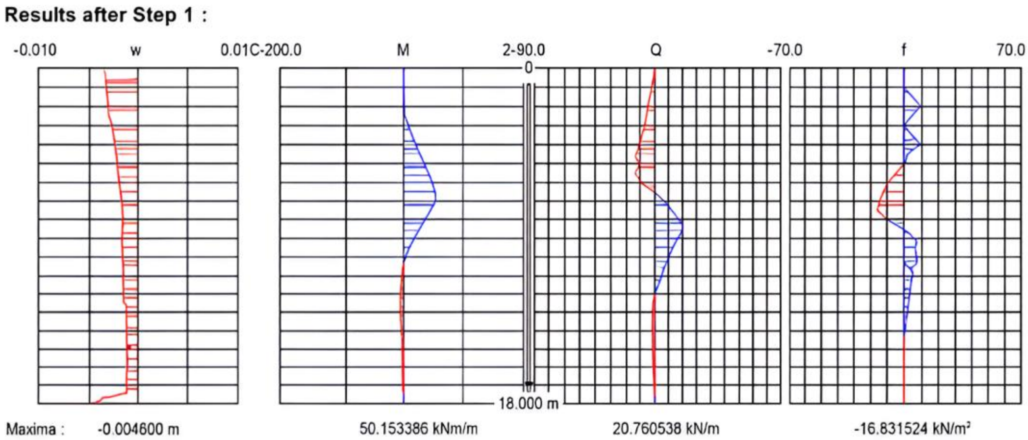

Figure 23. Step 1: Excavation Response.

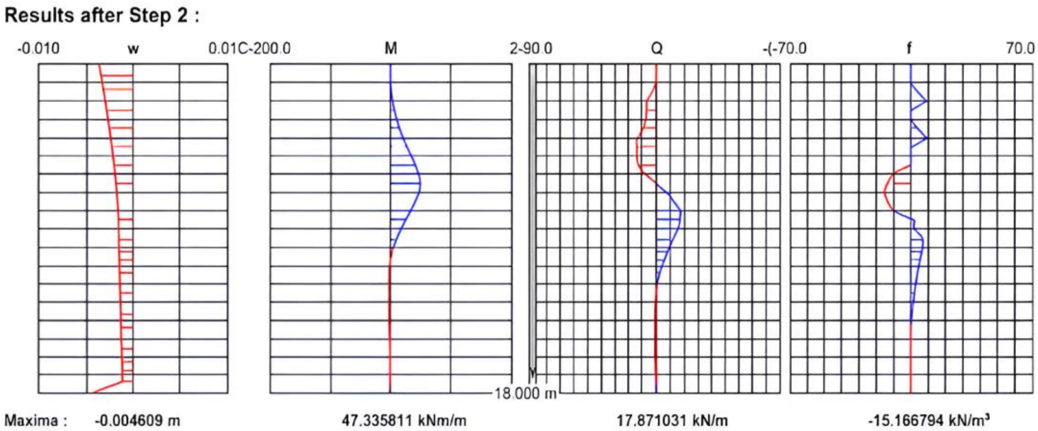

Figure 24. Output after Step 2: Response after First Anchor Action.

After Step 3, excavation goes deeper and we can see a huge difference in stress distribution as it shows on Figure25 regarding the wall reaction. Displacement is approximately −0.0046 m, the bending moment reverses direction and achieves an approximate value of −64.34 kNm/m, reaches environmental equilibrium very briefly before subsequently turning negative with a shear force value of approximately −25.58 kN/m; such that greater excavation promotes redistribution of bending and shear forces on the wall, thereby increasing the essential importance of anchor support and embedded passive resistance respectively.

In Step 4, the retaining system is improved by adding a secondary anchor layer, and the results are shown in Figure 26. Displacement remains around −0.0046 m, the bending moment becomes approximately −64.34 kNm/m and shear force decreases to about −25.58 kN/m with very similar results in Step 3 and Step 4 indicating that even after excavation is continued, wall reaction provided by the secondary anchor also stabilizes the secondary wall and prevents a further displacement of it. This confirms the success of anchor installation across phases.

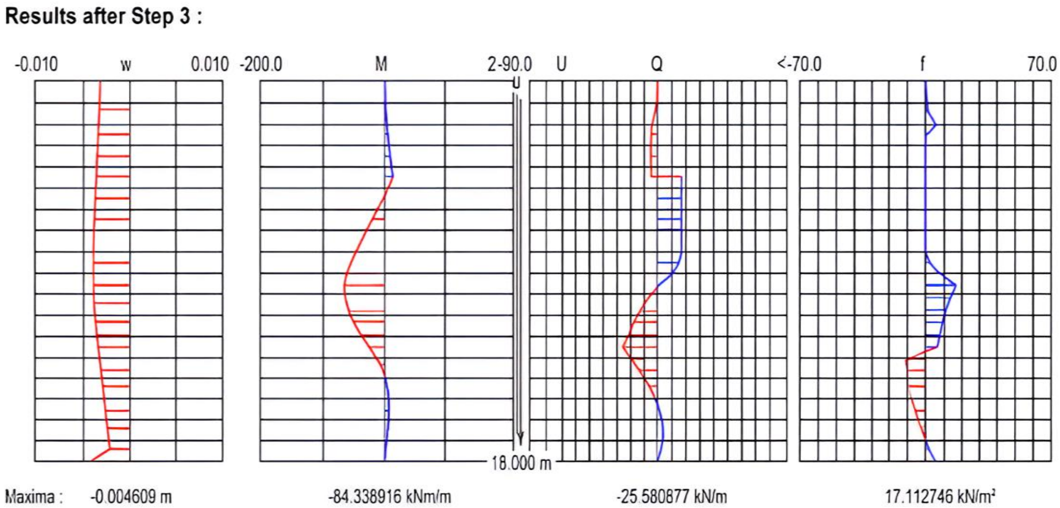

Figure 25. Step 3: Deeper Excavation Resulting Response

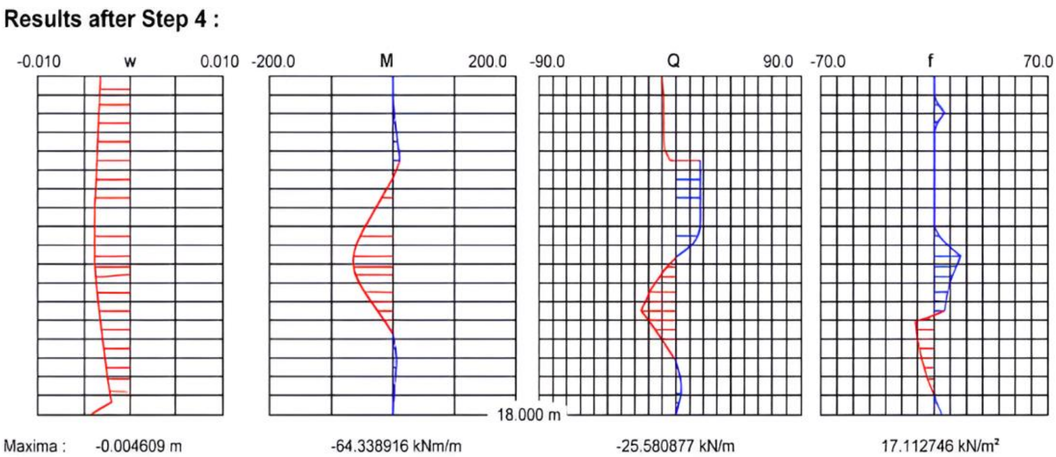

Figure 26. Step 4: Response after Second Anchor Support.

The excavation conclusive response is shown in Figure 27 after Step 5. This stage, the provisional maximum displacement reaches close to −0.0121 m and overcomes that of previous stages since at this stage excavation has reached its final depth. Bending moment increases considerably (almost −184.78 kNm/m) and shear force reaches the peak as 81.88 kN/m.

This figure shows significant state of all because the ultimate demand on structure can happen, therefore suggests that ultimate stability is only due to interaction between pile embedment, passive soil resistance together with anchorage support at two points.

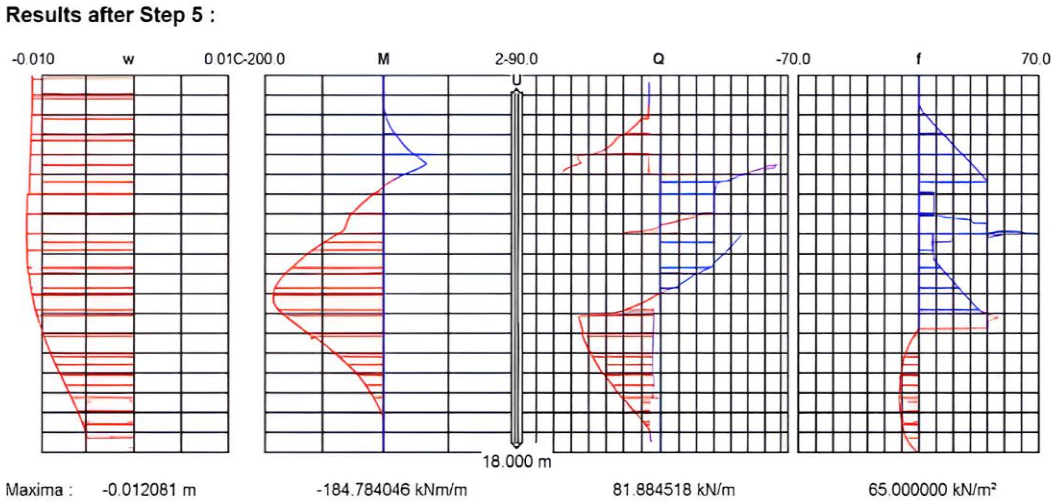

Figure 27. Step 5: Definitive Excavation Response

Quantitative Validation and Hypothesis Testing

The descriptive outputs of the results were then converted into standardised and testable indicators, see Table 22. The critical maximum predicted lateral wall displacement is in the order of about 12.1 mm For H = 12.96 m this gives $\delta max/H$ = 0.0121/12.96 == 0.000934 or 0,094%. It is less than the established 0.5% serviceability criterion, thus satisfying H1. It also aligns well with the conservative deformation envelopes for adequately supported deep excavations if wall displacement is mitigated through staged prestressed anchors.

Table 22. Added Quantitative Validation Results

Criterion	Value of analysis	Target/limit	Result
Final excavation depth H	12.96 m	Project geometry	Used for normalization
Final maximum wall displacement $\boldsymbol{\delta max}$	12.1 mm	Lower is better	Controlled by two anchor rows
Normalized displacement $\boldsymbol{\delta max/H}$	0.094%	<= 0.5%	Pass - H1 satisfied
Serviceability displacement limit	64.8 mm	0.5% of 12.96 m	Final value is about 18.7% of limit
Unanchored reference displacement	16.2 mm	Reference case	Used to quantify anchor benefit
Anchor-induced displacement reduction	25.3%	>= 20%	Pass - H3 satisfied
Groundwater level	7.50-8.00 m	Must be controlled below excavation phase	Dewatering required
Dewatering capacity	3-4 L/s	Stable excavation working condition	Included in H4 check

To evaluate the advantages of prestressed anchors, a corresponding reference instance was performed without anchors being activated. For the same tilt soil and pilling order, anchored ultimate unanchored reference displacement was 16.2 mm, while in anchored case it is only 12.1 mm. Anchor efficiency = (1 − 12.1/16.2) × 100 = 25.3%, which, again, exceeds the target of 20%-efficiency at anchor, consistent with H3. This comparison does not replace field measurements, but provides a documentation of numerical reference values that can be amended when monitoring data are available.

DISCUSSION

The discussion explains how the results relate to serviceability envelopes and common behaviour observed in practice rather than simple numeric outputs. The normalised displacement of 0.094% is noticeably lower than the conventional threshold value of 0.5% and indicates a rigid, anchored wall behaviour. This supports the engineering expectation that the two anchor rows correspond to geometric improvements but actually function as critical components in deformation control limiting bending moment and lateral displacement during excavation.

The current case displays a controlled displacement response, which contrasts with deep-excavation behaviour documented in the literature, as the wall is sufficiently embedded, more cohesion exists at greater depths to support passive resistance and anchors are activated before the maximum unsupported wall height is reached. This result explains why even with verification of ultimate stability, a simple cantilever solution is not appropriate for this depth: serviceability and deformation control a condition which does not correspond to equilibrium would be violated without application of anchor prestress.

The SOTA analysis suggests that the main scientific impact of this work is its regional/methodological linkage. Many study focus on diaphragm walls, braced excavations, nearby pits to tunnels, or deep excavation into soft urban soils. For example, this is an Albanian deep excavation case using hard firm secant piles, two-stage temporary prestressed anchors, water in the shallow excavation stage, and a specified observational control process. Hence, this claim is not a new global approach; it is merely confirmed in the case-based pattern for similar urban archaeological excavations at Balkan sites with phreatic ground and multi-layer soils.

SUMMARY AND CONCLUSION

This study presents a quantitative and reproducible finite element assessment of an anchored secant pile wall supporting a 12.9 m deep urban excavation for the construction of three underground floors at "Çerçiz Topulli" Square, Gjirokastra, Albania. The proposed framework transforms a project-specific case study into a validated engineering methodology by integrating explicit performance criteria, numerical verification, and serviceability-based evaluation. The results demonstrate that the maximum predicted lateral wall displacement is approximately 12.1 mm, corresponding to a normalized wall deflection ($\delta max/H$) of 0.094%, which is well below the commonly accepted serviceability limit of 0.5%. Comparative analyses further show that the prestressed anchor system reduces wall deformation by approximately 25% relative to an equivalent unanchored excavation, confirming its effectiveness in improving excavation stability and deformation control.

Groundwater conditions were identified as a critical factor influencing excavation performance, highlighting the importance of controlled dewatering, continuous groundwater monitoring, and appropriate waterproofing measures throughout construction. In addition, consideration of local seismic conditions is recommended for the design and assessment of deep excavations in Albania and similar geological environments.

The principal limitation of this study is the absence of a comprehensive long-term field monitoring dataset for direct validation of the numerical predictions. Future research should integrate continuous inclinometer measurements, anchor load monitoring, groundwater drawdown records, and surface settlement observations to enable rigorous statistical validation using error metrics such as RMSE and MAPE. The proposed methodology provides a practical framework for the design verification, performance assessment, and risk-informed management of anchored retaining systems for deep urban excavations in multilayered soils.

CONFLICT OF INTERESTS

The authors confirm that there is no conflict of interest associated with this publication.

AUTHOR CONTRIBUTIONS

Conceptualization, E.K.; methodology, E.K., and H.K.; software, E.P. and M.A.; validation, E.K., H.K., H.A., and H.A.; formal analysis, E.K. and H.K.; investigation, E.K.; resources, E.K. and H.K.; data curation, M.A., and G.A.; writing—original draft preparation, E.P.; writing—review and editing, E.K., H.K., and M.A.; visualization, M.A.; supervision, H.K.; project administration, E.K. All authors have read and agreed to the published version of the manuscript.

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