Compare the Design Analysis and observed Performance of Pile foundation

Bhupendra Kumar Sah[1], Vanshika Muchhara[2]
[1]PG Student, Department of Geotechnical Engineering, Parul University PIT
[2]Assistant Professor, Department of Civil Engineering, Parul University PIET
Mail- bhupendra.geotech@gmail.com[1]

Abstract
When dealing with unstable slopes, you can change the shape of the slope, add reinforcements, or just stay away from it. The slope may be fortified if evasion and/or geometric modifications are not workable alternatives. Drilled shafts, soil nails, tieback anchors, and micropiles are just a few of the various technologies that may be used to reinforce slopes. Among these methods, using heaps has been shown to be both efficient and cost-effective. Both limit equilibrium (LE) and geomechanical numerical models are being used in the study of pile-reinforced slopes (the finite element method, FEM, and the finite difference method, FDM). Geomechanical numerical modelling has become more popular in recent years, although its benefits, limits, and accuracy remain open to debate among designers. Design Comparisons of Slope Stabilisation Techniques is a report on the findings of a comparative analysis conducted by the Deep Foundation Institute's Deep Foundations for Landslides/Slope Stabilisation Committee. Three instances were analysed using various analytical methodologies, both coupled and uncoupled, to compare the existing methods of advanced numerical modelling for pile-reinforced slopes (LE, FEM, and FDM). Based on the findings, suggestions for the best approach to stability analysis are provided. The results of the analyses are provided, together with lessons learned and recommendations for optimal pile position and pile length.

Keywords: Slope stability, limit equilibrium, pile reinforced slope, numerical modeling in geotechnicalengineering, constitutive models.

Introduction

The use of retaining structures to address settlement and bearing capacity issues is on the rise as more and more large-scale excavations are carried out in metropolitan areas. In geotechnical engineering, retaining walls with tieback anchors are often used, particularly around deep excavations [1]. Because of the necessity to protect neighbouring buildings and structures, the service performance of tiebacks and walls is crucial when preserving extensive excavations in urban settings. During the course of the previous seven decades, this method has evolved for use in design, building, and testing [2]. While there are many sets of rules and suggestions for design and construction published in the literature, they are often dismissed as merely suggestive. There needs to be more systematic analysis of design processes and outcomes. To guarantee a safe, successful, and cost-efficient design, one must also have the necessary knowledge and expertise based on observed performances under local ground circumstances. With the development of geotechnical engineering design standards in several nations, our knowledge of designing tieback-anchored walls has increased. Throughout the last three decades, several scientists and engineers have worked to refine design methodologies in order to provide practical advice (see, for example, Briaud and Lim [3] and Lambe and Hansen [4]). To better the design of tieback walls and give optimisation techniques for design, Weatherby [5] investigated the behaviour of tieback H-beam walls. Suggestions were made about apparent earth pressures, limiting equilibrium approaches for design, and the management of wall and ground movements. Khoshnevisan et al. [6] suggested an optimization-based method for safe, cost-effective, and resilient geotechnical design (RGD). It was shown that the modified RGD methodology is more cost-effective than the conventional design approaches for complicated geotechnical conditions and buildings. The aforementioned research shows that there is still a problem with the design of tieback-anchored walls.

Since Peck [7] (see, for instance, O'Rourke [8]), researchers have put in a lot of time and effort to analyse field data to reveal the results of excavations and retaining structures. Finno and others (9); Ou and co. (10); Finno et al. (11); Wei and Tan (in). Due to the increasing use of tieback-wall-supported excavations, the existing literature frequently includes analyses of their mechanical behaviour. Seo et al. provide a sand-anchored tieback-anchored wall as an illustration [ot of time and effort to analyse field data to reveal the results of excavations and retaining structures. Finno and others (9); Ou and co. (10); Finno et al. (11); Wei and Tan (12). Due to the increasing use of tieback-wall-supported excavations, the existing literature frequently includes analyses of their mechanical behaviour. Seo et al. provide a sand-anchored tieback-anchored wall as an illustration [13] and carried out twelve distinct model experiments. Based on the model testing, prediction methods were provided for analysing the behaviours of an anchored wall in sand. Broad field execution information and three-layered mathematical models were utilised in the examinations by Finno and Roboski [14–16] of a profound tieback removal in Chicago Earth. The maximum horizontal ground motions for retaining structures were calculated as a function of the safety factor against basal heave and the depth of the excavation. Although there is a wealth of information available on the service behaviour of retaining walls, there is a dearth of information about the performance of ground anchors.

This research looks at a deep excavation in Shenyang, China, and how its tieback-anchored pile wall was built, tested, and is being monitored. The design process, design methods, and design philosophy for tieback walls are all dissected in great depth. Comparisons are established between the design outcomes and the FEM calculations and excavation case data. An example of geotechnical structure design is used to compare and contrast the Chinese and European regulations for such design. An excellent reference for structural design is the observed performance of a tieback-anchored pile wall in sand after evaluating test data and in situ measurements. The results of this research may be used in the planning, design, and construction of anchored pile walls for use in deep excavations.

Literature Review

There has been research on the design approach of tieback-anchored walls and cantilever pile walls for deep excavation for around 60 years. Two limit states, the ultimate limit state and the serviceability limit state, were established and evolved from two issues, the stability problem and the elasticity problem, that Terzaghi [17] (1943) introduced in the field of classical geotechnical engineering. Historically, geotechnical design in Europe, North America, and elsewhere has relied heavily on global or total safety considerations. Table 1 displays the typical global safety factors. Bearing capacity, service condition, and temporary works all had various safety factor values applied to them.

In the 1950s, as the theory of soil mechanics began to take shape, it was thought that overall safety factors couldn't meet the needs of limit-state-based geotechnical design. Researchers like Taylor [18] and Brinch Hansen [19, 20] were the first to think of separate safety factors or partial variables.Because of this, a lot of researchers looked at the best partial factor values and how geotechnical design could be used.Simpson [21] explained the rationale behind and the minimum necessary condition for the partial factors.

The flaws in the European code EN 1997-1 were looked at with the help of four designs.Because he didn't know how much design resistance structural elements or the ground zone would offer, he decided that a resistance model was needed to figure out the partial variables.He suggested changing the actual boundaries instead of the security factors to show a few things, like the soil and water levels.Orr [22] talked about how to choose characteristic strength values and partial factors for Eurocode 7 geotechnical designs.A sound method was developed for extrapolating the characteristic values from the data. Case studies were also used to show how important the partial component is in geotechnical design. Becker [23] defined and looked into the various reliability-based design techniques, as well as the advantages and disadvantages of each. The European and North American design processes' differences and similarities were discussed.

The European geotechnical design code and the probability limit-based partial coefficient approach [24] were the main things that Li et al. talked about.comparing geotechnical design codes from China and Europe. The differences and similarities between the Chinese and European writing systems are discussed in this article. However, additional research is required into the arguments between the Chinese code and the European code.

Several methods, such as physical models, finite element analyses, and field investigations, have been used to study the pullout capacity, load changes during excavation, prestress loss, and antifatigue performance of tieback anchors used to strengthen the ground. Kim [25] tried seven ground secures, including four pressure-type anchors and three strain-type secures, for pullout strength in worn soil. Anchor prestress losses were measured over the course of seven days of relaxation. In addition, a correlation between the amount of prestress loss expected and the amount of creep deformation was presented. The initial prestress loss was not taken into account in the anticipated prestress loss, which instead focused on the influence of creep movement. To study the effectiveness and predictability of tieback walls supporting deep excavations, Konstantakos [26] chose 39 examples. Tieback creep and prestress loss were identified as significant influencing co

mponents of pile wall deformation after comparing different observations from excavation case histories. Three groups of researchers (Tamano et al. [27], Ugur Terzi et al. [28], and Hsu [29]) have undertaken several field experiments in a variety of geotechnical settings. Many ground anchors were studied for their anchoring capacity and service performances.

Basics of Slope Stability

Slope collapses have cost billions and killed millions worldwide. Many "multiple-hazard disasters" include landslides or mass waste. So, to better foresee and prevent these kinds of tragedies, public understanding of slope safety assessment (for natural or man-made slopes) must be improved. Slope stability analysis is the best way to anticipate if a slope's soil will break under stress.

Stability occurs when driving forces equal opposing forces. The opposing forces of soil strength maintain the soil or rock in place while gravity drags it down the slope. Geotechnical engineers struggle with stability analysis because soil characteristics impact both forces. Studying slope failure mechanisms has improved soil behaviour the most. Choosing the optimum analytical method for each issue still needs development.

Types of Slope Failure

Stable means secure against movement of the earth's mass. Slope failure, also known as mass wasting, is the movement of earth, soil, rock, or debris downslope in a vertical and/or horizontal direction. According to Figure 1, these shifts may be broken down into six categories based on the nature of the failure.

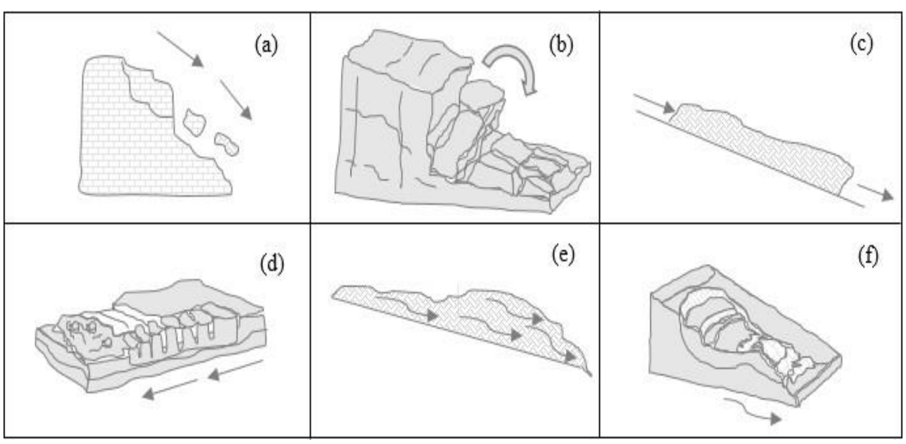

Figure 1. Slope Movements Based on Classification

Table 1. Landslide Triggering Factors

General Process	Source Of The Triggering	Natural Factors	Anthropogenic Factors
Generally Increase The Stress	Toe Removal	Waves, Current, And Rivers That Could Erode The Toe.	Drawdown Of Lakes Or Reservoirs Through Excavations.
	Lateral Material Removal	Solution For The Kart Terrain	Mining
	Addition Of Surcharges	Rain Or Snow; The Flow Of Surface Or Ground Water; The Slide; Vegetation; Volcanic Activity; Earthquake;	Construction Loads
			Fill Placement
			Waste Dumps
			Stockpiling
Generally Decrease Strength	TransitoryStresses	Rain Or Snow; The Flow Of Surface Or Ground Water; The Slide; Vegetation; Volcanic Activity; Earthquake;	Explosions (Construction Related)
		Explosion From Volcanic Activity Storms	Vibrations From Pile Driving OrHeavy Traffic
	Uplift Or Tilting	Tectonic Forces	Cutting
		Volcanic Activities	
		Earthquake	
		Melting Of Ice Sheets	
	Material Characteristics	Soils That Are Weak And Vulnerable Saturation Chemical And Physical Weathering Arrangement And Fabric Clay's Hydration, Which May Result In A Loss Of Cohesion	Disturbance Due To Construction May Affect Sensitive Material
			Dewatering Will Cause Water Table Fluctuations
	Mass Characteristics	Sheared Zones Are Discontinuities Like Faults, Fractures, And Fissures.	Fractures Caused By Construction Processes

Finite Element Method

The Finite Element Method (FEM) is "a numerical approach" that may be used to tackle geotechnical issues, according to the Department of the U.S. Army Corps of Engineers' Engineering and Design Geotechnical Analysis using the Finite Element Method. This method has improved our ability to examine more intricate geotechnical issues such as slope and embankment deformations, stress and movement determination in tunnelling and excavations, and earth pressure structures.

Discretization, or the process of breaking down the problem's geometry into smaller and smaller pieces, is the foundation of FEM. Many writers have talked about how FEM is better than other methods for solving slope stability problems.

Finite Difference Methods

FDMs are based on finite differences. This solution calculates stresses and strains at each time step using forward, backward, or central differences. Mathematical physics problems were solved using the finite difference method (FDM) in the 1930s. FDM does not need soil stress-strain relationship assumptions like LEM. This approach solves more complex problems and yields more accurate results. Carter et al. (2000) say the FDM is easier to use for soil modelling inputs than the FEM. Nonetheless, linear or somewhat nonlinear difficulties are weaker given the strategies utilised. FEA and FDM are numerical methods that need constitutive models to characterise the soil's reaction to applied stresses.

Reinforced Slopes

If the estimated safety factor of an unreinforced slope is less than the minimum safety standards, there are several ways to fix it.These remedial strategies include avoidance, a decrease in shear stress (or driving forces), and an improvement in shear strength (or resisting forces). Depending on slope stability and condition, maintenance, observation, or doing nothing may be employed. Maintenance and surveillance will concentrate on slow-moving landslides and their short-term treatment. But, after years of observation, inactivity may appear rational. But the long-term impacts must be known. Table 2 reviews slope stability approaches.

Table 2. Remediation Methods for Slope Stability Problems

Category	Procedures	Limitations
Do nothing	“No action”	Ineffective if catastrophic failure occurs over a long period of time; applicable to slopes that move slowly or are stable and accelerate.
Avoid the problem	Relocate project	Ought to be contemplated during arranging
		Cost impact on the chosen location
		If the design is finished, it is not cost-effective.

Maintenance	Activities such as removing materials, closing areas, etc.	Only applicable to slow-moving landslides; if it fails, it will need to be done over.
Monitoring	Inclinometers, surface survey points, etc.	restricted to landslides that move slowly
Decrease driving forces	Change the grade	Could have an impact on portions of adjacent roadways. There isn't always enough physical room for a change in geometry.
	Drain surface	will only eliminate seepage or surface infiltration caused by surface infiltration.
	Drain subsurface	Will is dependent on the sliding mass's permeability.
	Reduce weight	requires light, right-of-way materials.
		Produce excavation debris that must be dealt with
Increase resisting forces	Use buttress or counterweight fills;toe berms	Doesn't work well in deep-seated landslides; requires a right-of-way; may need a solid foundation.
	Use structural system	Could not withstand significant deformations. Should penetrate well below the sliding surface
	Install anchors	requires a solid foundation to withstand the tension of the anchor's shear forces.
	Drain subsurface	will be determined by the sliding mass's permeability.
	Use reinforcedbackfill	Requires durability of reinforcement
	Install in situ reinforcement	requires piles, micropiles, and nails that can last for a long time.
	Use biochemical stabilization	restricted to the slope's height; influenced by climate
	Treat chemically	The effectiveness over the long term is still being evaluated. It could affect the environment.
	Use electrosmosis	Constant direct current and maintenance are required. May be expensive.
	Use thermal stabilization	Require an expensive and meticulously designed system

Pressure-based Method

This method analyses passive piles under lateral stresses. This method assumes that the soil surrounding the pile undergoes plastic deformation and plastic flow. The first step meets Mohr-Coulomb yield requirements and ground viscosity. So, plastic deformation is like hard soil, and plastic flow is like soft soil undergoing creep deformation. As Ito and the heaps were considered to be solid, and the earth was expected to be able to plastically distort around them, the approach given here has limited applicability since it doesn't reflect the real soil/heap connection. According to plastic twisting theory, horizontal power rises with c. The theory of plastic flow states that when yield stress and plastic viscosity rise, the lateral force will increase, but the difference will be minimal. Both models estimate lateral forces within one order of magnitude of data. When piles are top-constrained, integrate equation (5) from plastic deformation theory down the soil strata to determine the lateral force.

Table 3. Pile Reinforced Slopes Analisys Methods

Soil type	FailureType	Model characterization
Cohesive Soil	Circular	Pressure based method/ Uncoupled
Clay, Claystoneand Silt stone	Circular	Displacement based/ Uncoupled
Cohesive Soil	Circular	Finite Difference Method/Pressurebased/ Uncoupled
Purely Cohesiveslope	Circular	
Upper soft Lowerstiff	Circular	Boundary Element/ Displacementbased/ Uncoupled
Upper stiff Lowersoft	Circular	
C=10kpaφ=20◦	Any	Coupled analysis and the 3D Shear Reduction Finite Element Method
C=4.7kpa φ=25◦	Log-spiral	Kinematic approach limit analysis/ Uncoupled
Anisotropic and non-homogeneous	Log-spiral	The Finite Element Method for Strength Reduction Based on Displacement and Uncoupled and Coupled
γ=20.0kn/m3C=10kpa φ=20◦	Log-spiral	ABAQUS Finite Element Method/Uncoupled
Granular and Fined-grain soils	Any	FLAC/coupled finite difference program
Cohesive Soil	Any	Strength Reduction Method/Coupled

For Coupled Analysis:

Coupled analyses need constitutive models and soil properties. When the slope and pile are modelled simultaneously, materials and conditions must be accurately described. Like the uncoupled analysis, an unreinforced analysis determines if the slope needs reinforcement. If reinforcement is required, set the pile in the recommended location to calculate the revised safety factor. Comparing the study's maximum bending and shear loads to the pile's nominal values verifies structural integrity. If the pile loses structural integrity, whether linked or unlinked, reinforcement, geometry, or material must be modified. To link slope stability study loads to LRFD or ASD analysis, factor or unfactor them before lateral response analysis. Displacement-based ASD analysis requires an ultimate or allowed pile analysis value.

Discussion of the results Unreinforced and Reinforced Analysis

SLOPE/W safety considerations The unreinforced scenario study varied by 8%. SLOPE/W and Slide 7.0 indicate a 1.26–1.27 unreinforced slope. Raise the slope below 1.5. "Method B" reinforced analysis can meet the safety factor with the shear force. The target safety factor equation uses resistive reinforcing loads. A and B need different shear forces for the same safety factor. "Method A" is 1.5-safe at 60 kN/m shear. B requires 87.5 kN/m (the ultimate). "Method Bs results are conclusive and "Method As satisfactory. underlined that the process is vital to correctly characterising the reinforcing force and that the difference between the two approaches may be considerable. Shear forces perpendicular to reinforcement and parallel to the critical surface do not vary. 99% sure. Reinforcement changes the critical failure point. Continuous sliding surface position distorts soil-pile interaction. Laterally-loaded piles must slide.

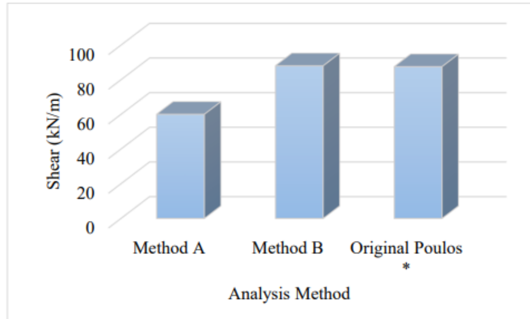

Figure 2. Shear Force to Achieve F.S.=1.5 among Methods

Sensitivity Analyses (Uncoupled)

After the holding force was calculated, further analysis was done using the matched calibrated condition. The goal of this study is to find out how the length of the piles affects the amount of reinforcing load that is needed for a given safety factor. No matter how large the shear, the desired values of the factor of safety were not always reached. In these cases, the optimal value has been found, which is the point at which the safety factor no longer helps. The outcomes of these studies are summarised in Table 4.

Table 4. Pile Lenght Influence on Required Shear Force

Lengthof Pile (L1, m)	Critical surface depth (L2,m)	**L2/L1	Required shear (kN/m)	F.S. achieved	Ratio Length/Shear
6*	6	1.01	>47	1.32	-
7*	7.2	1.01	>46.4	1.44	-
8*	8.09	1.02	>84	1.489	-
9	5.4	0.61	88	1.52	10%
12	5.4	0.44	89	1.49	13%

14	5.4	0.39	91	1.53	16%
18	5.4	0.32	87	1.54	20%

*Not 1.5 could be achieved.
**L2/L1, ratio sliding surface/ pile length

The least steep is 9.0m. Under-0.6 pile lengths failed the safety factor (considered not embedded in the firm stratum). These heaps were tip-critical. Piles did not impact shear. Safety factor shear/length% is length/shear ratio. If F.S. = 1.5, heaps of 8.0–9.0m are optimal. 8-meter piles fail below the reinforcing, showing appropriate safety.

A similar study placed piles of different lengths with the same required shear from the previous analysis at various points from the toe to the top of the slope to determine how pile placement influences safety.

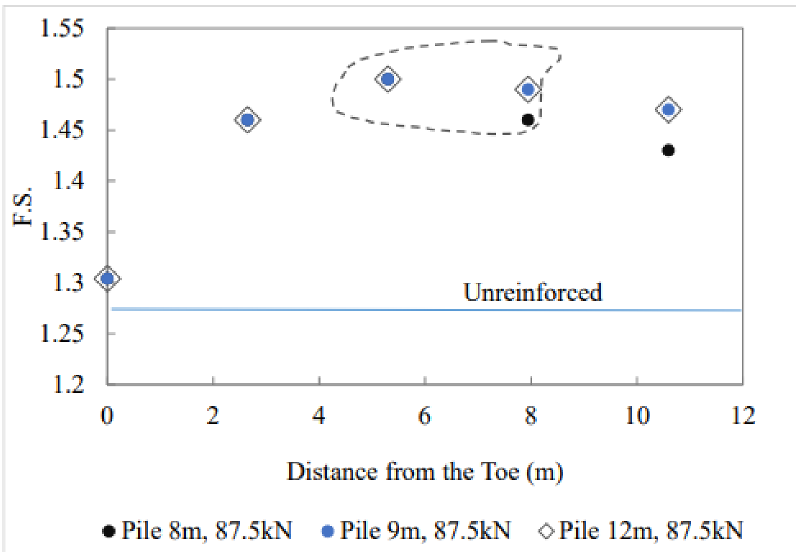

Figure 3. Factor of Safety vs Pile Location

Table 5. Summary of Parameters for Pile Lateral Analysis

Pile diameter (Bored concrete pile)	1.2m
Reinforcementequivalent area	2.5% of Ac
Yield stress ofreinforcement	260Mpa
Young’s modulus of thesoil	Varying from 5Mpa at a rateof 3Mpa/m
Unstable soil limiting lateral pressure	As presented by (Reese et al.2004)
Stable soil limiting lateral pressure	4xHorizontal earthcoefficient

Table 6. Distribution of Lateral Loads for Pile Design

Variable	Value
Fh	61kN (per width of soil, Unfactored)
s	3m
r	1 rows
l	5.3m
Ph	62kN/m per pile

The results of the displacement-based method and the stability analysis with a uniform load on the pile are the same. The depths at which the greatest values for both situations may be found are about the same. To make sure that the pile is structurally sound, the results are compared to the nominal ultimate bending moment and shear for the reinforced concrete section. For flexure, the safety factor should be >1.67, and for shear, it should be >1.5. In this scenario, the maximum shear is not found at the sliding depth. As a result, the maximum shear should be checked against the calculated shear resistance of the pile in every given place in order to ensure the structure is sound.

Table 7. Lateral Analisys Results (Unfactored loads)

Parameter	**Distributed load approach**	**Displacement based (90mm)**
Unfactored nominal capacityof pile	3,412 kN.m	3,389 kN.m
Maximum applied moment	308 kN.m	236 kN.m
Unfactored resisting shear force	1012 kN	1015 kN
Maximum shearforce at critical circle	135 kN	83 kN

Unreinforced Analysis

Based on a two-dimensional elasto-plastic finite element study, Phase2 shear strength reduction calculated the unreinforced factor of safety for this case. FLAC finite difference programme analysis yields more results. Shear Strength Decrease determines safety factor (SSR). The Mohr-Coulomb failure criteria characterise soil behaviour. This criterion is straightforward to apply and accurate in describing soil behaviour. Its proportions and geotechnical recognition make it significant. Phase2 FE SSR findings. SSR technique contours of maximum solid displacement at failure match LE critical failure circle superposition.

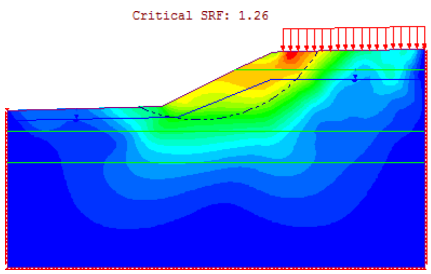

Figure 4. SSR Unreinforced Analisys with Phase2.

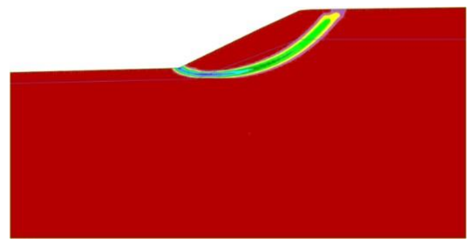

Figure 5. SSR Unreinforced Analisis with FLAC SRF=1.26

Table 8. Summary Unreinforced Analyses

Programs	**SLOPE/W**	**Slide 7.0**	**Phase2**	**FLAC**
Unreinforced F.S.	1.25 9	1.268	1.262	1.263

The summary from the unreinforced analysis clearly indicates that results from LE analyses can be reproduced by SSR FE and FD analyses with high agreement. In this case, layered slope with different soil properties, the difference among results is less than 1%, thus allmethods are relatively accurate and reliable.

Reinforced Analysis

Pile models in Phase 2 included a structural interface that simulated slip between the reinforcement (liner) and the soil via the presence of joints on each side of the reinforcement (liner).

Table 9. FLAC and Phase 2 of the SSR Investigation of Pile Properties

Pile length	9m
Pile spacing	3m
Pile diameter	1.2m
Young's modulus	40,000,000
Poisson's ratio	0.21

FE research's greatest shear force authorised Phase 2 and LE programmes in another LE study. With a certain shear pressure, LE (Slide 7.0) and FE (Phase 2) provide 1.392 and 1.38 wellbeing factors, respectively. The FE study demonstrates that the reinforcement's shear resistance in the studied configuration (9m, position, and diameter) is insufficient to meet the required factor of safety, yet the two assessments are extremely equivalent for the same shear. that a 9-meter pile required 61kN (allowable) and 87.5kN (final) shear forces to reach 1.5. This (coupled) investigation reveals that changing the heap geometry (length and width), heap area inside the slant, or heap support enhances shear opposition and security.

Sensitivity Analyses (Coupled)

Phase 2 showed that pile position influences slope safety. Base-to-top pile heights vary. SSR FE analysis recommends slope-center pile placement. suggests placing smaller heaps at Le/4 (one-fourth of the slope from the toe). The length of the pile centres the slope's sweet spot. For the same slope, the coupled analysis recommended placing the pile closer to the centre, with a slightly greater safety factor than ones towards the quarter of the slope (for shorter piles). Given the frictional properties of the slope material, placing piles in the middle of the slope will always enhance the safety factor, especially with an adequate embedment depth, whereas placing piles near the toe is never desirable.

The Mill Creek Landslide (Mitigation)

State Highway 15 southbound embankment landslip at Tioga Reservoir in north-central Pennsylvania 2011's rains and snowmelt left scars on the Pennsylvania Department of Transportation. Driven steel H-piles were the most cost-effective and least intrusive road slide stabilisation method. Laboratory testing, site stability, and recommended solutions This area has alluvial and colluvial sands, perhaps with glacial lake deposits. The embankment was medium- to heavy-weight silty sand and gravel. 17 borings, 10 inclinometers, and a subsurface geophysical survey measured the soft glacial lake deposit depth. Analyse independently to choose the reinforcing alternative. GSTABL7 with STDwin (Gregory, 2005) assessed slope stability, whereas LPILE v5.0 analysed pile design (Ensoft, 2005). Slope stability experiments with a safety factor of 1.0 based on back-calculated soil characteristics The safety factor was 1.3 for each steel H-pile's shear force after strengthening. This research examines coupled and uncoupled components. Table 10 summarises soil qualities.

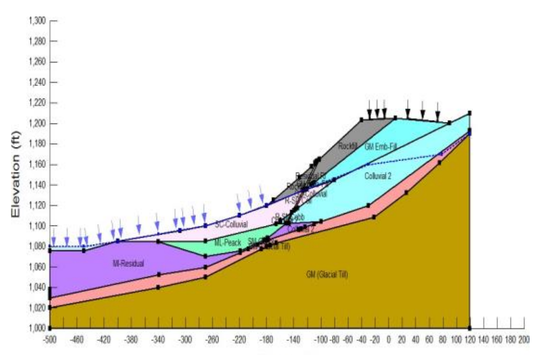

Figure 6. Subsurface Profile along the Critical cross-section at Station 1303+00

From the laboratory testing, the glacial lake deposit was found to have residual and peak strengths that were fixed to determine the additional soil properties from the back analysis.

Table 10. Soil Properties Mill Creek Landslide

Soil type	Total Unit Weight (pcf)	c' (kPa)	Φ' (°)
Embankment Fill	120	0	32
Colluvial	120	0	30
Peak ML	120	0	25
SM cobbl	125	0	36
Residual ML	120	0	16
Glacial	130	0	38
Rockfill	130	0	45
Colluvial 2	120	0	34
Residual Colluvial	120	0	28
Residual SM cobbl	125	0	30
Residual Rockfill	130	0	38

Unreinforced Analysis

In the absence of in-situ knowledge of the scar, the initial unreinforced study will confirm the likely position of the failure surface given the provided soil conditions. the SLOPE/W results, while the Slide 7.0 results. Both sets of data show that the values are quite close to one another. In addition, the predicted in situ scar, which is a tiny area of residual soils, is not followed by the critical sliding surface.

Although F.S. less than 1.0 were discovered to be crucial from the general analysis, further information is given the position of the 1.0 unconstrained failure. Both analyses followed the pattern of the constrained analysis for the position of the failure surface with F.S. around one.

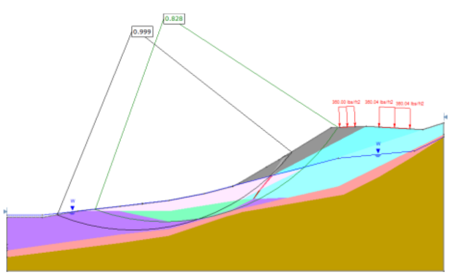

Figure 7. F.S.=1.0 of Unconstrained Analysis Slide 7.0

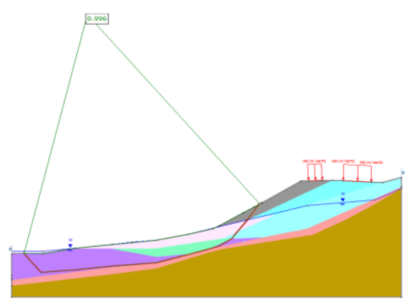

Figure 8. Factor of Safety of Constrained Analysis Slide 7.0

On the other hand, the results from the constrained analysis are presented in Figures 58 and 59. The target value F.S. equal to 1.0 is achieved by following the scar developed at the time of failure.

Reinforced Analysis

After the results of the unreinforced analysis are in, a reinforcing analysis is done using a failure surface that has been calibrated to a factor of safety. A 1.30 safety factor is the goal. Shear forces from the latest studies are compared to the original values, and the section chosen as the best feasible choice (H-Piles 12x53) is examined. Method B will be utilised to compute the reinforcing load since the original algorithm (GSTABL7 by Gregory (2005)) incorporates the stabilising force into the numerator when calculating the safety factor. With SLOPE/W and Slide 7.0, the goal value of 65 kip was met. At the shear rate of 75 kip, the safety factor was also calculated. Since the two values are so close, the shear value stated earlier will be used in the pile's design (for comparison purposes). The results of the strengthened analysis are shown in Table 11.

Table 11. Summary of Shear Forces and FOS

Condition	Shear force (Kips)	FOS
Constrained (SLOPE/W)	75	1.310
Constrained (Slide 7)	75	1.305
Original report (GSTABL7)	75	1.300

Designing Resisting Pile

The resisting pile will be analysed using two methods: (1) the traditional method, in which the shear force from the stability analysis is applied to the pile as a distributed load, and (2) a displacement-based method, in which the soil displacement recorded by the installed inclinometers is utilised. In the first technique the distributed lateral load at the failure plane (Ph) is calculated using the appropriate shear force. This method is based on one provided by the Ohio Department of Transportation (ODOTO, 2011). The stability analysis loads were used as factored loads for the strength limit state in the pile design using load and resistance factor design (LRFD).

Table 12. Distribution of Lateral Loads for Pile Design

Variable	Value (Original report)
Fh	75kips
s	6ft

r	4rows
l	35ft
Ph	536lbs/in

To account for the fact that the piles were installed in a dynamic landslide, the lateral resistance calculated using p-y models was multiplied by a factor of p. P was multiplied by 0.231. The outcomes of the previous study's and this one's LPILE pile analyses for strength limit state are shown in Table 12. Based on these findings, we may be confident in the stability of the piles. It was determined that the resistive moment of the pile was 2,106 kip in when the maximum applied moment was 1,788.4 kip in. Yet, the greatest shear load delivered to the pile was 44.0 kip, and its calculated resisting shear force was 98 kip. the shear and moment diagrams derived from the lateral stability study.

Table 13. Results of Lateral Analyses For Strength Limit State

Parameter	**Previous study (LPILE v5.0)**	**Present study (LPILE v09.010)**
Resisting moment of pile*	1,983kip· in	2106kip· in
Maximum applied moment	1,740.0kip.in	1,740.4kip.in
Resisting shear force*	98kips	98kips
Maximum shear force	41.0kips	41.8kips

*Determination of nominal values is presented in Appendix II.

Reinforced Analysis

This model is indicative of an ongoing failure, making Phase2 analysis inappropriate. that when reinforcement has been added, the critical SSR value is almost identical to that of the unreinforced analysis. Future studies will utilise FLAC's couple analysis results as a benchmark against which to evaluate the uncoupled segment.

Summary Results Coupled and Uncoupled Analyses

The compiled data for this issue is shown in Table 14. Based on the findings, uncoupled analysis may be preferable in situations where piles are situated in a dynamic failure zone. This is because it is no longer necessary to speculate on the precise position of the sliding surface. When inclinometer data is available, confirming the findings of a pile structural analysis using the distributed load approach may be done reliably by using the displacement-based method.

Table 14. General Summary

Condition	UnreinforcedF.S.	Reinforced F.S.	Shear Force (kN/m)	Max. Shear from LPILE (kN/m)	Max. Moment (kN.m)
Zicko et al. (2011)	1.0	1.30	75	48	1,740.0
Slide 7	1.0	1.305	75	48.1(1) 50.1(2)	1,740.4(1) 1,084.3(2)
SLOPE/W	1.0	1.31			
Phase2	0.7	-	-	-	-

Conclusions

1. This study assessed pile-reinforced slope stability techniques. Linked (one study) and uncoupled (two trials) approaches were compared. Sensitivity testing helped pile-fortify slopes. It appears:
2. Accurate and reliable unreinforced slope investigations should be linked and uncoupled.
3. Reinforced slope displacement-based analysis is less conservative than distributed load uncoupled analysis. Ground conditions are important for displacement-based methods. Safety factors were comparable. Research shows that pairing works best. Sliding depth, soil empirical representation (p-y curves for lateral analysis), modifiers, and geotechnical and structural safety problems should not be anticipated. Iterations without pile shear assumptions determine optimum length and pile location using combined analysis.
4. Displacement and slope instrumentation aid uncoupled pile mitigation (active landslide).
5. Uncoupled analysis uses fewer parameters. Nevertheless, when the time and energy necessary to run two separate analyses and categorise the soil to build p-y curves for the lateral study are considered, the two procedures are equivalent, and basic FE or FD models may be developed quickly.
6. Paired analysis requires detailed scenario descriptions. Accurate soil, constitutive model, and characteristics
7. Repeat shear resistance or soil

displacement with limit equilibrium or laterally loaded pile analysis.

8. This study's sensitivity analysis focuses on pile location rather than pile spacing, head condition, length, or rows. Most pile-reinforced investigations found comparable results.
9. Slope material affects surface positioning. If the sliding depth is moderate, piles must be installed from the quarter to the higher slope depending on the materials' frictional properties (pure friction; middle, with cohesion and friction; generally, pure cohesion from the bottom to the top; generally, the toe, but not very important to the improvement in safety with location). Soft clay, solid clay, and sand influenced sliding depth equally.
10. Sensitivity analysis is best since slope design affects pile length. Uncoupled analysis determines the safest depth. The sliding depth-to-pile length ratio was 0.3 for a 1.4 safety factor and 0.45 for a 1.5 factor. An analytical sensitivity study showed that shear resistance abruptly rises at the optimal pile length.

References

[1] Adekunte, D. Madsen, N. Margetson, and L. Quinn, "Pre-dicted and observed performance of an anchored retaining wall in granite," in Ground Anchorages and Anchored Structures in Service 2007: Proceedings of the Two Day International Conference Organised by the Institution of Civil Engineers, pp. 427–437, Thomas Telford Publishing, London, UK, November 2007.

[2] Veder, Excavation of Trenches in the Presence of Bentonite Suspension for the Construction of Impermeable and Load- Bearing Diaphragms. Grouts and Drilling Muds in Engineering Practice, Butterworths, London, UK, 1963.

[3] J.-L. Briaud and Y. Lim, "Tieback walls in sand: Numerical simulation and design implications," Journal of Geotechnical and Geoenvironmental Engineering, vol. 125, no. 2, pp. 101–110, 1999.

[4] P. C. Lambe and L. A. Hansen, Design and Performance of Earth Retaining Structures. American Society of Civil Engineers Geotechnical Publication No. 25, 1990.

[5] E. Weatherby, "Behavior of tiedback, h-beam walls and recommendations for their design," in Proceedings of the 2010 Earth Retention Conference - Earth Retention Conference 3, pp. 170–187, USA, August 2010.

[6] S. Khoshnevisan, L. Wang, and C. H. Juang, "Response surface- based robust geotechnical design of supported excavation– spreadsheet-based solution," Georisk: Assessment and Manage- ment of Risk for Engineered Systems and

Geohazards, vol. 11, no. 1, pp. 90–102, 2017.

[7] R. B. Peck, “Deep excavation & tunneling in soft ground. Stateof-the-art-report,” in Proceedings of the 7th International Conference of Soil Mechanics and Foundation Engineering, International Society for Soil Mechanics and Geotechnical Engineering, pp. 225–281, Mexico, 1969.

[8] T. D. O’Rourke, “Ground movements caused by braced excava- tions,” Journal of Geotechnical and Geoenvironmental Engineer- ing, vol. 107, no. 9, pp. 1159–1178, 1981.

[9] R. J. Finno, D. K. Atmatzidis, and S. B. Perkins, “Observed per- formance of a deep excavation in clay,” Journal of Geotechnical Engineering, vol. 115, no. 8, pp. 1045–1064, 1989.

[10] C.-Y. Ou, J.-T. Liao, and H.-D. Lin, “Performance of diaphragm wall constructed using top-down method,” Journal of Geotech- nical and Geoenvironmental Engineering, vol. 124, no. 9, pp. 798–808, 1998.

[11] R. J. Finno, J. T. Blackburn, and J. F. Roboski, “Three- dimensional effects for supported excavations in clay,” Journal of Geotechnical and Geoenvironmental Engineering, vol. 133, no.1, pp. 30–36, 2007.

[12] Y. Tan and B. Wei, “Observed behaviors of a long and deep excavation constructed by cut-and-cover technique in Shanghai soft clay,” Journal of Geotechnical and Geoenvironmental Engi- neering, vol. 138, no. 1, pp. 69–88, 2011.

[13] H. Seo, B. S. Chang, S. S. Jeong, and S. I. Kim, “Evaluation of the sequential behavior of tieback wall in sand by small scale model tests,” Journal of the Korean Geotechnical Society, vol. 15, no. 3, pp. 113–129, 1999.

[14] R. J. Finno and J. F. Roboski, “Three-dimensional responses of a tied-back excavation through clay,” Journal of Geotechnical and Geoenvironmental Engineering, vol. 131, no. 3, pp. 273–282, 2005.

[15] J. Roboski and R. J. Finno, “Distributions of ground movements parallel to deep excavations in clay,” Canadian Geotechnical Journal, vol. 43, no. 1, pp. 43–58, 2006.

[16] J. F. Roboski, Three-Dimensional Performance And Analyses of Deep Excavations [M.S. thesis], Northwestern University, 2004.

[17] K. Terzaghi, Theoretical Soil Mechanics, John Wiley & Sons, New York, NY, USA, 1943.

[18] W. Taylor, Fundamentals of Soil Mechanics, John Wiley & Sons, New York, NY, USA, 1948.

[19] J. Brinch Hansen, “Earth Pressure Calculations,” Tech. Rep., The Danish Technical Press, Copenhagen, Denmark, 1953.

[20] J. Brinch Hansen, “Limit design and safety factors in soil mechanics,” Limit Design and Safety Factors in Soil Mechanics. Danish Geotechnical Institute, Copenhagen, Bulletin No. 1, 1956.

[21] B. Simpson, “Partial factors: where to apply them,” in Proceed- ings of the LSD 2000: International Workshop on Limit State Design in Geotechnical Engineering, pp. 125–136, 2000.

[22] T. L. L. Orr, “Selection of characteristic values and partial factors in geotechnical designs to Eurocode 7,” Computers & Geosciences, vol. 26, no. 3-4, pp. 263–279, 2000.

[23] D. E. Becker, “Eighteenth Canadian geotechnical colloquium: Limit states design for foundations. Part I. An overview of the foundation design process,” Canadian Geotechnical Journal, vol. 33, no. 6, pp. 956–983, 1996.

[24] Y.-S. Li, X. Duan, Y. Li, and X.-P. Li, “Comparative research on design approaches on retaining structure for deep foundation pits in Chinese and European geotechnical design codes,” Chinese Journal of Geotechnical Engineering, vol. 36, pp. 77–81, 2014 (Chinese).

[25] N.-K. Kim, “Performance of tension and compression anchors in weathered soil,” Journal of Geotechnical and Geoenvironmen- tal Engineering, vol. 129, no. 12, pp. 1138–1150, 2003.

[26] D. C. Konstantakos, “Online database of deep excavation performance and prediction,” in 6th International Conference on Case Histories in Geotechnical Engineering, 2008.

[27] T. Tamano, H. Q. Nguyen, M. Kanaoka, Y. Fuseya, and W. Tonosaki, “Field Tests and Numerical Investigation of Splits Anchors in Structured Clay,” 2006.

[28] N. Uğur Terzi, G. I. Daldal, and S. Yildirim, “Monitoring a grouted anchor in a reinforced structure,” Experimental Techniques, vol. 35, no. 2, pp. 47–54, 2011.

[29] S.-T. Hsu, “Behavior of pressure-grouted anchors in gravel,” Canadian Geotechnical Journal, vol. 49, no. 6, pp. 719–728, 2012