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Design Considerations for Tied Back Soil Landslide Suppressor Walls

사면붕괴 억제 타이백 벽체 설계에 대한 고찰

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ABSTRACT

This paper reviews many of the design considerations surrounding the topic of tiedback landslide suppressor walls primarily for soils app1ications. The design requires combining knowledge of many aspects of soil mechanics and geology to obtain a design a wall that fits site specific conditions. Many of the aspects necessary to complete the design are stil1 not comprehensively studied or understood. This paper provides an overview of the more traditional aspects of tieback wall design and a discussion of newer issues such as suppressor wall earth pressures and rotation of stresses due to tiebacks. An overview is also provided regarding the effect of seismic forces.

요 지

본 논문은 지반 적용을 위한 사면붕괴 억제 타이백 벽체설계의 주제에 관한 여러 가지 설계고려사항을 검토해보았다. 설계는 특정 지역 조건에 맞는 벽체를 설계하기 위한 토질역학과 지질학의 복합된 지식을 요구한다. 설계를 완성하기 위한 많은 측면들이 여전히 완벽하게 연구되어 있지 않다. 본 논문은 기존의 타이백 벽체설계에 관한 개요와 suppressor 벽체 지중압력/타이백에 기인한 응력 회전과 같은 새로운 이슈의 토론 등을 기술하였다. 지진력의 효과에 관해서도 서술하였다.

Keywords : Tieback wall design, Seismic force, Rotation of stress

1. Introduction

Landslide damage in the me United States exceeds $1 billion annually (Schuster et al., 1982). The importance of stabilizing slopes is becoming increasingly important as expensive developments push further into steep gradient, high risk regions with the desire to utilize more land. A variety of methods have been available both externally and internally to remediate slopes. Some of the more common include (Weatherby et al., 1982):

(1) Concrete retaining walls
(2) Buttress fills
(3) Reinforced earth walls
(4) Excavation of the sliding soil
(5) Relocation of the structure
(6) Regrading
(7) Canti1evered Wall
(8) Soil reinforcement
(9) Drainage
(10) Tiedback Wall

However, tiedback walls provide some significant benefit over other a1ternatives, especially in tight working areas. Tiedback walls can be installed with a minimum of disturbance to the up-slope areas, require less soil movement, and therefore, can often be built cheaper. However, because of the addition of tieback forces, the design of these walls

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can be different than other retaining wall type structures. This paper will focus on design considerations for the construction of tiedback landslide suppressor walls for soil applications.

2. Design Considerations

2.1 Tiedback Wall Applications

Tiebacks can be used to support a variety of different types of wall configurations. Tiebacks are used in combination with cantilevered landslide suppressor walls or attached to non-imbedded walls. Some examples include: 1) Soldier pile and lagging, 2) Sheet pile, and 3) Intermittent Drilled Piers. In each case, the tieback provides the same general function. However, the analysis of the wall can vary with assumed stress states regarding the tiebacks. One method includes heavy pre-tensioning of the strands to actually pull the wall back into the slope significantly and produce passive pressures. Another type of analysis assumes a reduction of the pretensioning to prevent significant movement of the wall but without having passive pressures develop. The analysis of these conditions will be covered later in the paper.

2.2 Geologic Aspects and Limitations

The failure modes for slopes are usually highly variable and site dependent. Every combination cannot be covered in detail, however, it is necessary to provide an overview of failure types to limit the focus of this paper. Detailed descriptions categorizing landslide failure types have been developed for rock, soil, and their combination. Rock can fail in methods such as toppling and block slides along discontinuity planes. Failure of these types can be remediated with rock bolts or anchors and knowledge of 3-D rock mechanics.

An emphasis must be placed on an in-depth site exploration detailing strata layers especially in situations where weathered rock exists that can act much like soil. As illustrated below in Fig. 1, distinctly different failure planes can develop if this interpretation is made incorrectly. However, this paper will focus on situations primarily retaining soil, with the understanding the anchors are often rooted in underlying rock.

For the purpose of building tieback soil retaining walls, the focus will be on either rotational or translational type slides. Examples of these are shown below in Fig. 2. Other types of landslides such as flows and lateral spreads are

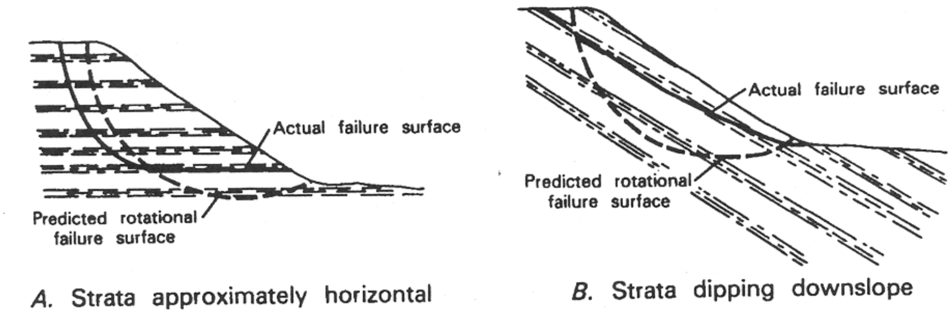


Fig. 1. Strata effects on failure surface (Schuster et al., 1982)

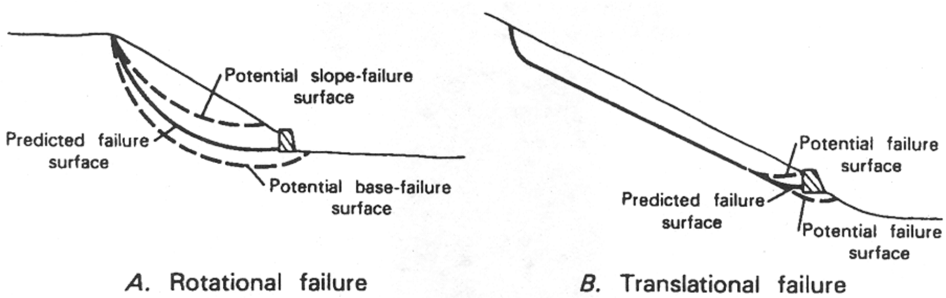


Fig. 2. Failure types (Schustert et al., 1982)

generally not suitable for remediation with walls due to the speed and extent over which they act (Schuster et al., 1982). In these cases deflector walls may be constructed, but a significantly different design method would be necessary.

2.3 Tieback Use

Tiebacks are generally not used in soft to medium clays do to concern with their long term strength. Creep and stress relaxation can lead to highly variable earth pressures. Therefore, Weatherby and Nicholson (1982) have developed some guidelines for the limiting values of soil strength for anchor placement. They suggest high organic content soils, normally consolidated clays, and cohesive soils with an unconfined strength less than 95.8 kN/m^2 and remolded strengths less than 47.9 kN/m^2 may be susceptible to creep. Tiebacks can be installed in soils with greater strengths and a consistency index above 0.8. Consistency index (I_C) can be defined as follows in Equation 1.

$$I_c = \frac{W_L - W}{W_L - W_P} \tag{1}$$

where, W_L=Liquid Limit, W=Natural water content, and W_p=Plastic limit.

Fig. 3 provides a simplified sketch of a tieback wall. The anchor length should be located in soils deemed both

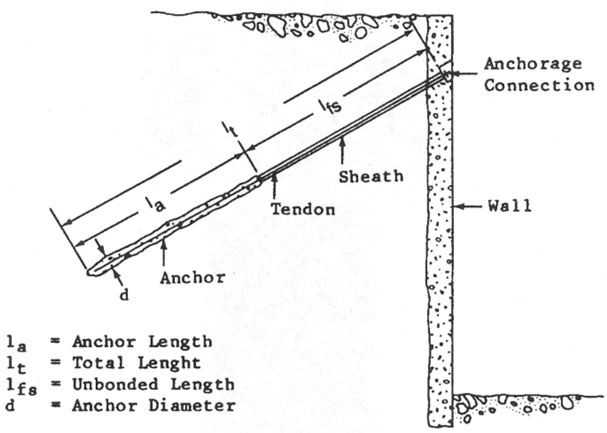


Fig. 3. Tieback components (Weatherby et al., 1982)

strong enough according to the above criteria and below the perceived failure plane. This assumption is analogous to the criteria developed in the design of tiebacks for excavations where the tiebacks must be placed behind the active soil wedge as defined by the assumed failure plane. If the bonded anchor lengths fall within the failure zone, they provide no benefit other than holding the failure block of soil together. Soil strength is not accounted for over the distance of the unbonded length. Therefore, the capacity of the tieback becomes a function of the bond developed only through the anchor length.

Numerous types of anchorage methods involving tiebacks exist for the different combination of requirements a specific site provides. Some of the considerations include:

1) Corrosion Protection
2) Length
3) Soil/Rock Strength and Properties
4) Installation Space Limitations
5) Angle of Inclination
6) Easement Location

These criteria are often interrelated. For example, length and angle of inclination must often be selected to fall within any easements onto adjacent properties where tie-backs may protrude. Fig. 4 illusnates how some of these functions are accounted for with a grout protected tieback.

2.4 Corrosion

Corrosion of me tendons becomes a concern in land-slide controlling walls as tiebacks are called upon to sustain strength in a long-tem use situation. Steel tendons in the ground mainly suffer from corrosion stemming from electrochemical reactions (Hobst et al., 1983). In this process, the metal surface reacts with soil moisture that acts as all electrolyte. Electrical current passing between susceptible surfaces on the metal causes corrosion. These chemical reactions can either be anodic or cathodic. Anodic reactions happen when ions of one metal pass into solution

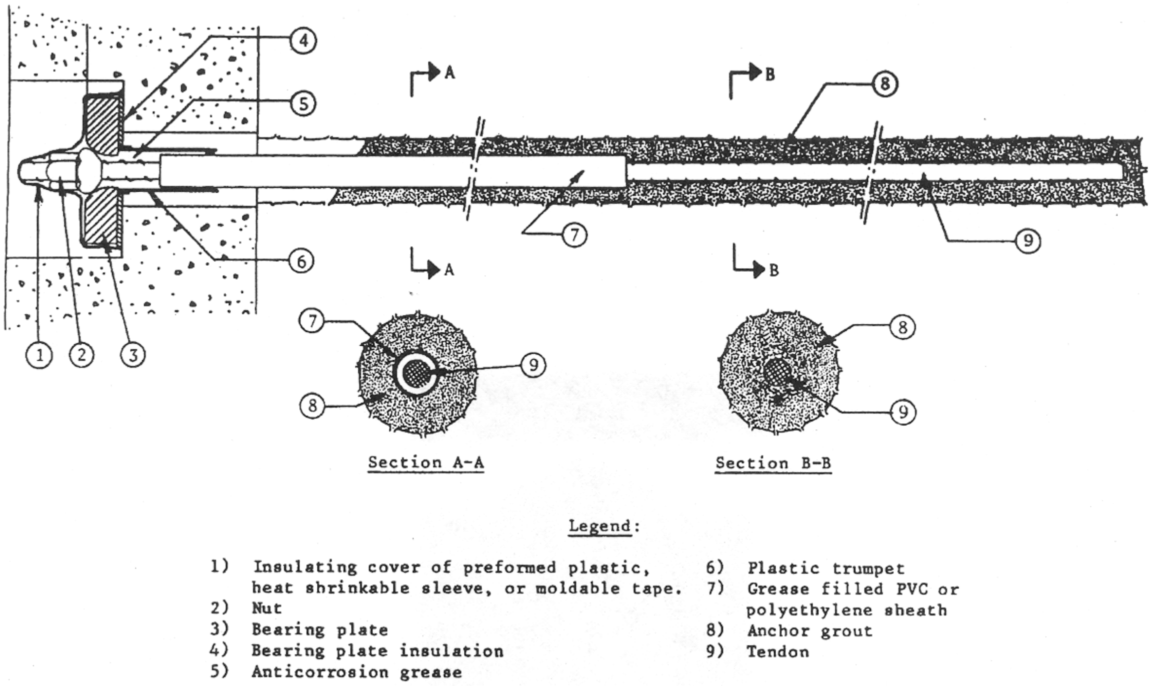


Fig. 4. Tieback details (Weatherby et al., 1982)

as free hydrated ions. Cathodic reactions occur when metal ions come from out of the solution to recombine with the metal. The cathodic reactions can reinforce the anodic reactions by drawing off electrons released by anodic reactions. After time, with a decrease in oxygen, the products become insoluble substances and further erosion is stopped.

Studies have shown no evidence of corrosion failure when the tieback tendon was encased in grout (Weatherby and Nicholson, 1982). However, if the soil surrounding the anchor has a pH less than 5, a resistivity less man 2000 ohm-cm, or with high sulfide levels, they suggest a local corrosion condition could develop on the tendon. In this situation, the tendon can be completely encapsulated inside a corrosion tube. If normal soil conditions exist, the focus of protection shifts to the unbonded length portion of the tendons. Fig. 4 above shows how a grease filled PVC pipe can be used to encapsulate this section.

Grout protected tiebacks should also be electrically isolated from the structures they are attached to. Fig. 4 above shows how anchorage insulation can be used to isolate the tendons. This becomes important as a condition referred to as a long-line differential corrosion cell can develop. This system becomes dangerous for two reasons. First, oxygen is not necessary. Therefore, corrosion will not be stopped in the manner described previously. Secondly, the relative size of the anode and cathode become extremely large. This occurs as the tendon at the top of the anchor zone becomes the anode and the rest of the wall becomes the cathode (Weatherby et al., 1982).

2.5 Location

The location of tiebacks and the wall itself are often a product of situational conditions. Fig. 5 below shows an example of a situation constrained by surrounding structures, a common situation when tiedback walls are required.

The number of rows of anchors required becomes a function of the depth of the slip surface and the capacity

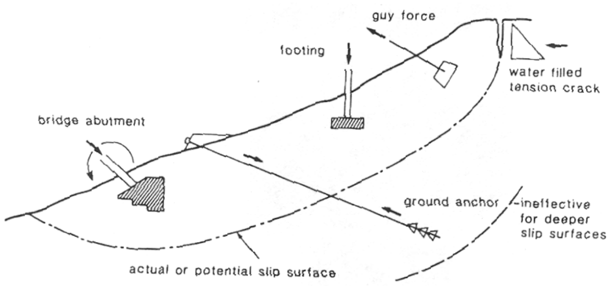


Fig. 5. Location constraints and forces (Bromhead, 1982)

of the anchors. It is suggested a minimum 4.6 m of overburden lie above the anchor. Weatherby and Nicholson (1982) suggest penetration of the failure surface with the wall becomes a function of continuity of the soil. For example, weathered rock may be simply tied back without wall penetration because the material will behave more as a block. However, as illustrated above, many tieback walls in soil function without this suggested penetration.

2.6 Length

The length and location of the tiebacks into the slope should be located so that the anchor is located beyond the critical failure surface. Fig. 5 emphasizes an important consideration when acknowledging failure surfaces when using tieback walls. The failure surface can actually be changed from the previous critical slip surface to a lower plane behind or below the tiebacks. Balanko et al., provide an excellent example of this phenomenon on their case study of the Edmonton Convention Center (1982). Weatherby and Nicholson (1982) suggest the length should be established so the most probable failure surface passing through the ends of the tiebacks, or behind them, would have a factor of safety equal to or greater than on the critical failure surface (with the tiebacks installed). They also provide a suggested upper bound of 45.7 m for unbonded anchor lengths.

2.7 Angle of Inclination

Most soil anchors are installed at an angle between 10 and 30 degrees. If anchors are applied at less than 10 degrees, special grouting techniques are required (Weatherby et al., 1982). However, wall placement and site conditions often require tiebacks to be installed at angles well above 30 degrees. For example, a solid rock anchor boundary might be available at a relatively deep level near a highway location placement. As a compromise between excessive anchor lengths and steep angles, a 45 degree soil anchor angle may be agreed upon. However, this steep angle brings up an important design consideration in tieback walls. The steeper the angle of inclination, the higher the vertical

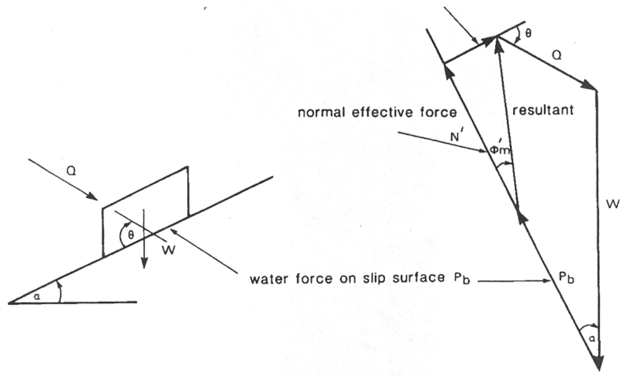


Fig. 6. Tieback force polygon (Bromhead et al., 1992)

force pulling down on me wall structure. The foundation of the wall must account for these extra vertical pretensioning forces. If the design takes place in soil or weak weathered rock that acts as a block. Fig. 6 below illustrates how a sliding block analysis can be used to determine an optimum angle of inclination.

3. Soil Property Analysis

3.1 Stress Conditions

In general, shear strength of soils can either be calculated using total or effective stresses. Because of the important influence of water, some authors suggest effective stresses are normally used in the calculation of stresses for landslide conditions. However, in many cases a total stress description of conditions must be used. Therefore, it is important to emphasize an analysis must be made using both limiting conditions based on site dependent situations. The total stress situations occur when the undrained shear strength Cu (With $\phi = 0$) provide a smaller strength than the drained analysis. This type of analysis generally corresponds to a short-term condition where pore pressures are not allowed to drain and usually occurs in low permeability materials like clay. However, if drainage produces a decrease in strength, the long term, drained situation becomes critical. For partially saturated soils, the prediction of porewater pressures becomes difficult and in-situ measurements must be taken to ascertain strength properties. The details of overconsolidation, sensitivity, and its effects in various types of soil must also be accounted for on a site-specific basis. This paper will not address these issues entirely to focus on design considerations more directly related to tiebacks. The 1994 FHWA manual provides an excellent coverage of various analyses and the assumptions regarding stress conditions necessary for each approach.

3.2 Orientation of Stresses

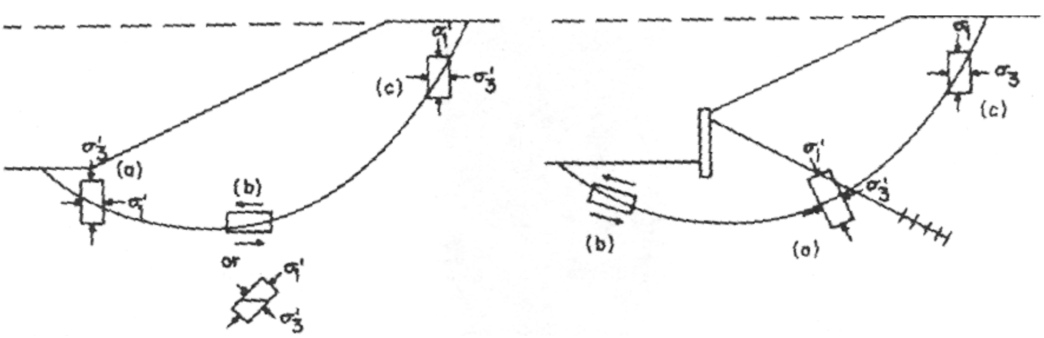
As mentioned above, tiebacks can provide some different design considerations than standard landslide suppressor walls depending on the pretensioning in the strands of the tieback. However, a general knowledge of rotational stress states is necessary even without me tieback loads. Fig. 7(a) shows how the movement along a slope can produce a reduction and rotation of principle stresses (Sangrey, 1982). It has been shown that these changes can significantly affect the strength properties of the soil. The FHWA manual cites the 1966 work of Duncan and Seed where they found the undrained strength of specimen (a) is 75 percent of element (c) in Fig. 7(a). Fig. 7(b) shows a very different condition of stresses induced by tiebacks especially in the case of high pretensioning.

As sketched, the stress distribution suggests the anchor force may impede the development of progressive failure. Sangrey (1982) states that the necessary research and field observations to account for rotation of stresses based on this additional rotation of stresses is not currently available. Although not mentioned, the applicability of this stress state seems to become a question of scale. For example, a smaller circular surface as sketched above might tend to act in the stress state shown. However, in consideration of a longer translational type failure, the rotations of stresses will primarily only act on the lower slope where the tiebacks are acting. This becomes an issue that needs to be addressed on a site dependent basis using engineering judgment.

The excavation and construction of the wall involves additional strength considerations. Sangey (1982) provides some of the following insights on the subject.

a) The compressive stresses imposed by the anchor system on the soil mass and failure surface will offset the effects of excavation.
b) The installation of the anchor will generally result in an increase in both the shear stress and normal stress in the soil surrounding the anchor.
c) The stresses imposed by the wall on the adjacent soils will be larger than those for slopes without a wall.
d) Locking off the tieback load will preload the soil and limit the potential strain in the soil.

In general these changes will provide a beneficial aspect to soil strengths. However, if the tiebacks are placed in soils instead of anchored in rock, Bromhead (1992) brings up some important considerations. The initial prestresses cause an increase in porewater pressure for soils surrounding the anchors. Over time, these stresses decreases, consolidation takes place with drainage, and the prestressed loads are lost. To maintain the benefits of prestressing, it is beneficial to periodically re-tension the



(a) (b)

Fig. 7. Stress rotations (Sangrey, 1982)

anchors as losses occur. However, increasing the load can at best only worsen the condition which gave rise to the problems initially, or at worst, lead to failure. This strength decreasing loop leads to the aforementioned, limiting strength criteria provided by Weatherby and Nicholson (1982) requiring anchors to be placed in either strong soil or rock. They also address the long tem relaxation, creep, and seating issue by suggesting a minimum unbonded length of 4.6 m.

3.3 Earth Pressure

Much debate has surrounded the issue of earth pressures in landslide suppressor walls and a number of methods have been suggested on how to determine them. The first suggests variations of conventional earth pressure theories based on wall movements. A second analysis can be done by back calculation of strength properties of a soil pertaining to information when the slope originally failed. The benefits of this are evident as it has been found laboratory strength values often do not coincide well with what apparently develops in the field. The lab values obtained often indicate higher strengths and hence, a dangerous situation can develop. However, the designer does not always have the luxury of analyzing previously failed slopes at the site and an empirical method is needed.

Wright et al., (1989) provided a paper on earth pressures in regards to simple cantilever piles without tieback systems. For this analysis, the authors analyzed slide suppressor walls located at 1/3 of the slope height or lower. He chose the second method of back-calculation to obtain earth pressures. With the knowledge of the slide geometry, the unit weight of the soil, and that the factor of safety for failure corresponds m a value of 1, he showed only one set of values of the infinite combinations of ϕ and c will produce the critical surface observed. Citing work done by Abrams and Wright using this method, they found differences between total and effective stress analyses were less than 20 percent. They then analyzed different failure surfaces using the following two methods: 1) Spencer's method with circular shear surfaces and 2) the "trial wedge" method using planar shear surfaces. Although Spencer's method produced slightly greater forces, the difference proved insiglificant and planar shear surfaces were relatively shallow failures. Because their case studies involved relatively small cohesion values, they proceeded with assuming zero cohesion using Rankine and Coulomb techniques. The authors made the typical assumption with Rankine pressures acting parallel to slope. However, in their Coulomb calculations an error seems to have been made in assuming the resultant pressures acted horizontally. Regardless, they proceeded with this analysis to find the earth pressures coefficients at least 0.8 and some exceeding 0.9 times the unit weight. Due to this fact, their final equation in regards to earth pressures simply acknowledged an equivalent fluid pressure equal to the unit weight of the given soil. The final pressure distribution is given below in Equation 2.

$$P = \frac{1}{2}\gamma H^2 \tag{2}$$

The assumptions of this analysis must be looked at further, but we will now move to case studies involving tiebacks.

Unfortunately, the addition of a tieback just makes the situation more difficult. Hovland and Willoughby detail the Geyser Power Rant case study in Northern California in their 1982 paper that includes a tieback landslide suppressor wall. The authors describe a retaining wall tied back with rock anchors as "a rather challenging earth pressure problem," They suggest because of movements within the slide and restraint at the top of the wall due to anchors, a limit equilibrium state of minimum active earth pressures cannot be assumed. Movements within the wall are likely to create a situation somewhere between the active and at rest earth conditions. They go on to state that estimation of earth pressures for complicated situations is likely to remain a semi- empirical procedure based on their literature review.

In their case study, for analysis reasons, they had the advantage of having a previous landslide on site and one occur during construction of a power plant on the slope when the calculated factor of safety was 1.41. They therefore proceeded with back calculation analyses similar to the second method mentioned above to determine a more appropriate analysis. The earth pressures developed are shown below in Fig. 8. Thee different situations were developed based on movements of the wall base and tieback. An equivalent fluid pressure of approximately 15.4 kN/m^3 was used which is above both the at-rest and active pressures suggested by the standard deformation criteria. They suggest the value comes from an average of active and at rest pressures, however this would correspond to a value around 2,873 N/m^2. Regardless of its interpretation, their value does coincide with the 0.8 to 1.0γ suggested by Hovland and Willoughby. However, on top of this calculation an additional cohesive block force corresponding to 22.5 kN/m^2 was added. Although not mentioned in the paper, this force seems to correspond to a passive type cohesive force following basic earth pressure theory derived from the prestresses in the tiebacks. This is a significant jump as now the earth pressure theory used is now combining differently factored active and passive type analyses to obtain the pressures on the wall.

The combination of these earth pressures suggest the values obtained from Eq. (2) and Wright et al., may be underestimating forces if used for tie-back retaining walls. This may be due to a number of reasons including:

1) Cohesion assumptions
2) Tieback forces and passive pressures
3) Deeper seated failures
4) Assumed Coulomb resultant angle

The zero cohesion assumption in a material with high cohesive strengths in a passive condition leads to a much smaller earth pressure. This cohesion must be accounted for in analyses with materials with a high cohesion. As evidenced by the three cases sketched in Fig. 8, the expected behavior of the tieback and seating of the wall must be calculated as they will provide significantly different earth pressures. As mentioned the work by Wright et al., was based solely on shallow failures. More deep seated failures analyzed by Spencer's method might provide larger earth pressures than suggested by the plane strain assumptions that they used.

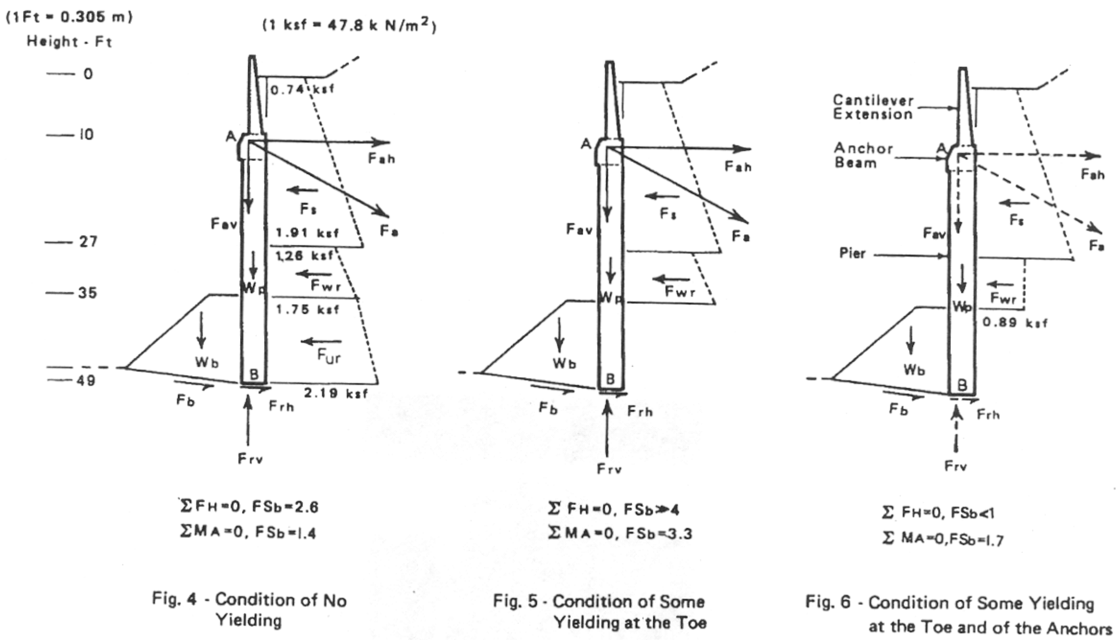


Fig. 8. Yielding condition effects on earth pressures (Hovland, 1982)

4. Other Considerations

4.1 Seismic Conditions

Earthquakes can often provide the additional inertial forces necessary to induce landslide behavior into slopes. The earth pressures induced on landslide suppressor walls due to earthquakes must therefore be accounted for in situations where seismic concerns exist. Note that these forces are in addition to the earth pressures discussed above. Various static and dynamic methods have been identified to analyze slope stability. The FHWA manual (1994) provides the following guidelines to be considered when undertaking a seismic evaluation:

1) Determine the critical horizontal seismic coefficient (k_{crit}) that generates a FS = 1.0 for the static critical failure surface; if $k_{design} < k_{crit}$, the slope can be expected to be stable during the design earthquake.
2) Find the maximum peak ground acceleration, A_{peak}, expected at the site
3) Then by comparing the calculated k_{crit} and assigned A_{peak} (in g's) the following conclusions in Table 1 may be made based on the 1984 Hynes-Griffin and Franklin report:

To establish the critical seismic coefficient, K_{crit}, a pseudostatic analysis is employed. This method modifies the limit equilibrium method to account for a seismic force that remains a static rather than dynamic force. The seismic force is assumed to act horizontally and is proportional to the weight of the sliding mass. This creates an inertial force kW which is the product of the weight, W, and seismic coefficient, k. A complete analysis is carried out and a plot should be developed presenting FOS vs. seismic coefficient. The aforementioned critical coefficient, kcrit can be determined where the curve crosses a factor of safety of unity as shown in Fig. 9. Where the projected seismic coefficient for a site is greater than the critical coefficient just determined, the FHWA manual suggests the use of the Newmark Sliding Block Method to determine displacements.

Table 1. Seismic damage (FHWA, 1994)

Condition	Remarks
$K_{crit}<1/2A_{peak}$	Slop can be expected to survive the design earthquake
$1/2A_{peak}<K_{crit}<A_{peak}$	Minor to major damage can be expected
$K_{crit}>A_{peak}$	Overall damage predicted, may with to consider a complete dynamic analysis

In terms of direct application to landslide suppressor walls, some difficulty may be encountered in identifying a critical failure surface. Different critical surfaces may often be encountered for each seismic coefficient (FHWA, 1994). This phenomena is probably encountered in situations with relatively uniform soil properties throughout the analyzed area. However, in the design of tiedback landslide suppressor walls, a stronger stratum is already identified for the location of the tiebacks. Therefore, the dimensions and placement of the wall should already encompass all critical failure surfaces if soil conditions are analyzed correctly.

4.2 Tieback Testing

Testing of tiebacks should be employed to verify they will hold the design load without excessive movements. This long tem strength of the bonds becomes particularly important for tieback landslide suppressor walls as many of the design assumptions accounted for above rely on maintaining the prestressing induced during construction.

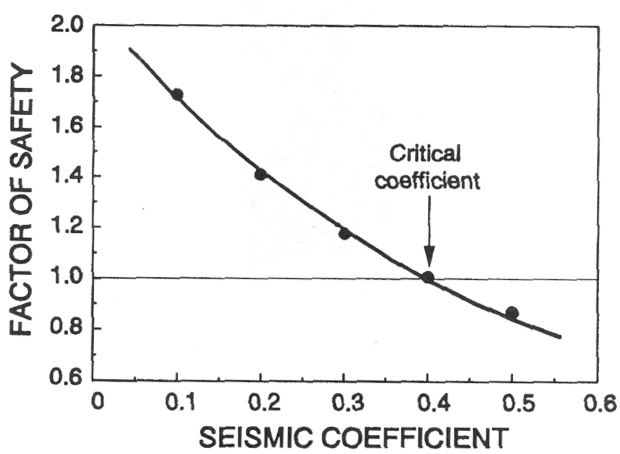


Fig. 9. Critical seismic coefficient

The three types of tests generally used to determine suitability include: performance, proof, and creep. In the cases where anchoring into rock, destructive tests are used in some cases and only a percentage of anchors (not on the structure) are tested (Hobst and Zajic, 1983). All tiebacks should be tested in situations where me tieback is anchored in soil. For the purpose of brevity, an overview will be provided for soils applications as rock anchors follow the same basic principles.

A hydraulic jack and pump are used to apply the test loads. These loads arc usually in the range of 133 to 200 percent of the design load. The movement of the tieback is monitored by a dial gauge or similar device independently supported on an additional setup. The extension of the hydraulic jack cannot be used as the additional deflection of the wall and structural elements would indicate additional movement. The total movement is made up of the following four increments (Weatherby et al., 1982): 1) Elastic elongation of the tendon 2) Residual movement of the anchor 3) Elastic movement in the anchor 4) Creep movements of the anchor and tendon. As a tieback is loaded, the anchor moves through the soil developing capacity. When the load is reduced to zero, a portion of the movement is elastic and recoverable. The portion of the movement mat is non- recoverable is referred to as residual anchor movement.

Performance tests are done by measuring the load applied to the tieback and its movement during the incremental loading and unloading. They are usually done on the first few tiebacks and a selected percentage of the remainder of the tiebacks (Weatherby et al., 1992). Because of the complete loading and unloading cycle, performance tests can be used to separate and identify the causes of tieback movement. They can also be used to verify that me unbonded length has been provided. Several design criteria are available for load testing and it should be emphasized arbitrary movement should not determine whether a tieback meets me standards. Different strains are necessary to develop capacity in different anchor types. One of the methods is provided below to illustrate the different test types. In non-cohesive soils and rocks, the maximum load applied during the tests is held constant for 10 minutes. If the movement is less than 1 mm, the test is discontinued. If the movement exceeds 1 mm, the loads are held for 1 hour and a creep curve can be developed for interpretation.

A proof test is a simpler version where movements are monitored only during incremental loading. Every tieback which is not performance tested should be proof tested. The duration the maximum load is held during a proof test is reduced to five minutes in a proof test. However, the allowable movement is also reduced to 0.76 mm.

If the movement exceeds this value, the load is maintained until the creep rate can be determined and compared to either performance or creep test values.

Creep tests are performed to evaluate long-term load carrying capacity in cohesive soils. These tests are often done during a separate testing program due to the length of time needed to obtain data. A plot of data is produced showing movement vs. log base time. The data is then compared to a characteristic creep curve as shown below m Fig. 10. Curve (c) indicates me tieback would creep to failure while curve (a) illustrates acceptable behavior. Intermediate behavior such as curve (b) must be analyzed on an individual case study.

5. Conclusion

Tiedback landslide suppressor walls can provide an effective solution to stabilize high risk slopes especially in situations where space becomes an issue. An appropriate geologic interpretation of failure planes and soil properties are necessary to assess tieback locations and

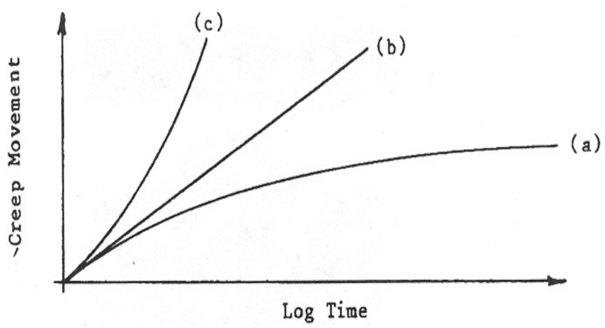


Fig. 10. Creep characteristic curves (Weatherby, 1982)

behaviors. This paper focused on situations retaining soil rather than anchoring rock.

Traditional tieback concerns such as corrosion, length, angle of inclination have been addressed with typical values and solutions. A correct geologic interpretation of failure planes and stress analyses are necessary for an adequate design. Seismic concerns need to be addressed when a site specific critical horizontal seismic coefficient is compared to expected ground accelerations in the area.

Earth pressures on tiedback landslide suppressor walls need to be accounted for differently than standard retaining walk. The prestressing in the tendons mobilizes a portion of the passive resistance of the soil behind the wall. The pressure diagrams become a semi-empirical method accounting for anticipated movement, equivalent fluid pressures, and a cohesive component where applicable.

The possible rotation of stresses depends upon the level of prestressing and the scale over which the tiebacks act. This needs to be accounted for on a site dependent basis and testing should be done accordingly.

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