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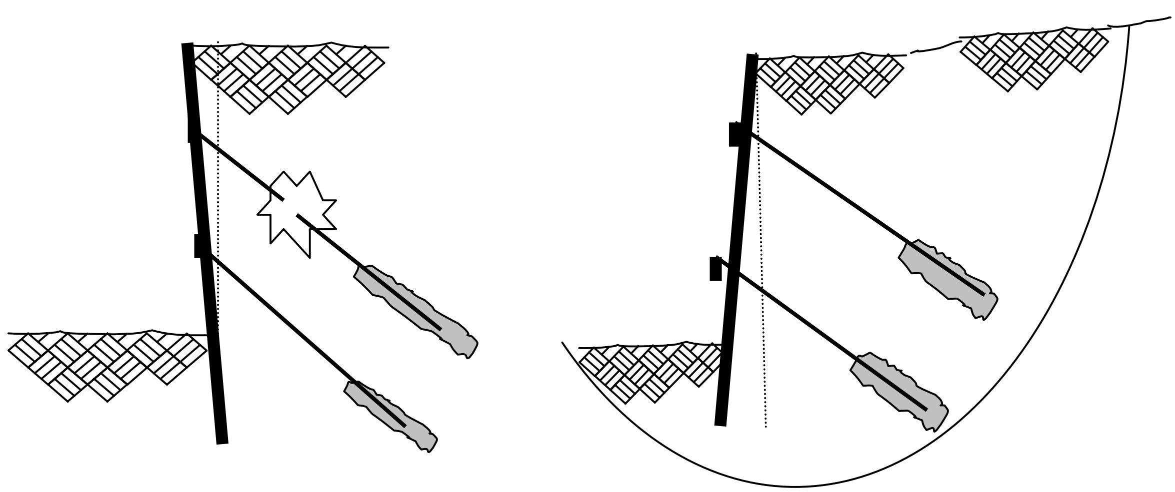
Innovations for Navigation Projects Research Program

Methods Used in Tieback Wall Design and Construction to Prevent Local Anchor Failure, Progressive Anchorage Failure, and Ground Mass Stability Failure

Ralph W. Strom and Robert M. Ebeling

December 2002



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Methods Used in Tieback Wall Design and Construction to Prevent Local Anchor Failure, Progressive Anchorage Failure, and Ground Mass Stability Failure

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ABSTRACT: A local failure that spreads throughout a tieback wall system can result in progressive collapse. The risk of progressive collapse of tieback wall systems is inherently low because of the capacity of the soil to arch and redistribute loads to adjacent ground anchors. The current practice of the U.S. Army Corps of Engineers is to design tieback walls and ground anchorage systems with sufficient strength to prevent failure due to the loss of a single ground anchor.

Results of this investigation indicate that the risk of progressive collapse can be reduced by using performance tests, proof tests, extended creep tests, and lift-off tests to ensure that local anchor failures will not occur and to ensure the tieback wall system will meet all performance objectives; by using yield line (i.e., limit state) analysis to ensure that failure of a single anchor will not lead to progressive failure of the tieback wall system; by verifying (by limiting equilibrium analysis) that the restraint force provided by the tieback anchors provides an adequate margin of safety against an internal stability failure; and by verifying (by limiting equilibrium analysis) that the anchors are located a sufficient distance behind the wall face to provide an adequate margin of safety against external stability (ground mass) failure.

Design measures that can be used to protect against local anchor failure are described, along with testing methods that can be used to ensure that anchor performance meets project performance objectives. Examples are given to demonstrate the yield line analysis techniques that are used to verify that the wall system under the "failed anchor" condition can safely deliver loads to adjacent anchors and to ensure that the failure of a single anchor will not lead to progressive wall failure are. Limiting equilibrium analysis procedures used for the internal and external stability of tieback wall systems are also described. Simple procedures applicable to "dry" homogeneous sites and general-purpose slope stability programs applicable to layered sites (with and without a water table) are also illustrated by example.

Contents

Preface

The study described in this report was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Innovations for Navigation Projects (INP) Research Program. The study was conducted under Work Unit (WU) 33272, "Soil-Structure Interaction Studies of Walls with Multiple Rows of Anchors."

Dr. Tony C. Liu was the INP Coordinator at the Directorate of Research and Development, HQUSACE; Research Area Manager was Mr. Barry Holliday, HQUSACE; and Program Monitors were Mr. Mike Kidby and Ms. Anjana Chudgar, HQUSACE. Mr. William H. McAnally of the ERDC Coastal and Hydraulics Laboratory was the Lead Technical Director for Navigation Systems; Dr. Stanley C. Woodson, ERDC Geotechnical and Structures Laboratory (GSL), was the INP Program Manager.

This report was prepared by Mr. Ralph W. Strom, Portland, OR, and Dr. Robert M. Ebeling, U.S. Army Engineer Research and Development Center (ERDC), Information Technology Laboratory (ITL). The research was monitored by Dr. Ebeling, Principal Investigator for WU 33272, under the supervision of Mr. H. Wayne Jones, Chief, Computer-Aided Engineering Division, ITL; Dr. Jeffery P. Holland, Director, ITL; and Dr. David R. Pittman, Acting Director, GSL.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL John W. Morris III, EN, was Commander and Executive Director.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	meters
foot-pounds (force)	1.355818	joules
inches	25.4	millimeters
inch-pounds (force)	0.1129848	joules
kips (force)	4.448222	kilonewtons
kips (force) per square foot	47.88026	kilopascals
kips (force) per square inch	6.894757	megapascals
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals

1 Introduction

1.1 Preventing Progressive Anchor System Failure–Corps Practice

This report assumes that the current Corps practice is to design tieback walls and ground anchorage systems with sufficient strength to prevent failure due to the loss of a single ground anchor.

1.2 Preventing Progressive Collapse of Tieback Wall Systems

A local failure that spreads throughout the structure is termed a progressive collapse. With tieback wall systems, the local anchorage failure could originate through any of the failure mechanisms described in Figures 1.1a, b, and c. The risk of progressive collapse of tieback wall systems is inherently low because of the capacity of the soil to arch and redistribute loads to adjacent ground anchors. This is illustrated for a soldier beam with concrete lagging system in Figures 1.2 and 1.3. Figure 1.2 illustrates redistribution for a wall with multiple rows of anchors. Figure 1.3 illustrates redistribution for a wall with a single row of anchors. The risk of progressive collapse can be further reduced by

- Designing to prevent local anchor failure under extreme loading conditions.

- Using performance tests, proof tests, extended creep tests, and lift-off tests to ensure that local anchor failures will not occur and that the tieback wall system will meet all performance objectives.

- Using yield line (i.e., limit state) analysis to ensure that failure of a single anchor will not lead to the progressive failure of the tieback wall system.

- Verifying by limiting equilibrium analysis that the restraint force provided by the tieback anchors provides an adequate margin of safety against an internal stability failure.

- Verifying by limiting equilibrium analysis that the anchors are located a sufficient distance behind the wall face to provide an adequate margin of safety against external stability (ground mass) failure.

Design measures that can be used to protect against local anchor failure are described in Chapter 2. Testing methods required to ensure that anchor performance meets project performance requirements are described in Chapter 3. Yield line analysis techniques are used to verify that the wall system under the "failed anchor" condition can safely deliver loads to adjacent anchors, and to ensure that the failure of a single anchor will not lead to progressive wall failure. This process is described in Chapter 4. The yield line analysis process is demonstrated using the "Granular Soil Design Example" of the Federal

Highway Administration publication FHWA-RD-97-130. Design computations for this example are provided in Appendix A.

Limiting equilibrium analysis procedures used for the internal and external stability of tieback wall systems are described in Chapter 5. The procedures used to evaluate stability as described herein are applicable to all tieback wall designs and therefore are not special to the “loss of anchor” analysis. Internal and external stability evaluation techniques were initially introduced in Strom and Ebeling (2001). The use of simple force equilibrium procedures with respect to tieback walls constructed at “dry” homogeneous soil sites were demonstrated by example in Strom and Ebeling (2002). These simple procedures for dry homogeneous soil sites are also demonstrated by example herein, and the results are compared with those obtained from general-purpose slope stability (GPSS) programs (e.g., CSLIDE and UTEXAS4).

CSLIDE and UTEXAS4 analyses are also used to demonstrate the stability evaluation process for walls constructed at partially submerged homogeneous soil sites. Internal stability analyses are used to verify that the anchor design loads are adequate. External stability analyses are used to ensure that the anchor location (i.e., distance behind the wall) will provide an adequate margin of safety against a ground mass stability failure.

Simple force equilibrium methods and GPSS analysis techniques used to evaluate internal and external stability are demonstrated with respect to a 30-ft- high[1] tieback wall system. Internal stability calculations for a 30-ft-high tieback system retaining a dry cohesionless soil are presented in Appendix B. Internal stability calculations for the same system retaining partially submerged cohesionless soil is presented in Appendix C. External stability calculations for the 30-ft-high tieback wall system with a single row of anchors for both dry and partially submerged conditions are provided in Appendix D. External stability calculations for the same system with two rows of anchors are demonstrated in Appendix E for the partially submerged condition. In Appendix F, the internal and external stability evaluation process for a tieback wall constructed in a layered soil system is demonstrated by example. The layered soil system includes a piezometric water surface (i.e., static water table) located in the soil retained by the wall.

The simple force equilibrium methods and GPSS analysis techniques demonstrated in this report represent the state of the practice as described in Strom and Ebeling (2001). Future studies using nonlinear finite element soil-structure interaction analyses will be conducted to verify the suitability of these methods with respect to the internal and external stability analysis of tieback wall systems.

[1] A table of factors for converting non-SI units of measurement to SI units is presented on page vii.

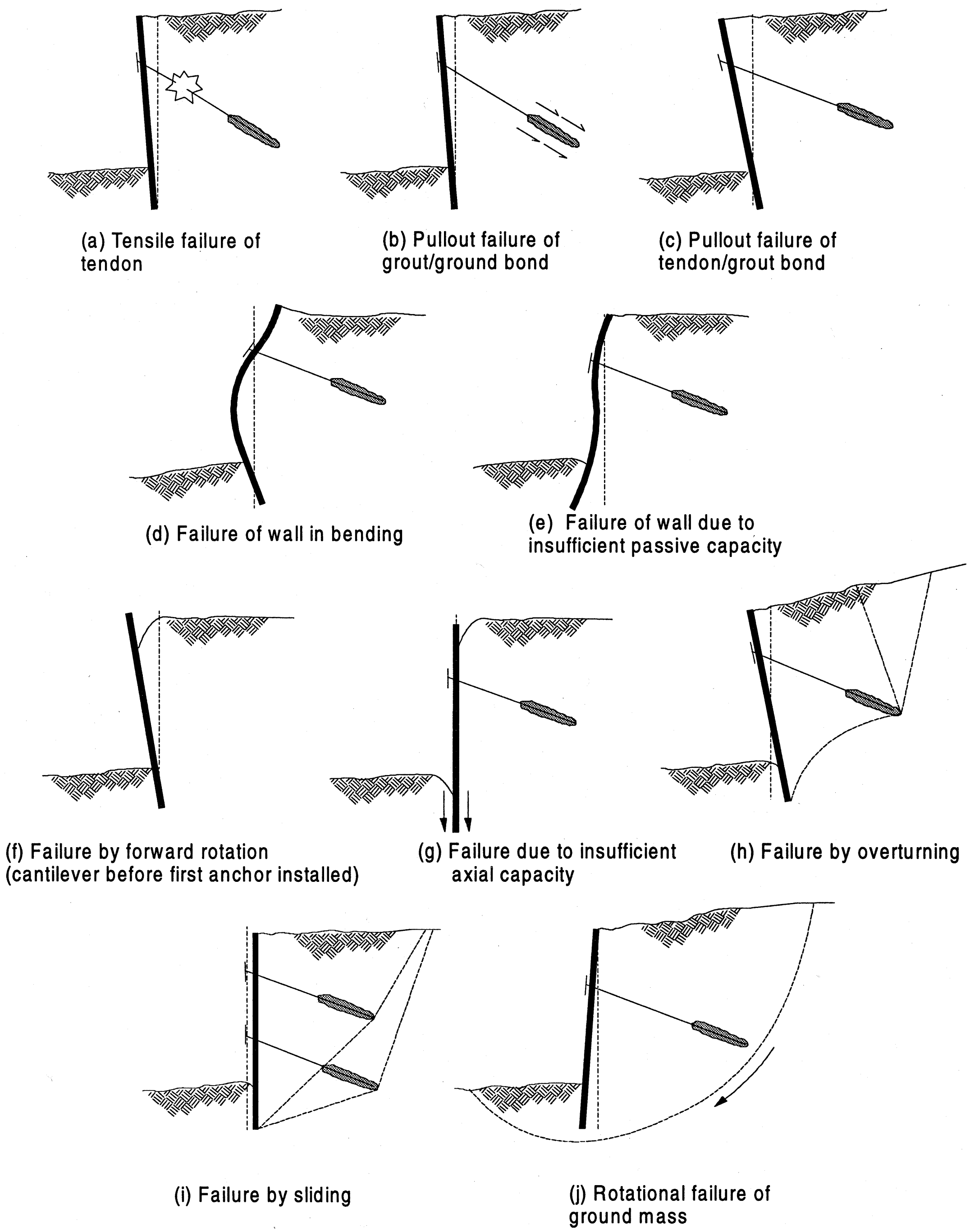


Figure 1.1 Failure conditions to be considered in design of anchored walls (after Figure 11, FHWA-SA-99-015)

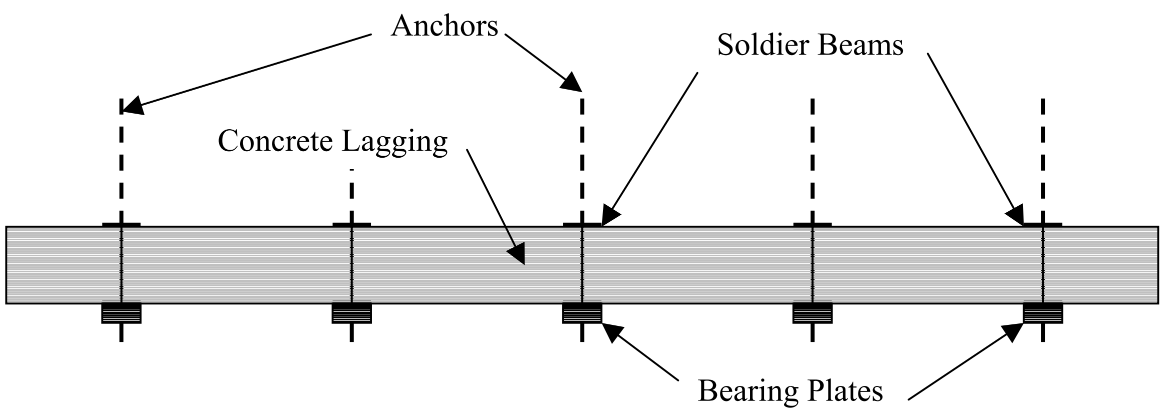


Figure 1.2a Soldier beams with concrete lagging – sectional plan

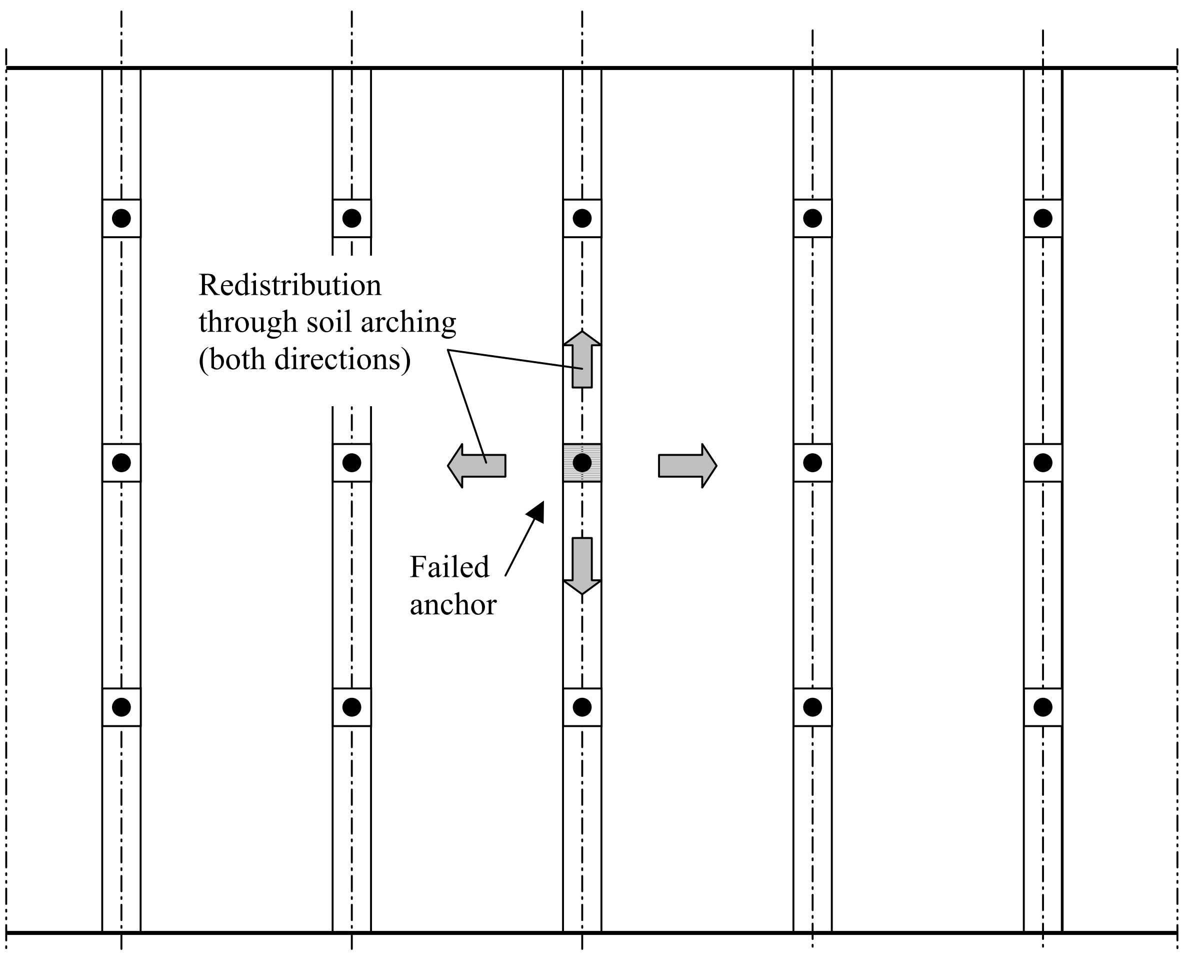


Figure 1.2b Soldier beams with concrete lagging–elevation, multiple rows of anchors, and redistribution through arching–both directions

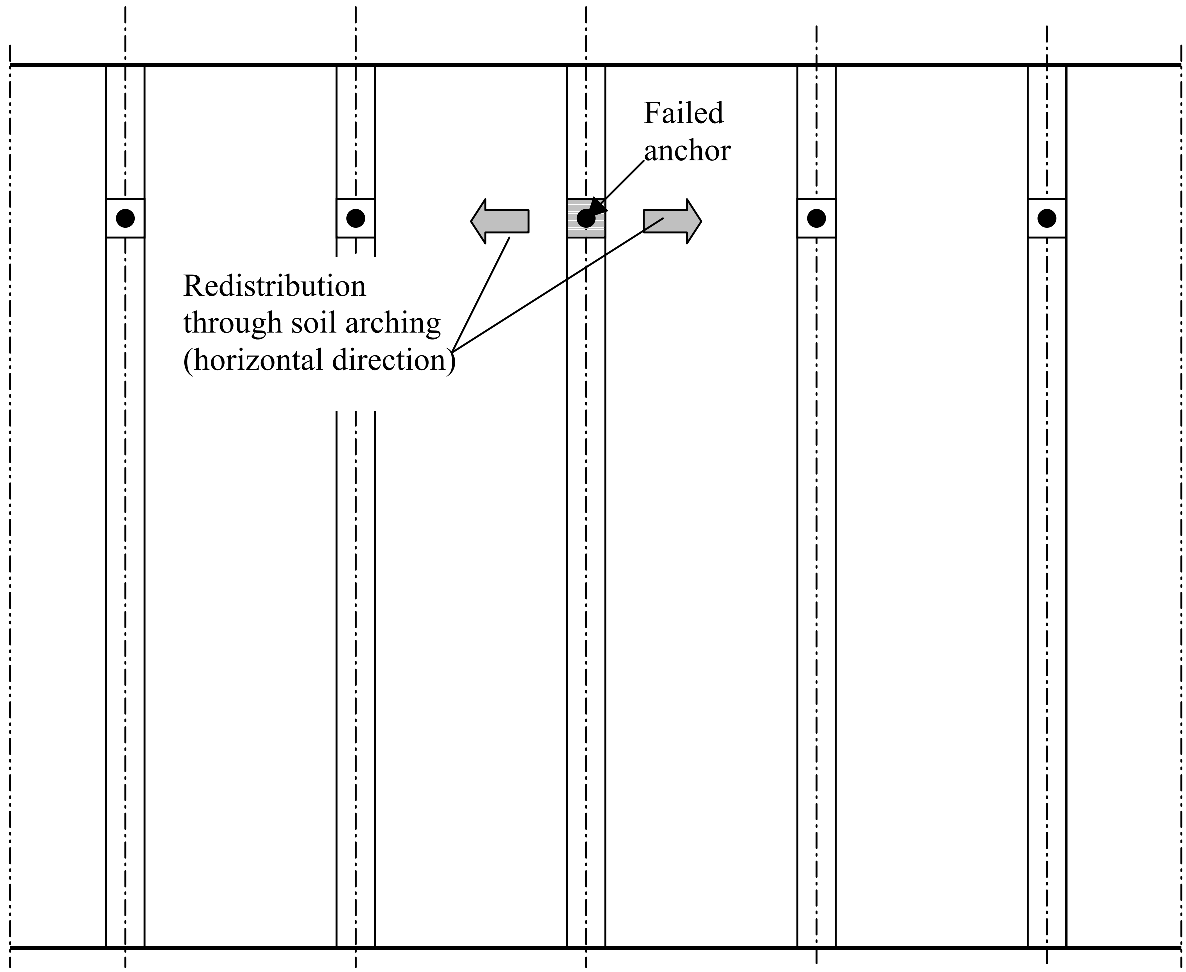


Figure 1.3 Soldier beams with concrete lagging–elevation, single row of anchors, and redistribution through arching–both directions

2 Preventing Local Anchor Failures

2.1 General

The risk of local anchor failures (i.e., loss of a single anchor) can be minimized through

- Proper design procedures.
- Performance testing, proof testing, and extended creep testing (see Chapter 3).
- Lift-off testing to verify lock-off load.
- Modifying design and/or installation procedures when testing indicates such modifications are necessary to meet project acceptance criteria (i.e., performance objectives).

2.2 Ground Anchor Design

2.2.1 Maximum design load

Often, the ground anchor test load is set equal to133 percent of the design load. The maximum test load should not exceed 80 percent of the specified minimum tensile strength (SMTS) of the prestressing steel (refer to paragraph 5.3.8 of FHWA-SA-99-015). Therefore, the maximum design load would be equal to 0.8/1.33, or 0.60 SMTS. For projects that must satisfy the Corps "loss of a single anchor" criterion, it is suggested that the ground anchor test load be set equal to 150 percent of the design load. This more stringent testing requirement reduces the chance for a single anchor failure. However, the maximum design load would be 0.8/1.50, or 0.53 SMTS rather than 0.60 SMTS. The single failed anchor condition is described below with respect to tieback walls with a single row of anchors and with respect to tieback walls with multiple rows of anchors. The information covered in the subsequent paragraphs assumes that loads from the failed anchor can be redistributed to adjacent anchors. Sometimes redistribution cannot occur due to tieback wall system discontinuities. Wall system discontinuity issues are addressed in Chapter 4.

2.2.1.1 Tieback walls with a single row of anchors

A tieback wall system with a single row of anchors is illustrated in Figure 1.3. The single-row arrangement is also common to the cap beam system described in Chapter 4. For walls with a single row of anchors, it can be conservatively assumed that the two anchors on each side of the failed anchor will pick up the additional load, with half the additional load going to each of the two adjacent anchors. As such, under the single failed anchor condition, the anchors adjacent to the failed anchor will have stress levels equal to 0.80 SMTS (i.e., 1.5×0.53 SMTS = 0.80 SMTS). It is considered acceptable for the failed anchor condition to allow the anchor-restraining forces to reach 0.80 SMTS. As such, internal stability analysis for the failed anchor condition may assume a 0.80 SMTS restraining force for the two anchors adjacent to the failed anchor. For other design conditions, the anchor restraining force should be no greater than 0.60 SMTS.

2.2.1.2 Tieback walls with multiple rows of anchors

For walls with multiple rows of anchors, it may be possible for the single failed anchor condition to assume a load distribution that is more favorable than that assumed for tieback walls with a single row of anchors. A tieback wall system with multiple rows of anchors is illustrated in Figure 1.2.

For an intermediate row (i.e., row other than a top or bottom row as shown in Figure 1.2a) it can be conservatively assumed that four adjacent anchors will pick up the failed anchor load, with 25 percent of the failed anchor load going to each of the four adjacent anchors. As such, under the single failed anchor condition the anchors adjacent to the failed anchor will have stress levels equal to 0.66 SMTS (i.e., 1.25 × 0.53 SMTS = 0.66 SMTS).

For a top or bottom row, it can be conservatively assumed that three adjacent anchors will pick up the failed anchor load, with 33 percent of the failed anchor load going to each of the three adjacent anchors. As such, under the single failed anchor condition, the anchors adjacent to the failed anchor will have stress levels equal to 0.70 SMTS (i.e., 1.33 × 0.53 SMTS = 0.70 SMTS).

2.2.2 Ground anchor lock-off load (after FHWA-SA-99-015)

After load testing is complete and the anchor has been accepted, the load in the anchor will be reduced to a specified load termed the “lock-off” load. When the lock-off load is reached, the load is transferred from the jack to the anchorage. The anchor in turn transmits this load to the tieback wall.

The designer selects the lock-off load and, as indicated in FHWA-IF-99-015, it generally ranges between 75 and 100 percent of the anchor design load for conditions where the anchor design load is based on an apparent earth pressure envelope. Lock-off loads of 75 percent of the design load may be used for temporary support of excavation systems where relatively large lateral movements are permitted. Since apparent earth-pressure diagrams result in total loads greater than actual soil loads, lock-off loads at 100 percent of the design load may result in some net inward movement of the wall. However, when critical structures (i.e., structures very sensitive to settlement) are founded adjacent to the excavation, a large lock-off load corresponding to approximately 100 percent of the design load is often required to limit soil movement. In certain cases lock-off loads greater than 100 percent of the design load may be required to stabilize a landslide. Under these circumstances, structural elements must be sized to transmit potentially large landslide forces to the ground.

When transferring the lock-off load to the anchorage, the load will inevitably be reduced owing to mechanical losses associated with the physical transfer of load between two mechanical systems (i.e., the jack and the anchorage). These losses are referred to as seating losses and are generally on the order of 1/16 in. (1.6 mm) for bar tendons and 1/4 in. (6.4 mm) for bare strand tendons (FHWA-DP-68-1R). For strand tendons, seating losses occur as the jack ram is retracted and the wedges are pulled in around the tendon. The wedges should be seated at a load no less than 50 percent of the ultimate load for the tendon. This will prevent possible strand slip through the wedges if the load in the tendon increases above the lock-off load during the service life. For epoxy-coated strands, the wedges must bite through the epoxy coating; this results in additional seating losses. To account for seating losses, after the tendon is loaded to the lock-off load, the jack ram is extended by an amount equivalent to the anticipated seating loss.

In the long term, the load will also reduce due to relaxation in the prestressing steel. Long-term load losses may be estimated as 4 percent for strand tendons and 2 percent for bar tendons (FHWA-DP-68-1R). Specific information on relaxation losses should be obtained from the tendon supplier. To account for these losses, the load that is transferred to the anchorage may be increased above the desired load based on results of a lift-off test. After the losses, the transferred load will reduce, presumably to the long-term load selected by the designer to meet project performance requirements.

2.2.3 Verifying lock-off loads through lift-off testing (after FHWA-SA-99-015)

After the load had been transferred to the anchorage, a lift-off test is performed. The purpose of a lift-off test is to verify the magnitude of the load in the tendon. For strand tendons, the lift-off test is performed by gradually applying load to the tendon until, for restressable anchor heads, the wedge plate lifts off the bearing plate (without unseating the wedges) or, for cases where the hydraulic head rests on the anchor head, the wedges are lifted out of the wedge plate. For bar tendons, the lift-off test is performed by gradually reapplying load to the tendon until the anchor nut lifts off the bearing plate (without turning the anchor nut). Lift-off is evident by a sudden decrease in the rate of load increase, as observed on the jack pressure gauge. The load measured during the lift-off test should be within 5 percent of the specified lock-off load. Where this criterion is not met, the tendon load should be gradually adjusted accordingly and the lift-off test repeated.

2.3 Acceptance Criteria (after FHWA-SA-99-015)

An anchor may be put into service at the lock-off load following testing if certain specified acceptability limits are satisfied. These criteria, which are described herein, prescribe acceptable limits of creep (i.e., movement during load holds) and elastic movement measured during anchor load tests. The creep and elastic movement criteria have been integrated into an acceptance decision tree that is described in this section. This decision tree describes procedures that are to be used in the event that a specific criterion is not satisfied.

2.3.1 Creep

Creep testing, either as part of a performance or proof test or as an extended creep test, is performed on each production anchor to evaluate creep movement of the anchor grout body through the ground. For an anchor to be accepted, total movements measured during load holds must be below a specified limit.

For performance and proof tests, the measured total movement for the required load holds at the test load should not exceed 1 mm for hold durations between 1 and 10 min. If the movements are less than 1 mm for this period, the anchor is considered to be acceptable with respect to creep. For load tests in which the measured total movement exceeds the criteria described above, the load is held for an additional 50-min time period. If the measured movement over this additional time period does not exceed 2 mm between 6 and 60 min, the anchor is considered acceptable with respect to creep.

For extended creep testing, the total movement for any load hold should not exceed 2 mm per logarithmic cycle of time (Post-Tensioning Institute, PTI 1996) over the final log cycle of time of each load increment. Sometimes the designer may consider it acceptable to incorporate into the tieback wall system an anchor that failed creep performance requirements. In such cases the design capacity for that anchor should be reduced to 50 percent of the load where acceptable creep movements were measured over the final log cycle of time. The total design force required to stabilize the cut must not be compromised, however. This means that additional anchors, or anchors with higher load capacity, will be required to make up for the capacity lost by the anchor that failed creep test requirements. Creep testing and extended creep testing is covered in Chapter 3.

2.3.2 Apparent free length

The apparent free length of a tendon forms the basis for evaluating the acceptability of a ground anchor with respect to elastic movement. The apparent free length is defined as the length of the tendon that is, based on measured elastic movements at the test load, not bonded to the surrounding ground or grout. The apparent free length, L_a, may be calculated using the following equation:

$$L_a = \frac{A_t E_s \delta_e}{P}\left(\frac{1}{10^9}\right) \qquad \text{(meters)}$$

where A_t = cross section area of the prestressing steel (m)

E_s = Young’s modulus of the prestressing steel (kPa)

δ_e = elastic movement at the test load (mm)

P = test load minus the alignment load (kN)

For proof tests where the residual movement is not measured or estimated, the apparent free length may be calculated using the total movement in place of the elastic movement. For long multi-strand tendons, it is likely that the elastic modulus of the multi-strand tendon will be less than the manufacturer's elastic modulus for a single strand. Because of this, PTI (1996) recommends that a reduction in the manufacturer's reported elastic modulus of 3 to 5 percent be allowed for satisfying the free-length criterion.

2.3.2.1 Minimum apparent free-length criterion

If the apparent free length is greater than the specified minimum apparent free length, it is assumed that the unbonded length has been adequately developed. The minimum apparent free length is defined as the jack length plus 80 percent of the design unbonded length. An apparent free length less than the specified apparent free length may indicate that load is being transferred along the unbonded length and thus within the potential slip surface assumed for overall stability of the anchor system. Alternately, an apparent free length less than the specified minimum apparent free length may be caused by friction due to improper alignment of the stressing equipment or tendon within the anchorage. Where test results do not satisfy the apparent free-length criterion, the anchor may be subjected to two cycles of loading from the alignment load to the test load in an attempt to reduce friction along the unbonded length. The apparent free length is then recalculated based on the elastic movement at the test load for the reloaded anchor. A value greater than the jack length plus 80 percent of the design unbonded length may be used to define the specified minimum apparent free length for cases in which the redistribution of friction along the unbonded length could cause unacceptable structural movement or where there is potential for prestressing loads to be transferred in the unbonded length by tendon friction.

2.3.2.2 Maximum apparent free-length criterion

The acceptance criterion based on maximum apparent free length was used in the past when load transfer along the bond length was assumed to propagate at a uniform rate as the applied load was increased. For that assumption, the maximum value of apparent free length was restricted to elastic movements of 100 percent of the free length plus 50 percent of the bond length plus the jack length. However, the concept of uniform distribution of bond is not valid for soil anchors and only approximates the behavior of most rock anchors. The primary use of this criterion is as an alternate acceptance criterion for proof tests in sound rock where creep tests are waived. Anchors that do not pass this preliminary criterion are subsequently creep tested to determine acceptability before a decision is made to reject the anchor.

2.4 Ground Anchor Acceptance Decision Tree (after FHWA-SA-99-015)

PTI (1996) developed a ground anchor acceptance decision tree that is shown as Figure 2.1.

The decision tree does not include the maximum apparent free-length criterion, as this criterion is not routinely used. The purpose of the decision tree is to provide recommendations as to the field procedures that should be followed in the event that an anchor does not satisfy specified acceptance criteria. Anchors that do not satisfy requirements for lock off at the design lock-off load may be locked off at a reduced load or replaced. The total design force required to stabilize the cut must not be compromised, however. This means that additional anchors, or anchors with higher load capacity, will be required to make up for the capacity lost by those anchors that are incorporated into the project at reduced load levels. These decisions are the responsibility of the design engineer.

Whether an anchor satisfies the minimum apparent free-length criterion is the first decision to be made using the decision tree. The ground anchor acceptance decision tree indicates that, for an anchor to be put into service at the design lock-off load, the elastic movement (i.e., minimum apparent free length) criterion must be satisfied. The following section provides information as to the recommended procedures to be used for an anchor that has passed the minimum apparent free-length criterion and for an anchor that has failed the minimum apparent free-length criterion.

2.4.1 Anchors that pass apparent free-length criterion

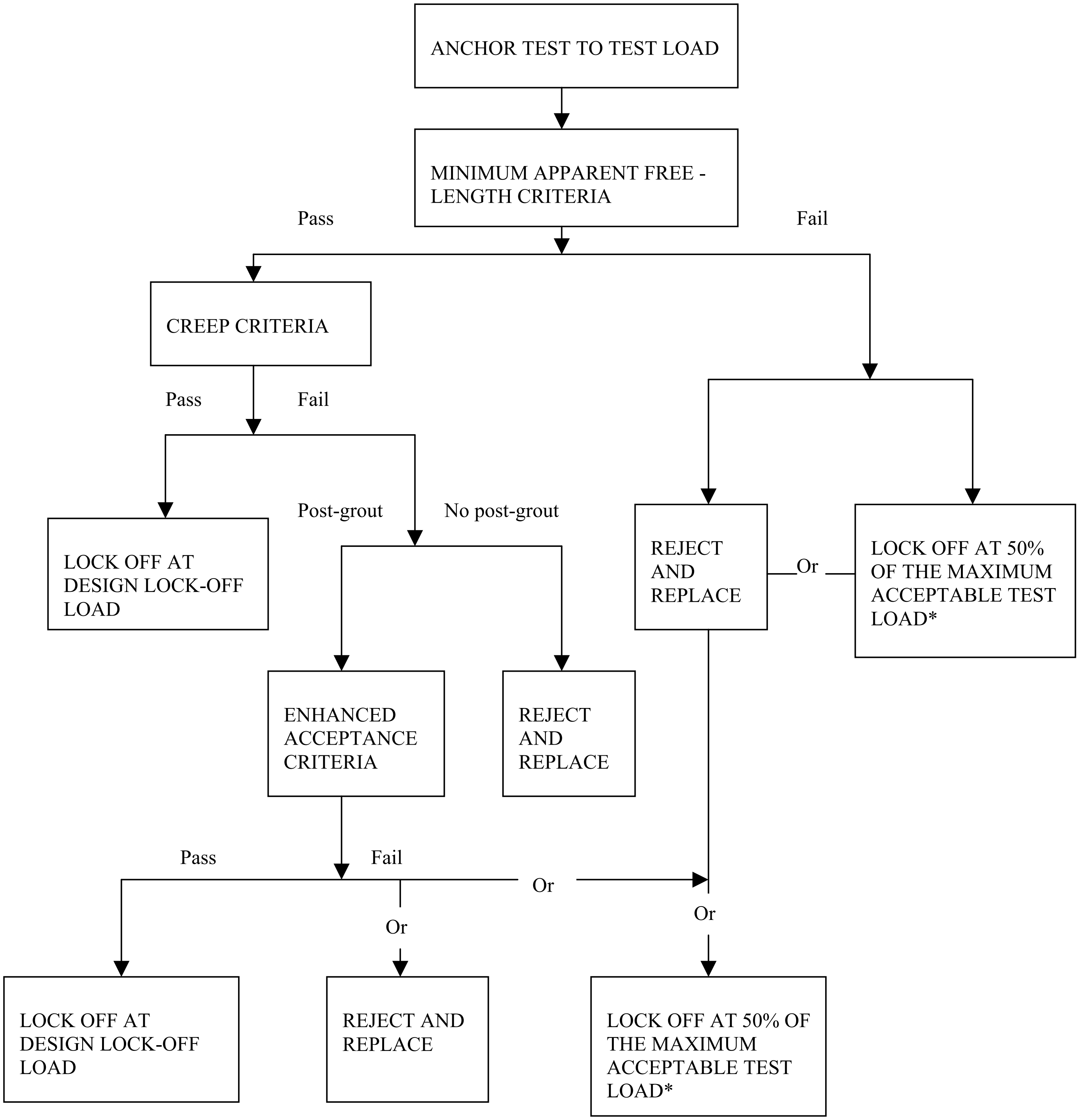
For anchors that pass the minimum apparent free-length criterion but do not pass the requirements of the creep test, the anchor may, if possible, be post-grouted. Those anchors that can be post-grouted will be retested and subject to an enhanced creep test and a more stringent acceptance criterion as compared to creep and extended creep tests. For this enhanced creep test, movements are monitored during a load hold at the test load for 60 min. The anchor may be locked off at the design test load if the total movement does not exceed 1 mm between 1 and 60 min. If the anchor does not satisfy this criterion, it can be either rejected and replaced or locked off at 50 percent of the load that the anchor holds without any detectable movement. If the anchor cannot be post-grouted, it may either be rejected and replaced or locked off at 50 percent of the load that the anchor holds without any detectable movement.

2.4.2 Anchors that fail minimum apparent free-length criterion

Anchors that fail the minimum apparent free-length criterion may be either locked off at a load no greater than 50 percent of the maximum load attained during testing, or rejected and replaced. As stated previously, the total design force required to stabilize the cut must not be compromised. Additional anchors, or anchors with higher load capacity, will be required to make up for the capacity lost by those anchors that are incorporated into the project at reduced load levels. Replacement anchors must satisfy all project specifications. Changes in ground anchor locations require approval from the design engineer. Where anchors are installed using prefabricated connections to steel beams or sheet piles, the failed anchor must be removed from the connection or a new connection must be fabricated. Connections may not be offset from the center of a soldier beam for a permanent anchor. Off-center connections will induce adverse bending and torsional stress on the soldier beam and bending stresses in the tendon. It is important that all field changes be made in consultation with, and with full approval of, the design engineer.

2.5 Modification of Design or Installation Procedures

Multiple failures early in construction or multiple failures of adjacent anchors should be cause to reassess subsurface conditions and/or design and installation procedures. Modifications to design and installation procedures commonly include (1) changing installation methods or anchor type, (2) increasing the anchor length or anchor bond length or changing the inclination of the anchor, or (3) reducing the anchor design load by increasing the number of anchors. A description of any proposed changes should be submitted to the owner in writing for review and approval prior to implementing the changes.



* Additional anchors, or anchors with higher load capacity, will be required to make up for the capacity lost by those anchors that are incorporated into the project at reduced load levels. These decisions are the responsibility of the design engineer.

Figure 2.1 Ground anchor acceptance decision tree (after FHWA-SA-99-015)

3 Anchor Testing

Tieback wall anchorages should be

- Tested to ensure that the margin of safety for all tieback anchors meets design requirements.

- Instrumented to permit the discovery of conditions that signal system distress.

3.1 General (after FHWA-RD-97-130)

Anchor testing is described in both FHWA-RD-97-130 and in FHWA-SA-99-015. The information presented in this section is taken from the FHWA-RD-97-130.

Each ground anchor is load tested to verify that it will develop the required load-carrying capacity in accordance with testing procedures described in the contract documents. Performance, proof, or creep tests are used. The "Specification for permanent ground anchors" (AASHTO-AGC-ARTBA 1990) describes each test. Typical testing setups are shown in FHWA-RD-82-047. Ground anchor failure criteria are based on a creep definition of failure. A creep failure occurs when the anchor movement exceeds a specified amount during a constant load hold period. Creep failure is different from a pullout failure. Creep failure occurs at a lower load than a pullout failure. The test load must be held constant to measure creep movements accurately. Pressure gauges are used to measure anchor loads for all three tests. Accurate pressure gauges are suitable for monitoring load during the load holds required for proof or performance tests. The following procedures will help ensure that the load tests are run well.

- Allow the grout to gain sufficient strength. (Grout strength tests are not always performed. If a prescriptive specification is used, the owner may want to specify grout strength testing to verify the contractor has mixed a quality grout.)

- Verify that the jack and pressure gauge have been calibrated in accordance with the specifications.

- Verify the jack pressures that correspond to the test loads.

- Fill out the ground anchor test sheet before starting a test. (AASHTO-AGC-ARTBA 1990) contains sample proof, performance, and creep test sheets.)

- Ensure an independent reference point is established to measure ground anchor movements.

- Ensure that the test equipment and dial gauge are aligned.

- Load test the anchors in accordance with the testing procedures described in the contract documents.

- Run performance tests on the first anchors installed on the project.

- Plot the anchor movements as the tests are performed. (Unusual behavior or errors in reading the dial gauge will be apparent if the data are plotted as the test is run.)

- Hold the ground anchor load constant during the load holds.

- Do not retest ground anchors. (PTI 1996 describes a procedure that can be used to allow post-grouted anchors to be retested if they fail the acceptance criteria. The approach taken in the PTI recommendations is sound, but it has not been verified extensively by experience.)

- Recognize that ground anchor failure will occur. (Failures are most likely to occur at the beginning of the job when the contractor is refining installation techniques. If frequent failures continue, the ground installation methods may have to be modified or changed.)

- Verify that an anchor passes the acceptance criteria when the test is completed.

- Stress the anchors (lock off) to the specified load. (The load will be between 75 and 100 percent of the design load.)

- Lift off the anchor and verify that the desired load has been locked off in the anchor before removing the test jack.

3.2 Details (after FHWA-SA-99-015)

The following detailed information on anchor testing is taken from FHWA-SA-99-015, except that testing is presented with respect to a test loading of 150 percent of the design load, instead of 133 percent.

For anchored system applications, each ground anchor is tested to loads that exceed the design load. Testing occurs after installation and prior to putting the anchor into service. This load testing methodology, combined with specific acceptance criteria, is used to verify that the ground anchor can carry the design load without excessive deformations and to verify the assumed load transfer mechanisms have been properly developed behind the assumed critical failure surface. After acceptance, the ground anchor is stressed to a specified load and the load is locked off.

3.2.1 Concepts for monitoring anchor bond zone capacity

The bond zone of an anchor develops resistance in the surrounding ground by straining in response to tensile loads applied to the anchorage. For anchor bond lengths in tension, the strains in the tendon are greatest at the top and decrease along the length of the anchor bond zone. The amount of load transfer to the ground at any particular strain will depend on the stress-strain characteristics of the ground. Figure 3.1 illustrates two possible skin friction versus strain diagrams for a ground anchor.

Curve A represents a soil or rock where very little strain is required to mobilize most of the skin friction. Curve B represents a weaker soil or rock where more strain is required to mobilize a peak skin friction and where continued straining results in a reduction of skin friction to a residual value. Early concepts for anchor testing were based on uniform propagation of load transfer down the bond length as tensile loads were increased. Figure 3.2 shows how the centroid of load, referred to as the "fictitious anchor point" (FAP), in the grout body was assumed to migrate toward the end of the tendon.

The assumption that all load transfer was mobilized when the FAP approached the midpoint of the bond length formed the basis for early acceptance testing. However, this concept of uniform load transfer is not valid for soil anchors and only approximates the behavior of most rock anchors. The current approach to monitoring bond zone capacity in soils has been used since the 1970s and is based on creep of the grouted body under a constant load. As shown in Figure 3.3a, the rate of creep of the bond zone is directly related to the applied load. Creep tests on numerous anchors have shown that when the creep rate exceeds 0.08 in. (2 mm) per log cycle of time, additional loads applied to the tendon will result in unacceptable continuing grout body movements. As shown in Figure 3.3b, a maximum load, T_c, defined as the critical creep tension, does exist for each bond zone. This critical creep tension corresponds to the load at which the creep rate exhibits a sharp upward break. Monitoring small creep movements (typically less than 1 mm) under constant applied tension loads requires appropriate testing equipment. Both the absolute value of the applied load and, more importantly, the ability to maintain a constant load for a substantial period of time must be addressed.

3.2.2 Anchor load testing

A unique aspect of ground anchors, as compared to other structural systems, is that every ground anchor that is to be part of the completed structure is load tested to verify its load capacity and load-deformation behavior before being put into service. The acceptance or rejection of ground anchors is determined based on the results of (1) performance tests, (2) proof tests, and (3) extended creep tests. In addition, shorter duration creep tests (as opposed to extended creep tests) are performed as a part of performance and proof tests. Proof tests are the most common and are performed on the majority of the ground anchors for a particular project. The number of performance and extended creep tests that are performed on a project depends upon whether the anchors are for a temporary support of excavation or permanent application. Testing also depends on the type of ground.

Every ground anchor is tested using one of the particular tests introduced above. The results of these tests are compared to specified acceptance criteria to evaluate whether the ground anchor can be put into service. The acceptance criteria are based on the allowable creep and elastic movements during load testing. A brief discussion of each type test follows. Testing requirements are specified in Chapter 4 of the *Post-Tensioning Manual* (PTI 1990).

3.2.2.1 Performance tests. Performance tests involve incremental loading and unloading of a production anchor. The performance test is used to verify anchor capacity, establish load-deformation behavior, identify causes of anchor movement, and verify the actual unbonded length is equal to or greater than that assumed in the anchor design. The results of a performance test may also be used to assist in the interpretation of the simpler proof test.

Performance tests are commonly performed on the first two or three production anchors installed and thereafter on a minimum of 2 percent of the remaining production anchors. Additional performance testing may be required where creep-susceptible soils are suspected to be present or where varying ground conditions are encountered. Where ground conditions are variable, performance test anchors should be located near geotechnical borings, if possible, to facilitate the interpretation of test measurements.

<u>3.2.2.1.1 Procedures for performance test</u>. The load schedule for a performance test is shown in the first three columns of Table 3.1.

The first step in a performance test comprises applying a nominal load to the anchor tendon. This load, termed the alignment load, typically varies between 2 and 10 percent of the design load (PTI 1990). The purpose of the alignment load is to ensure that the stressing and testing equipment is properly aligned. The displacement zeroing equipment is zeroed upon the stabilization of the alignment load, *AL*, as shown in Figure 3.4.

During the first cycle, the load is raised to 25 percent of the design load, and the incremental movement is recorded (i.e., Point 1 in Figure 3.4). The load is then reduced back to the alignment load. This procedure is repeated, using load increments as shown in Table 3.1, until the maximum testing load, referred to as the test load, is achieved. The test load normally vary from 120 to 150 percent of the design load, with 133 percent being commonly used for permanent applications and 120 percent being commonly used for temporary applications. [Note: 150 percent is suggested for critical Corps walls designed for “loss of anchor” conditions.] A test load of 150 percent may also be required for anchors in potentially creeping soils. In those cases where it is impossible to establish a fixed reference point for measurements, a test load equal to 150 percent of the design load is required (PTI 1990). This is increased to 200 percent for anchors in potentially creeping soils.

At the test load, a constant load is held for 10 min prior to reducing the load to the lock-off load. During this 10-min load hold period, movements are measured and recorded as 1, 2, 3, 4, 5, 6, and 10 min. The purpose of this load hold is to measure time-dependent (i.e., creep) movements of the anchor. This portion of the performance test is referred to as a creep test. If the total movement between 1 and 10 min exceeds the specified creep movement, the test load is maintained for an additional 50 min, and total movement is recorded at 20, 30, 40, 50, and 60 min. If the results of a creep test for a specific anchor indicate that creep movements are excessive relative to specified criteria, the anchor may be incorporated into the structure at a reduced load, the anchor may be replaced, or, in the case of post-groutable anchors, the anchor may be regrouted and then retested. Additional anchors, or anchors with higher load capacity, will be required to make up for the capacity lost by anchors that are incorporated into the project at reduced load levels.

3.2.2.1.2 Recording of performance test data. The magnitude of each load is determined from the jack pressure gauge. During creep testing, a load cell is monitored to ensure that the jack load remains constant. The load-deformation data obtained for each load increment in a performance test are plotted as shown in Figure 3.5.

Movement is recorded at each load increment and for the alignment load. The total movement (δ_t) that is measured consists of elastic movement and residual movement. Acceptance criteria for anchors require that the elastic movement of the anchors be known. Elastic movement (δ_e) results from elongation of the tendons and elastic movements of the ground anchor through the ground. Residual movement (δ_r) includes elongation of the anchor grout and movement of the entire anchor through the ground. The residual for a given increment of load is the movement that corresponds to the net "irrecoverable" movement that occurs upon application of a load increment and the subsequent relaxation of the load to the alignment load (see Figure 3.4 for definition of δ_{r6}). The elastic movement is therefore the difference between the total movement measured at the maximum load measured for a cycle and the movement measured at the alignment load (see Table 3.1). Although not used for anchor acceptance, residual movement is an indicator of the stress-strain behavior of the ground-grout bond in the anchor bond zone.

During the creep test portion of the performance test, the movement measured at specific times (i.e., 1, 2, 3, 4, 5, 6, and 10 min) is recorded. The time at which the total movement is measured for the test load (i.e., time at which Point 6 in Figure 3.4 is measured) represents the start time for the creep test. The movement from 1 to 10 min after this starting time is recorded and compared to the acceptance criteria with respect to creep. If the creep acceptability criterion is not satisfied, the test load is held on the anchor for an additional 50 min. The total amount of movement between 10 and 60 min is recorded and compared to specified criteria.

Creep acceptability criteria were established for anchors using bare prestressing strand. For epoxy-coated filled strand tendons, the creep movements of the strand itself are significant during load testing. The creep movements of the strand should be deducted from the total movement measured during a load test so that the creep movements within the ground can be accurately calculated.

3.2.2.1.3 Analysis of performance test data. One of the acceptability criteria for ground anchors is based on measured elastic movements of the ground anchor during load testing. The elastic movements calculated from a load increment during a performance test are evaluated using the equations shown in Table 3.1. These elastic movements should be calculated for each load cycle and plotted versus each load, as shown in Figure 3.5. The residual movement curve should also be plotted. For a soil anchor to be considered acceptable with respect to elastic movements, the elastic movement at the test load must exceed a specified minimum value. The acceptability criteria with respect to elastic movement are described in Chapter 2.

3.2.2.2 Proof tests

The proof test involves a single load cycle and a load test hold at the test load. The magnitude of the applied load is measured using the jack pressure gauge. Load cells are required only for creep tests in soils where the performance tests show a creep rate exceeding 1 mm per log cycle of time. The proof test provides a means for evaluating the acceptability of anchors that are not performance tested. Data from the proof test are used to assess the adequacy of the ground anchors considering the same factors as for performance test data. Where proof test data show significant deviations from previous performance test data, an additional performance test is recommended on the next adjacent anchor.

The proof test is performed in accordance with the procedure outlined in Table 3.2. The total movement from each load cycle in a proof test should be plotted as shown in Figure 3.6.

If an unloaded cycle is included (Step 4 in Table 3.2), residual movements and elastic movements should be calculated for the test load. This calculation is the same as that previously described for performance tests. If an unloaded cycle is not performed, an estimate of residual movement can be based on performance tests on other production anchors from the same project.

3.2.2.3 Extended creep testing

An extended creep test is a long-duration test (e.g., approximately 8 hr) that is used to evaluate creep deformations of anchors. These tests are required for anchors installed in cohesive soil having a plasticity index (PI) greater than 20 or liquid limit (LL) greater than 50. For these ground conditions, a minimum of two ground anchors should be subjected to extended creep testing. Where performance or proof tests require extended load holds, extended creep tests should be performed on several production anchors.

3.2.2.3.1 Procedures for extended creep test. The test arrangement for an extended creep test is similar to that used for performance or proof tests. The increments of load for an extended creep test are the same as those for a performance test. At each load cycle, the load is held for a specific period of time and the movement is recorded. During this observation period, the load should be held constant. The load is assumed to remain reasonably constant if the deviation from the test pressure does not exceed 50 psi (0.35 MPa). The loading schedule and observation periods for each load cycle in an extended creep test for a permanent anchor are provided in Table 3.3.

Information on the extended creep test for temporary anchors is provided in FHWA-RD-82-047.

3.2.2.3.2 Recording and analysis of extended creep test data. The test data for an extended creep test should be plotted as shown in Figure 3.7.

The creep movement at any time is the difference between the total movement and the movement measured at 1 min. Creep curves for a typical extended creep test are as shown in Figure 3.7. Each curve is for a separate load hold. The creep rate is defined as the slope of the curve per log cycle of time.

Extended creep test data are used in evaluating the acceptability of an anchor with respect to the creep acceptance criteria. Creep rates should be evaluated for each of the curves shown in Figure 3.7. These creep rates are compared to the maximum specified rate.

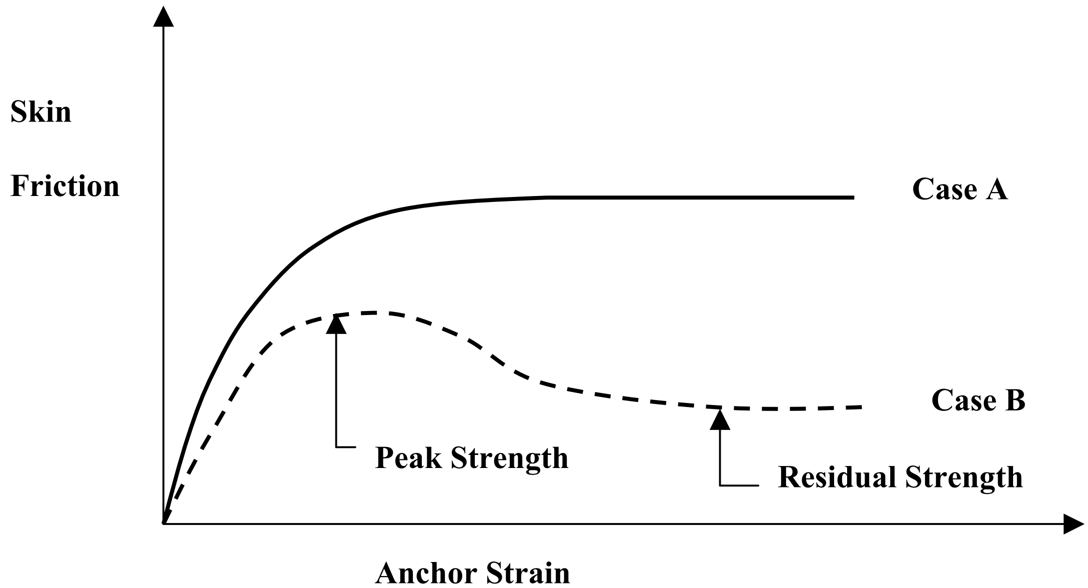


Figure 3.1 Skin friction versus strain diagrams for ground anchors (after FHWA-SA-99-015)

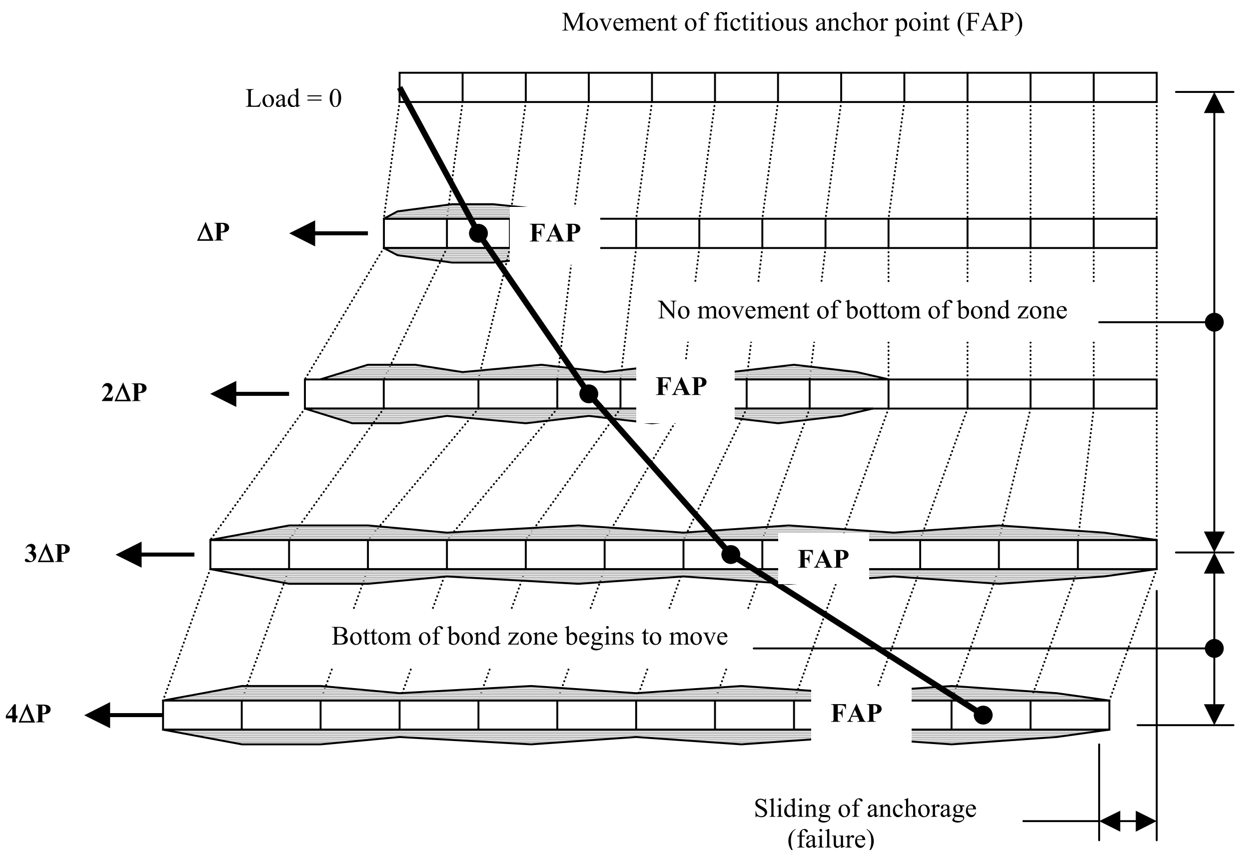
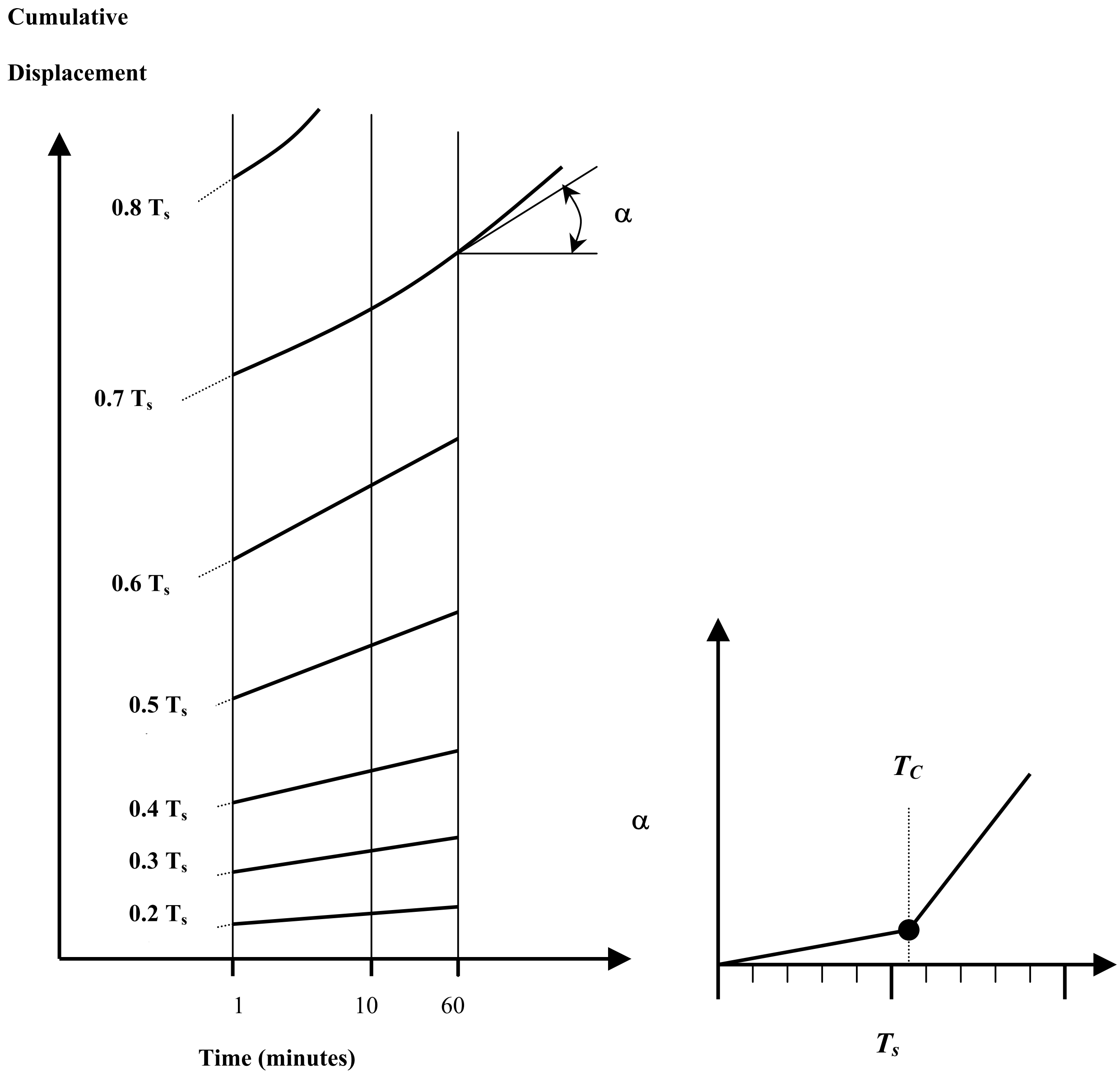


Figure 3.2 Stress propagation in bond length of ground anchor (after FHWA-SA-99-015)



Note: T_s = Ultimate strength of tendon

a. Creep curves

b. Critical creep tension

Figure 3.3 Evaluation of critical creep tension (after FHWA-SA-99-015)

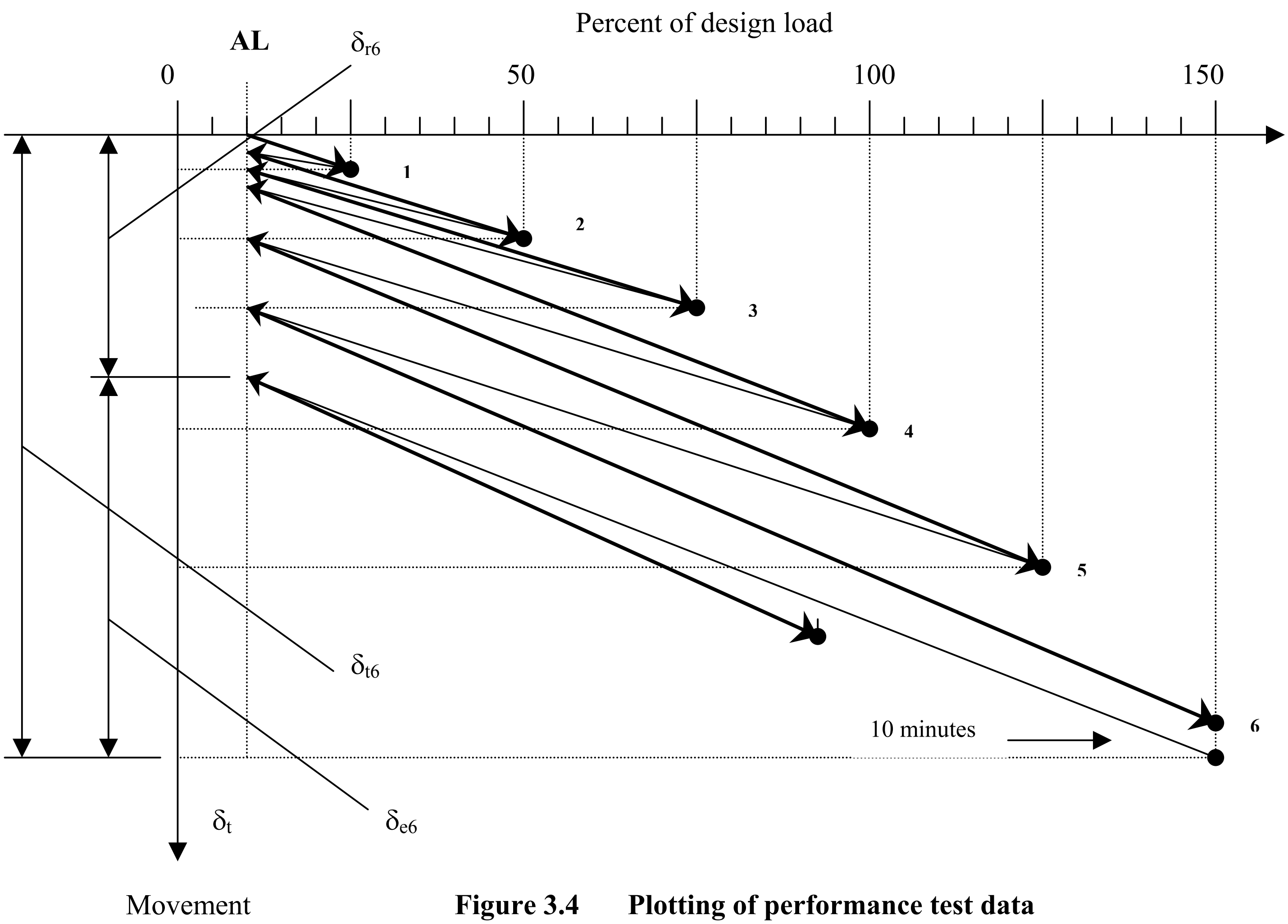


Figure 3.4 Plotting of performance test data (after FHWA-SA-99-015)

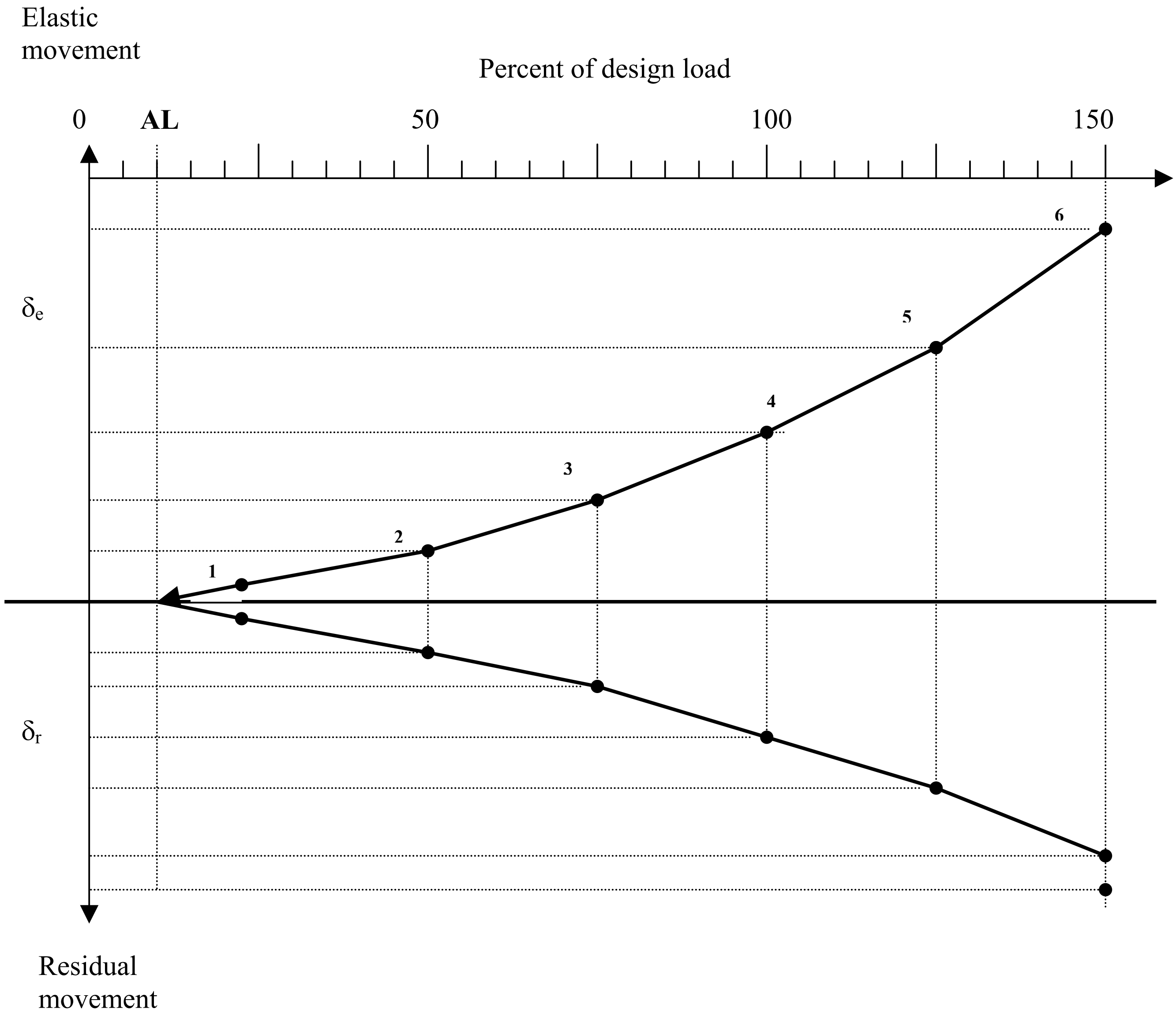


Figure 3.5 Plotting of elastic and residual movement for a performance test (after FHWA-SA-99-015)

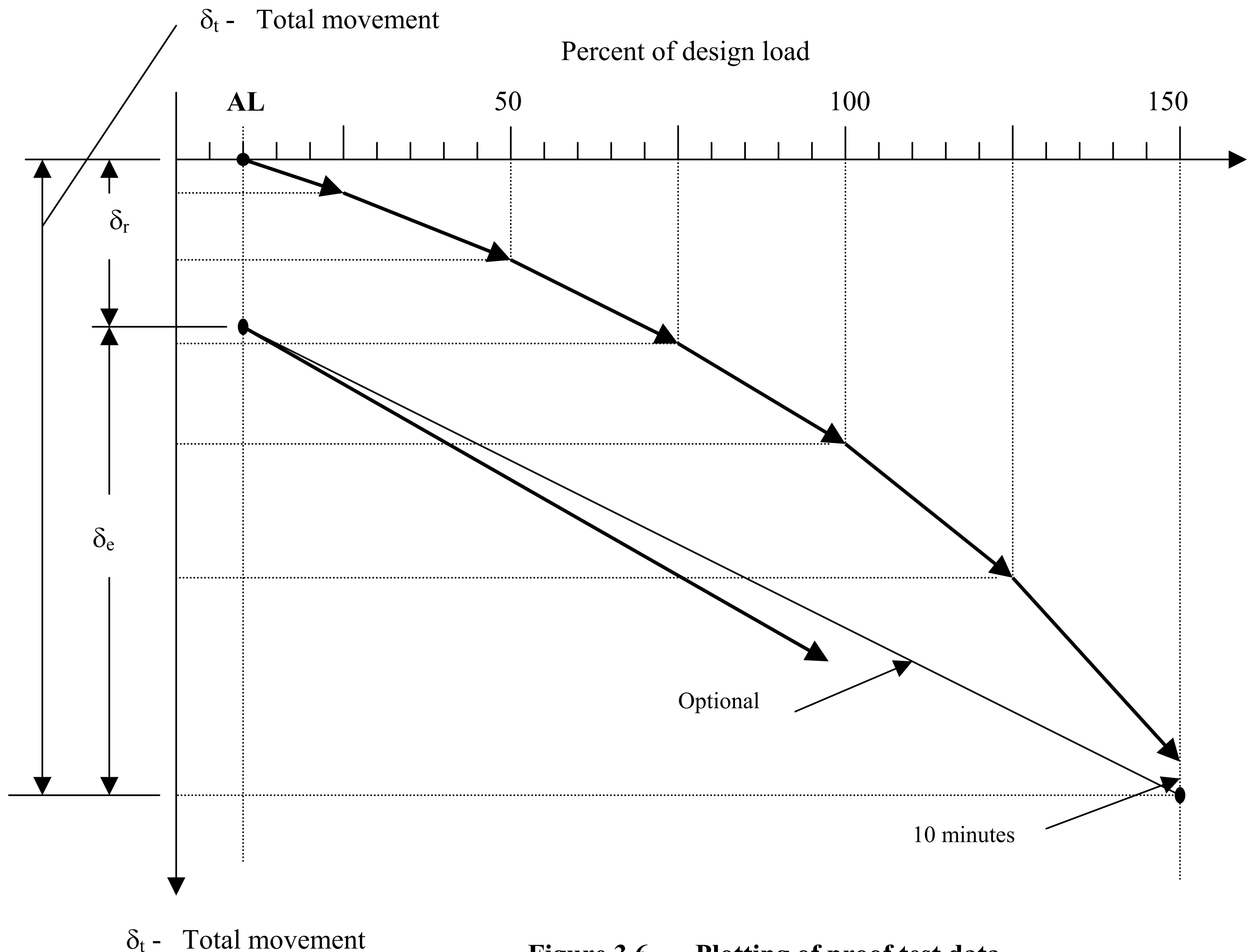


Figure 3.6 Plotting of proof test data (after FHWA-SA-99-015)

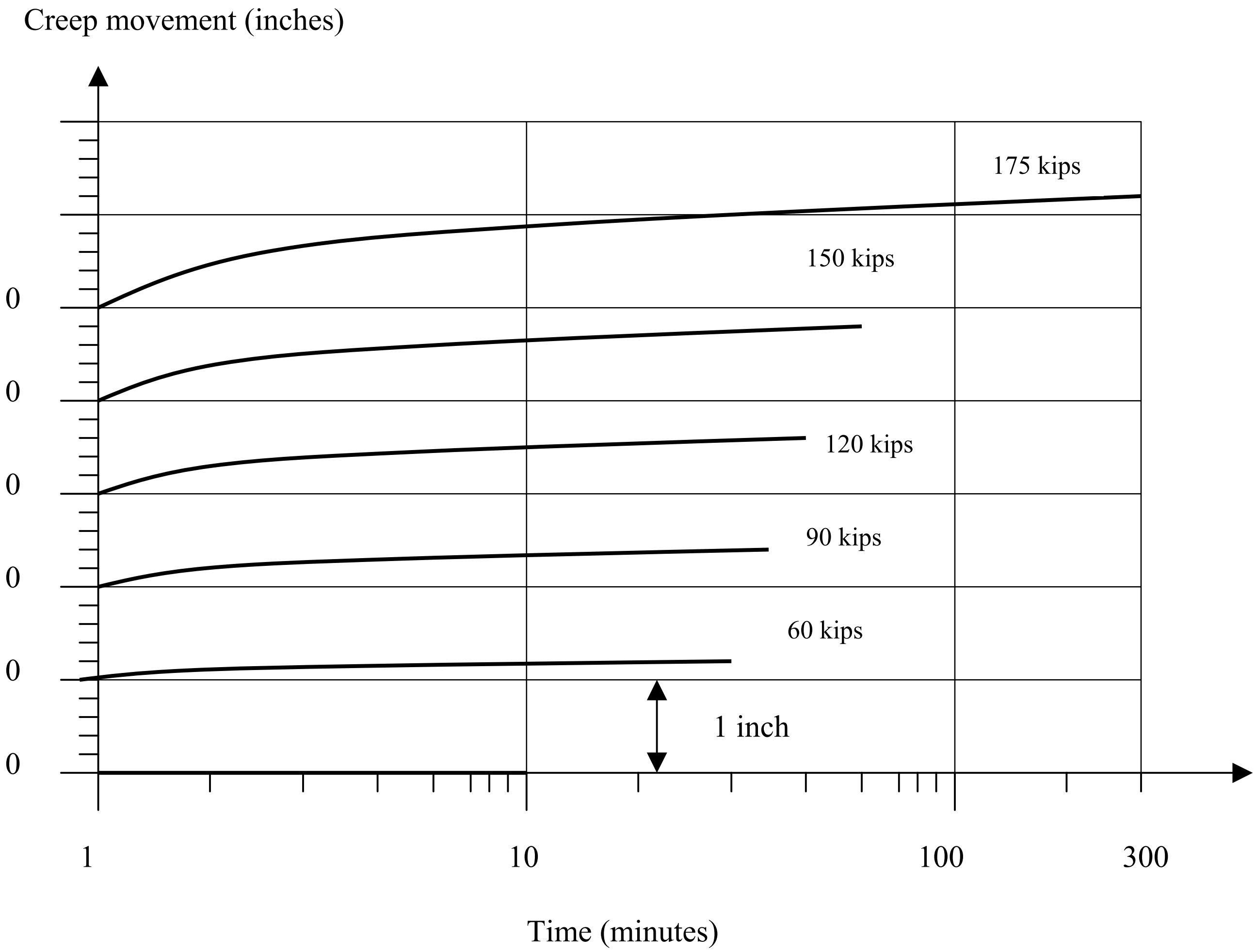


Figure 3.7 Plotting of extended creep test data (after FHWA-SA-99-015)

Table 3.1 Steps for the Performance Test (after FHWA-SA-99-015)

Step	Loading	Applied Load	Record and Plot Total Movement (δ_{ti})	Record and Plot Residual Movement (δ_{ri})	Calculate Elastic Movement (δ_{ei})
1	Apply alignment load (AL)				
2	Cycle 1	0.25 DL	δ_{t1}		$\delta_{t1} - \delta_{r1} = \delta_{e1}$
		AL		δ_{r1}	
3	Cycle 2	0.25 DL	δ_2		
		0.50 DL	δ_{t2}		
		AL		δ_{r2}	$\delta_{t2} - \delta_{r2} = \delta_{e2}$
4	Cycle 3	0.25 DL	δ_3		
		0.50 DL	δ_3		
		0.75 DL	δ_{t3}		
		AL		δ_{r3}	$\delta_{t3} - \delta_{r3} = \delta_{e3}$
5	Cycle 4	0.25 DL	δ_4		
		0.50 DL	δ_4		
		0.75 DL	δ_4		
		1.00 DL	δ_{t4}		
		AL		δ_{r4}	$\delta_{t4} - \delta_{r4} = \delta_{e4}$
65	Cycle 5	0.25 DL	δ_5		
		0.50 DL	δ_5		
		0.75 DL	δ_5		
		1.00 DL	δ_5		
		1.25 DL	δ_{t5}		
		AL		δ_{r5}	$\delta_{t5} - \delta_{r5} = \delta_{e5}$
7	Cycle 6	0.25 DL	δ_6		
		0.50 DL	δ_6		
		0.75 DL	δ_6		
		1.00 DL	δ_6		
		1.25 DL	δ_6		
		1.50 DL	δ_{t6} zero reading for creep test		
8	Hold load for 10 minutes while recording movement at specified times. If the total movement measured during the hold exceeds the specified maximum value then the load hold should be extended to a total of 60 minutes.				
9	Cycle 6 cont.	AL		δ_{r6}	$\delta_{t6} - \delta_{r6} = \delta_{e6}$
10	Adjust to lock-off load if test results satisfy acceptance criteria.				

Note: AL = Alignment Load, DL = Design Load, δ_i = total movement at a load other than maximum for cycle, i = number identifying a specific load cycle.

Table 3.2 Test Procedure for Ground Anchor Proof Test (After FHWA-SA-99-015)

Step 1	Apply the alignment load at which movement is assumed equal to zero.
Step 2	Successively apply and record total movements for the following load increments to the test load: 0.25 DL, 0.50 DL, 0.725 DL, 1.00 DL, 1.25 DL, 1.50 DL (i.e., the test load). Note that the test load for an anchor for a temporary support of application may be set at 1.20 DL.
Step 3	Hold test load for ten minutes and record residual movement.
Step 4	(Optional) Unload to alignment load and record residual movement.
Step 5	If the test results satisfy acceptance criteria, reduce load to lock-off load (or if Step 4 was used, increase load to lock-off load), otherwise follow guidance.

Table 3.3 Load Schedule and Observation Periods for Extended Creep Test for Permanent Anchor (After FHWA-SA-99-015)

Loading Cycle	Maximum Cycle Load	Total Observation Period (minutes)	Movements Measured at Following Times (minutes)
1	0.25 DL	10	1, 2, 3, 4, 5, 6, 10
2	0.50 DL	30	1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30
3	0.75 DL	30	1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30
4	1.00 DL	45	1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 45
5	1.25 DL	60	1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 45, 60
6	1.50 DL	300	1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 45, 60, 300

4 Preventing Progressive Anchor Failure

4.1 Introduction

To prevent progressive failure of a tieback wall system, the support provided by the anchors must be adequate to stabilize the "active" soil-pressure block (i.e., driving side soil wedge) retained by the wall. The practice of the Corps is to evaluate the progressive failure potential assuming that a single anchor has failed. Progressive failure analyses must consider

- The stability of the "active" soil-pressure block considering the reduction in anchorage system restraint due to the loss of a single anchor.

- The added loads (i.e., earth pressures, hydrostatic pressures, etc.) on the remaining soil anchors due to the loss of a single anchor.

- The added loads on tieback wall system components due to the loss of a single anchor.

Stability of the active soil-pressure block is covered in Chapter 5. Evaluating the anchors and tieback wall system due to the added earth-pressure demands resulting from a single anchor failure is described below. In this evaluation, it is assumed that the total loading on the wall due to earth pressure, hydrostatic pressure, etc., is unaltered due to the loss of a single anchor. However, it is recognized that earth-pressure distributions can change as a result of soil arching that takes place in the soil retained by the tieback wall system. The redistribution of loads as a result of a single anchor failure is illustrated in Figure 4.1. This figure represents a soldier beam system with concrete lagging.

Potential failures in the anchors adjacent to a "failed anchor" must be evaluated. Tensile failure of the tendon, pullout failure of the grout/ground bond, and pullout failure of the tendon/grout bond should all be evaluated with respect to the "single anchor failure" condition.

The tieback wall system itself must also be evaluated for its capacity to deliver soil loads under a single failed anchor condition to the tieback anchors. The wall system evaluation will depend on the type of wall used. The purpose of the evaluation is to ensure the additional earth-pressure demands on the wall system resulting from a single anchor failure can be safely transmitted to adjacent tieback anchors. All potential modes of wall system failure must be considered. With respect to a soldier beam and lagging system, the concrete facing may fail in flexure or shear, the connection of the facing to the soldier beams may fail, or the soldier beams may fail in flexure.

Each of the above wall system failure mechanisms can be evaluated by yield line analysis, which is a form of limit state analysis. In the following yield line analysis, the tieback wall is subjected to increasing load until yield lines or plastic hinge regions develop and a collapse mechanism occurs. As used in the following soldier beam and lagging system example, the minimum earth pressures that (under a single failed anchor condition) cause a particular failure mechanism to develop are determined. The failure mechanism earth pressures are then compared to the earth pressures used for design. If the failure mechanism earth pressures are less than the design earth pressures, a potential for failure exists.

The yield line analysis process illustrated below is similar to that used in the design of soil nail walls and described in FHWA-SA-96-069R. The yield line analysis process is demonstrated for a soldier beam with a permanent concrete facing system. The example used follows the "Granular Soil Design Example" of FHWA-RD-97-130. The original design for this system was accomplished using apparent pressures per FHWA-RD-97-130. Computations for the original design are provided in Appendix A. A uniform pressure distribution is used to design the permanent cast-in-place (CIP) facing. This is in accordance with the procedures outlined in FHWA-RD-97-130. Wood lagging that spans horizontally between soldier beams is used during construction to temporarily support the excavation. The design of the wood lagging, although not germane to the following yield line analysis, is generally in accordance with empirical procedures described in FHWA-SA-99-015.

As indicated previously, upon the loss of a single anchor, the earth pressures in the retained soil will tend to redistribute toward anchor locations due to soil arching. This is illustrated using a beam on elastic foundation analysis by assuming the load is delivered from the anchors through the soldier beams and permanent CIP facing to the retained soil. The analysis was performed for this soldier beam example using CBEAMC (Dawkins 1994). The earth-pressure distribution and facing bending moments for the case where all anchors carry load is illustrated in Figure 4.2. As can been seen, the earth-pressure distribution is nearly uniform, and the facing bending moments are approximately equal to those determined assuming uniform earth-pressure distribution. The earth-pressure distribution and facing bending moments for the case where a single anchor has failed is illustrated in Figure 4.3. This illustrates that the earth pressures in the failed anchor span concentrate at adjacent anchor locations, and bending moments at the adjacent anchor locations increase.

In general it can be stated that, as the flexural stiffness of the tieback wall facing system decreases with respect to the soil subgrade reaction modulus, the pressure distribution behind the tieback wall will become highly nonuniform with earth pressures concentrating at soldier beam locations. The loss of an anchor will in effect double the facing span, which in turn will significantly reduce the flexural stiffness of the facing. The nonuniformity of the facing earth-pressure distribution should be recognized in a loss of anchor evaluation because the magnitude of the earth-pressure load that can be delivered by the facing to the soldier beams will be significantly larger than that determined based on a uniform pressure distribution. However the actual earth-pressure distribution used in the analysis, to be legitimate, should be based on the back analysis of case histories and full-scale laboratory testing, calibrated finite element modeling, experience, and judgment.

The soldier beams must then be evaluated for the additional load that will be delivered by the facing under a "loss of anchor" condition. This may result in flexural yielding in the soldier beams. The soldier beams, in a manner similar to the CIP facing, must be evaluated for the load they can deliver to the tieback anchors. Yielding in the soldier beams will result in earth-pressure concentrations at nearby tieback locations. FHWA-RD-97-130 indicates that, although the soldier beams can yield as earth pressures increase, they would not be subjected to progressive failure since the earth pressures will redistribute to the tieback anchor supports.

4.2 Wall Evaluation

The soldier beam system with permanent CIP facing is illustrated in Figure 4.4. Using yield line analysis, evaluations will be made of the CIP facing, the facing to soldier beam connection, and the soldier beams with respect to their capacity to deliver the extra earth-pressure loads resulting from single anchor failure. It is assumed the total earth pressure resisted by the wall will not change as a result of the loss of a single anchor. Therefore, the one-way shear that must be transmitted from the facing to the soldier beams must increase from 3.84 to 7.68 kips per foot (see Figure 4.5).

4.2.1 Flexural capacity of CIP facing

The nominal moment capacity required to meet the original design objectives is equal to 10.53 ft-kips per foot of wall facing, assuming the facing spans horizontally between soldier beams (see calculations in Appendix A). In the original design, the earth pressures were assumed to be uniform (see Figure 4.5). In the yield line evaluation (i.e., limit state evaluation) it is assumed that, should an anchor fail, the facing will have to span 16.0 ft rather than 8.0 ft. The large facing displacements associated with flexural yielding will cause the soil to arch in the horizontal direction and, as such, the earth pressures will concentrate near the soldier beam supports. Therefore a triangular earth-pressure distribution as indicated in Figure 4.5 is used for the yield line analysis. This distribution approximates that determined in the beam on elastic foundation analysis. It should be noted, however, that the soil-pressure distribution assumed for the case where all anchors carry load (Figure 4.2) and the case where a single anchor has failed (Figure 4.3) can be determined only by an appropriate nonlinear soil structure interaction analysis that has been calibrated and verified by experimental investigation. It should also be recognized that the retained earth will have a great capacity to arch and redistribute earth-pressure loads; as a result, failure of the CIP facing is unlikely. In the yield line analysis it is assumed that plastic hinges will form at the supports and center span, as indicated in Figure 4.5. It is also assumed, per yield line practice, that the shear at the positive moment hinge point (center span point where loss of anchorage occurs) is equal to zero. Using this information, the earth-pressure intensity (w_u) required to develop plastic hinging is equal to

$$w_u = \frac{12M_N}{(S_H)^2} = \frac{12(10.53)}{(8)^2} = 1975\,psf$$

Using this value, the shear (V_u) that can be delivered to the soldier beams on each side of the failed anchor is 1.975 (4) = 7.90 kips per foot. Assuming the total load on the wall system is unchanged, the shear that must be delivered to the soldier is equal to 0.961 (8) 7.68 kips (recall the 8.0-ft anchor spacing between soldier beams as illustrated in Figure 4.2). Since 7.90 kips > 7.68 kips, it can be assumed the CIP facing has adequate flexural capacity.

4.2.2 Shear capacity of CIP facing

The shear capacity (V_u) of the concrete facing, assuming the depth to the reinforcing steel is 8 in., is equal to

$$V_u = 2\sqrt{f_c'}bd = 2\sqrt{4000}(12)(8)\left(\frac{1}{1000}\right) = 12.14\,\text{kips per foot of wall}$$

Since 12.14 kips > 7.68 kips, it can be assumed the CIP facing has adequate shear capacity.

4.2.3 Punching shear capacity of soldier beam headed-stud to CIP facing connection

Shear cone analysis is used to determine the capacity of the studs in punching shear. This was done for the original design (see Appendix A). Punching shear controlled over tensile strength and resulted in 5/8-in.-diam studs spaced at 12 in. along the length of the soldier beam exterior flange face (see Figure 4.5). This arrangement must be investigated for the loss of a single anchor condition. As presented in Appendix A, the shear cone surface area (A_o) is equal to

$$A_O = \sqrt{2}l_e(\pi)(l_e + d_h) = 80.9\ \text{in}^2$$

where

l_e = stud length - head thickness = 4.0 - 0.3125 - 3.688 in.

d_h = stud head diameter = 1.25 in.

and the punching shear (T_{SP}) capacity is equal to

$$T_{SP} = 2.67\sqrt{f_c'}A_O = 13.66\ \text{kips}$$

The stud reaction (R_{stud}) (i.e., the tensile force the facing exerts per foot on the soldier beam), using a tributary area approach and assuming that the soldier beam with the failed anchor provides no support, is

$$R_{stud} = (0.075 + 0.886)\ (4 + 8) = 11.53 \text{ kips} < 13.66 \text{ kips} \qquad \text{OKAY}$$

4.2.4 Tensile strength capacity of headed-stud connections

The fracture capacity of the headed studs in tension (T_{ST}) must also be checked (see Appendix A). This is accomplished by simply multiplying the stud cross-sectional area times the tensile yield strength of the steel (i.e., 60,000 psi), or

$$T_{ST} = A_{stud}(F_y) = 0.307(60) = 18.41 \text{ kips}$$

Stud capacity is controlled by shear cone punching shear (see Section 4.2.3).

4.2.5 Flexural capacity of soldier beams

It will be assumed for this part of the yield line evaluation that the soldier beam at the failed anchor location will not carry any lateral earth pressure. Therefore, all the lateral earth pressure carried by that soldier beam must be transferred to the adjacent soldier beams. This assumption is conservative. The plastic moment capacity of the HP 12×53 Grade 50 soldier beams is equal to the plastic section modulus times the yield capacity in flexure, or

$$M_p = Z_P(F_Y) = 74.0(50{,}000) = 3.7x10^6 \text{ in.-lb}$$

On a per-foot of wall basis, assuming 8.0 ft of wall tributary to the soldier beam (i.e., 4.0 ft on each side of soldier beam) for the condition where all the anchors perform as intended,

$$M_{PO} = 3.7x10^6 / 8(12) = 3.85x10^4 \text{ ft-lb/foot of wall}$$

On a per-foot of wall basis for the single anchor failure condition, assuming 12.0 ft of wall tributary to the soldier beam (i.e., 4.0 ft on the side away from the failed anchor location and 8.0 ft on the side towards the failed anchor location),

$$M_{PF} = 3.7x10^6 / 12(12) = 2.57x10^4 \text{ ft-lb/foot of wall}$$

The M_{PF} value above for the single anchor failure condition will be used to determine the earth pressure the soldier beams can support, assuming that plastic hinging occurs at the base of the cantilever span. (Referring to Figure 4.6.)

Using the same earth-pressure distribution assumed for the original design and summing moments about the upper anchor point for the cantilever span, the maximum pressure (p_C) that the soldier beam can accommodate can be determined as shown below.

$$2.5 p_C (1/2) 2.5 + (1/2) 5.0 p_C (2.5 + 5/3) = M_{PF}$$

$$13.5417 p_C = M_{PF}$$

$$p_C = \frac{M_{PF}}{13.5417} = \frac{2.57x10^4}{13.5417} = 1{,}898 \text{ psf}$$

The maximum earth-pressure demand for the failed anchor condition is equal to (12/8) times 961.1 psf = 1,442 psf. Since 1,898 psf > 1,442 psf, the flexural capacity of the soldier beams on each side of the failed anchor is adequate to carry the additional earth pressures associated with the failed anchor condition. The additional capacity due to cantilever action of the failed anchor soldier beam, as shown in Figure 4.7, could also be considered in the yield line analysis.

The M_{PF} value above for the single anchor failure condition will also be used to determine the earth pressure the soldier beams can support, assuming that plastic hinging occurs in the continuous span between the upper and lower anchors. Again, referring to Figure 4.6,

$$5.5 p_i (1/2) 5.5 = 2M_{PF}$$

$$p_i = \frac{2M_{PF}}{15.125} = \frac{2\left(2.57x10^4\right)}{15.125} = 3398 \quad \text{psf} \; > 1{,}442 \text{ psf} \quad \text{OKAY}$$

The capacity of the soldier beams in shear will not be a problem, and therefore computations are not provided.

4.3 Anchor Evaluation

It has been demonstrated by yield line analysis that the tieback wall system has the capacity under the failed anchor condition to deliver the additional earth-pressure loads to adjacent anchors. It now must be demonstrated that the adjacent anchors have sufficient capacity to accommodate the load carried by the failed anchor. As stated above, anchor failures due to tensile failure of the tendon, pullout failure of the grout/ground bond, and pullout failure of the tendon/grout bond must be evaluated with respect to the single anchor failure condition.

As indicated in Chapter 2, for projects that must satisfy the Corps' loss of a single anchor criterion, it is suggested that the ground anchor test load be set equal to150 percent of the design load. The maximum design load would then be equal to 0.8/1.50, or 0.53 SMTS. The example presented in Appendix A has two rows of anchors, and it is assumed that one of the anchors in the top row fails. Therefore, per Section 2.2.1, it is conservatively assumed that the three anchors adjacent to the failed anchor will pick up the load no longer carried by the failed anchor (two anchors on each side of the failed anchor and the anchor immediately below the failed anchor). For this particular example, the three adjacent anchors, assuming they each pick up one third of the failed anchor load, would be stressed to 1.33 (0.53) SMTS, or 0.70 SMTS. For the failed anchor condition, it is considered acceptable to use anchor-restraining forces as high as 0.80 SMTS in two-dimensional (2-D) internal stability analysis. A larger anchor-restraining force is permitted for the failed anchor condition because

- The total restraint provided by the anchors to the 3-D ground mass failure wedge would not be significantly reduced, and
- Ground anchor stress levels at 0.80 SMTS are considered acceptable for extreme loading conditions.

4.3.1 Tensile capacity of anchors

Referring to the original design calculations in Appendix A, the two anchors on each side of the failed anchor and the anchor immediately below the failed anchor would be required to resist 87.5 (1.33) = 116.4 kips. The SMTS of a 1.25-in.-diam Grade 150 anchor is equal to 187.5 kips. The load in the three anchors adjacent to the failed anchor is equal to (116.4/187.5), or 0.62 SMTS, which is acceptable for the loss of single anchor condition.

4.3.2 Pullout capacity of grout/ground bond

The bond length, or contact length between the anchor grout and soil, is usually estimated by dividing the allowable anchor load (i.e. anchor load at 60 percent of ultimate) by the estimated ultimate transfer capacity for the particular soil, and then multiplying by a factor of safety of 2. Performance testing, proof testing, and extended creep testing as appropriate must be performed in the field to ensure the grouted anchor length will provide a tieback wall that meets all performance objectives, including those established for a loss of single anchor condition. Anchor grout/ground bond length must be increased, or other measures taken, when testing indicates that performance objectives will not be met. Performance objectives should be established for the loss of single anchor conditions, and testing should measure anchor performance at load levels representing the loss of single anchor condition. Testing is described in Chapter 3.

4.3.3 Pullout capacity of tendon/grout bond

The same provisions described above for grout/ground bond apply to tendon/grout bond.

4.4 Defensive Design Considerations

This section briefly describes defensive design measures that can be taken to reduce the risk of progressive tieback wall failure. Characteristics of the tieback wall system can be as important to performance as the loadings used for design. Important tieback wall system characteristics include continuous and redundant load paths (achieved by tying wall elements together to behave as a unit), as well as ductile members and connections. The ability for the completed tieback wall system to meet performance objectives will also depend on the quality of the design, materials, and construction.

4.4.1 Redundant load paths and cap beams

Under a loss of anchorage condition, it is important that there is a continuous alternate (i.e., redundant) load path to prevent progressive collapse and to provide overall ground mass stability for the tieback wall system. Soldier beam and sheet-pile systems with continuous walers, and tremie concrete/slurry wall systems with adequate connections at panel joints, will generally have the redundancy required to prevent a progressive failure. Other systems may require a cap beam to provide necessary system redundancy.

The cap beam system offers excellent protection against progressive failure for tieback wall systems that lack redundancy and for systems that must be supported at the top by a single row of anchors. The cap beam provides continuous support for the vertical tieback wall elements whether they are soldier beams or CIP reinforced concrete panels constructed by slurry trench methods. The cap beam also allows anchors to be spaced at desired intervals, regardless of spacings assigned to the vertical elements. The cap beam is generally designed as a continuous CIP reinforced concrete beam with the capacity to transfer load from the vertical elements to tieback anchors. A continuous cap beam system can easily be designed for the loss of a single anchor condition by providing additional design capacity to redistribute the load carried by a failed anchor to the anchors located on each side of the failed anchor. This is illustrated in Figure 4.8 for a soldier beam-type system.

The cap beam system is also excellent for those excavations adjacent to existing waterways where it is necessary to install the anchors in the dry while maintaining an existing pool as the riverward excavation and tieback wall construction takes place.

A cap beam system was employed in the construction of a tremie concrete/slurry trench guard wall for the Bonneville Navigation Lock (Maurseth and Sedey 1991). The guard wall was 3.5 ft thick and consisted of steel piles (soldier beams) as the main structural members, with the tremie concrete acting as lagging. The exposed height of the wall varied from 30 to 68 ft. Pile sizes ranged from W36 × 194 on 6-ft centers to W36 × 300 with variable thickness and cover plates (up to W36 × 848 equivalent) on 4-ft centers. The guard wall is a permanent structure featuring a reinforced CIP cap beam and permanent soil anchors. The cap beam was designed as a continuous beam located at the top of the guard wall and used to transfer loads from the soldier beams to the tieback anchors. Should a tieback fail, the resulting additional load could be safely carried by adjacent tiebacks.

A cap beam was also used for the Monongahela River Locks and Dams 2 left abutment tieback retaining wall system. This particular tieback wall was a secant pile system with

5-ft-diam reinforced concrete caissons (soldier piles) spaced at 7 ft on center with 4-ft-diam unreinforced concrete caissons placed between the reinforced caissons to serve as lagging. The tieback wall had two rows of post-tensioned anchors, located approximately 4 ft and 12 ft below the top of the cap beam. The reinforced CIP cap beam was approximately 6 ft wide by 2 ft deep. It was continuous except for discontinuities at expansion joints and where the tieback retaining wall intersected the existing dam and abutment. The cap beam was designed for a loss of single anchor condition, with the critical design condition occurring when an anchor in one of the rows located next to a discontinuity was assumed to have failed.[2]

Since continuity is such an important attribute, wherever possible, an attempt should be made in cap beam design to eliminate expansion and contraction joints. Instead of joints, additional reinforcement should be provided in the cap beam to accommodate forces that can develop due to temperature and shrinkage restrained volume change effects. In certain circumstances, as with the Monongahela River Locks and Dams 2 Project, it may not be possible to maintain continuity in such a manner that the lateral earth-pressure loads that develop during excavation can be delivered continuously by the cap beam (or other load path elements) to undisturbed ground at each end of the excavation. Discontinuities can result when a tieback wall abuts an existing structure, or where an abrupt change in direction of the tieback wall occurs. In such circumstances it may be advisable to provide redundancy to the anchorage system located on each side of the discontinuity. This can be accomplished by providing additional anchor capacity (i.e., increasing the factor of safety), providing redundant anchors, or providing details that will allow the insertion of replacement anchors at the first sign of distress.

4.4.2 Ductile members and connections

Both primary and redundant load paths should contain ductile members and connections. Connections, if possible, should be capable of developing the strength of the connecting members. Members themselves should be designed to yield initially in flexure so as to protect against a brittle shear failure. Also, the design should protect against compression failures, rebar splice failures, rebar anchorage failures, and other brittle failure mechanisms.

4.4.3 Quality of the design, materials, and construction

All tieback wall designs should be subject to an independent peer review of both the structural and geotechnical aspects of the design. In addition, inspections of the construction, anchor installation, and testing should be performed by engineers experienced in tieback wall design and construction to ensure that the completed system will meet all performance objectives.

[2] Personal Communication, 2001, Rich Allwes, Engineer, U.S. Army Engineer District, Pittsburgh, PA.

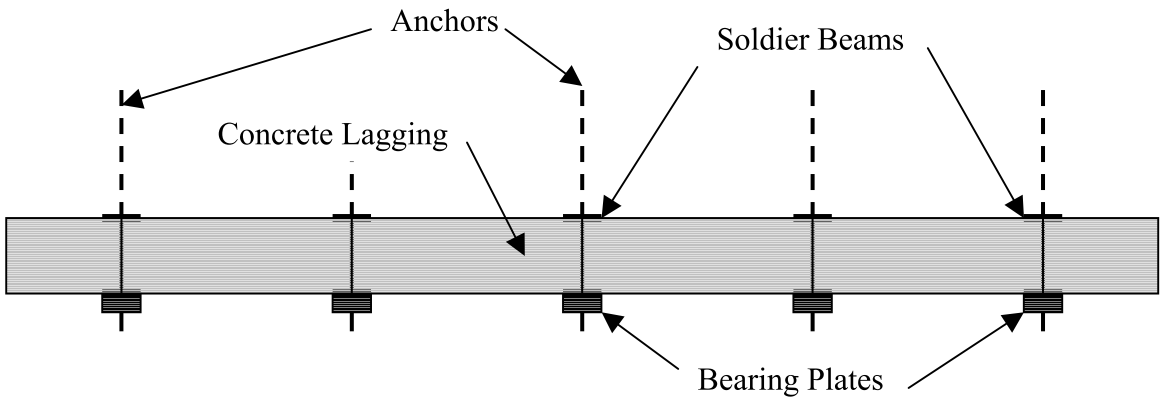
4.5 Research and Development Needs

The capacity of the tieback wall system to redistribute earth-pressure loads under a single failed anchor condition can be investigated using yield line analysis procedures, as described above. It should be noted, however, that FHWA-RD-97-130 does not consider progressive collapse to be a likely mode of failure since, as system components deform, the earth pressures redistribute and concentrate at tieback anchor supports. However, the yield line analysis procedure does allow the designer to check various potential modes of failure should a concern arise.

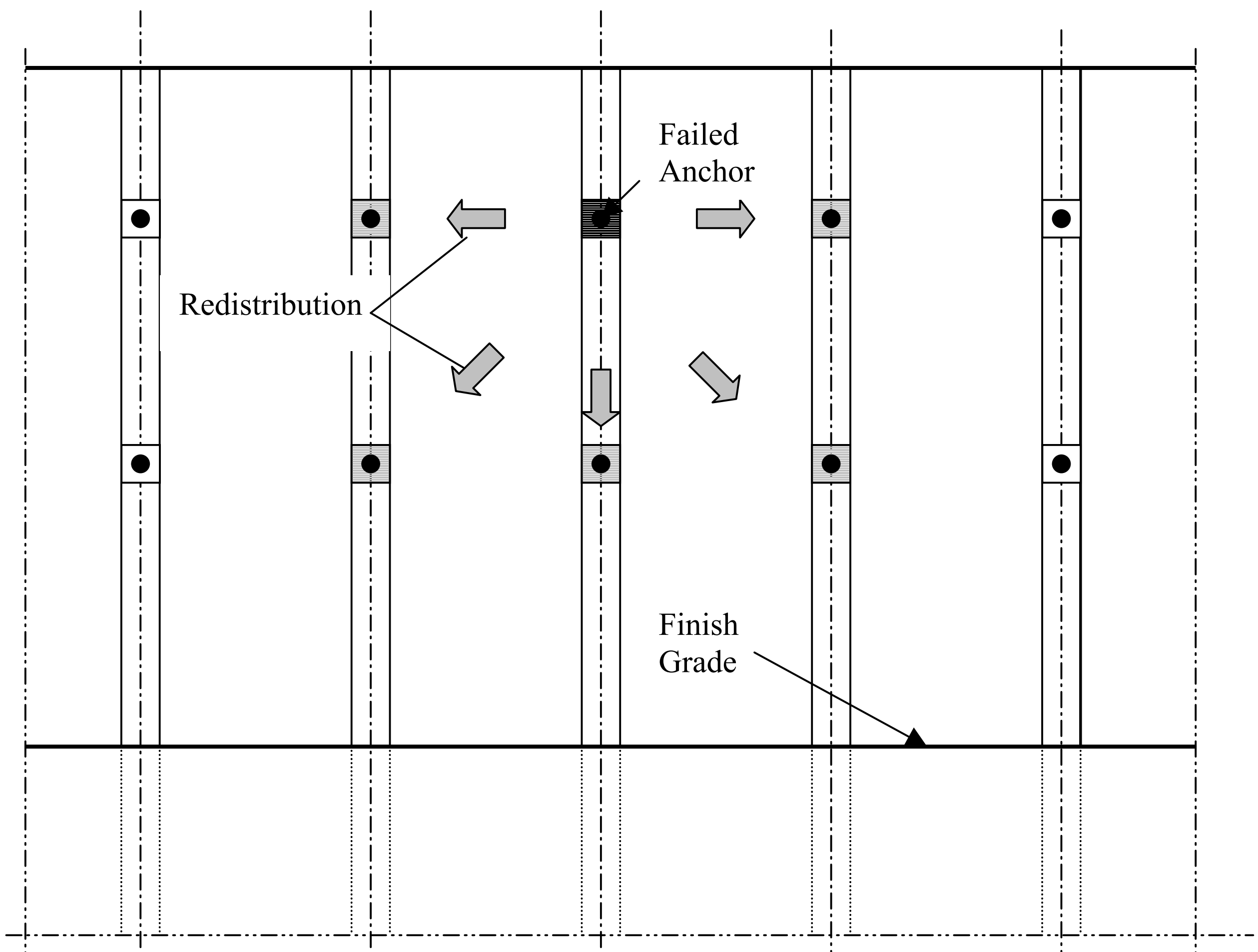
The yield line analysis methodologies described herein with respect to tieback wall systems assume that, under the failed anchor condition, the earth pressures in the vicinity of the failed anchor will redistribute and concentrate at adjacent tieback anchor supports. Lateral earth pressures will be essentially nonlinear and will be dependent on many factors, including soil type, movement, facing stiffness, tieback spacing, and tieback prestress levels. Although the earth pressures used in the preceding limit state analysis for the failed anchor condition assume a specified displacement response, it is impossible at this time to reasonably predict the displacement responses that will occur in the facing and tieback wall system. As such, it is also impossible to predict how earth-pressure distributions will change when an anchor fails.

Additional research using three-dimensional nonlinear soil-structure interaction finite element analyses is needed to validate the use of the limit state analysis process illustrated in this chapter, especially with respect to the type of earth-pressure distributions that should be assumed for various failed anchor conditions. Research should be directed toward producing appropriate limit state evaluation procedures for tieback wall systems, procedures that can be used to assess the failed anchor condition. The limit state procedures presented in FHWA-SA-96-069R for soil nail walls can serve as a basis for the limit state analysis of tieback wall systems. As with soil nail systems, the soil structure interaction model used for the limit state evaluation of tieback wall systems should be verified and calibrated with full-scale test results. The calibration process for soil nail wall systems is described in Seibel (1996).

An alternative approach would be to assume for the failed anchor condition a uniform pressure distribution behind the facing (i.e., uniform rather than the triangular distribution shown in Figure 4.5). Design based on limit state analysis for the failed anchor condition using a conservative uniform pressure distribution will likely produce a thick, heavily reinforced facing and result in a high cost that cannot be justified by rational analysis.



a. Sectional plan



b. Elevation—multiple rows of anchors; redistribution

Figure 4.1 Soldier beams with concrete lagging

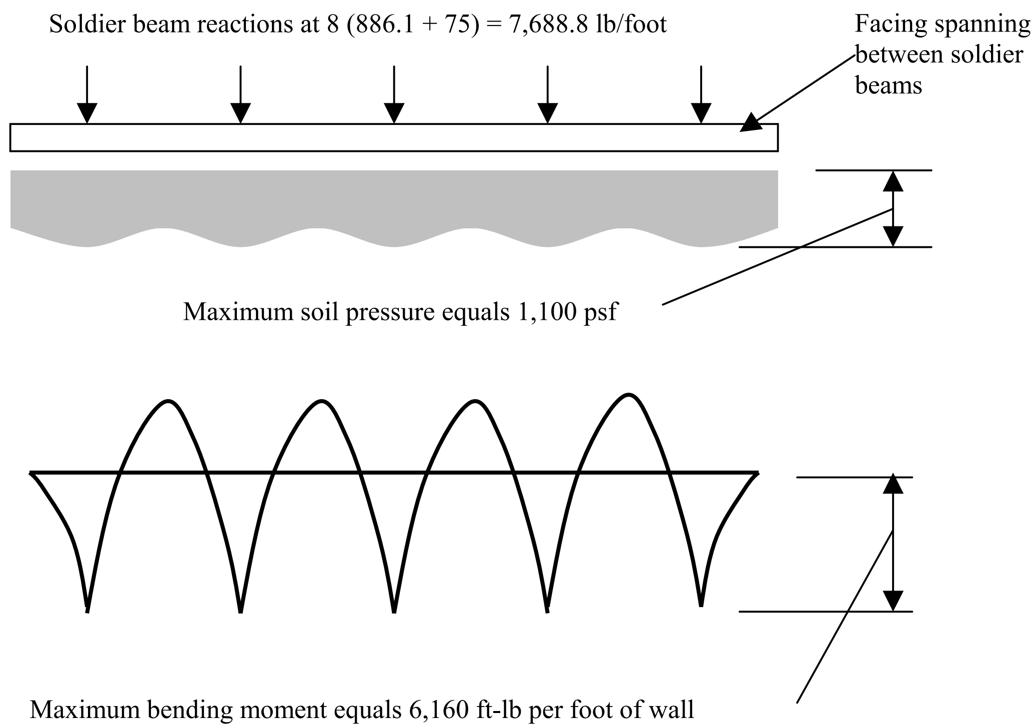


Figure 4.2 Tieback wall facing pressures and moments; no loss of anchorage condition

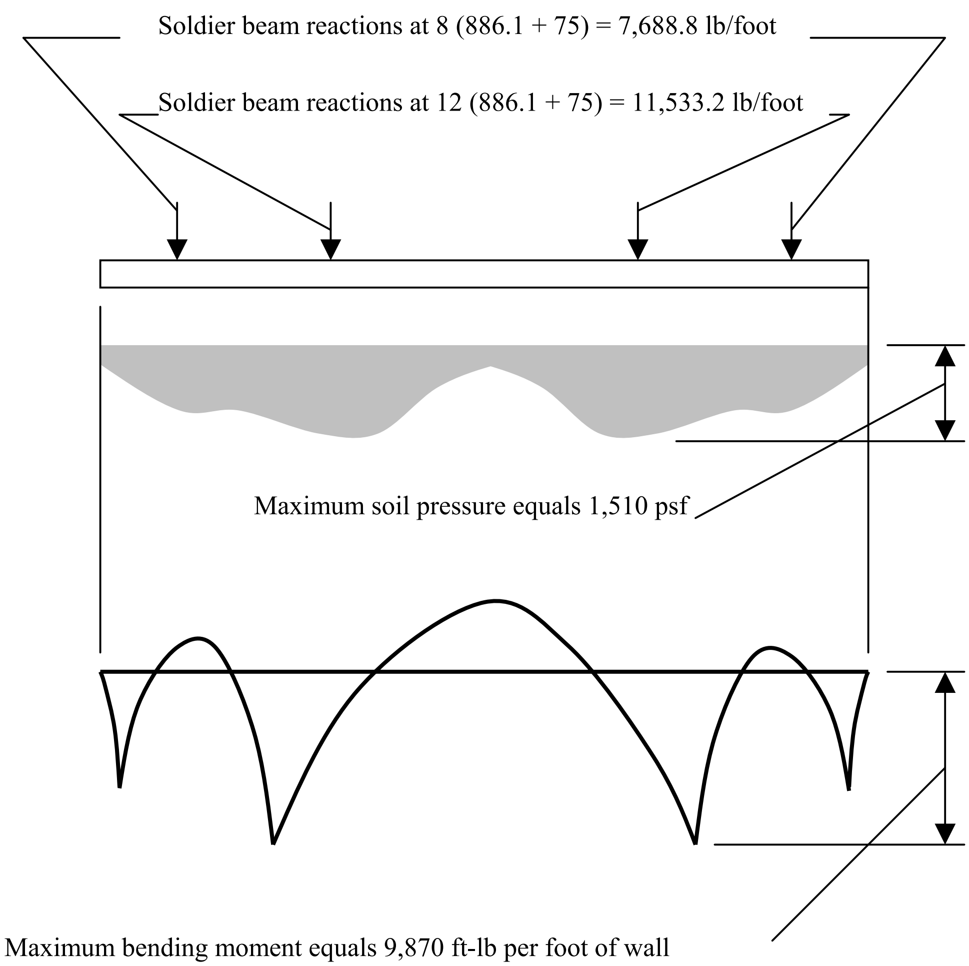


Figure 4.3 Tieback wall facing pressures and moments; loss of anchorage condition

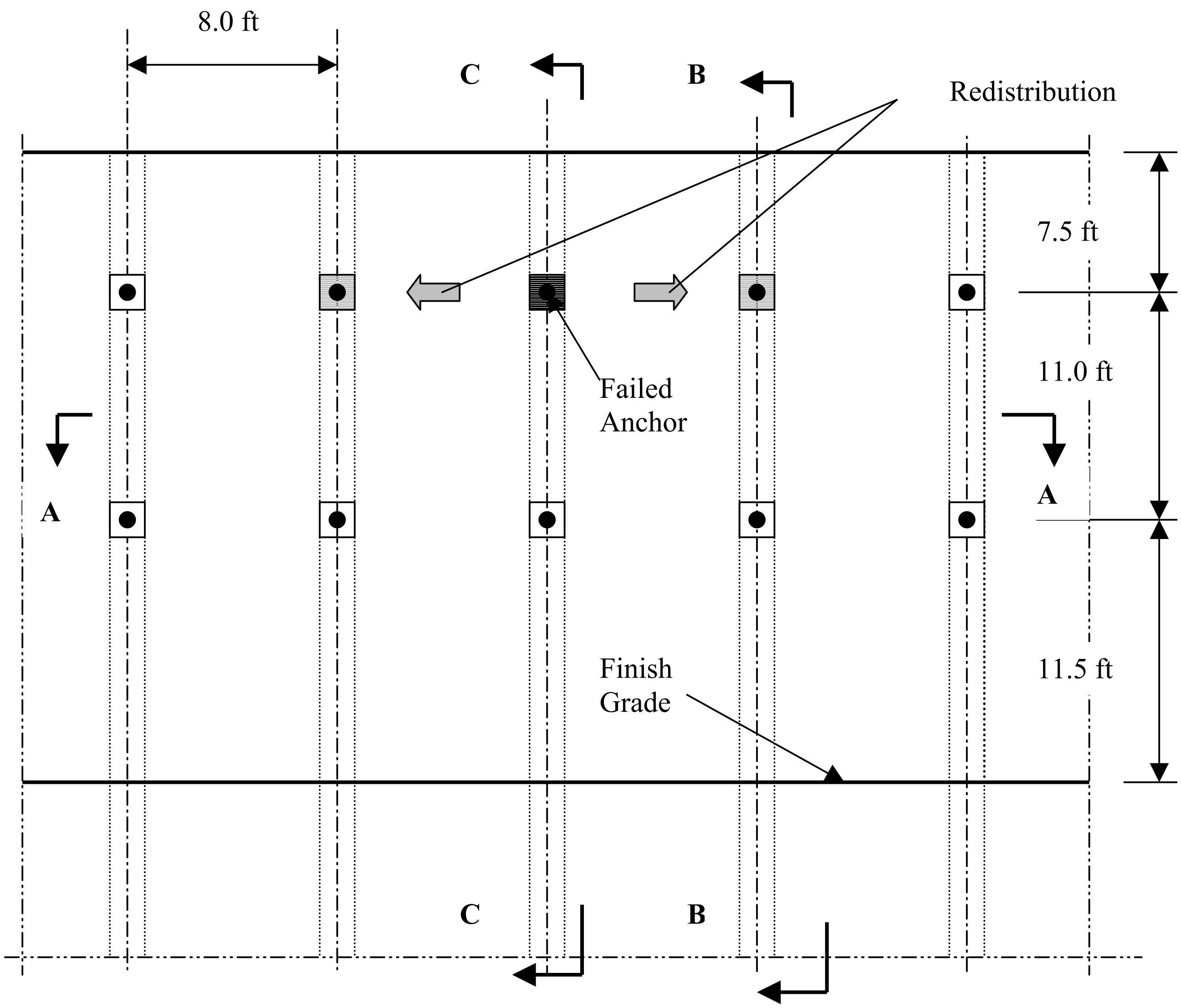
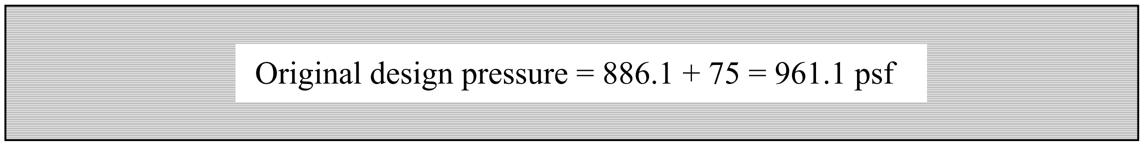
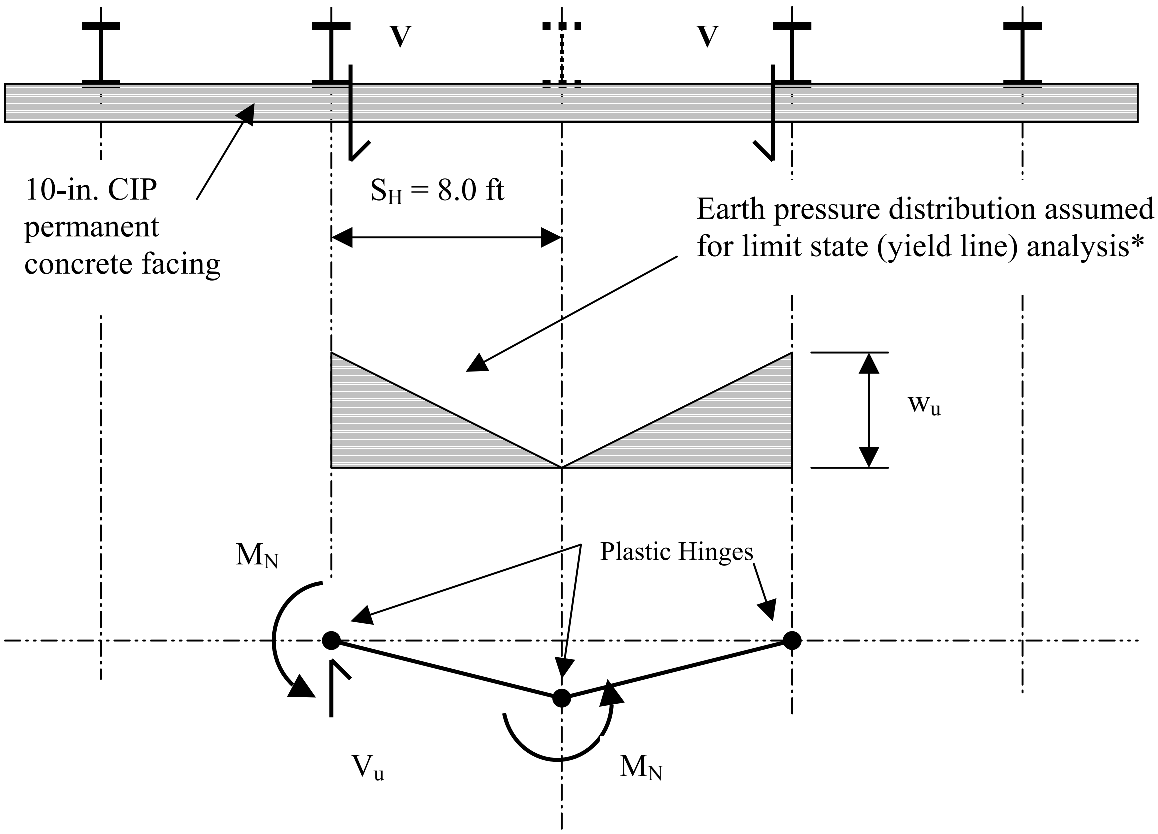


Figure 4.4 Soldier beams with concrete facing—elevation view; yield line analysis example



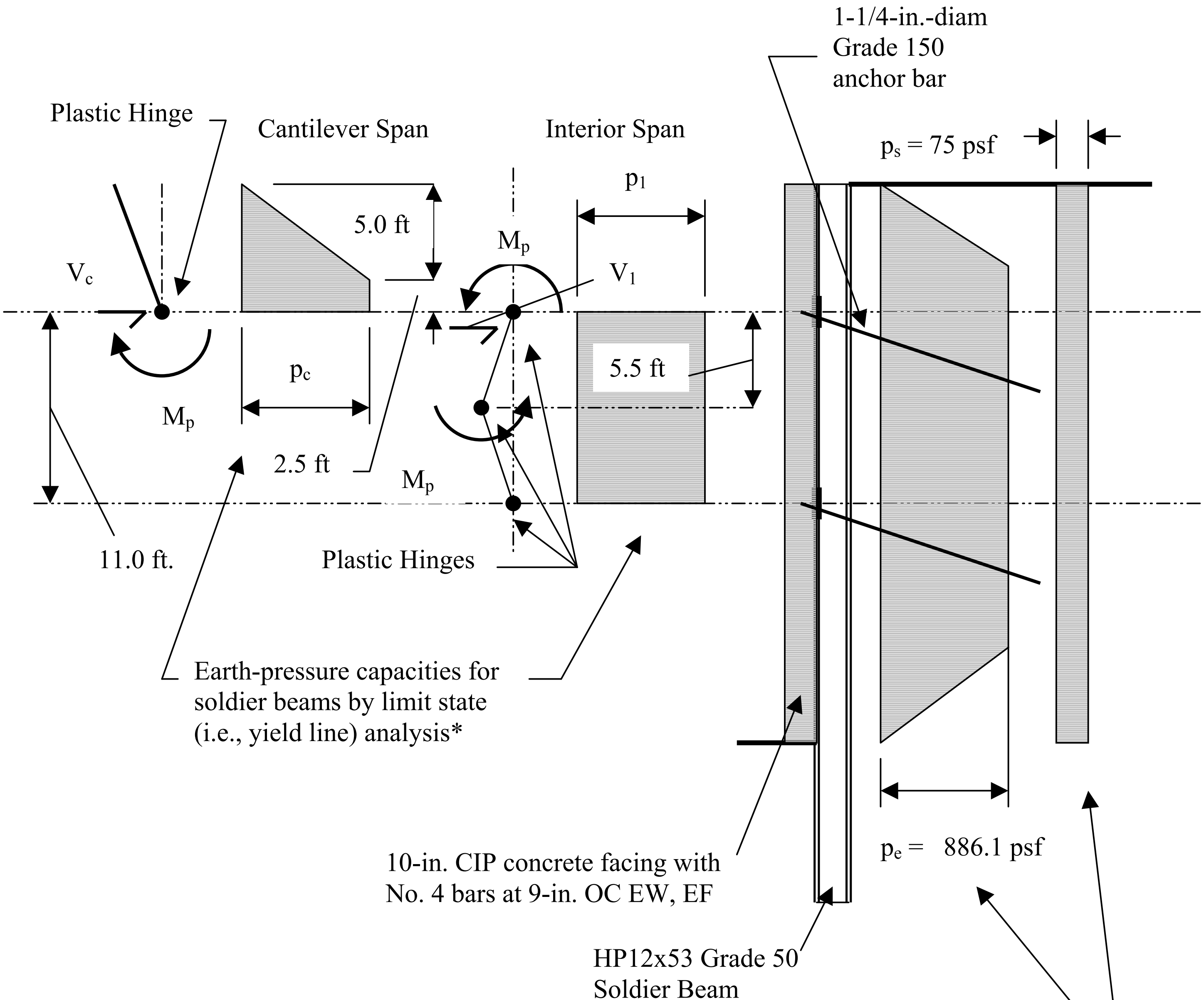
V = 0.961 (4) = 3.84 kips (original design)

V = 0.961 (8) = 7.68 kips (loss of anchor condition)



* Soil-pressure distribution assumes that significant soil arching will occur after an anchor fails (i.e., zero soil pressure at failed anchor location and a significant soil-pressure increase at the adjacent anchor locations). This is an extreme oversimplification of the earth pressure redistribution that occurs behind a tieback wall as the wall system undergoes large plastic deformation. This assumption is based solely on judgment and not supported by testing or analytical studies.

Figure 4.5 Section A-A, yield line analysis for facing



* The earth pressure distributions used to determine limit state earth-pressure capacities for the cantilever and interior spans are assumed to be similar to those used in the original design.

Figure 4.6 Section B-B, yield line analysis for soldier beams

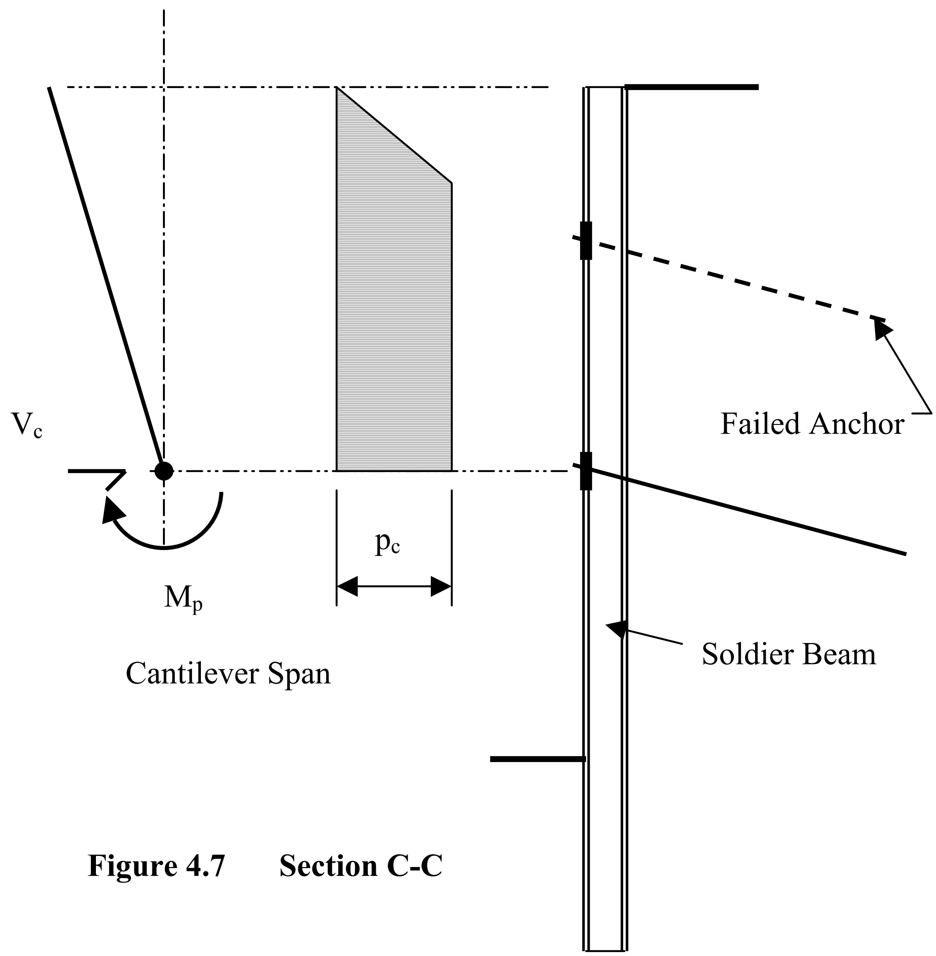


Figure 4.7 Section C-C

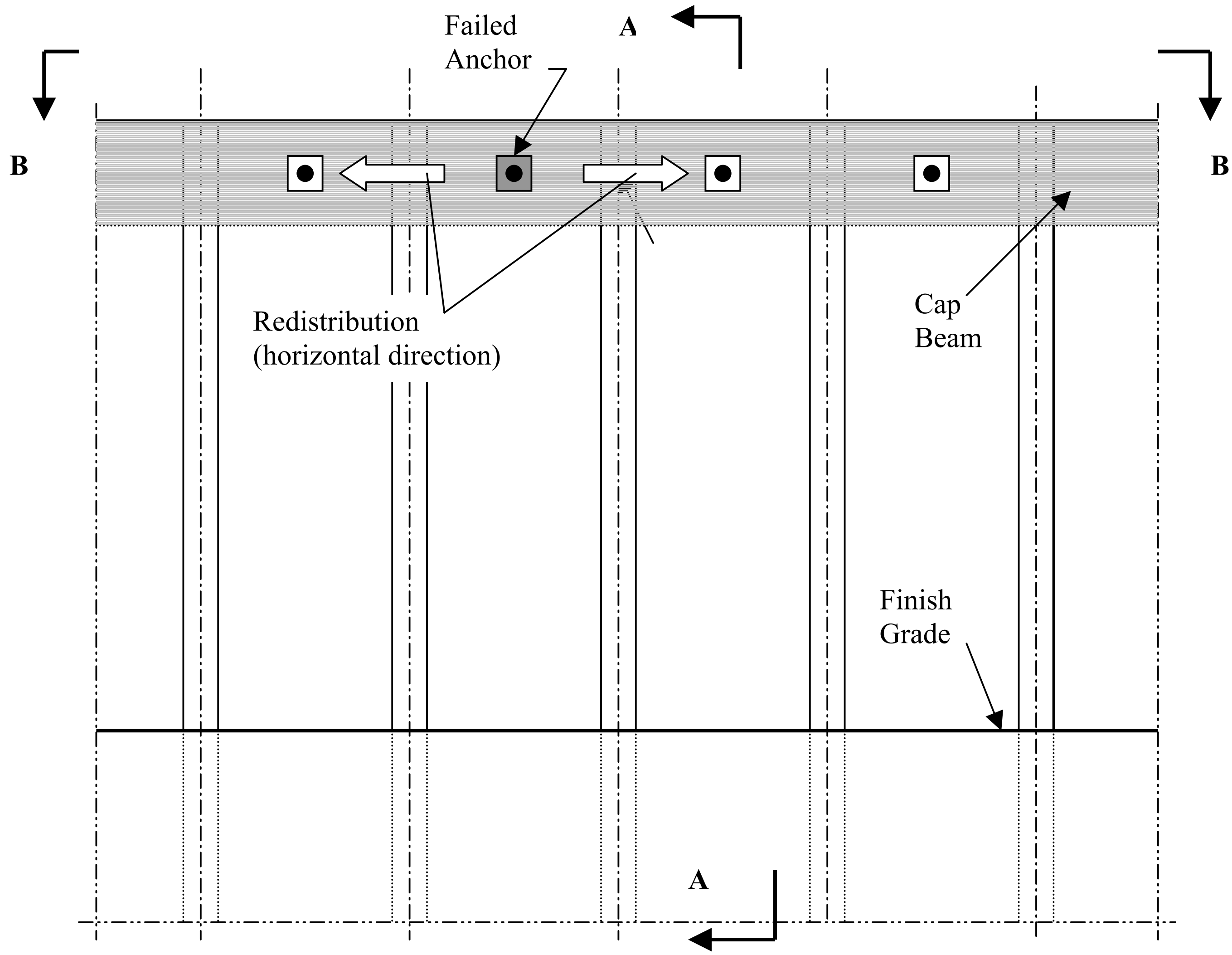


Figure 4.8 Soldier beams with concrete lagging–elevation; single row of anchors; redistribution by cap beam

5 Internal and External Stability

5.1 Introduction

The internal and external stability of tieback wall systems must be evaluated as part of any tieback wall design. Internal stability evaluations are used to ensure that the total load on which the original wall system design was based (i.e., usually an apparent pressure diagram) is suitable and, as such, that the ground anchors have sufficient capacity to prevent a structural failure. External stability evaluations are needed to verify that the anchor location is adequate to prevent ground mass instability. Simple force equilibrium methods are available to evaluate internal and external stability. The simplified approach, however, is limited to walls with reasonably homogeneous soil profiles and for conditions where a water table is not present in the retained soil. Complicated soil stratification, irregular ground surfaces, irregular surcharge loadings, and conditions in which a water table is present usually require the use of general-purpose slope stability (GPSS) programs.

As stated above, simple force equilibrium methods and GPSS methods can be used to evaluate internal stability. These methods are described herein and illustrated by example for a uniform cohesionless soil profile in Appendixes B and C. Appendix B covers the condition where the retained soil is dry, and Appendix C covers the condition where a water table is present in the retained soil.

Simple force equilibrium methods and GPSS methods can also be used to evaluate external stability and to establish anchor location. These methods are also described herein and illustrated by example for a uniform cohesionless soil profile in Appendix D for a tieback wall supported by a single row of anchors and in Appendix E for a tieback wall supported by two rows of anchors. Both “dry” and “partially submerged” conditions are covered, to illustrate the influence of the presence of a water table in the retained soil on anchor location. Internal and external stability analyses by GPSS methods for a layered soil system are presented in Appendix F.

A 30-ft-high tieback wall is used to demonstrate the internal and external stability evaluation process. The wall represents a continuous reinforced concrete slurry wall system. This simplifies the analysis because only those failure planes passing below the wall must be considered in the stability analyses. With soldier beam systems, failure surfaces passing through the soldier beams at the excavation level, and failure surfaces passing through the soldier beams between the beam tip and excavation level, must also be considered. Stability analyses unique to soldier beam systems are described in FHWA-RD-98-065. The FHWA-RD-98-065 report also covers those procedures that are applicable to the tremie concrete/slurry wall system. Much of the information presented in this section comes from FHWA-RD-98-065. Two GPSS programs are used in the evaluations: CSLIDE and UTEXAS4 (Wright 2001).

The design of an anchored wall concentrates on achieving a tieback wall system that is safe against a range of potential failure conditions. These conditions are described in FHWA-SA-99-015 and illustrated in Figure 5.1.

The stability analyses presented herein focus on whether the shear strength of the soil mass and the location and magnitude of the resultant forces provided by the ground anchors are sufficient to provide an acceptable factor of safety. An adequate level of serviceability with respect to various external failure modes is also required.

For temporary support of excavation anchored systems constructed in soft to medium clay soils, external stability should be evaluated using short-term (i.e., undrained) strength parameters and temporary loading conditions. For permanent anchored wall systems constructed in soils, external stability for both short- and long-term conditions should be checked. For systems constructed in stiff clays, i.e., overconsolidated clays, external stability for short-term conditions may not be critical, but long-term conditions, using drained shear strength parameters, may be critical. External stability of wall supported by rock anchors is normally adequate; however, if the rock mass has planes of weakness that are orientated in a direction that may affect stability, external stability should be checked for failure surfaces passing along those weak planes.

5.2 Factors of Safety for Internal and External Stability Evaluations

It has been indicated (in FHWA-RD-97-130) that a factor of safety of 1.3 applied to the shear strength of the soil will give lateral earth-pressure loads similar to those estimated using Terzaghi, Peck, and Mesri's (1996) apparent earth-pressure diagrams. This premise is demonstrated in FHWA-RD-97-130 and in FHWA-RD-98-065 and is used in various limiting equilibrium analyses as an alternate method for determining the "total design load" for tieback walls where stringent displacement control is not important. The later referenced report suggests that when displacement control is important, a factor of safety approaching 1.5 (applied to the shear strength of the soil) should be considered in the design. The authors of FHWA-SA-99-015 indicate that the minimum acceptable factor of safety is 1.3 and suggest that higher factors of safety be considered when deformation control is important.

Based on the above, a factor of safety equal to 1.3 is recommended for Corps "safety with economy" designs (i.e., designs where stringent displacement control is not a project performance requirement). For "stringent displacement control" designs and for other permanent applications that are critical, a factor of safety equal to 1.5 is recommended. These recommendations apply to both the internal and external stability evaluations of tieback walls.

5.3 Evaluation of Internal Stability

Limiting equilibrium methods are used in internal stability analyses to determine, or verify, the total force required to provide stability to the vertical cut. This is accomplished by investigating various potential failure surfaces that pass in front of the anchor bond zones. The process is used to ensure that the restraint force required to provide internal stability can be accommodated by the anchors. For the internal stability analysis, since the potential failure surfaces pass in front of the anchor bond zone, the entire ground anchor load acts as a stabilizing force to the soil mass above the failure plane. The internal stability process is described in detail in subsequent paragraphs and demonstrated in the appendixes.

Both simple force equilibrium methods and GPSS methods can be used for the internal stability evaluation. In most GPSS programs, the tiebacks can be modeled as high-capacity reinforcement. The axial force in the high-capacity reinforcement is described along the length of the anchor and in the anchor bond zone. The axial force in the reinforcement is assumed to vary linearly, from the full anchor capacity for all positions in front of the anchor bond zone to zero force at the end of the anchor bond zone. The UTEXAS4 program has the capability to apply the tieback forces to the base of each slice the anchor intersects and to the boundaries between each slice the anchor intersects, or to apply the tieback forces only to the base of each slice the anchor intersects. The first option is considered by Wright (2001) to be a more realistic representation of how reinforcement forces are distributed to the soil.

5.4 Evaluation of External Stability

To evaluate the stability of an anchored system, potential failure surfaces passing behind or through the anchor bond zones need to be checked. For walls with multiple levels of anchors, failure surfaces that pass behind each anchor bond zone should be checked (Figure 5.2). In checking a failure surface that passes behind a level of anchors, the failure surface may cross in front or through the anchor bond zone of other level(s) of anchors. In this case, the analysis is amended to include a portion of the restraint force from the other anchor(s). If the failure surface passes in front of the anchor bond zone, the full design load for that anchor is modeled as a restraint force. If the failure surface crosses the anchor bond zone, a proportional magnitude of load assuming that the anchor bond stress is distributed uniformly over the anchor bond length may be assumed. In most ground this is a reasonable assumption. However, in ground that becomes much weaker with depth, the ground anchor may develop most of its load-carrying capacity near the front of the anchor bond length. Under such circumstances, with respect to external stability, the anchor may act like a shorter anchor (FHWA-RD-97-130). In such cases, a more suitable bond stress distribution model (i.e., other than uniform) will be required.

Where stability requirements cannot be met, the anchors may be lengthened or methods to improve anchor bond or load transfer mechanisms may be used.

Both simple force equilibrium methods and GPSS methods can be used for the external stability evaluation.

5.5 Using GPSS Programs for Evaluating Internal and External Stability

General-purpose slope stability programs offer a means to determine requirements for the equilibrium of anchored wall systems. GPSS programs solve for the minimum resistance along potential failure surfaces passing through the soil. These potential failure surfaces may be circular or noncircular. Many GPSS programs include capabilities to consider variable slope geometry, layered soil profiles, groundwater table and seepage effects, internal loads, and surface loads. Virginia Polytechnic Institute and State University evaluated various GPSS programs. These programs can be used for the analysis of reinforced and unreinforced slopes, including tieback wall systems. The report "Comparison of computer programs for analysis of reinforced slopes" (Pockoski and Duncan 2000) compares the features of the GPSS programs and the results from a series of example problems, including three tieback wall examples. The reliable use of these programs, however, can be complicated by an incomplete understanding of the limitations of the GPSS programs, by a lack of understanding of the algorithms used to find the critical failure surface, or by undocumented errors in programming. It is prudent when performing internal and external stability analyses to progress from simple force equilibrium procedures to the more complex GPSS procedures, to ensure that the final results obtained from the GPSS programs are reasonable. This approach has been taken with respect to the examples presented in the appendixes. The summary section at the end of this chapter describes other methods that can be used to ensure that the results obtained from GPSS analyses are valid.

The factor of safety in GPSS analyses is based on the ratio of the soil strength available to the soil strength required for equilibrium. The general procedure for determining the factor of safety is to construct a trial failure surface through the soil mass and then divide the soil above the failure plane into several vertical slices. Subdividing the soil mass into slices allows GPSS programs to include the effects of soil layering, effects of water pressure, variable geometry, and surface loads. GPSS programs require a variety of assumptions to solve the equilibrium equations for the soil above the potential failure surface. Methods available may solve for the factor of safety by requiring force equilibrium ($\Sigma F = 0$), moment equilibrium ($\Sigma M = 0$), or a combination of moment and force equilibrium. Most solution techniques require assumptions with respect to the interslice force angle. General features for some common methods of slices are given in Table 5.1.

Many GPSS programs are coded to allow the user to select a specific method of analysis from a list of several methods, such as those described in Table 5.1.

5.6 Internal Stability of Anchored Wall Systems–Analysis Details

The internal stability evaluation of anchored wall systems is described herein with respect to the simple force equilibrium method of FHWA-SA-99-015. It is also described with respect to CSLIDE and UTEXAS4, two GPSS programs used by the Corps of Engineers. The internal stability analysis process is illustrated by example with respect to a 30-ft tieback wall (see Appendixes B and C).

5.6.1 Simplified limiting equilibrium approach (FHWA-SA-99-015, paragraph 5.2.8)

As described in FHWA-SA-99-015, a sliding wedge force equilibrium method may be used to evaluate the total horizontal load required to provide stability to a vertical cut. An example failure surface, free body diagram, and force vector diagram are shown in Figure 5.3 for a wall of height (H) with a soil behind and in front of the wall characterized by an effective stress friction angle (ϕ'_{mob}). It is assumed that the critical potential failure surface passes in front of the anchor bond zone such that the full anchor loads contribute to wall stability. The shear strength is factored by the target factor of safety such that $\phi'_{mob} = \tan^{-1}(\tan\phi' / FS)$. Mobilized passive resistance is assumed to develop over the wall embedment depth (d). For the assumed failure surface, an interface friction angle (δ_{mob}) equal to ϕ'_{mob} may be used to calculate the passive earth-pressure coefficient.

In the analysis (P_{REQ}) represents the external horizontal force required to provide stability to the vertical cut. This force represents the combined resistance provided by the horizontal component of the anchor force ($T \cos i$) and the lateral resistance provided by the embedded portion of the wall, (SP_H). The assumption that (P_{REQ}) is horizontal implies that the vertical resistance provided by the soldier beam (SP_V) is equal in magnitude and opposite in sign to the vertical component of the ground anchor loads ($T \sin i$). The required force (P_{REQ}) is then calculated as

$$P_{REQ} = \frac{1}{2}\gamma H^2 \left[\frac{(1+\xi)^2}{\tan(\alpha)} - K_{pmob}\xi^2 \left(\sin(\delta_{mob}) + \frac{\cos(\delta_{mob})}{\tan(\alpha - \phi_{mob})} \right) \right] \tan(\alpha - \phi_{mob}) \qquad (5.1)$$

where all the terms are defined in Figure 5.3.

The solution is found iteratively by adjusting the angle of the potential failure surface (α) and the wall embedment (d) until the greatest P_{REQ} is found. The value for K_{pmob} in Equation 5.1 is based on the assumption that the failure surface beneath the bottom of the cut on the passive portion of the soil has a log spiral shape. The passive coefficient, K_{pmob}, can be obtained for a log spiral solution using information provided in Ebeling and Morrison (1992). This same information is also provided in NAVFAC (1982) and in FHWA-RD-98-065. This load (P_{REQ}) can be converted to an apparent pressure envelope for calculating ground anchor loads. Detailed discussion on the use of this simplified method is provided in FHWA-RD-98-065.

5.6.2 CSLIDE

5.6.2.1 General. The computer program CSLIDE (Pace and Noddin 1987) is often used by the Corps to assess the sliding stability of concrete structures. The limit equilibrium analysis procedure implemented in CSLIDE is based on principles that consider the shear strength of the soil and/or rock in the analysis. A factor of safety is applied to the factors that affect the sliding stability and are known with the least degree of certainty; these factors are the material strength properties.

A state of limiting equilibrium is said to exist when the resultant of the applied shear force is equal to the maximum shear strength along a potential failure surface. The ratio of the maximum shear strength to the applied shear stress along a potential failure surface is defined as the factor of safety (*FS*), as defined in Equation 5.2 below.

$$FS = \frac{\tau_F}{\tau} \tag{5-2}$$

where

τ_F = maximum shear strength
τ = shear stress required for equilibrium

Two simplifying assumptions are used in the CSLIDE analysis:

- The interface between adjacent wedges is a vertical plane.
- The failure surface is composed of linear segments.

The fundamental assumptions used in the CSLIDE analysis are

- The factor of safety is as defined by Equation 5.2.
- The sliding mechanism can be adequately represented by two-dimensional analysis.
- The maximum available shear resistance is defined by Mohr-Coulomb failure criteria.
- The assumed failure surface is kinematically possible.
- Force equilibrium is satisfied; moment equilibrium is not considered.
- The shearing force acting parallel to the vertical interface of any two wedges is negligible. There is no interaction of vertical effects between wedges; therefore, wall friction cannot be included in the analysis.
- The factor of safety for each wedge is identical.
- The effects of displacements on the magnitudes of active and passive forces developed are not considered.
- There can be only one structural wedge because concrete structures transfer significant shearing forces across vertical internal boundaries.

The advantage of CSLIDE over the simplified limiting equilibrium approach is that it can easily accommodate soil layering, irregular ground surface profiles, water pressures, and surcharge loadings. Wall friction, however, cannot be included in the analysis. Therefore, the results will generally be more conservative than those obtained from a simplified limiting equilibrium analysis, or other GPSS analyses. In CSLIDE, moment equilibrium is not considered.

5.6.2.2 Using CSLIDE for the internal stability of tieback wall systems. A limiting equilibrium (i.e., GSSP analysis) of a tieback wall system using CSLIDE can be performed using the tieback wall as the structural wedge. Shear strength equal to zero is assigned to the structural wedge, so all sliding resistance will be provided by the passive soil wedge. For internal stability evaluation, the desired factor of safety is input into the CSLIDE analysis. A factor of safety approximately equal to 1.3 is commonly used for "safety with economy" designs, with higher factors of safety used for "stringent displacement control" designs. The base of the structural wedge is located at the failure plane level under consideration (i.e., not the actual tieback wall toe location). CSLIDE will indicate the *Sum of Forces on System*. This represents the lateral force needed to achieve system equilibrium at the specified factor of safety. The base of the structural wedge is varied (i.e., embedment depth to height ratio varied) until the maximum lateral force needed to achieve system equilibrium is determined. As with the simplified limiting equilibrium approach, this force represents the combined lateral resistance provided by the horizontal component of the anchor force and by the embedded portion of the wall. The use of CSLIDE for internal stability analysis is demonstrated in Appendix B for dry or moist soil conditions and in Appendix C for submerged soil conditions.

5.6.3 UTEXAS4

5.6.3.1 General. UTEXAS4 (Wright 2001) is a GPSS program that permits the user to obtain factors of safety using a procedure of slices. The Spencer's Method (default method) (Spencer 1967), the Bishop's Simplified Procedure (circles only) (Bishop 1955), the Janbu Simplified Method (Corps of Engineers' Modified Swedish Procedure (HQDA 1970) when side forces are horizontal), or the Lowe and Karafaith (1960) Method can be selected for the analysis. For circular and noncircular failure surfaces, the surface can be defined or determined by a search for the failure plane producing the lowest factor of safety. The Spencer's Method is the only method that requires both force and moment equilibrium. All other methods require only force equilibrium.

5.6.3.2 Using UTEXAS4 for the internal stability of tieback wall systems. The Janbu Simplified Method with the side force inclination set equal to zero (Corps of Engineers' Modified Swedish Procedure) and planar failure surfaces was used to determine the total force required to stabilize the cut. A factor of safety equal to 1.3 (applied to the shear strength of the soil) was assumed for the analysis. The wall penetration depth was set equal to that determined previously using CSLIDE. The load required to stabilize the cut (i.e., tieback design load) was compared to the total lateral earth load determined by the apparent pressure diagram (Terzaghi, Peck, and Mesri 1996). It was also compared to the load required to stabilize the cut obtained from the simple force equilibrium method (FHWA-RD-98-065) for dry site conditions and to the results obtained from the Corps' CSLIDE analysis for both dry and partially submerged conditions. The results are presented in Appendix B for dry site conditions and in Appendix C for partially submerged conditions. It should be noted that Wright (2001) does not recommend the Janbu Simplified Method. It usually tends to underestimate the factor of safety (i.e., overestimate the force required to stabilize the cut). However, FHWA-RD-97-130 (paragraph 4.3.3) recommends that in sandy ground, a method such as the Janbu Method (i.e. one that uses force equilibrium and planar failure surfaces) be considered for the internal stability evaluation.

An attempt was made to perform the internal stability analysis with UTEXAS4 using the Spencer Procedure. However, the default range of acceptable side force inclination in UTEXAS4 is from +80 deg to –10 deg from horizontal. This range is too restrictive for the internal stability evaluation of reinforced slopes, since the side force inclinations tend to be very negative due to the large side forces imparted by the reinforcement. It should be noted that Pockoski and Duncan (2000) modified the newest version of UTEXAS4 so the user could specify more negative side force inclination values for use in the internal stability evaluation of tieback wall systems. This problem was avoided in the examples contained in Appendixes B and C by using the Corps of Engineers' Modified Swedish Procedure (Simplified Janbu Procedure) and assuming all the side forces to be horizontal. The internal stability evaluation contained in Appendix F for the layered soil system included a Spencer Procedure analysis but applied a uniform surcharge to the wall face, rather than modeling the tieback reinforcement directly. The uniform surcharge model is per Method 1 of FHWA-RD-97-130.

5.7 External Stability of Anchored Wall Systems–Analysis Details

The external stability evaluation of anchored wall systems is described herein with respect to the simple force equilibrium method of FHWA-RD-98-065. It is also described with respect to CSLIDE and UTEXAS4, two GPSS programs used by the Corps. The external stability analysis process is illustrated with respect to a 30-ft tieback wall (see Appendix D). It is also illustrated in Appendix F with respect to a layered soil system.

5.7.1 Simplified limiting equilibrium approach (FHWA-RD-98-065, paragraph 3.5.2)

5.7.1.1 General. A simplified force equilibrium approach can be used to check external stability of a tieback wall. This approach is described in FHWA-RD-98-065 and is limited to walls with reasonably homogeneous soil profiles. For complicated stratification, irregular ground surface, or irregular surcharge loading, the lateral force required to stabilize the excavation must be determined by a GPSS analysis.

The external stability of an anchored wall system is determined by assuming the potential plane of sliding passes behind the anchor and below the bottom of the wall. Since anchors are spaced at a horizontal distance, *S*, the potential failure surface may assume a three-dimensional (3-D) shape rather than the 2-D shape used as an idealized basis for the following analysis. When a 2-D surface is used to approximate a 3-D failure surface, it is commonly assumed that the idealized 2-D failure plane intersects the ground anchor at a distance *S*/3 from the back of the anchor, as shown in Figure 5.4.

The stability for the soil mass is determined by requiring horizontal and vertical force equilibrium. The soil mass under consideration is the soil prism ABCDEG, as shown in Figure 5.4.

5.7.1.2 Simplified force limit equilibrium approach for homogeneous soil sites. Forces on the soil mass are shown in Figure 5.5, and the force vectors on area ABDEG are shown in Figure 5.6.

The soil mass acts downward with a magnitude equal to its weight. On the left face, the mobilized passive soil resistance, K_{mob}, acts at a mobilized angle of interface friction, δ_{mob}. Active soil pressure is assumed to act on the right vertical face. On the bottom, soil resistance acts at an angle ϕ_{mob} from the perpendicular to the failure plane. The forces will sum to zero in the horizontal and vertical directions for a safety factor equal to one and a friction angle ϕ_{mob}. Additional details pertaining to the force equilibrium analysis can be found in FHWA-RD-98-065. Equation 5.3 (Equation 3.22 of FHWA-RD-98-065) is used to determine the friction angle ϕ_{mob} needed to produce force equilibrium for the soil mass ABDEG. In Equation 5.2, the friction angle ϕ is replaced by the mobilized friction angle, ϕ_{mob}. The resulting factor of safety based on strength, $FS_{STRENGTH}$, is equal to tan (ϕ) / tan (ϕ_{mob}). A value of $FS_{STRENGTH}$ equal to 1.3 is often used in practice, according to FHWA-RD-98-065 (paragraph 3.3.1, page 35), and such a factor of safety would be appropriate for "safety with economy" type designs.

$$\left(1+\xi+\lambda\right)X - K_{Pmob}\xi^2\sin(\delta_{mob}) + \frac{K_{Pmob}\xi^2\cos(\delta_{mob}) - K_{Amob}\lambda^2}{\tan(\phi_{mob}-\alpha)} = 0 \qquad (5.3)$$

where $X = x/H$ $\quad \lambda = y/H$ $\quad \xi = d/H$

The dimensions x, y, d, and H are shown in Figure 5.6.

In the Appendix D example, Equation 5.3 is solved to find the anchor location (i.e., x and y dimensions) required to meet minimum factor of safety requirements. Minimum factor of safety requirements are met when tan ϕ / tan ϕ_{mob} is equal to 1.3.

5.7.2 CSLIDE

A cursory description of the CSLIDE program (Pace and Noddin 1987) has been provided above under the section dealing with internal stability analysis. For external stability analysis the location of the anchor is established, and the active failure wedge adjacent to the wall defined by the failure plane angle (α) and the depth to the anchor (y). This process is demonstrated in Appendix D for dry and partially submerged soil. The analysis is performed with the selected α and y geometric information describing the anchor location, and a factor of safety is determined. If the anchor location fails to meet factor of safety performance requirements, the anchor location is moved back from the wall and the stability evaluation process repeated until the anchor location meets acceptance criteria. Since wall friction cannot be included in the analysis, the results in terms of acceptable anchor location will generally be more conservative than those obtained from the simplified limiting equilibrium analysis. In addition, moment equilibrium is not considered and, therefore, the CSLIDE analysis should be used only to verify the results of a more sophisticated GSSP analysis such as UTEXAS4. It should be noted that the presence of water in a cohesionless soil would require that the anchor be located farther behind the wall than would be the case for dry or moist soil conditions.

5.7.3 UTEXAS4

A cursory description of the UTEXAS4 GPSS program (Wright 2001) has been provided above under the section dealing with internal stability analysis. The Spencer Method was used for the external stability analysis. An external stability analysis with respect to a uniform cohesionless soil was performed for a tieback wall supported by a single row of anchors (Appendix D) and a tieback wall supported by two rows of anchors (Appendix E). Wall penetration was as assumed for the previous external stability analyses. Only noncircular (planar) failure surfaces were considered for the external stability evaluation of the tieback wall supported by a single row of anchors. Both noncircular and circular failure surfaces were considered (per Figure 5.2) for the external stability evaluation of the tieback wall supported by two rows of anchors. The “search” option was used for the Spencer Method analysis to determine the potential failure surfaces with the lowest factors of safety. All the slope stability methods available in UTEXAS4 were used to evaluate the external stability of the layered soil system tieback wall of Appendix F.

5.8 Cohesive Soils

Limit equilibrium methods can be used to evaluate the total earth load for anchored systems in purely cohesive soils. For temporary anchored systems in soft to medium clays with $N_s > 4$, computed earth loads were compared using Henkel’s Method, Rankine Method, and limit equilibrium solutions. These results are shown in Figure 5.7.

The stability number, N_s, is defined as

$$\frac{\gamma H}{s_u} \qquad (5.4)$$

Limit equilibrium methods are summarized in Table 5.1. Results indicate that limit equilibrium methods compare favorably to Rankine analyses where the failure surface intersects the corner of the wall. When the failure surface extends below the excavation (e.g., $d/H = 0.2$ in Figure 5.7), Henkel’s and Bishop’s Methods are in reasonable agreement and are upper bounds. For cases where the critical potential failure surface extends below the base of the excavation and where $N_s > 5$, the Rankine analysis results are unconservative. For those cases, either Henkel’s Method or limit equilibrium analysis methods should be used to evaluate the total earth load. The total load should then be redistributed into an apparent pressure diagram using the Terzaghi, Peck, and Mesri (1996) diagram for soft to medium clays.

5.9 Summary and Conclusions

For complicated stratification, irregular ground surface, or irregular surcharge loading, the lateral force required to stabilize the excavation must be determined by a GPSS analysis. A limited number of stability analyses were performed (see Appendixes B-F). The authors of this report want to emphasize that this should in no way be considered an all-inclusive, exhaustive study of the GPSS analyses related to tieback wall systems. These limited analyses, however, are presented to demonstrate that the reliable use of these programs can be complicated by an incomplete understanding of the limitations of a particular GPSS program, or by a lack of understanding of the algorithms used to find the critical failure surface, including numerical algorithm(s) required for implementation of the theoretical formulation in the computer program, as well as search algorithms devised during computer programming.

As was stated earlier, it is prudent when performing internal and external stability analyses to progress from simple force equilibrium procedures to the more complex GPSS procedures, to ensure that the final results obtained from the GPSS programs are reasonable. Checking by various other GPSS methods is always advisable, since there are definitely some unusual things that can happen internally within the GPSS programs, especially with respect to tieback wall systems when searching for the critical failure plane (i.e., plane with the lowest factor of safety). Checking is important because most of the GPSS programs, which trace their origin to the 1960s and 1970s, were not developed with tieback wall applications in mind. It should be noted that, with respect to tieback wall applications, it is not uncommon to discover undocumented errors in programming.

Pockoski and Duncan (2000) provide some tips for coping with and resolving difficulties with GPSS programs. These tips cover various slope stability applications, including reinforced slopes and tieback wall systems. Some of their key recommendations are presented below.

- During the GPSS program analysis, use different methods (Spencer's Method, Bishop Modified Method, etc.) to compute the factor of safety. When one method has a high degree of numerical problems and nonconvergence, another method having a more simple side force assumption may provide a more reliable estimate of the factor of safety.

- If possible, use different computer programs to compute the factor of safety. Because computer analysis of reinforced slopes is a relatively new topic, there is no accepted convention of applying the reinforcement forces to slice boundaries and slip surfaces, and different computer programs handle reinforcement forces differently. Different methods of applying the reinforcement forces result in different side force inclinations, and sometimes better convergence.

- During the actual search, be sure to search thoroughly for the most critical slip surface (minimum factor of safety).

- The first step would be to search a wide area for the most critical surface center. Use a grid spacing small enough to give a complete picture of the search area, but large enough that many analyses can be performed quickly.

- Refine the grid size in a second phase of the search in the area around the lowest factor of safety of the initial search. However, do not immediately jump to a small grid spacing directly around the lowest grid point. Local minima may be present, and can mislead the search. Instead, reduce the size of the grid in several steps, re-centering on the minimum if necessary.

- If the convergence criterion in the program can be controlled by the user, use a small final tolerance (e. g., 0.0001) on the factor of safety. A coarser tolerance may result in a false indication that convergence has been reached, and erroneous search results.

- Examine the program output carefully for warnings of problems with convergence or search results, and consider carefully how these may have influenced the search for the critical failure surface.

- Realize that some reinforced slopes may be impossible to analyze by limit equilibrium methods.

Based on the limited GPSS program internal and external stability evaluations performed in conjunction with this report effort, the concerns expressed above with respect to the Pockoski and Duncan (2000) study mirror the concerns of the authors of this report.

The authors of this report also found the Corps program CSLIDE to be a useful GPSS tool. This has been demonstrated with respect to the various tieback wall stability evaluations presented in the appendixes. CSLIDE is useful when performing preliminary internal stability evaluations needed to determine total design load, and also when performing preliminary external evaluations needed to establish anchor locations. It also offers a simple means for checking the results of the more comprehensive GPSS analyses to determine whether or not the comprehensive GPSS analyses have converged to a reasonable/believable result. The advantage of CSLIDE over other simplified limiting equilibrium approaches is that it can easily accommodate soil layering, irregular ground surface profiles, water pressures, and surcharge loadings. However, since wall friction cannot be included in the analysis, the results will generally be more conservative than those obtained from simplified limiting equilibrium analyses and most GPSS analyses.

5.10 Recommendations for Research

Although CSLIDE was not specifically developed for tieback wall application, its potential for use in this area should be researched further. Some improvements to CSLIDE that may have merit include

- Increased capacity to accommodate a greater number of soil layers. (Currently, CSLIDE is limited to five soil layers.)

- The capacity to easily define "by x- and y-coordinates" various failure planes of interest. This would allow easy investigation of external stability for the types of linear failure planes illustrated in Figure 5.2.

- Windows-based input and output capabilities.

- Improved plot capabilities.

Based on the limited number of stability analyses performed by the report authors, it appears that additional research related to GPSS program analysis of tieback walls is warranted. Studies similar to those by Pockoski and Duncan (2000) directed solely toward tieback wall applications should be conducted to identify those GPSS programs that have the greatest potential with respect to the internal and external stability evaluation of Corps tieback wall systems. In addition, specific areas of difficulty with respect to the GPSS program evaluation of tieback wall systems need to be resolved. Remedies include, but are not limited to, the following:

- Improved side force inclination capability.

  An attempt was made to perform internal stability analysis with UTEXAS4 using the Spencer Procedure. However, the default range of acceptable side force inclination in UTEXAS4 is from +80 deg to –10 deg from horizontal. This range is too restrictive for the internal stability evaluation of reinforced slopes, since the side force inclinations tend to be very negative due to the large side forces imparted by the reinforcement.

- Improved tieback anchor modeling capability.

  In most GPSS programs, the tiebacks can be modeled as high-capacity reinforcement. The axial force in the high capacity reinforcement is described along the length of the anchor and in the anchor bond zone. The axial force in the reinforcement is assumed to vary linearly from the full anchor capacity for all positions in front of the anchor bond zone, to zero force at the end of the anchor bond zone. The UTEXAS4 program has the capability to apply the tieback forces to the base of each slice the anchor intersects and to the boundaries between each slice the anchor intersects, or to apply the tieback forces only to the base of each slice the anchor intersects. These methods and others that may be applicable to tieback wall systems should be investigated.

- Improved convergence capability.

  Convergence with respect to slopes with abrupt changes in geometry (e.g., tieback wall systems) by GPSS programs that consider both force and moment equilibrium is often difficult. Also, slopes that depend entirely on reinforcement for stability are generally more difficult to analyze due to increased numerical problems and nonconvergence. Some tieback slopes may be virtually impossible to analyze by limit equilibrium methods (Pockoski and Duncan 2000).

- Improved capability to analyze layered soil systems that may include water in the retained soil.

  Layered soil systems that contain clay layers are difficult to evaluate with respect to tension cracks and water pressures that can develop in tension cracks. Since many Corps tieback walls will have loading conditions that involve a differential head, this aspect of design needs special consideration.

It is not possible to predict wall system displacements by limit equilibrium analysis. The serviceability performance of a given tieback wall system will depend on its ability to limit deformations in the wall and retained soil. Deformation prediction is therefore an important research objective. It should be remembered earth pressures behind a tieback wall will be essentially nonlinear and dependent on many factors, including soil type, wall fixity and restraint, factors of safety, tieback size and spacing, tieback prestress levels, construction sequencing, and overexcavation at anchor locations. Research, in addition to that described above, using nonlinear soil-structure interaction finite element analyses, is needed to validate the use of the design and analysis tools illustrated in the various Strom and Ebeling reports (2001, 2002, and this report). The objective of this additional research is to determine if there are simple procedures that can be used to predict the displacement response of those tieback wall systems that must meet “stringent displacement control” performance objectives.

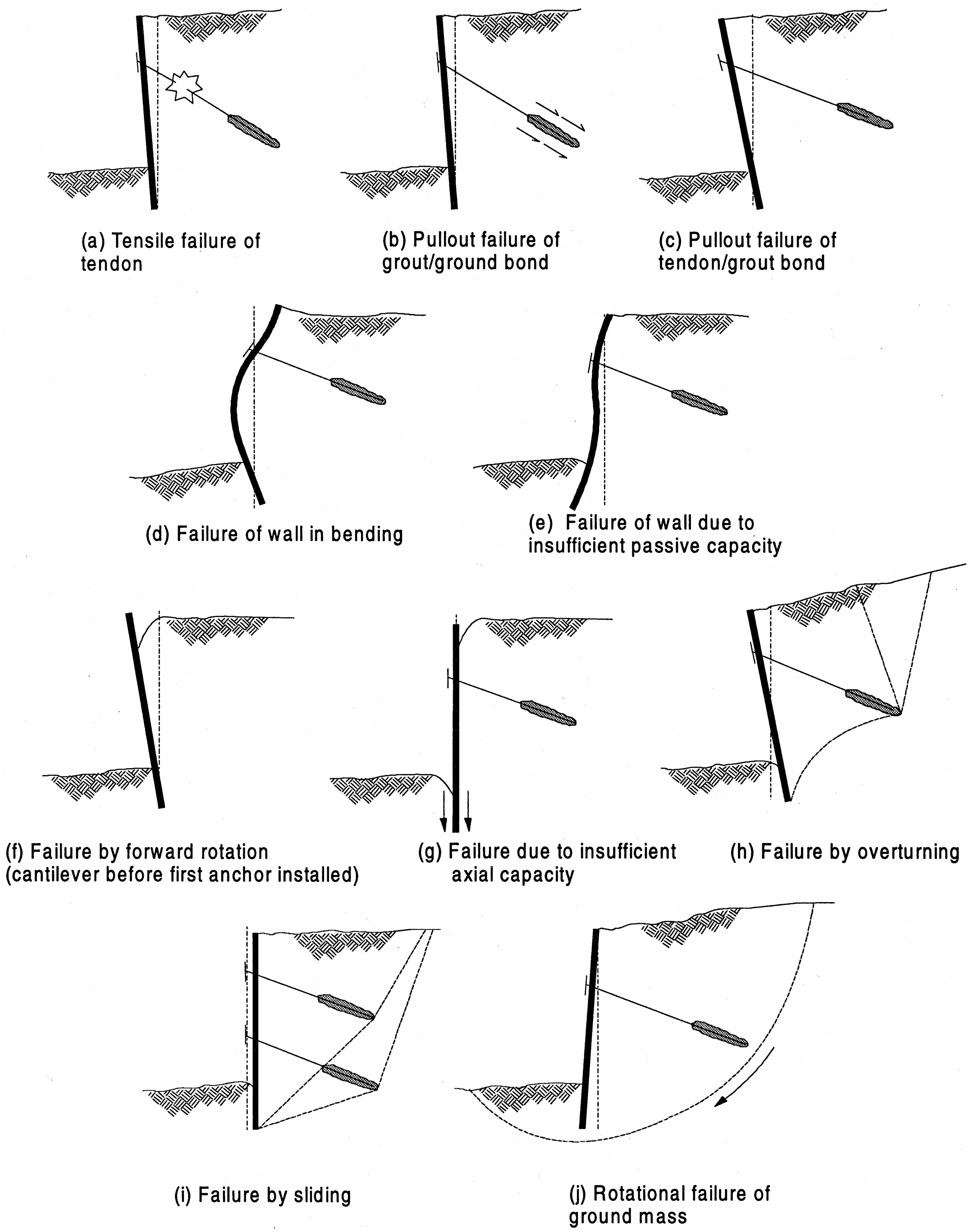


Figure 5.1 Potential failure conditions to be considered in design of anchored walls (after FHWA-SA-99-015, Figure 11)

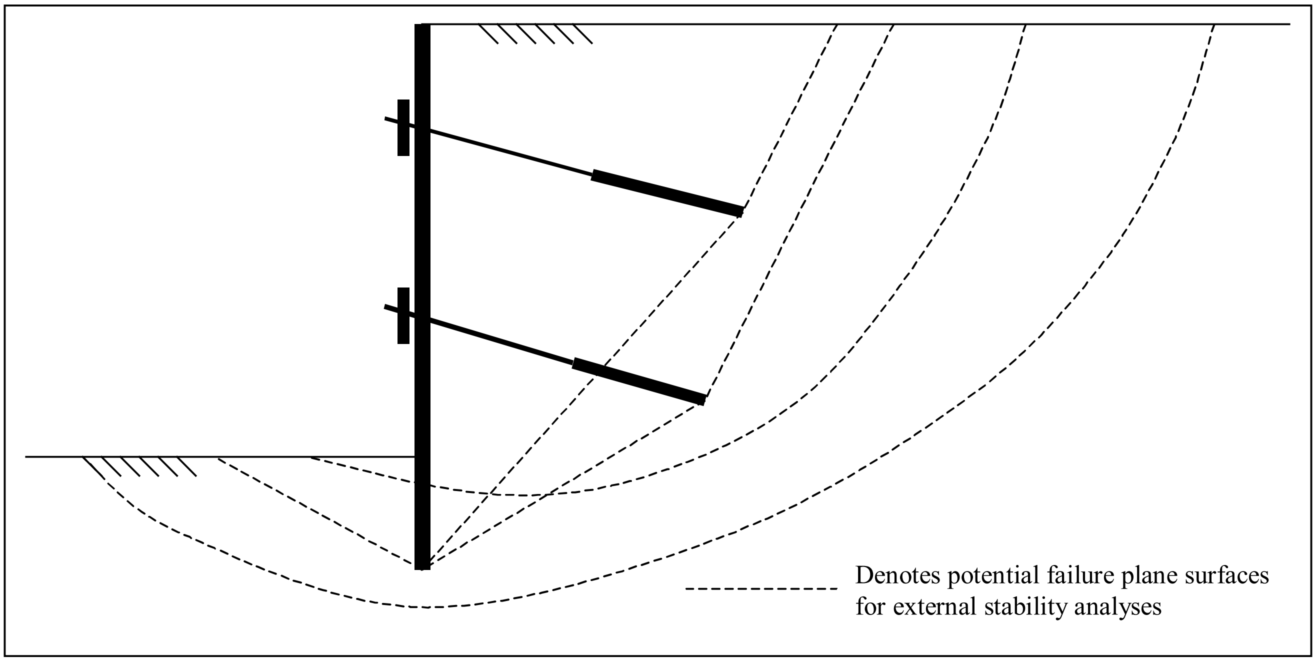
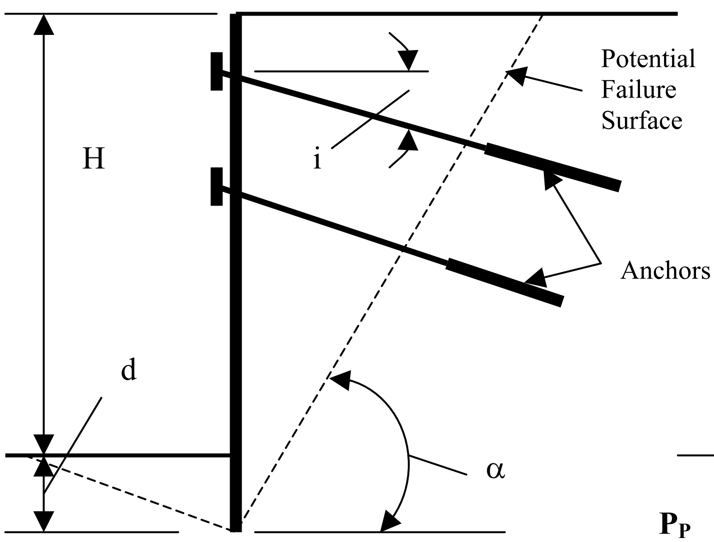
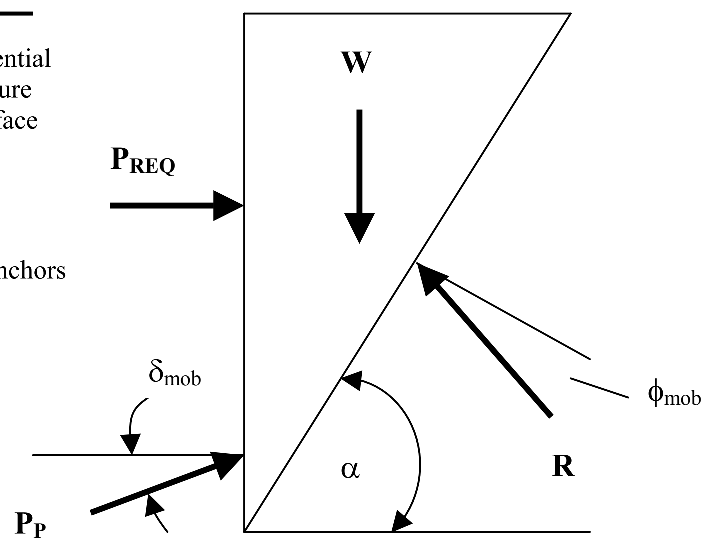


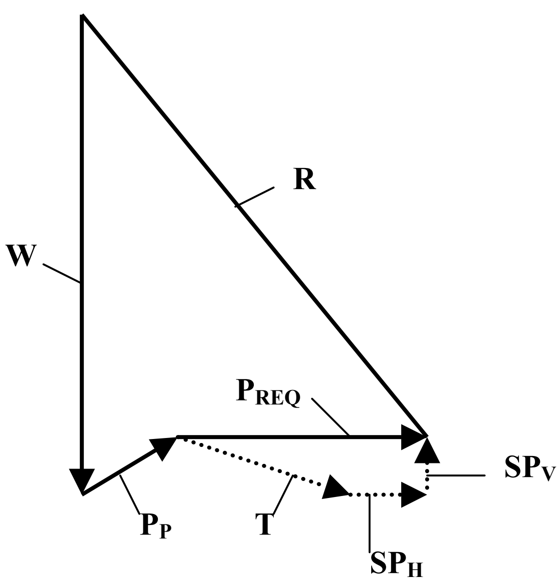
Figure 5.2 Failure surfaces for external stability evaluations (after FHWA-SA-99-015, Figure 52)



a. Example ground anchor wall system



b. Free body diagram



c. Force vectors

W = weight of soil mass
R = frictional component of soil strength
P_P = passive earth resultant force
T = total anchor force
SP_H = horizontal resistant force from wall
SP_V = vertical resistant force from wall
ϕ_{mob} = mobilized friction angle of soil
δ_{mob} = mobilized interface friction angle of soil/wall
i = inclination of anchor
α = inclination of potential failure surface
ε = d/H

Figure 5.3 Force equilibrium method for anchored walls (after FHWA-RD-98-065)

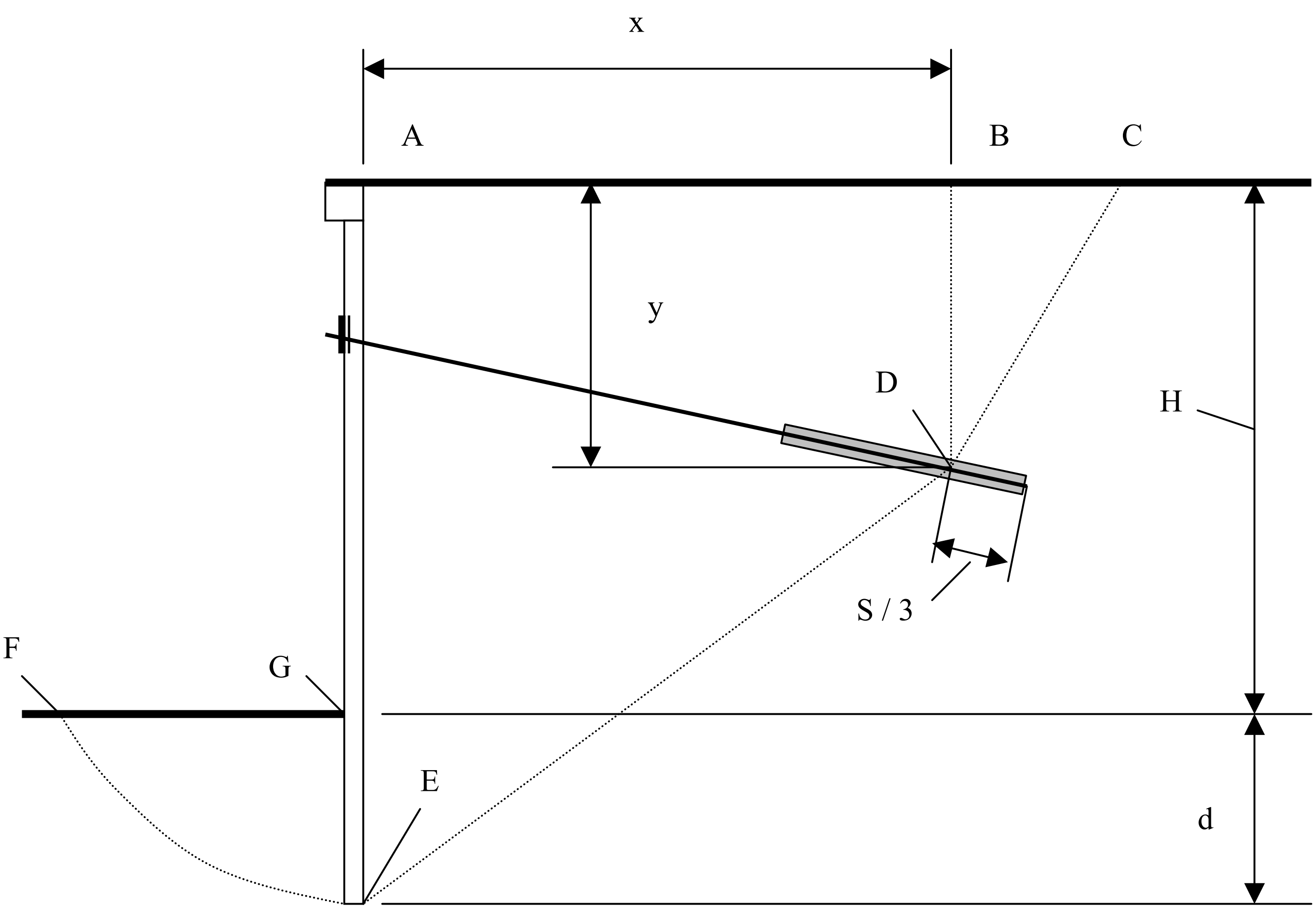


Figure 5.4 External stability—simple force equilibrium model stage

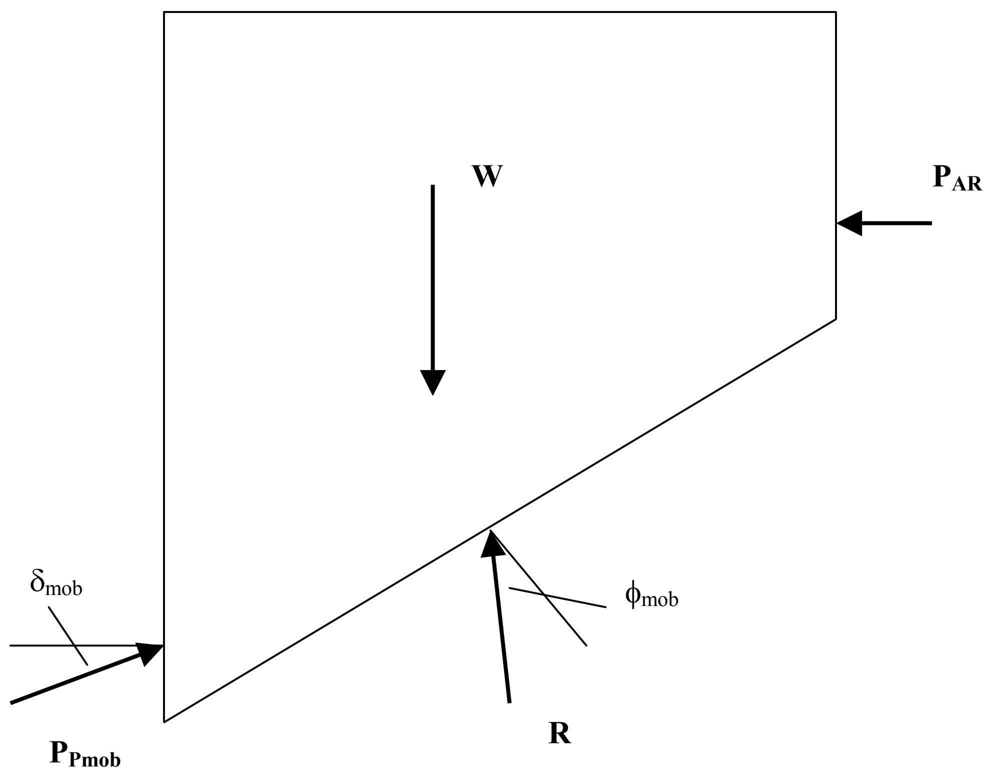


Figure 5.5 Force-body diagram

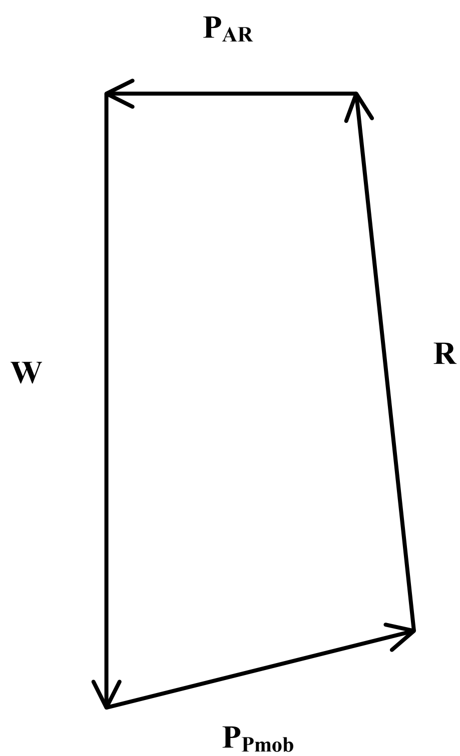
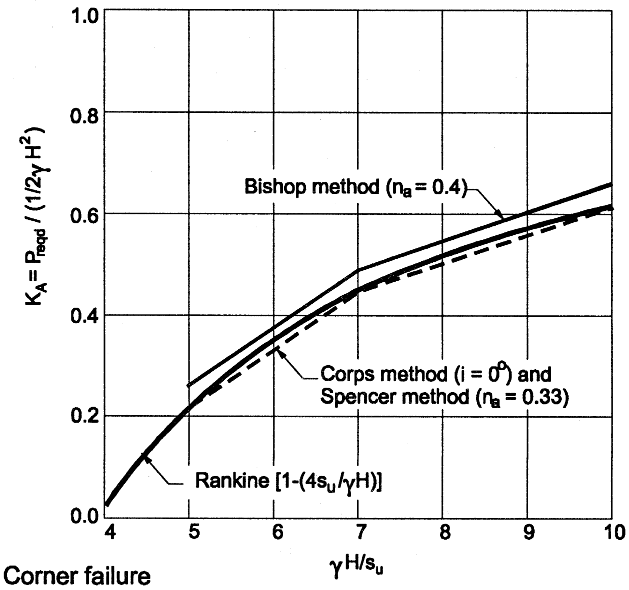
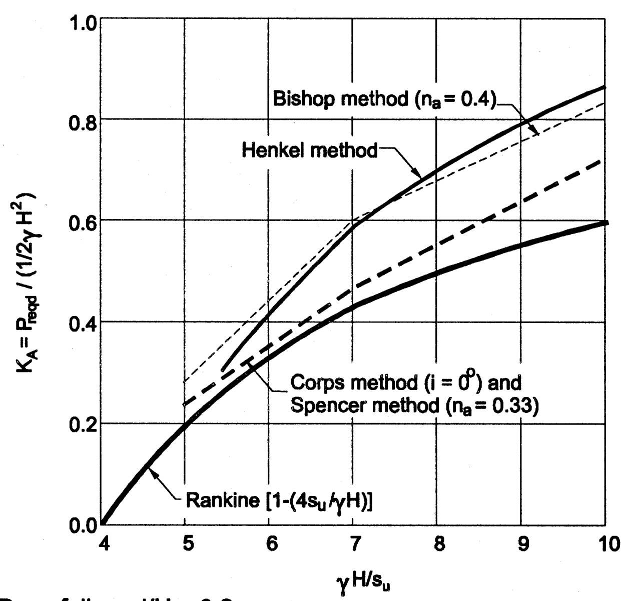


Figure 5.6 Force vectors acting on area ABCDEG



(a) Corner failure



(b) Base failure d/H = 0.2

Figure 5.7 Comparison of limit equilibrium methods for cohesive soils (after FHWA-SA-99-015 and FHWA-RD-98-065)

Table 5.1 Assumptions and Features of General-Purpose Slope Stability Methods of Analysis (after FHWA-RD-98-065)

Method of Analysis	Shape of Failure Surface	Equilibrium Equations	Assumptions for Interslice Forces
Bishop	Circular	$\Sigma F_V = 0$ $\Sigma M_{Overall} = 0$	Horizontal
Janbu (simplified)	Circular, noncircular	$\Sigma F_V = 0$ $\Sigma F_H = 0$	Horizontal
Janbu (rigorous)	Circular, noncircular	$\Sigma F_V = 0$ $\Sigma F_H = 0$ $\Sigma M = 0$	User-defined line of thrust
Corps of Engineers' Modified Swedish Procedure	Circular, noncircular	$\Sigma F_V = 0$ $\Sigma F_H = 0$	User-defined interslice force angle
Lowe and Karafaith	Circular, noncircular	$\Sigma F_V = 0$ $\Sigma F_H = 0$	Interslice force angle defined by slope of top and bottom of each slice
Spencer	Circular, noncircular	$\Sigma F_V = 0$ $\Sigma F_H = 0$ $\Sigma M = 0$	Constant interslice force angle
Morgenstern and Price	Circular, noncircular	$\Sigma F_V = 0$ $\Sigma F_H = 0$ $\Sigma M = 0$	User-defined variation in interslice force angle
Corps of Engineers' CSLIDE General Wedge Equations	Noncircular	$\Sigma F_H = 0$	User-defined wedge geometry

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Appendix A
Design Calculations for Soldier Beam Wall

A.1 Introduction

The design calculations presented herein reflect an approach often used for the design of soldier beam tieback wall systems with post-tensioned anchors and a permanent a cast-in-place (CIP) reinforced concrete facing.

A.2 Design Calculations

The following design calculations are in accordance with the design procedures contained in the cohesionless soil example of FHWA-RD-97-130 (pp 181-183, calculation steps 1-6a) for the two-tier tieback wall. An elevation view of the example wall is illustrated in Figure A.1.

A horizontal cross section through the wall (Section A-A) is shown in Figure A.2, and a vertical cross section (Section B-B) is shown in Figure A.3. The design soil loadings are also contained in these figures.

Details related to the design of the permanent CIP reinforced concrete facing are provided in Figure A.4.

Subsequent calculations representing the original design, which is a beam on rigid supports design using apparent soil pressures per Weatherby, 1998, are from a Mathcad file. These calculations were expanded to cover the design of the CIP reinforced concrete facing and headed stud attachments. These calculations form the basis for the original tieback wall design and the basis for the limit state evaluation used in Chapter 4. The limit state evaluation in Chapter 4 is provided to demonstrate the unlikelihood that a progressive tieback wall system failure will occur as a result of a single anchor failure.

File: BAA3 \ FHWA 130 Granular Soil Example

Tieback Wall Design Example
Multiple Anchor Locations
Two Foot Surcharge

1. Soil Properties
   Soil Classifications ML,SP,GP
   Friction Angle = 29 Degees
   Total weight (γ) = 108 pcf
   SPT (blows per foot) = 14

Determine Ground Anchor Load, Soldier Beam Moments and Subgrade Reaction per Linear Foot of Wall

2. Develop Earth Pressure Diagram

   a. Determine earth pressure factor (EPF)
   (per Figure 30, FHWA-RD-97-130)

   $EPF := 23.3$ psf

   a. Check of earth pressure factor (EPF)
   (per Terzaghi and Peck)

   $\phi := 29 \cdot deg$ $\delta := 0 \cdot deg$

   $k_a := \tan\left(45 \cdot deg - \frac{\phi}{2}\right)^2$ $k_a = 0.347$

   $\gamma := 108$ pcf

   $EPF := 0.65 \cdot k_a \cdot \gamma$ $EPF = 24.358$ psf Checks - OKAY

   Use $EPF := 23.3$ psf in computations

   c. Total load to stabilize cut (T_L) per foot run of wall

   $H := 30$ feet $T_L := EPF \cdot H^2$ $T_L = 2.097 \cdot 10^4$ pounds per foot of wall

d. Earth pressure to stabilize cut (p_e), Refer to Figure C-3

$H_1 := 7.5 \quad H_2 := 11 \quad H_3 := 11.5$ Refer to Figure C-1

$$p_e := \frac{T_L}{H - \frac{1}{3} \cdot H_1 - \frac{1}{3} \cdot H_3} \qquad p_e = 886.056$$ psf Refer to Figure C-3

e. Surcharge pressure (SP=2 feet of soil), Refer to Figure C-3

$SP := 2 \cdot (108) \quad SP = 216$ psf

f. Lateral surcharge pressure (p_s), Refer to Figure C-3

$$p_s := \tan\left(45 \cdot deg - \frac{\phi}{2}\right)^2 \cdot SP \qquad p_s = 74.946$$ psf

3. Calculate Bending Moment at Upper Ground Anchor (M_1)
   Contribution to M_1 for p_e is per Figure 29, FHWA-RD-97-130

$$M_1 := \left[\left(\frac{13}{54}\right) \cdot H_1^{\,2} \cdot p_e + \left(p_s \cdot H_1 \cdot \frac{H_1}{2}\right)\right] \frac{1}{1000}$$

$M_1 = 14.107$ ft-kips per foot of wall

4. Calculate the Ground Anchor Loads by the Tributary Area Method
   Contribution to T_1 and T_2 for p_e is per Figure 29, FHWA-RD-97-130

$$T_1 := \left[\left(\frac{2}{3}\right) \cdot H_1 + \left(\frac{1}{2}\right) \cdot H_2\right] \cdot \frac{p_e}{1000} + \left(H_1 + \frac{H_2}{2}\right) \cdot \frac{p_s}{1000}$$

$T_1 = 10.278$ kips per foot of wall - horizontal component

$$T_2 := \left[\left(\frac{1}{2}\right) \cdot H_2 + \left(\frac{23}{48}\right) \cdot H_3\right] \cdot \frac{p_e}{1000} + \left(\frac{H_2}{2} + \frac{H_3}{2}\right) \cdot \frac{p_s}{1000}$$

$T_2 = 10.599$ kips per foot of wall - horizontal component

5. Calculate the Subgrade Reaction at Base of Wall (R_B)

Contribution to R_B for p_e is per Figure 29, FHWA-RD-97-130

$$R_B := \left[\left(\frac{3}{16}\right)\cdot H_3\right]\cdot\left(\frac{p_e}{1000}\right) + \left(\frac{H_3}{2}\right)\cdot\frac{p_s}{1000}$$

$R_B = 2.342$ kips per foot of wall - horizontal component

6. Calculate Maximum Bending Moment Below Upper Anchor

Use greater of MM_1 or MM_2

Contribution to MM_1 and MM_2 for p_e is per Figure 29, FHWA-RD-97-130

$$MM_1 := \left(\frac{1}{10}\right)\cdot H_2^{\,2}\cdot\left(p_e + p_s\right)$$

$MM_1 = 1.163\cdot10^4$ ft-kips per foot

$$MM_2 := \left(\frac{1}{10}\right)\cdot H_3^{\,2}\cdot\left(p_e + p_s\right)$$

$MM_2 = 1.271\cdot10^4$ ft-kips per foot **Use**

Ground Anchor Design for Driven Soldier Pile Wall
Refer to Figure C-1

1. Determine upper ground anchor location

a. Ground anchor elevation = Ground surface elevation - H_1
= 45 - 7.5 = Elevation 37.5

b. Center of anchoring strata = 20 feet

c. Install anchor at flat angle to keep downward load on the soldier beam low. Assume a 57-foot long ground anchor.

d. Assuming a 24-foot long bond length, caculate the ground anchor inclination (α)

$$\text{Sin } \alpha_1 = \frac{(37.5 - 20)}{\left(57 - \frac{24}{2}\right)} = 0.389$$

$$\alpha_1 := \operatorname{asin}(0.389) \cdot \frac{180}{\pi}$$

$\alpha_1 = 22.892$ Degrees

Use α_1 = 20 Degrees for Constructability

e. Unbonded length (57 ft.-25 ft. = 33 ft)
15 Feet minimum recommended by PTI

2. Determine upper ground anchor load (T_{1D}). Assume a soldier beam spacing of 8 feet center to center for the driven soldier beams

$$\alpha_1 := 20 \cdot \deg$$

$$T_{1D} := T_1 \cdot \frac{(8)}{\cos(\alpha_1)}$$

$T_{1D} = 87.5$ kips

Upper Anchor Design Load = 87.5 kips
Use a 1.25-inch daimeter Grade 150 bar
Ultimate Capacity of 1.25-inch daimeter Grade 150 bar = 187.5 kips

Allowable design load is: $0.6 \cdot (187.5) = 112.5$ kips

3. Determine lower ground anchor location.
   a. Ground anchor elevation = Ground surface elevation -H_1 - H_2

      = 45.0 - 7.5 -11 = Elevation 26.5 feet

   b. Center of anchoring strata = 20 feet

   c. Use a ground anchor tendon with an anchor bond length of 24 feet and an unbonded length of 15 feet. Total ground anchor length = 39 feet.

   d. Assuming a 24-foot long bond length, caculate the ground anchor inclination (α)

$$\text{Sin } \alpha_2 = \frac{(26.5-20)}{\left(39-\frac{24}{2}\right)} = 0.241$$

$$\alpha_2 := \text{asin}(0.241)\cdot\frac{180}{\pi} \qquad \alpha_2 = 13.946 \quad \text{Degrees}$$

Use α_2 = 15 Degrees for Constructability

   e. Unbonded length = 15 Feet (minimum recommended by PTI)

4. Determine lower ground anchor load (T_{2D}). Assume a soldier beam spacing of 8 feet center to center for the driven soldier beams

$$\alpha_2 := 15\cdot\text{deg}$$

$$T_{2D} := T_2\cdot\frac{(8)}{\cos(\alpha_2)} \qquad T_{2D} = 87.783 \quad \text{kips}$$

Lower Anchor Design Load = 87.5 kips
Use a 1.25-inch daimeter Grade 150 bar
Ultimate Capacity of 1.25-inch daimeter Grade 150 bar = 187.5 kips

Allowable design load is: $0.6 \cdot (187.5) = 112.5$ kips

Soldier Beam Design for Driven Soldier Pile Wall

1. Assume: Driven soldier beams are spaced at 8-feet on center

2. Determine design bending moment (M_D) and determine size of the soldier beams. Maximum bending moment is M_1 at 14.1 ft-kips per foot of wall.

$M_D := M_1 \cdot (8)$ $\quad M_D = 112.852$ ft-kips

For Grade 36 Steel

$f_b := 20$ (Allowable bending stress, ksi)

Determine section modulus requires (S_{reqd})

$$S_{reqd} := \frac{\left[M_D \cdot (12)\right]}{f_b} \qquad S_{reqd} = 67.711 \text{ in}^3$$

HP14x73 Grade 36

For Grade 50 Steel

$f_b := 27$ (Allowable bending stress, ksi)

Determine section modulus requires (S_{reqd})

$$S_{reqd} := \frac{\left[M_D \cdot (12)\right]}{f_b} \qquad S_{reqd} = 50.157 \text{ in}^3$$

HP12x53 Grade 50 Use

Design of permanent facing

$$p_s := 75 \cdot \frac{lbf}{ft^2} \qquad p_e := 886.1 \cdot \frac{lbf}{ft^2} \qquad s := 8.0 \cdot ft$$

Determine maximum bending moment in facing
See Table 8.8, Strom and Ebeling 2002a

Determine service load moment demand

$$M_F := \left(\frac{1}{10}\right) \cdot \left(p_e + p_s\right) \cdot s^2 \qquad M_F = 6.151 \cdot 10^3 \circ lbf \cdot \frac{ft}{ft}$$ 6.15 ft-kips per foot of wall

$\Gamma_{EH} := 1.5$ AASHTO load factor for horizontal earth pressure load

$\phi_b := 0.90$ AASHTO strength reduction factor for bending

Determine nominal moment demand,
and reinforcement that will provide the required capacity
See Figure C-4 for facing reinforcement and terminology

$$M_{Nreq} := \left(\frac{\Gamma_{EH}}{\phi_b}\right) \cdot M_F \qquad M_{Nreq} = 1.025 \cdot 10^4 \circ lbf \cdot \frac{ft}{ft}$$

$t_F := 10 \cdot in$ $\quad d := 8 \cdot in$ $\quad A_s := 0.27 \cdot in^2$ No. 4 bars at 9", EF

$f_c := 4000 \cdot psi$ $\quad f_y := 60000 \cdot psi$ $\quad b := 12 \cdot in$

$$M_N := A_s \cdot f_y \cdot \left[d - \frac{\left(A_s \cdot f_y\right)}{1.7 \cdot f_c \cdot b}\right] \qquad M_N = 1.053 \cdot 10^4 \circ lbf \cdot ft$$ Provided - OKAY

Use yield line analysis to determine capacity of system
to withstand the loss of a single anchor
See Section 4.2.1 and Figure 4.3b

Flexural yield line analysis for permanent facing
Assume triangular distribution

$S_H := 16 \cdot ft$ Facing horizontal span at missing anchor
Two times the regular 8-foot spacing between anchors

$$w_u := \frac{(48 \cdot M_N)}{S_H^{\,2}} \qquad w_u = 1.975 \cdot 10^3 \, \frac{lbf}{ft}$$

Investigate the capacity of the HP12x53 Grade 50 soldier beams to
tranfer load from failed anchor to adjacent anchors.
See Section 4.2.5 and Figure 4.3c

Determine plastic moment capacity of soldier beams

Assume earth pressure loading has same distribution when soldier beams yield
as assumed for the original design

$Z_p := 74.0 \cdot in^3$ Plastic section modulus of HP12x53
AISC Manual of Steel Construction ASD

$F_y := 50000 \cdot psi$

$M_p := Z_p \cdot F_y \qquad M_p = 3.7 \cdot 10^6 \, in \cdot lbf$ Plastic moment capacity

Determine earth pressure required to develop
plastic moment capacity at Section A-A for cantilever span
See Section 4.2.5 and Figure 4.3c

$$p_c := \frac{M_p}{108.4 \, ft^3} \qquad p_c = 2.844 \cdot 10^3 \, \frac{lbf}{ft^2}$$

Determine maximum shear associated
flexural yielding at Section A-A for cantilever span

$V_c := p_c \cdot (2.5 \cdot ft) \cdot (8.0 \cdot ft) + p_c \cdot (2.5 \cdot ft) \cdot (8.0 \cdot ft) \qquad V_c = 1.138 \cdot 10^5 \, lbf$ (113.8 kips)

Determine earth pressure required to develop
plastic moment capacity at Section A-A for interior span
See Section 4.2.5 and Figure 4.3c

$$p_c := \frac{M_p}{60.5 \cdot ft^3} \qquad p_c = 5.096 \cdot 10^3 \circ \frac{lbf}{ft^2}$$

Determine maximum shear associated
flexural yielding at Section A-A for cantilever span

$$V_1 := p_c \cdot (5.5 \cdot ft) \cdot (8.0 \cdot ft) \qquad V_1 = 2.242 \cdot 10^5 \circ lbf$$ (224.2 kips)

Potential increase in load deliverable to upper anchors
supporting soldier beams on each side of the failed anchor
See Figure C-3

Upper anchor from cantilever

$$H_1 := 7.5 \cdot ft \qquad H_2 := 11.0 \cdot ft \qquad p_s := 75 \cdot \frac{lbf}{ft^2} \qquad p_e := 886.1 \cdot \frac{lbf}{ft^2}$$

$$\Delta T_{1C} := \frac{V_c}{8 \cdot ft} - \left(\frac{2}{3}\right) \cdot H_1 \cdot p_e - H_1 \cdot p_s \qquad \Delta T_{1C} = 9.229 \cdot 10^3 \circ \frac{lbf}{ft}$$ (9.23 kips / ft)

Upper anchor from interior span

$$\Delta T_{11} := \frac{V_1}{8 \cdot ft} - \left[\left(\frac{H_2}{2}\right) \cdot p_e - \left(\frac{H_2}{2}\right) \cdot p_s\right] \qquad \Delta T_{11} = 2.357 \cdot 10^4 \circ \frac{lbf}{ft}$$ (23.6 kips / ft)

Soldier beams have more than enough capacity to deliver
failed anchor load to adjacent anchors

Determine nominal shear capacity of facing

$f_c := 4000$ $\quad b := 12 \cdot in$ $\quad d := 8 \cdot in$

Nominal shear capacity per foot of facing

$V_N := 2\sqrt{f_c} \cdot psi \cdot (b \cdot d) \cdot \frac{1}{ft}$ $\quad V_N = 1.214 \cdot 10^4 \frac{lbf}{ft}$

Determine stud capacity

Stud Punching (5/8 - inch diameter studs x 4 inches long)

$t_h := 0.3125 \cdot in$ $\quad l_e := 4.0 \cdot in - t_h$ $\quad l_e = 3.688 \, in$ $\quad d_h := 1.25 \cdot in$

$A_O := \sqrt{2} \cdot l_e \cdot \pi \cdot (l_e + d_h)$ $\quad A_O = 80.892 \, in^2$

Determine stud capacity in punching

$T_{SP} := 2.67 \cdot \left(\sqrt{f_c} \cdot psi\right) \cdot A_O$ $\quad T_{SP} = 1.366 \cdot 10^4 \, lbf$

Stud Tension (5/8 - inch diameter studs)

$d_s := 0.625 \cdot in$ $\quad F_U := 60000 \cdot psi$

$A_{stud} := \frac{\pi \cdot (d_s)^2}{4}$ $\quad A_{stud} = 0.307 \, in^2$

Determine stud capacity in tension

$T_{ST} := A_{stud} \cdot F_U$ $\quad T_{ST} = 1.841 \cdot 10^4 \, lbf$

Allowable capacity of 1/2-inch diameter studs - punching

$T_{SPA} := 0.67 \cdot T_{SP}$ $\qquad T_{SPA} = 9.152 \cdot 10^{3} \cdot lbf$ $\qquad$ Governs

Allowable capacity of 1/2-inch diameter studs - tension

$T_{STA} := 0.5 \cdot T_{ST}$ $\qquad T_{STA} = 9.204 \cdot 10^{3} \cdot lbf$

By tributary area method determine stud reaction per foot of facing
Assume all tieback anchors carry load, i.e. no loss of anchor.

$p_s := 75 \cdot \frac{lbf}{ft^2}$ $\qquad p_e := 886.1 \cdot \frac{lbf}{ft^2}$ $\qquad s := 8.0 \cdot ft$

$R_{stud} := \left(p_s + p_e\right) \cdot 8 \cdot ft$ $\qquad R_{stud} = 7.689 \cdot 10^{3} \cdot \frac{lbf}{ft}$

Determine stud spacing

$S_{stud} := \frac{T_{SPA}}{R_{stud}}$ $\qquad S_{stud} = 14.284 \cdot in$ $\qquad$ Space studs at 12-inches on center

By tributary area method determine stud reaction per foot of facing
Assume loss of single anchor condition

$R_{stud} := \left(p_s + p_e\right) \cdot 12 \cdot ft$ $\qquad R_{stud} = 1.153 \cdot 10^{4} \cdot \frac{lbf}{ft}$

Capacity of stud

$S_{stud} := 12 \cdot in$

$R_{cap} := \frac{T_{SP}}{S_{stud}}$ $\qquad R_{cap} = 1.366 \cdot 10^{4} \cdot \frac{lbf}{ft}$

Stud capacity greater than demand due to loss of single anchor condition

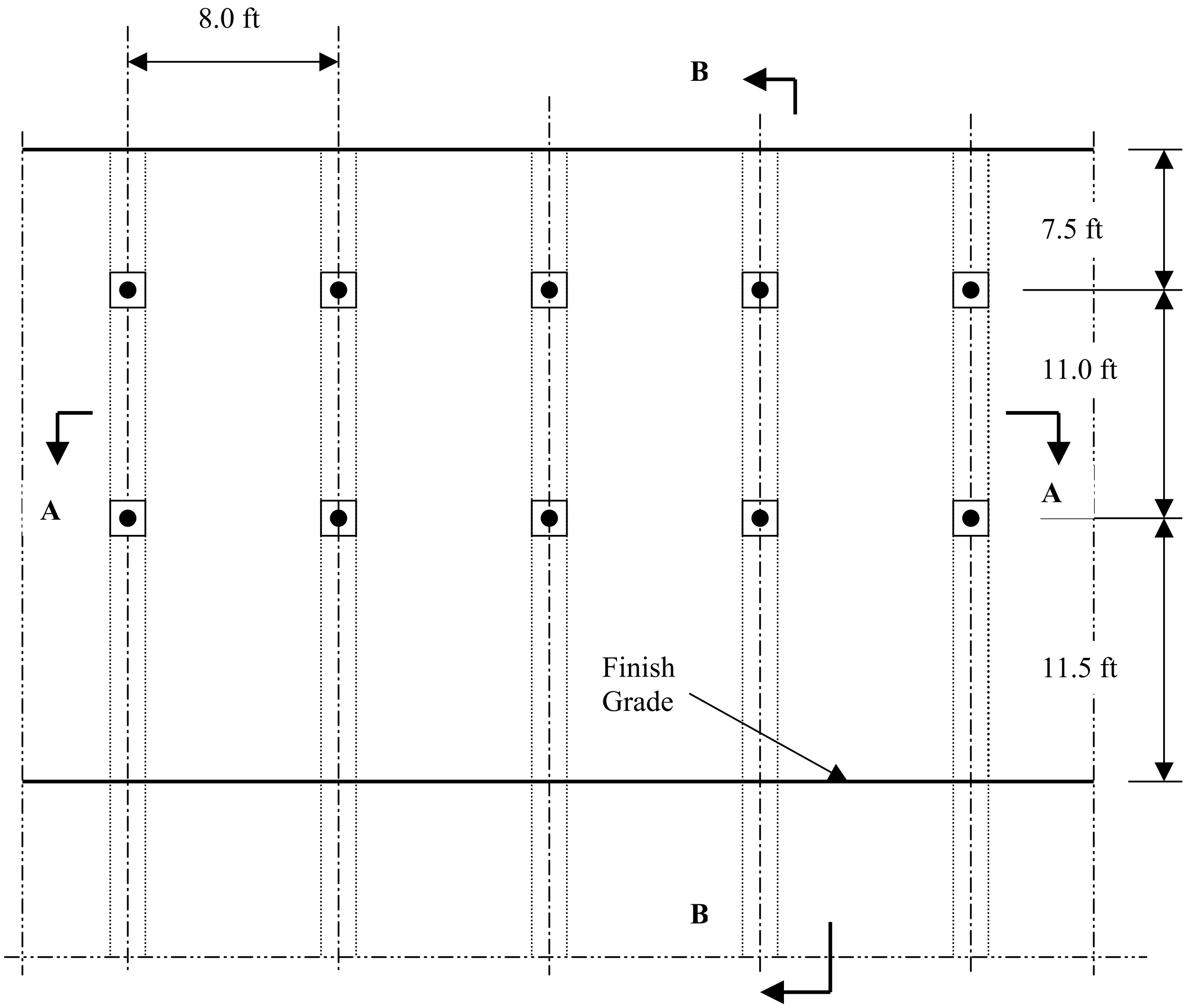


Figure A.1 Soldier beams with concrete facing—elevation view

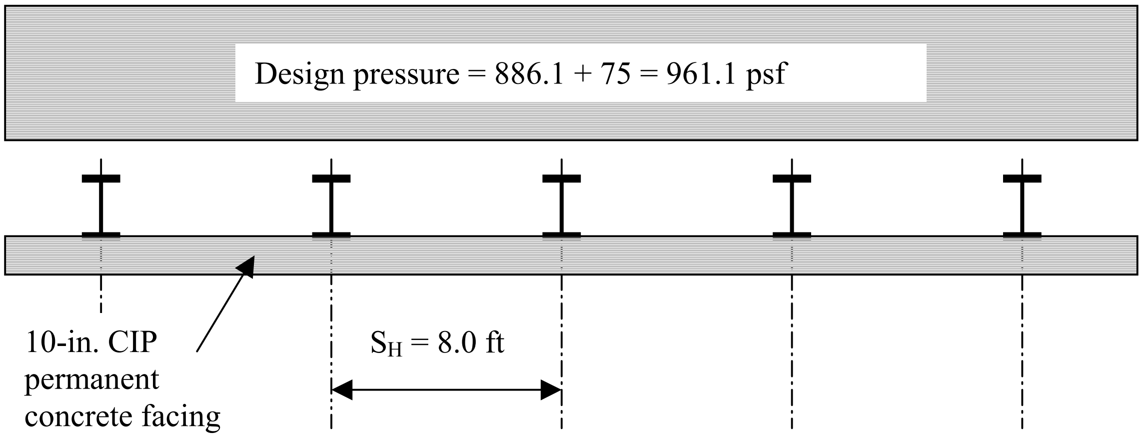


Figure A.2 Section A-A—yield line analysis for facing

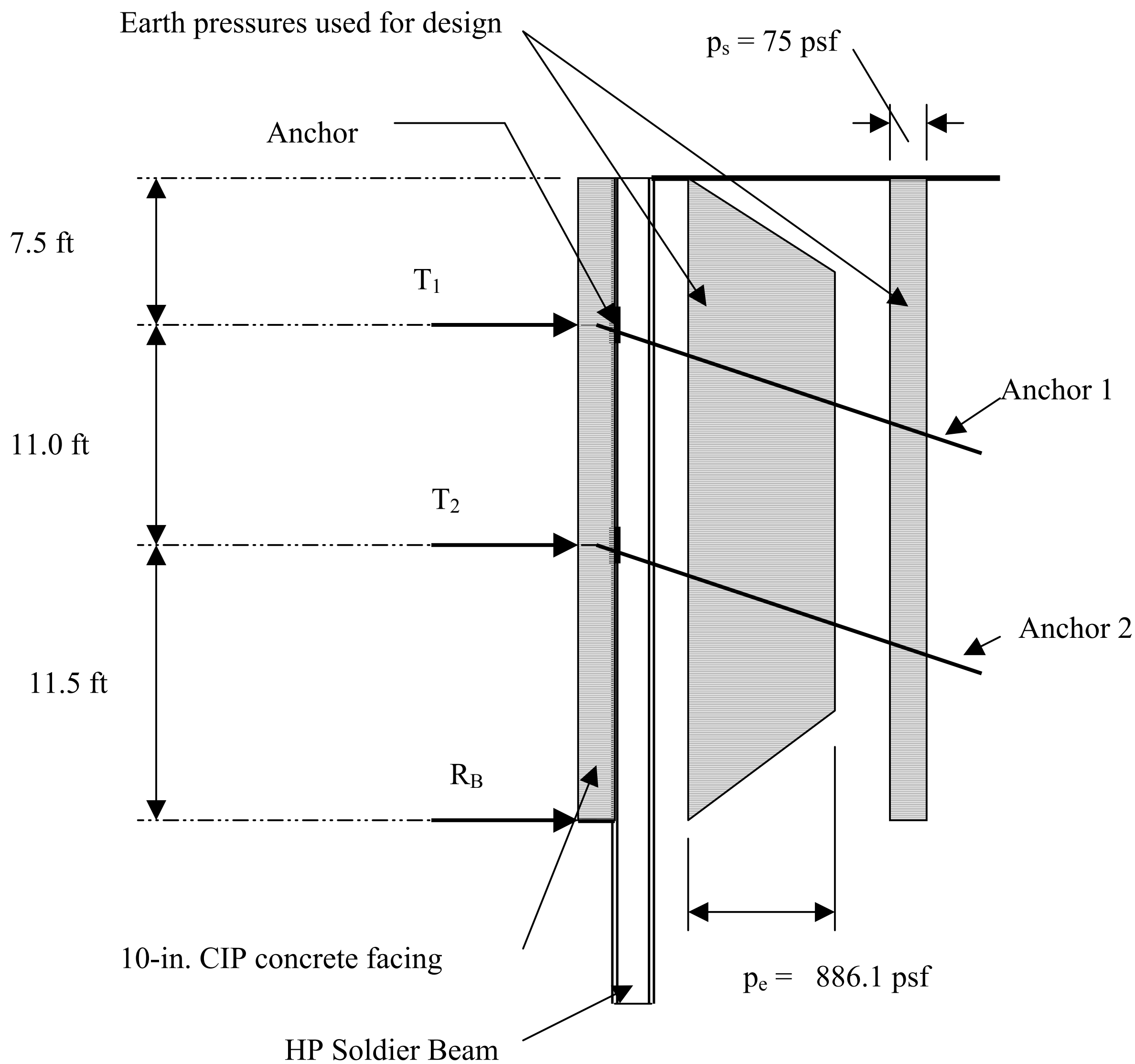
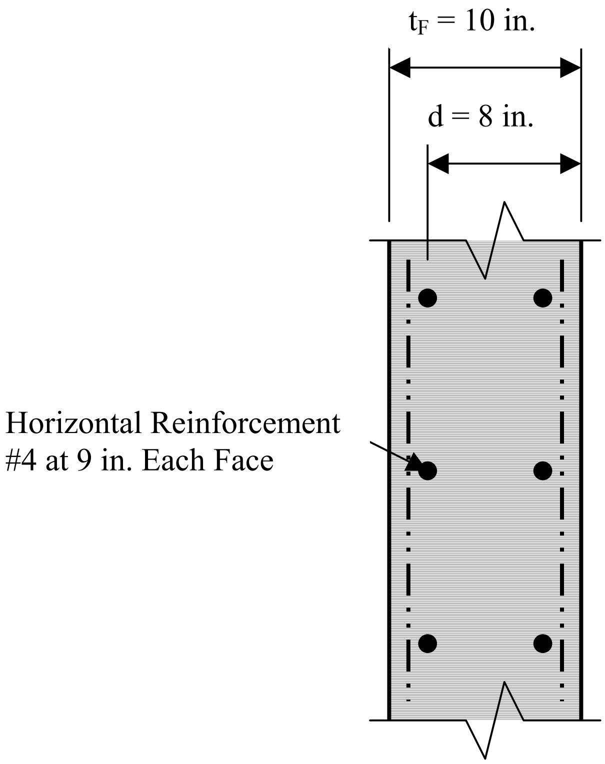


Figure A.3 Section B-B



Nomenclature used for facing design

f'_c =	Concrete compressive strength
f_y =	Yield strength of reinforcement
t_F =	Facing thickness
d =	Depth to reinforcing steel
A_s =	Reinforcing steel area
ϕ_b =	Strength reduction factor for bending
Γ_{EH} =	Load factor for horizontal earth pressure
M_F =	Facing service load moment demand
M_{Nreq} =	Required nominal moment capacity
M_N =	Nominal moment capacity provided
b =	Facing section width

Figure A.4a Vertical section through CIP facing

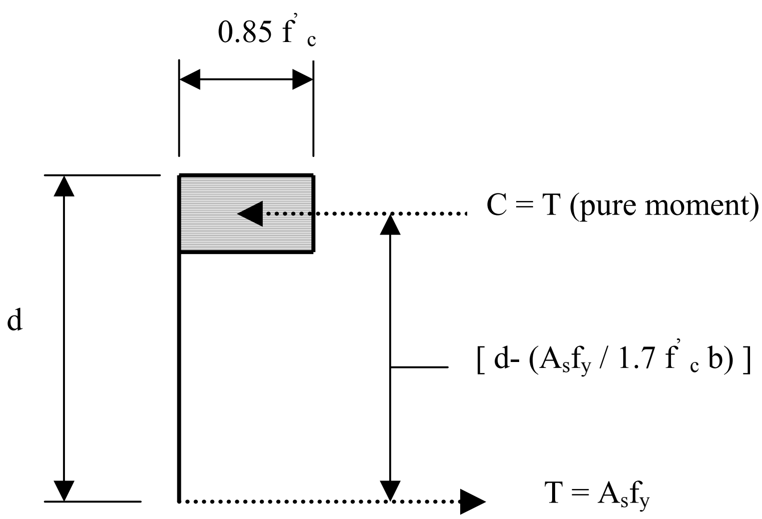


Figure A.4b Facing equivalent stresses and forces for nominal moment calculations

Appendix B
Internal Stability, 30-ft-High Wall–Retained Soil Dry

B.1 General

General-purpose slope stability (GPSS) programs can used to evaluate the internal stability of a single anchor wall for the original design condition and for the single failed anchor condition required by Corps practice. The 30-ft-high wall of FHWA-RD-97-130 (“Design Manual for Permanent Ground Anchor Walls”) is used to demonstrate both simple analysis and GPSS analysis procedures (see paragraph 4.3.2 of FHWA-RD-97-130) for conditions where the soil retained by the wall is dry. The results from the GPSS software CSLIDE and UTEXAS4 (Wright 2000) are compared with the results that would be obtained using the Terzaghi and Peck (1996) apparent earth pressure diagram. The results are also compared with the results obtained from the simple force equilibrium method of FHWA-RD-98-065 (also referred to in this appendix as Long et al. 1998). The 30-ft-high wall used for the evaluation is illustrated in Figure B.1. The mobilized friction angle for each limiting equilibrium analysis is equal to $\tan^{-1} \phi_{mob} = \tan \phi / FS$, where FS is the factor of safety.

B. 2 Internal Stability Analyses (adapted from Long et al. 1998)

Internal stability analyses are used to determine the total force required (P_{reqd}) to provide tieback wall system equilibrium under conditions where a factor of safety of 1.3 (“safety with economy design”) is applied to the shear strength of the soil. P_{reqd} is equal to the mobilized soil force (P_{soil}) or equal to the sum of the lateral resistances provided by the tieback anchor (P_{tie}) and the wall toe (P_{LL}). The resistance to vertical movement (P_V) that is provided by wall end bearing is assumed equal and opposite to the vertical component of the anchor load. This is illustrated in Figure B.2 for an assumed failure surface passing between the wall toe and the bottom of the excavation.

Referencing the free-body diagram in Figure B.2, if the retained soil mass moves downward with respect to the tieback wall, downdrag forces result, and the load exerted by the soil (P_{soil}) is angled downward (at an angle $+\delta$). If the wall components settle more than the soil mass, P_{soil} is directed upward (at an angle $-\delta$). The portion of the wall below the failure surface provides resistance to lateral and vertical movement of the tieback wall. Resistance to lateral movement is quantified as P_{LL}, while resistance to vertical movement is provided by P_V. The specific magnitude, orientation, and location of the force P_{soil} (retained soil side) depends on such details as the lateral load (P_{LL}), the relative movement between the wall and the soil, and the anchor force and inclination. The diagram illustrates the stability in terms of the soil force, P_{soil}, and in terms of the individual forces P_{tie}, P_{LL}, and P_V, applied to the soil anchor and structural components below the failure plane. The result of the individual forces (P_{tie}, P_{LL}, and P_V) equals P_{soil}. The result of the individual forces is termed P_{reqd} when the soil force is based on mobilized shear strength parameters.

Since the embedded portion of a tieback wall has a vertical capacity greater than the applied vertical load, the wall will settle less than the retained soil, and the resultant P_{soil} will be orientated at $+\delta$. If the effects of wall friction are neglected (a conservative approach), P_{soil} is horizontal (i.e., $\delta = 0$). The elevation at which P_{soil} acts is dependent on the interaction between the wall and soil mass. Based on estimates from full-scale measurements for deep braced cuts in sand, the elevation of P_{soil} was found to be on the order of 50 percent of the wall height (Terzaghi and Peck 1996, page 352).

B.3 Terzaghi and Peck Apparent Earth Pressure Diagram

A mobilized friction angle (ϕ_{mob}) for use in limiting equilibrium analyses can be determined by equating the total earth pressure load determined by classical earth pressure methods to the apparent earth pressure load for sands per Terzaghi and Peck. The ϕ_{mob} value is one that can used in limiting equilibrium analyses to obtain a total load (P_{Reqd}) equal to that produced by the Terzaghi and Peck apparent pressure diagram.

By Rankine classical earth pressure methods

$$P_{\mathrm{Re}qd} = 0.50 k_{amob} \gamma H^2 \tag{B.1}$$

By Terzaghi and Peck

$$P_{\mathrm{Re}qd} = 0.65 k_a \gamma H^2 \tag{B.2}$$

By setting Equation B.1 equal to Equation B.2 (i.e., 0.50 k_{amob} = 0.65 k_a), ϕ_{mob} can be determined:

$$\tan^2(45 - \frac{\phi_{mob}}{2}) = 1.3 \tan^2(45 - \frac{\phi}{2})$$

And

$$\phi_{mob} = 2\left\{\left[45^o - \tan^{-1}\left(\sqrt{1.3}\tan\left(45 - \frac{\phi}{2}\right)\right)\right]\right\} \quad \text{(see Eq. 4.12 of FHWA-RD-97-130)}$$

The above equation is solved below for ϕ_{mob} for an actual soil friction angle ϕ equal to 30 deg. The value obtained for ϕ_{mob} is 22.288 deg. This results in a factor of safety of 1.341 applied to the shear strength of the soil.

$$\phi := 30 \cdot deg$$

$$\phi_{mob} := 2 \cdot \left(45 \cdot deg - atan\left(\sqrt{1.3} \cdot tan\left(45 \cdot deg - \frac{\phi}{2}\right)\right)\right)$$

$$\phi_{mob} = 23.288 \circ deg$$

$$SF := \frac{tan(\phi)}{tan(\phi_{mob})} \qquad SF = 1.341$$

B.4 FHWA-RD-98-065 (Long et al. 1998) Simple Force Equilibrium Method

Using a factor of safety of 1.3 applied to the shear strength of the soil, the simple internal stability analysis procedure described in FHWA-RD-98-065 (Long et al. 1998) is used to determine the force required to provide internal stability to the 30-ft-high wall. The Mathcad calculations follow:

Long, et al, 1998 Internal Stability
Determine (P_{reqd}) the force required to provide stability
to the vertical cut using limiting equilibrium methods

File: BAA 3 \ Internal Stability 2 July 4, 2002

Equation 3.19, FHWA-RD-98-065

$\gamma_{moist} := 115 \cdot \frac{lbf}{ft^3}$ $H := 30 \cdot ft$ $\phi := 30 \cdot deg$

$\gamma_{sat} := 134.4 \cdot \frac{lbf}{ft^3}$ $\gamma_{w} := 62.4 \cdot \frac{lbf}{ft^3}$

$\gamma_{bouy} := \gamma_{sat} - \gamma_{w}$ $\gamma_{bouy} = 72 \circ \frac{lbf}{ft^3}$

$SF := 1.3$ Safety Factor = 1.3 applied to shear strength of soil

Assume mobilized interface friction (δ_{mob}) is equal
to the mobilized internal friction angle (ϕ_{mob})

$\phi_{mob} := atan\left(\frac{tan(\phi)}{SF}\right)$ $\phi_{mob} = 23.947 \circ deg$

From Figure 27, FHWA-RD-98-065, for mobilized $-\delta$ / ϕ = -1, K_{pmob} = 4.0

$K_{pmob} := 4.0$ $\delta_{mob} := 23.941 \cdot deg$ $\beta := 0 \cdot deg$

Try different values for the failure surface angle (α) and for the embedment depth to height ratio (ξ) to find the maximum value for P_{reqd}

Use Table 4 of FHWA-RD-98-065 to find the proper range

$\alpha := 54 \cdot deg, 55 \cdot deg .. 60 \cdot deg$

Try $\xi := 0.09$

$$P_{reqd}(\alpha) := \frac{1}{2} \cdot \gamma_{moist} \cdot H^2 \left[\frac{(1+\xi)^2}{\tan(\alpha) - \tan(\beta)} - K_{pmob} \cdot \xi^2 \left(\sin(\delta_{mob}) + \frac{\cos(\delta_{mob})}{\tan(\alpha - \phi_{mob})} \right) \right] \cdot \tan(\alpha - \phi_{mob})$$

$P_{reqd}(\alpha) =$ (lbf/ft)

$2.392 \cdot 10^4$
$2.398 \cdot 10^4$
$2.401 \cdot 10^4$
$2.401 \cdot 10^4$
$2.397 \cdot 10^4$
$2.391 \cdot 10^4$
$2.381 \cdot 10^4$

Try $\xi := 0.11$

$$P_{reqd}(\alpha) := \frac{1}{2} \cdot \gamma_{moist} \cdot H^2 \left[\frac{(1+\xi)^2}{\tan(\alpha) - \tan(\beta)} - K_{pmob} \cdot \xi^2 \left(\sin(\delta_{mob}) + \frac{\cos(\delta_{mob})}{\tan(\alpha - \phi_{mob})} \right) \right] \cdot \tan(\alpha - \phi_{mob})$$

$P_{reqd}(\alpha) =$ (lbf/ft)

$2.393 \cdot 10^4$
$2.398 \cdot 10^4$
$2.4 \cdot 10^4$
$2.399 \cdot 10^4$
$2.395 \cdot 10^4$
$2.388 \cdot 10^4$
$2.377 \cdot 10^4$

Try $\xi := 0.10$

$$P_{reqd}(\alpha) := \frac{1}{2}\cdot\gamma_{moist}\cdot H^2\left[\frac{(1+\xi)^2}{\tan(\alpha)-\tan(\beta)} - K_{pmob}\cdot\xi^2\left(\sin\left(\delta_{mob}\right)+\frac{\cos\left(\delta_{mob}\right)}{\tan\left(\alpha-\phi_{mob}\right)}\right)\right]\cdot\tan\left(\alpha-\phi_{mob}\right)$$

$P_{reqd}(\alpha) =$ $\frac{lbf}{ft}$

$2.394\cdot10^4$
$2.4\cdot10^4$
$2.403\cdot10^4$
$2.402\cdot10^4$
$2.399\cdot10^4$
$2.392\cdot10^4$
$2.381\cdot10^4$

Maximum total lateral load
required to stabilize cut = 24,030 lb.
with angle of surface failure (α) = 56 deg.
and depth of failure surface = 0.10 (30) = 3.0 feet
Embedment depth to height ratio (ξ) equals 0.10

B.5 CSLIDE

The Corps program CSLIDE can be used to assess the stability of a tieback wall system. It is based on the equations of horizontal and vertical equilibrium applied to the soil wedges. It does not include the equation of moment equilibrium between wedges. CSLIDE can accommodate water loads, surcharge loads, and layered soil systems. Since there is no interaction of vertical shear force effects between wedges, the passive resistance must act horizontally (i.e., $\delta_{mob} = 0$) rather than at an angle $\delta_{mob} > 0$. This will result in a conservative factor of safety. Also, the CSLIDE program satisfies force equilibrium only. Moment equilibrium is not considered.

The use of the CSLIDE program is demonstrated below with respect to the 30-ft wall example. In this case, various values of ξ (i.e., embedment depth, d, to height, H, ratio) are used to determine the maximum lateral force needed to provide equilibrium to the system, considering a factor of safety of 1.3 is applied to the shear strength of the soil (see Figure B.3).

The maximum lateral force determined by CSLIDE analysis represents the force that must be provided by the tiebacks to achieve system equilibrium. The maximum lateral force occurred at an embedment depth to height ratio (ξ) equal to 0.200. The CSLIDE results for this ξ-value are provided below. By inputting into CSLIDE a factor of safety of 1.3 for both the upper and lower bounds, the lateral force needed to achieve system equilibrium is output (see CSLIDE output below).

CSLIDE Input **ξ= d / H = 0.200**

```
10010 TITL 30 FOOT WALL FILE: WAL1.IN
10020 STRU  4      .150
10030        -1.00       -36.00
10040        -1.00         0.00
10050         0.00         0.00
10060         0.00       -36.00
10090 SOLT  1  1       30.00      0.00      0.115       0.00
10095      -100.00       0.00
10110 SORT  1  1       30.00      0.00      0.115       -30.00
10120       150.00       -30.00
10130 SOST        0.00     0.00
10135 WATR       -40.00   -40.00     0.0625
10140 METH  2
10160 FACT        1.3        1.3      1.00
10200 END
```

```
 -----------------------
   STATIONARY SOLUTION
 -----------------------
 30 FOOT WALL FILE: WAL1.IN    OUTPUT FILE: WAL100

 MULTIPLE FAILURE PLANE ANALYSIS
```

WEDGE NUMBER	FAILURE ANGLE (DEG)	TOTAL LENGTH (FT)	WEIGHT OF WEDGE (KIPS)	SUBMERGED LENGTH (FT)	UPLIFT FORCE (KIPS)
1	-57.0	42.938	48.443	.000	.000
2	.000	1.000	5.400	.000	.000
3	33.0	11.009	3.184	.000	.000

WEDGE NUMBER	NET FORCE ON WEDGE (KIPS)
1	-31.491
2	.000
3	4.898

```
SUM OF FORCES ON SYSTEM ---   -26.593 ←  Lateral force needed to
                                         achieve system equilibrium

 FACTOR OF SAFETY ----------  1.300


 * NOTE *   THE SOLUTION HAS NOT CONVERGED.
```

B.6 UTEXAS4 Analyses for Dry Conditions

The results from the UTEXAS4 program for partially submerged conditions are presented below. The Corps' Modified Swedish force equilibrium procedure (Simplified Janbu Procedure) was selected for cohesionless soils in accordance with the recommendation of FHWA-RD-97-130 (paragraph 4.4.3). The failure plane was selected to pass just below the bottom of the wall and the side force inclination set at 0 deg (i.e., horizontal). The bottom of the wall was set at elevation -36.0 per the CSLIDE analysis results. Wright (1999) indicates that the Simplified Janbu Procedure usually tends to underestimate the factor of safety and therefore is not recommended. Various anchor force values were tried until a factor of safety equal to 1.3 was obtained.

Input

```
GRA
HEA
INTERNAL STABILITY 30-FOOT HIGH WALL  FILE: TBW6.DAT
SINGLE ANCHOR - FAILURE PLANE THROUGH ANCHOR
RETAINED SOIL DRY

PRO
     1 1 Cohesionless retained soil
          -100  0
             0  0

     2 2 Concrete tieback wall
              0 0
              1 0

     3 1 Cohesionless material below finish grade
              0 -30
             50 -30


MAT
     1 Cohesionless soil
          115 = unit weight
          Conventional shear strengths
            0 30
          No pore water pressures
     2 Concrete
          145 = unit weight
          Very strong

REINFORCEMENT LINES
       1 0 1
     -40 -18.0 0     0
     -28 -15.6 26000 0     Note: The anchor forces in "bold" were varied
       0 -10.0 26000 0     until a factor of safety equal to 1.3 was obtained


LAB
INTERNAL STABILITY ANALYSIS - SINGLE ANCHOR
ANA
```

```
     Noncircular Search
     -20.00   0.00
       0.00 -36.00  FIX
      10.00 -30.00

       5.00   1.00
PRO
   C = Corps of Engineers Modified Swedish Procedure
   0
SAV
   4

COM
```

Output

```
Procedure of Analysis: Corps of Engineers' Modified Swedish
Specified side force inclination: 0.00
Will save the following number of shear surfaces with the lowest
factors of safety:4

TABLE NO. 41
****************************************
*  Critical Noncircular Shear Surface  *
****************************************

***** CRITICAL NONCIRCULAR SHEAR SURFACE *****
X:     -20.27     Y:       0.00
X:       0.00     Y:     -36.00
X:      18.56     Y:     -30.00

Minimum factor of safety: 1.303
Side force inclination: 0.00


TABLE NO. 57
**********************************************************************
**
* Check of Computations by Force Equilibrium Procedure (Results are for
*
* the critical shear surface in the case of an automatic search.)
*
**********************************************************************
**

Summation of Horizontal Forces: 1.97757e-011

Summation of Vertical Forces: 1.16529e-011

Mohr Coulomb Shear Force/Shear Strength Check Summation: 5.22538e-004
```

UTEXAS4 critical failure plane results for the dry condition are illustrated in Figure B.4.

B.7 Results Summary

Table B.1 Results Summary for Dry Condition

Calculation Method	P_{reqd} [1] (lb/ft)	FS on Soil Strength	Angle of Failure Surface α (deg)	Embedment Depth to Height Ratio (ε)	Embedment Depth (ft)
Apparent Pressure	22,400	1.34	56.7	NA	NA
Simple Force Eq.	24,030	1.30	56.0	0.10	3.0
CSLIDE	26,593	1.30	57.0	0.20	6.0
UTEXAS4	26,000	1.30	60.6	0.20	6.0 [2]

Notes: (1) P_{reqd} = Force required to stabilize the cut
(2) CSLIDE embedment depth used in UTEXAS4 GPSS analysis

The apparent pressure diagram approach assumes that the failure plane occurs at final grade. Since the other calculation methods determine P_{reqd} based on the most critical depth, the results from those methods will be somewhat more conservative. The passive earth pressures used in the Simple Force Equilibrium Method are based on the log spiral method and consider a mobilized interface friction angle that is equal to the mobilized angle of internal friction. The earth pressures used in CSLIDE are based on Rankine soil pressure theory and do not account for the interface friction that can be mobilized along the wall face. For this reason, the embedment depth producing the largest P_{reqd} is greater, and the actual P_{reqd} is greater than that determined by the Simple Force Equilibrium Method.

The UTEXAS4 analysis used the Corps of Engineers' Modified Swedish Procedure (Simple Janbu Procedure) with the side force inclination assumed as zero. Because of this, and since the embedment depth in the UTEXAS4 was the same as that used in the CSLIDE analysis, the results obtained for P_{reqd} were similar to those obtained from the CSLIDE analysis. The results for the UTEXAS4 solution are plotted in Figure B.4.

The designer of the above tieback wall system should, for this particular tieback wall system, ensure that the tiebacks under "loss of single anchor" conditions have a capacity approximately equal to 26,000 lb per foot of wall to stabilize the cut and satisfy internal stability requirements.

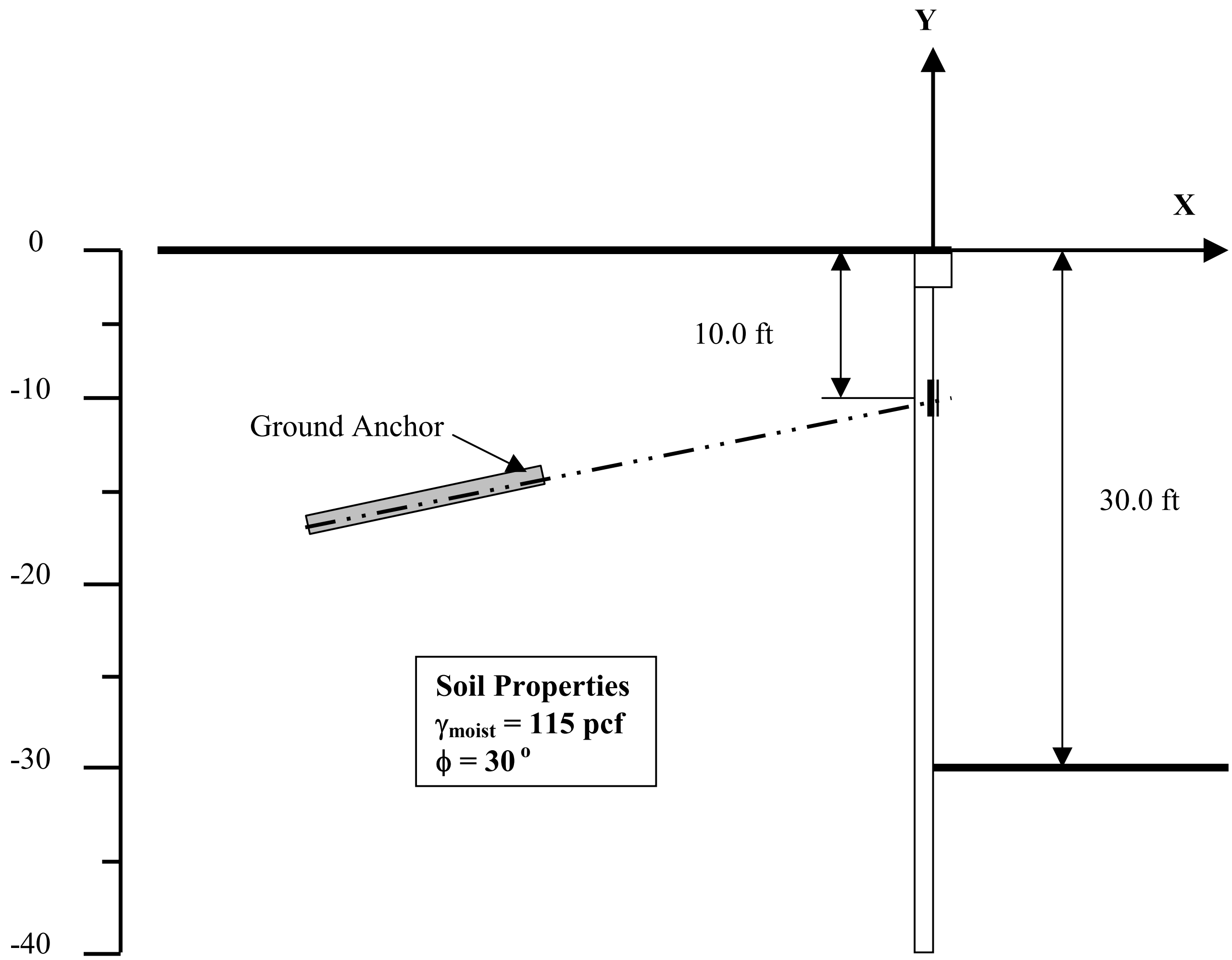


Figure B.1 30-ft-high wall--elevation at final excavation stage

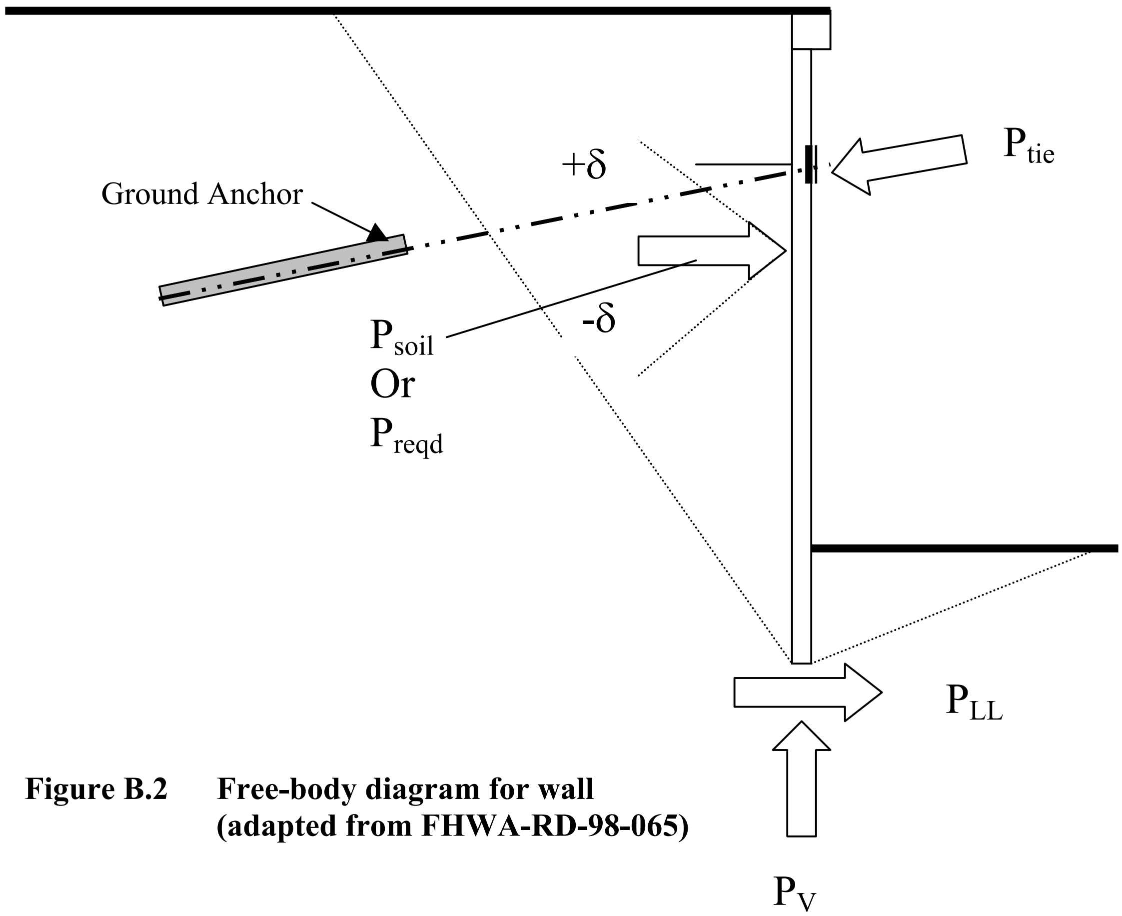


Figure B.2 Free-body diagram for wall (adapted from FHWA-RD-98-065)

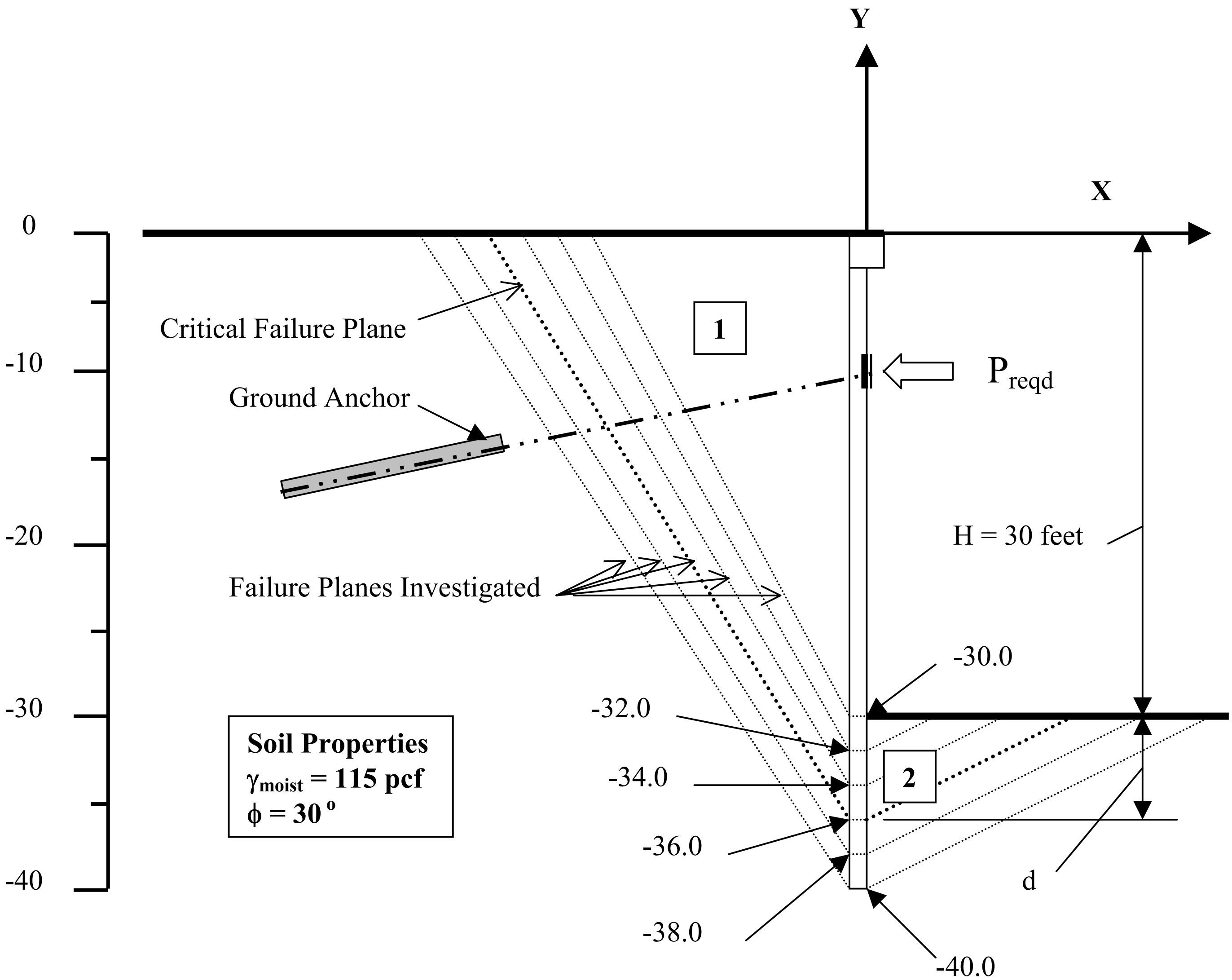


Figure B.3 30-ft-high wall--
CSLIDE failure plane analysis

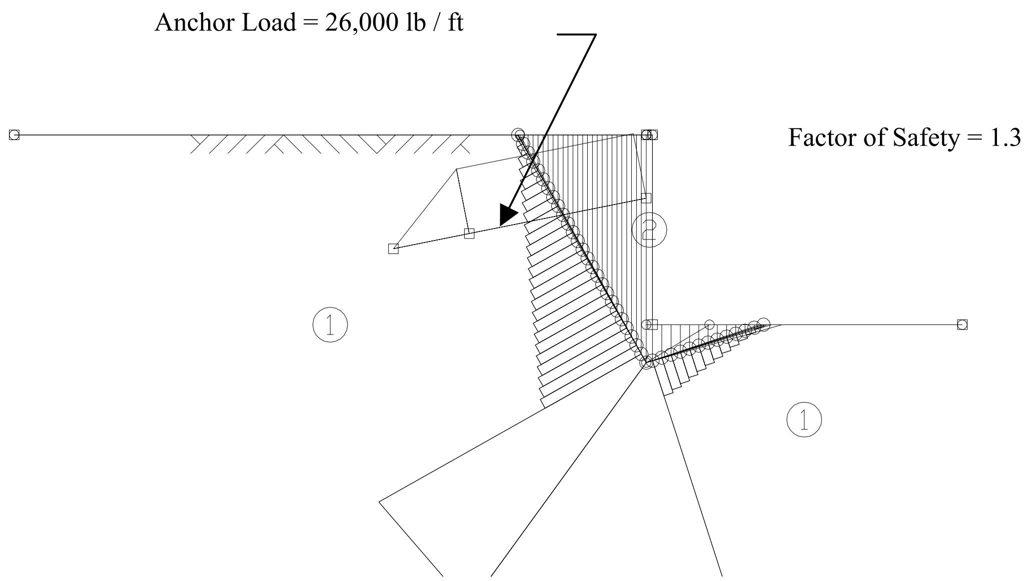


Figure B.4 **30-ft-high wall– UTEXAS4 results, dry conditions**

Appendix C
Internal Stability, 30-ft-High Wall–Partially Submerged

C.1 General

The 30-ft-high wall used in Appendix B is also used to evaluate partially submerged conditions in the retained soil. The partially submerged condition with a hydrostatic water table in the retained soil side (only) is illustrated in Figure C.1. The general-purpose slope stability programs CSLIDE and UTEXAS4 (Wright 1999) are used for the analysis, with the results summarized in Section C.4.

The assumptions with respect to the partially submerged condition are that there is no seepage through or around the wall (i.e., the wall is impervious and the substrate into which the wall is embedded is impervious).

For those cases where the substrate is pervious, the designer may wish to consider the effect of seepage on net water pressures. This is generally accomplished by flow net analysis. For information regarding flow net construction and seepage analysis, designers should refer to FHWA-SA-99-015 (paragraph 5.2.9) and Terzaghi, Peck, and Mesri (1996, pp 214-18).

When seepage occurs there is always the potential for piping. A discussion on the mechanics of piping, and methods used to calculate a factor of safety with respect to piping failure can be found in Terzaghi, Peck, and Mesri (1996, pp 222-23). Upward seepage on the excavated side of the tieback wall can also lead to bottom blowout, boiling, and heave. These problems can be aggravated where pervious strata underlie impervious soils at finish grade. Additional information on piping, bottom blowout, boiling, and heave, as well as measures used to protect against these types of failure, can be found in ASCE (1997).

It is not uncommon for designers to increase wall embedment to provide ample factors of safety against seepage-related failures. Information on factors of safety commonly used by designers to protect against piping, blowout, and boils can be found in Cedergren (1977).

C.2 CSLIDE Analysis Results

C.2.1 Partially submerged conditions

The results from the CSLIDE program for partially submerged conditions in the retained soil side are presented below. Recall that CSLIDE assumes that the interface friction angle is equal to zero. By inputting into CSLIDE a factor of safety of 1.3 for both the upper and lower bounds, the lateral force needed to achieve system equilibrium is output.

For this and the following UTEXAS4 analyses, the toe of the wall is assumed to extend 6 ft below finish grade (that is, el -36).

CSLIDE Input

```
10010 TITL 30 FT WALL HALF SUBMERGED FILE: WAL3.IN OUTPUT FILE: WAL300
10020 STRU  4      .150
10030        -1.00       -36.00
10040        -1.00         0.00
10050         0.00         0.00
10060         0.00       -36.00
10090 SOLT  1  1       30.00      0.00      0.115       0.00
10095      -100.00       0.00
10100 SOLT  2  1       30.00      0.00      0.1344     -18.00
10105      -100.00      -18.00
10110 SORT  1  1       30.00      0.00      0.115       -30.00
10120       150.00        -30.00
10130 SOST          0.00     0.00
10135 WATR        -18.00   -36.00     0.0625   0
10140 METH  2
10160 FACT          1.3         1.3      1.00
10200 END
```

CSLIDE Output

```
 -----------------------
   STATIONARY SOLUTION
 -----------------------
 30 FT WALL HALF SUBMERGED FILE: WAL3.IN

 MULTIPLE FAILURE PLANE ANALYSIS

 HYDROSTATIC WATER FORCE COMPUTED FOR WEDGES

 WEDGE     FAILURE       TOTAL       WEIGHT     SUBMERGED       UPLIFT
 NUMBER     ANGLE        LENGTH     OF WEDGE     LENGTH          FORCE
            (DEG)         (FT)       (KIPS)       (FT)          (KIPS)
 ---------------------------------------------------------------------
   1        -57.0       21.469       12.111         .000          .000
   2        -57.0       21.469       38.375       21.469        12.076
   3         .000        1.000        5.400        1.000          .563
   4         33.0       11.009        3.184         .000          .000

     WEDGE      NET  FORCE
     NUMBER      ON WEDGE
                  (KIPS)
 -------------------------

       1         -7.873
       2        -30.793
       3           .000
       4          4.898

 SUM OF FORCES ON SYSTEM ----    -33.767 ← Lateral force needed to
                                          achieve system equilibrium
FACTOR OF SAFETY -----------      1.300
```

```
 * NOTE *  THE SOLUTION HAS NOT CONVERGED.
```

C.2.2 Independent check of CSLIDE partially submerged condition results

By the simple resolution of forces on the "active" and "passive" failure planes (i.e., on the retained side and excavated side, respectively), a check of the CSLIDE results can be made. Forces are resolved as illustrated in Figure C.2. The calculations for the CSLIDE check are provided below. For the partially submerged condition, an effective weight for the partially submerged backfill on the retained soil side is determined in accordance with the procedures described in Ebeling and Morrison (1992, Figure 4.13). The resolution of forces on the active and passive failure planes is illustrated in Figure 3.4 of the same report.

Check of CSLIDE results for 30 Foot High Tieback Wall

FILE: CSLIDE Check 2A July 10, 2002

$SF := 1.3$ Factor of safety

$\phi := 30 \cdot deg$ $\quad \phi_{mob} := atan\left(\frac{\tan(\phi)}{SF}\right)$ $\quad \phi_{mob} = 23.947 \circ deg$

Half submerged retained soil condition

$\gamma_{sat} := 134.4 \cdot \frac{lbf}{ft^2}$ $\quad \gamma_w := 62.4 \cdot \frac{lbf}{ft^2}$ $\quad \gamma_{bouy} := \gamma_{sat} - \gamma_w$

$\gamma_{bouy} = 72 \circ \frac{lbf}{ft^2}$ $\quad \gamma_{moist} := 115 \cdot \frac{lbf}{ft^2}$

$H_1 := 36 \cdot ft$ $\quad H_{T1} := 18.0 \cdot ft$ $\quad H_{B1} := 18.0 \cdot ft$

$\gamma_e := \left(\frac{H_{B1}}{H_1}\right)^2 \cdot \gamma_{bouy} + \left[1 - \left(\frac{H_{B1}}{H_1}\right)^2\right] \gamma_{moist}$ $\quad \gamma_e = 104.25 \circ \frac{lbf}{ft^2}$

See Figure 4.13 Waterfront Retaining Wall Manual

For left side "active" wedge (CSLIDE wedges 1 + 2 on the retained side)

$$\alpha_1 := 45 \cdot deg + \frac{\phi_{mob}}{2} \qquad \alpha_1 = 56.973 \cdot deg$$

$$W_1 := \left[\left(\frac{H_1}{2} \right) \cdot \frac{H_1}{\tan(\alpha_1)} \right] \cdot \gamma_e \qquad W_1 = 4.391 \cdot 10^4 \cdot lbf$$

$$F_{h1} := \frac{W_1}{\tan(\alpha_1)} \qquad F_{h1} = 2.855 \cdot 10^4 \cdot lbf$$

$$F_{w1} := \gamma_w \cdot \left(\frac{H_{B1}^{\,2}}{2} \right) \qquad F_{w1} = 1.011 \cdot 10^4 \cdot lbf$$

$F_1 := F_{h1} + F_{w1}$ $\qquad F_1 = 3.866 \cdot 10^4 \cdot lbf$ Checks with CSLIDE

For wedge 2 on the excavated side

$$H_2 := 6.0 \cdot ft$$

$$\alpha_2 := 45 \cdot deg - \frac{\phi_{mob}}{2} \qquad \alpha_2 = 33.027 \cdot deg$$

$$W_2 := \left[\left(\frac{H_2}{2} \right) \cdot \frac{H_2}{\tan(\alpha_2)} \right] \cdot \gamma_{moist} \qquad W_2 = 3.184 \cdot 10^3 \cdot lbf$$

$$F_{h2} := \frac{W_2}{\tan(\alpha_2)} \qquad F_{h2} = 4.898 \cdot 10^3 \cdot lbf$$

$$F_{w2} := 0 \cdot lbf \qquad F_2 := F_{h2} + F_{w2}$$

$F_2 = 4.898 \cdot 10^3 \cdot lbf$ Checks with CSLIDE

$P_{reqd} := F_1 - F_2$ $\qquad P_{reqd} = 3.376 \cdot 10^4 \cdot lbf$ Checks with CSLIDE

C.2.3 Check of CSLIDE results—partially submerged condition on the retained soil side

Using the procedures on which the CSLIDE program is based, a numerical check is performed to illustrate the CSLIDE analytical process. Recall that CSLIDE assumes that the interface friction angle is equal to zero. In the CSLIDE method, forces are resolved in directions normal to and tangential to the failure plane.

For ***Wedge 1*** on the retained soil side (see Figure C.3):

Sum of forces tangential to failure plane.

The shear force required for equilibrium, T_1, is given as

$$T_1 = W_1 \sin \alpha_1 - P_1 \cos \alpha_1$$

The sum of forces normal to failure plane, N_1, is given as

$$N_1 = W_1 \cos \alpha_1 + P_1 \sin \alpha_1$$

Calculating the mobilized resistance, T_{F1}, tangential to failure plane:

$$T_{F1} = N_1 \tan \phi_{mob}$$

$$T_{F1} = T_1$$

$$N_1 \tan \phi_{mob} = [W_1 \cos \alpha_1 + P_1 \sin \alpha_1] \tan \phi_{mob}$$

Setting resistance equal to force in the tangential direction and solving for P_1,

$$(W_1 \cos \alpha_1 + P_1 \sin \alpha_1) \tan \phi_{mob} = W_1 \sin \alpha_1 - P_1 \cos \alpha_1$$

$$P_1 = -\frac{W_1 \sin \alpha_1 - W_1 \cos \alpha_1 \tan \phi_{mob}}{\sin \alpha_1 \tan \phi_{mob} + \cos \alpha_1}$$

The Mathcad computations for P_1 are provided below.

Check of CSLIDE results for 30 Foot High Tieback Wall

FILE: CSLIDE Check 3A July 10, 2002

$SF := 1.3$ Factor of safety

$\phi := 30 \cdot deg$ $\phi_{mob} := atan\left(\frac{tan(\phi)}{SF}\right)$ $\phi_{mob} = 23.947 \cdot deg$

Half submerged retained soil condition

$\gamma_{sat} := 134.4 \cdot \frac{lbf}{ft^2}$ $\gamma_w := 62.4 \cdot \frac{lbf}{ft^2}$ $\gamma_{bouy} := \gamma_{sat} - \gamma_w$

$\gamma_{bouy} = 72 \cdot \frac{lbf}{ft^2}$ $\gamma_{moist} := 115 \cdot \frac{lbf}{ft^2}$

$H_{T1} := 18 \cdot ft$ $H_{B1} := 18.0 \cdot ft$

For left side "active" soil wedge number 1on the retained side
(CSLIDE Wedge 1)

$\alpha_1 := 45 \cdot deg + \frac{\phi_{mob}}{2}$ $\alpha_1 = 56.973 \cdot deg$

$W_1 := \left[\left(\frac{H_{T1}}{2}\right) \cdot \frac{H_{T1}}{tan(\alpha_1)}\right] \cdot \gamma_{moist}$ $W_1 = 1.211 \cdot 10^4 \cdot lbf$ Agrees with CSLIDE

$L_1 := \frac{H_{T1}}{sin(\alpha_1)}$ $L_1 = 21.469 \cdot ft$ Agrees with CSLIDE

$$P_1 := \frac{-\left(\left(W_1 \cdot sin(\alpha_1) - W_1 \cdot cos(\alpha_1) \cdot tan(\phi_{mob})\right)\right)}{sin(\alpha_1) \cdot tan(\phi_{mob}) + cos(\alpha_1)}$$

$P_1 = -7.873 \cdot 10^3 \cdot lbf$ Agrees with CSLIDE

For ***Wedge* 2** on the retained soil side (see Figure C.3):

Sum of forces tangential to failure plane.

The shear force required for equilibrium, T_2, is given as

$T_2 = W_2 \sin \alpha_1 - P_2 \cos \alpha_2$

The sum of forces normal to failure plane, N_2, is given as

$N_2 = W_2 \cos \alpha_2 - U_2 + P_2 \sin \alpha_2$

Calculating the mobilized resistance, T_{F2}, tangential to failure plane:

$T_{F2} = N_2 \tan \phi_{mob}$

$T_{F2} = T_2$

$N_2 \tan \phi_{mob} = (W_2 \cos \alpha_1 - U_2 + P_2 \sin \alpha_2) \tan \phi_{mob}$

Setting resistance equal to force in the tangential direction and solving for P_2,

$(W_2 \cos \alpha_2 - U_2 + P_2 \sin \alpha_2) \tan \phi_{mob} = W_2 \sin \alpha_2 - P_2 \cos \alpha_2$

$$P_2 = -\frac{W_2 \sin \alpha_2 - [W_2 \cos \alpha_2 - U_2] \tan \phi_{mob}}{\sin \alpha_2 \tan \phi_{mob} + \cos \alpha_2}$$

The Mathcad computations for P_2 are provided below.

For left side "active" soil wedge number 2 on the retained side
(CSLIDE Wedge 2)

$$\alpha_2 := 45 \cdot deg + \frac{\phi_{mob}}{2} \qquad \alpha_2 = 56.973 \cdot deg$$

$$W_2 := H_{T1} \cdot \left(\frac{H_{B1}}{\tan(\alpha_2)}\right) \cdot \gamma_{moist} + \left[\left(\frac{H_{B1}}{2}\right) \cdot \frac{H_{B1}}{\tan(\alpha_2)}\right] \cdot \gamma_{sat}$$

$W_2 = 3.838 \cdot 10^4 \cdot lbf$ Agrees with CSLIDE

$$L_2 := \frac{H_{B1}}{\sin(\alpha_2)} \qquad L_2 = 21.469 \cdot ft$$ Agrees with CSLIDE

$$U_2 := \gamma_w \cdot H_{B1} \cdot \left(\frac{L_2}{2}\right) \qquad U_2 = 1.206 \cdot 10^4 \cdot lbf$$ Agrees with CSLIDE

$$P_2 := \frac{-\left[(W_2) \cdot \sin(\alpha_2) - \left[(W_2) \cdot \cos(\alpha_2) - U_2\right] \cdot \tan(\phi_{mob})\right]}{\sin(\alpha_2) \cdot \tan(\phi_{mob}) + \cos(\alpha_2)}$$

$P_2 = -3.078 \cdot 10^4 \cdot lbf$ Agrees with CSLIDE

For ***Wedge 4*** on the retained soil side (see Figure C.4):

Sum of forces tangential to failure plane.

The shear force required for equilibrium, T_4, is given as

$$T_4 = W_4 \sin \alpha_4 - P_4 \cos \alpha_4$$

The sum of forces normal to failure plane, N_4, is given as

$$N_4 = W_4 \cos \alpha_4 + P_4 \sin \alpha_4$$

Calculating the mobilized resistance, T_{F4}, tangential to failure plane:

$$T_{F4} = N_4 \tan \phi_{mob}$$

$$T_{F4} = T_4$$

$$N_4 \tan \phi_{mob} = (W_4 \cos \alpha_4 + P_4 \sin \alpha_4) \tan \phi_{mob}$$

Setting resistance equal to force in the tangential direction and solving for P_4,

$$(W_4 \cos \alpha_4 + P_4 \sin \alpha_4) \tan \phi_{mob} = W_4 \sin \alpha_4 - P_4 \cos \alpha_4$$

$$P_4 = \frac{W_4 \sin \alpha_1 - W_4 \cos \alpha_4 \tan \phi_{mob}}{\sin \alpha_4 \tan \phi_{mob} + \cos \alpha_4}$$

The Mathcad computations for P_1 are provided below.

For right side "passive" wedge number 4 below excavation
(CSLIDE Wedge 4)

$$\alpha_4 := 45\cdot deg - \frac{\phi_{mob}}{2} \qquad \alpha_4 = 33.027\,deg \qquad H_4 := 6.0\cdot ft$$

$$W_4 := \left[\left(\frac{H_4}{2}\right)\cdot\frac{H_4}{\tan(\alpha_4)}\right]\cdot\gamma_{moist} \qquad W_4 = 3.184\cdot 10^3\,lbf$$ Agrees with CSLIDE

$$L_4 := \frac{H_4}{\sin(\alpha_1)} \qquad L_4 = 7.156\,ft$$ Agrees with CSLIDE

$$P_4 := \frac{-\left(\left(W_4\cdot\sin(\alpha_4) + W_4\cdot\cos(\alpha_4)\cdot\tan(\phi_{mob})\right)\right)}{\sin(\alpha_4)\cdot\tan(\phi_{mob}) - \cos(\alpha_4)}$$

$$P_4 = 4.898\cdot 10^3\,lbf$$ Agrees with CSLIDE

SUM OF FORCES ON SYSTEM

$$SUM := P_1 + P_2 + P_4 \qquad SUM = -3.376\cdot 10^4\,lbf$$ Agrees with CSLIDE

The results agree with those of CSLIDE output (see Section C.2.1).

C.3 UTEXAS4 Analysis Results

C.3.1 Partially submerged conditions on the retained soil side

The results from the UTEXAS4 program for partially submerged conditions on the retained soil side (only) are presented below. The Corps of Engineers' Modified Swedish force equilibrium procedure (Simplified Janbu Procedure) was selected for cohesionless soils in accordance with the recommendation of FHWA-RD-97-130 (paragraph 4.4.3). The failure plane was selected to pass just below the bottom of the wall and the side force inclination set at 0 deg (i.e. horizontal). Wright (2001) indicates that the Simplified Janbu Procedure usually tends to underestimate the factor of safety and therefore is not recommended. Various anchor force values were tried until a factor of safety equal to 1.3 was obtained.

Input

```
GRA
HEA
EXTERNAL STABILITY 30-FOOT HIGH WALL      FILE: TBW8.DAT
SINGLE ANCHOR - FAILURE PLANE THROUGH ANCHOR
RETAINED SOIL HALF SUBMERGED

PRO
     1 1 Cohesionless retained soil above water table
          -100  0
             0  0

     2 2 Cohesionless retained soil below water table
          -100  -18
             0  -18

     3 3 Concrete tieback wall
              0 0
              1 0

     4 4 Cohesionless material below finish grade
              0 -30
             50 -30


MAT
     1 Cohesionless soil
         115 = unit weight
         Conventional shear strengths
            0 30
         No pore water pressure
     2 Cohesionless soil
          134.2 = unit weight
          Conventional shear strengths
            0 30
          Piezometric Line
            1
     3 Concrete
          145 = unit weight
          Very strong
     4 Cohesionless soil
          115 = unit weight
          Conventional shear strengths
            0 30
       No pore water pressure

PIE
     1 PIEZOMETRIC LINE FOR RETAINED SOIL
         -100 -18
            0 -18

REINFORCEMENT LINES
       1 0 1
     -50 -20.0 0     0
     -38 -17.6 34500 0
       0 -10.0 34500
LAB
```

Note: The anchor forces in “bold” were varied until a factor of safety equal to 1.3 was obtained

```
INTERNAL STABILITY ANALYSIS - SINGLE ANCHOR
ANA
     Noncircular Search
     -30.00   0.00
       0.00 -36.00 FIX
      15.00 -30.00

       5.00   1.00
PRO
    C = Corps of Engineers Modified Swedish Procedure
    0
SAV
       4

COM
```

Output

```
Procedure of Analysis: Corps of Engineers' Modified Swedish
Specified side force inclination: 0.00
Will save the following number of shear surfaces with the lowest
factors of safety: 4

TABLE NO. 41
****************************************
*  Critical Noncircular Shear Surface  *
****************************************

***** CRITICAL NONCIRCULAR SHEAR SURFACE *****
X:     -18.67     Y:       0.00
X:       0.00     Y:     -36.00
X:      18.53     Y:     -30.00
```

Minimum factor of safety: **1.298**

```
Side force inclination: 0.00

TABLE NO. 57
**********************************************************************
**
* Check of Computations by Force Equilibrium Procedure (Results are for
*
* the critical shear surface in the case of an automatic search.)
*
**********************************************************************
**

Summation of Horizontal Forces: 1.16783e-011

Summation of Vertical Forces: 1.60014e-011

Mohr Coulomb Shear Force/Shear Strength Check Summation: 5.61131e-004
```

UTEXAS4 critical failure plane results for the partially submerged condition are illustrated in Figure C.5.

C.4 Results Summary

Table C.1 Results Summary for Partially Submerged Condition

Calculation Method	P_{reqd} (lb/ft)	FS on Soil Strength	Angle of Failure Surface α (deg)	Notes
CSLIDE	33,767	1.3	57.0	(1)(2)
UTEXAS4	34,500	1.3	62.5	(1)(3)

Notes:

(1) P_{reqd} = force required to stabilize the cut.

(2) CSLIDE assumes the interface friction angle (δ) is equal to zero.

(3) The UTEXAS4 analysis used the Simple Janbu Procedure with a friction angle (δ) equal to zero (i.e., side force inclination is horizontal).

The earth pressures used in CSLIDE are based on Rankine soil pressure theory and do not account for the interface friction that can be mobilized along the wall face. The UTEXAS4 analysis used the Corps of Engineers' Modified Swedish Procedure (Simple Janbu Procedure) with the side force inclination assumed as zero. Because of this, the UTEXAS4 results were approximately the same as CSLIDE analysis results with respect to the total force (P_{reqd}) required to stabilize the cut.

The ease with which a CSLIDE analysis can be performed and the output verified makes it a promising tool for validating the results of more comprehensive GPSS analyses, such as UTEXAS4.

The designer of the above tieback wall system should ensure that the tiebacks under "loss of single anchor" conditions have a capacity approximately equal to 35,000 lb per foot of wall. This is needed to safely stabilize the cut and to satisfy internal stability requirements. It should be noted that the total force required to stabilize the cut for this particular wall, which has the retained soil half submerged, is 35,000 lb per foot, compared to 26,000 lb per foot for a similar wall where the water table is below finish grade (see Appendix B analyses).

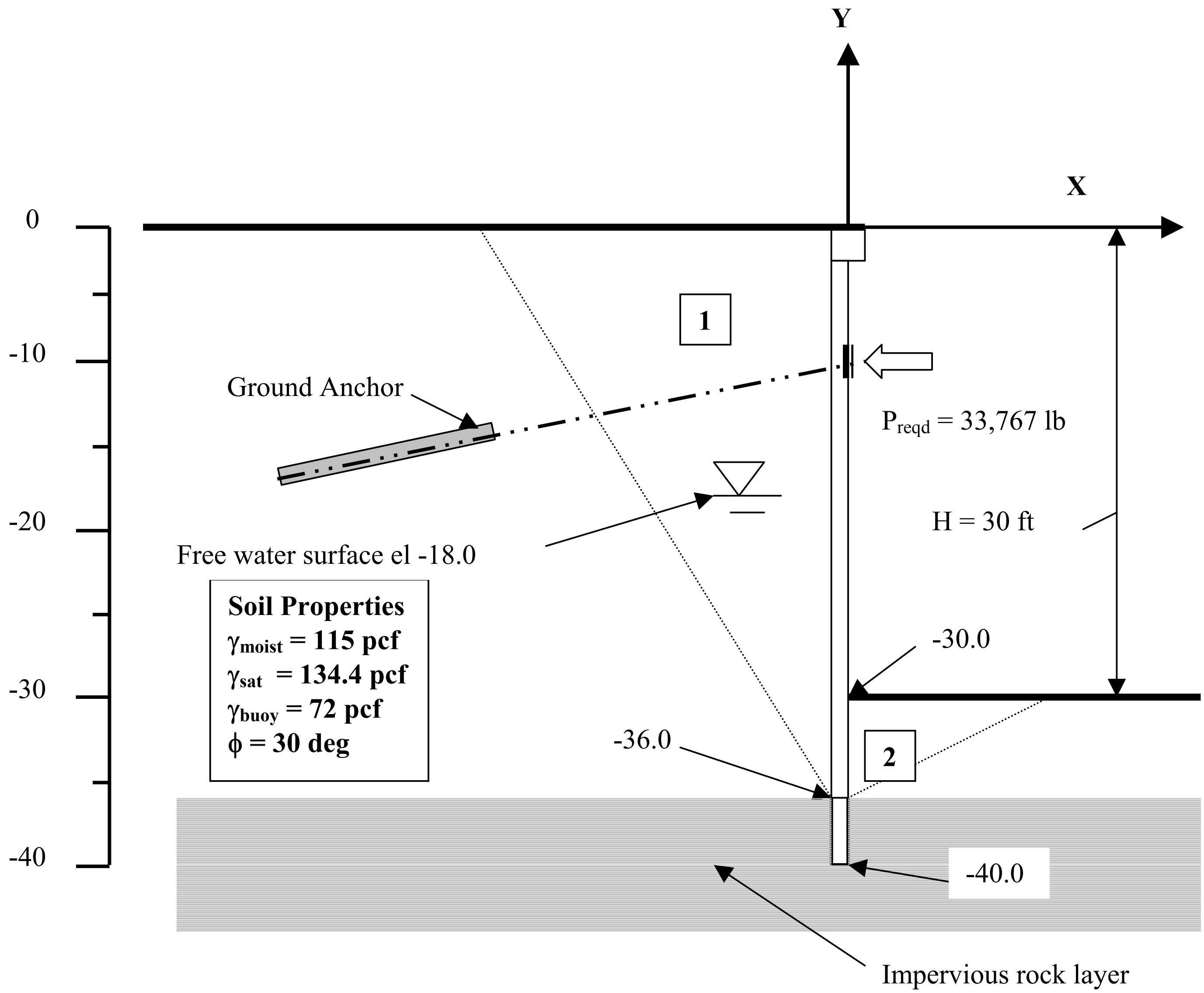


Figure C.1 30-ft-high wall, CSLIDE failure plane analysis—partially submerged condition on the retained soil side (only)

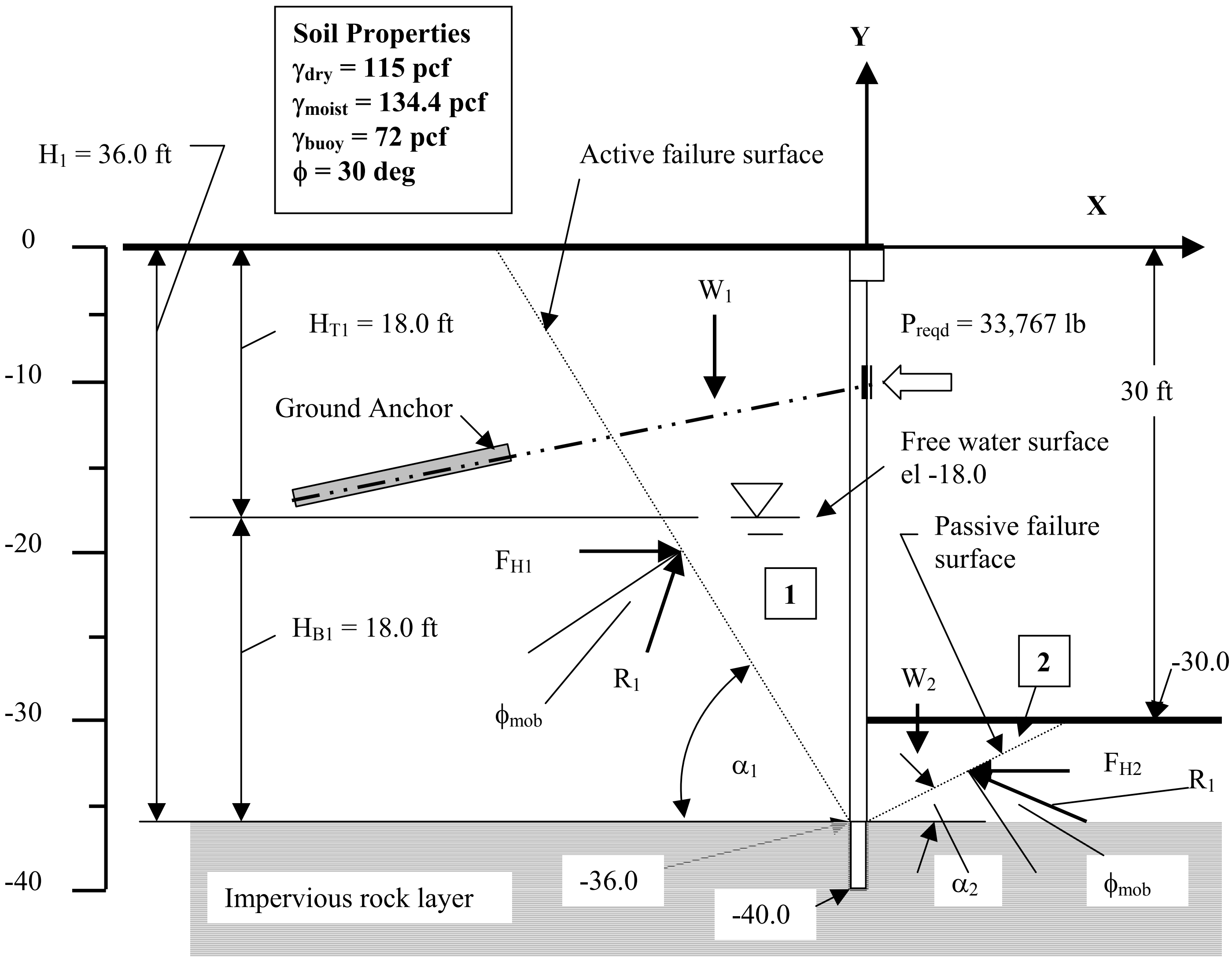


Figure C.2 30-ft-high wall, failure plane analysis check—partially submerged condition on the retained soil side (only)

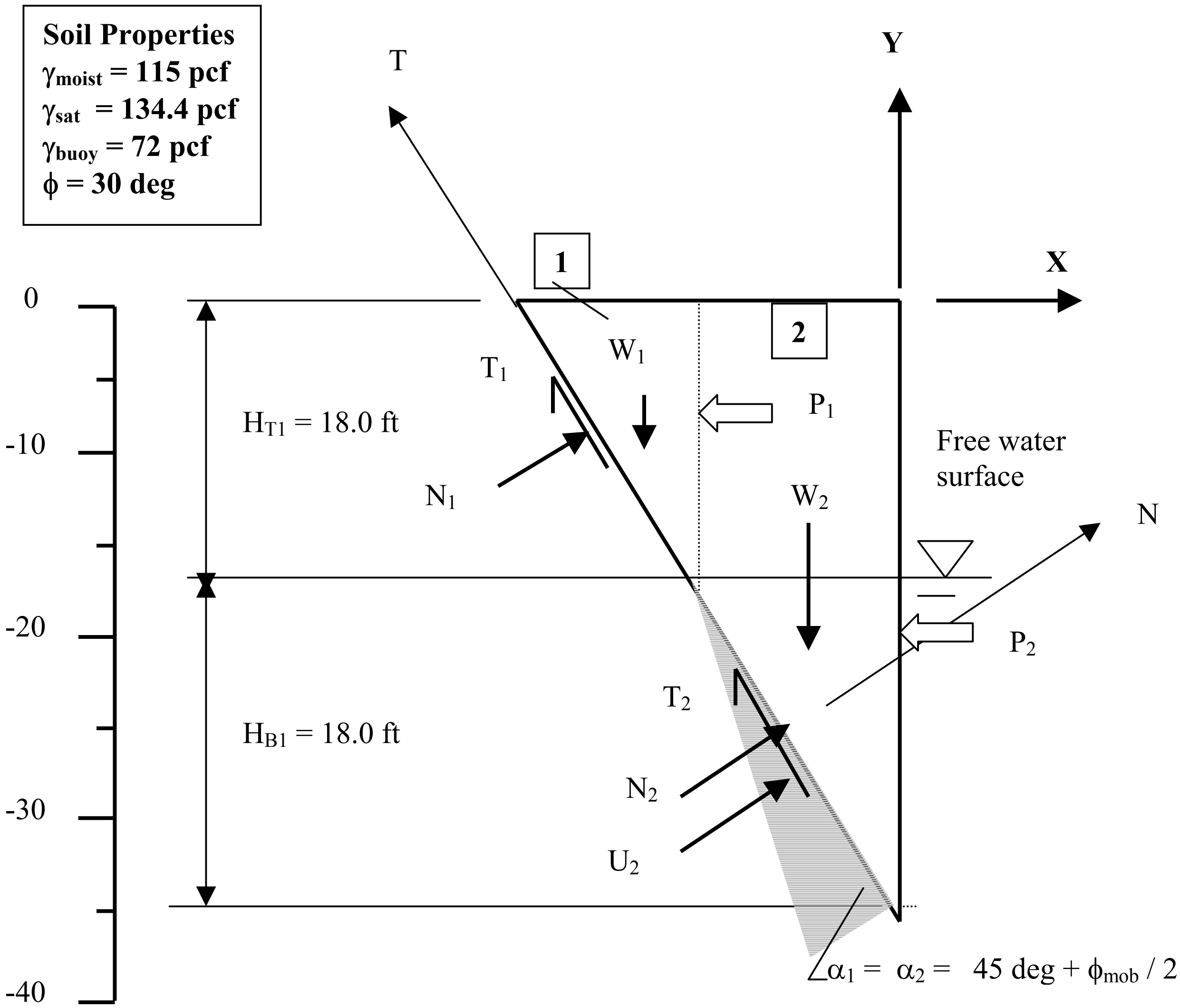


Figure C.3 30-ft-high wall, active wedges 1 and 2—failure plane analysis check, partially submerged condition on the retained soil side (only)

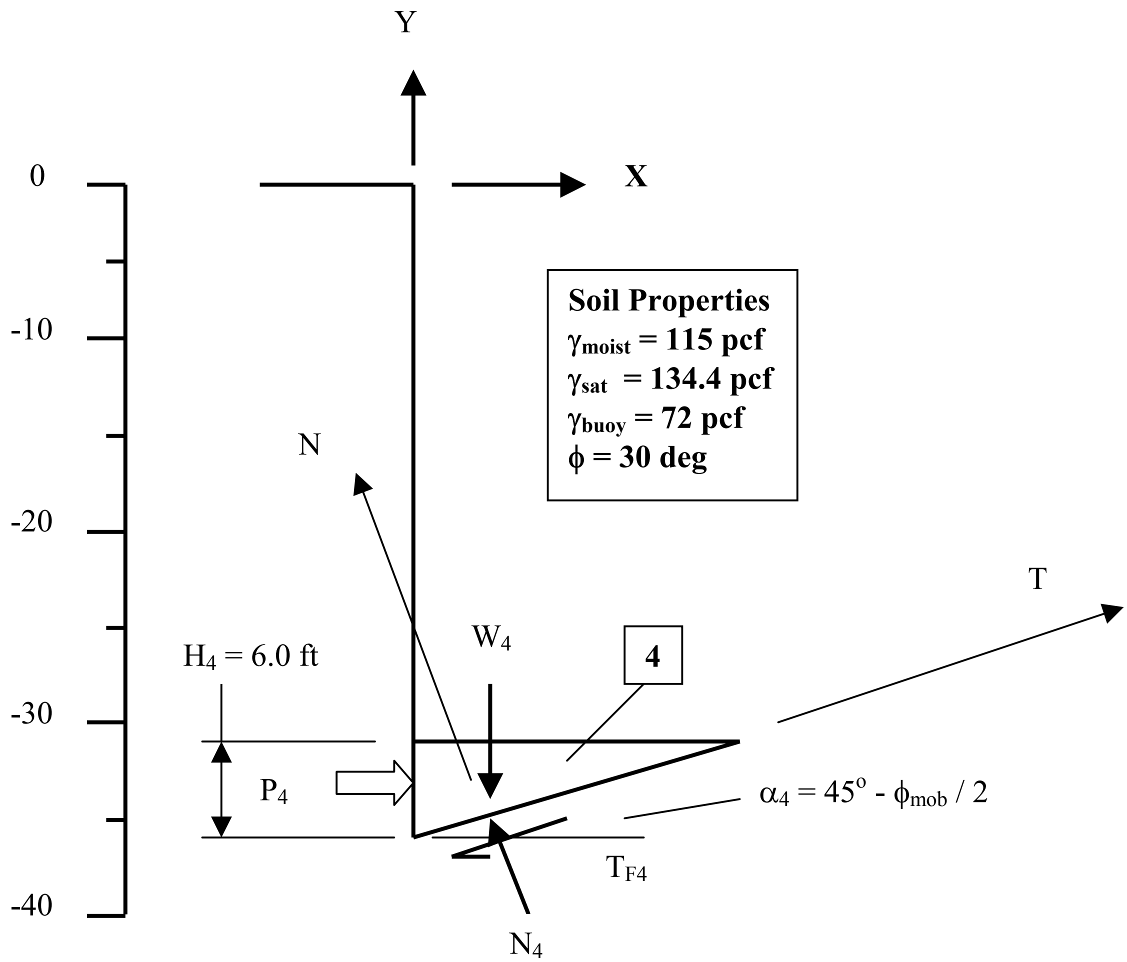


Figure C.4 30-ft-high wall, passive Wedge 4—failure plane analysis check, partially submerged condition on the retained soil side (only)

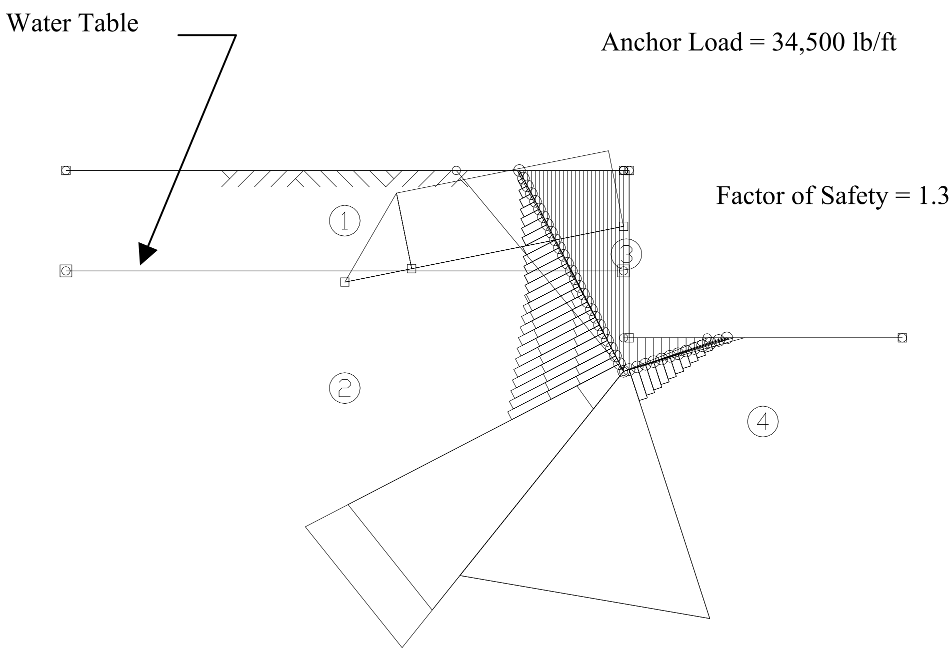


Figure C.5 30-ft-high wall, UTEXAS4 results—partially submerged condition on the retained soil wide (only)

Appendix D
External Stability, 30-ft-High Wall–Single Anchor

D.1 General

The simplified external stability approach described in FHWA-RD-98-065 and in Strom and Ebeling (2001) is used to check the external stability for a 30-ft-high wall, as illustrated in Figure D.1.

This simplified approach is limited to walls with reasonably homogeneous soil profiles. For complicated stratification, irregular ground surface, or irregular surcharge loading, the lateral force required to stabilize the excavation must be determined by slope stability analysis using general-purpose slope stability (GPSS) programs. In this approach, potential failure surfaces (slip surfaces) completely external to the ground containing the tieback anchors are examined using conventional slip surface limiting equilibrium slope stability models. The GPSS programs CSLIDE and UTEXAS4 (Wright 2001) are used for the external stability analyses, and the results are compared with those obtained from the simplified FHWA-RD-98-065 analysis.

The external stability of an anchored wall system is determined by assuming the potential plane of sliding passes behind the anchor and below the bottom of the wall. Since anchors are spaced at a horizontal distance, *S* (in-plan), the potential failure surface may assume a three-dimensional (3-D) shape rather than the 2-D shape used as an idealized basis for the following analysis. When a 2-D surface is used to approximate a 3-D failure surface, it is commonly assumed that the idealized 2-D failure plane intersects the ground anchor at a distance *S*/3 from the back of the anchor, as shown in Figure 2-34 of Strom and Ebeling (2002). The stability for the soil mass is determined by requiring horizontal and vertical force equilibrium. The soil mass under consideration is the soil prism ABCDEG, as shown in Figure D.1. For the external stability analysis of the 30-ft wall, it will be assumed the wall is terminated at the top of rock (el -36.0). The wall (for demonstration purposes) is assumed to be supported by a single anchor, and the external stability analyses are needed to determine the position of the anchor zone required to provide a minimum factor of safety of 1.3 on all potential failure planes.

D.2 Simplified External Stability Approach for Homogeneous Soil Sites

Forces on the soil mass are shown in Figure D.2, and the force vectors on area ABCDEG are shown in Figure D.3.

The soil mass acts downward with a magnitude equal to its weight. On the right face, the mobilized passive soil resistance, K_{mob}, acts at a mobilized angle of interface friction,

δ_{mob}. (See comments regarding $\delta = \phi$ and $\delta_{mob} = \phi_{mob}$ in Strom and Ebeling (2002, Chapter 3.) Mobilized "active" soil pressure is assumed to act on the left vertical face. On the bottom, soil resistance acts at an angle ϕ_{mob} from the perpendicular to the failure plane. Note in Figure D.2 that the interface friction angle (δ_{mob}) is assumed to be zero for the active soil pressure force P_{AR} (i.e., P_{AR} is based on Rankine "active" pressure). The forces will sum to zero in the horizontal and vertical directions for a safety factor equal to 1 and a friction angle ϕ_{mob}. Additional details pertaining to the force equilibrium analysis can be found in FHWA-RD-98-065. Equation 3.22 of FHWA-RD-98-065 is used to determine the failure plane angle (α) that will produce the required factor of safety. This failure plane angle is used to establish the anchor location needed to meet minimum factor of safety requirements for external stability. In Equation 3.22 (below), the friction angle ϕ is replaced by the mobilized friction angle ϕ_{mob}. The minimum factor of safety based on strength, $FS_{STRENGTH}$, is equal to tan (ϕ) / tan (ϕ_{mob}). A value of $FS_{STRENGTH}$ equal to 1.3 is often used in practice according to FHWA-RD-98-065 (paragraph 3.3.1, pg 35), and such a factor of safety would be appropriate for "safety with economy" type designs.

$$(1+\xi+\lambda)X - K_{Pmob}\xi^2 \sin(\delta_{mob}) + \frac{K_{Pmob}\xi^2 \cos(\delta_{mob}) - K_{Amob}\lambda^2}{\tan(\phi_{mob} - \alpha)} = 0$$

where [Eq. 3.22, FHWA-RD-98-065]

X = x/H
λ = y/H
ξ = d/H

The dimensions x, y, d, and H are shown in Figure 35 of the FHWA-RD-98-065 report, and the particular values used for the 30-ft wall are shown in Figure D.1.

Equation 3.22 of the FHWA-RD-98-065 report (shown above) is solved to find the failure plane angle (i.e., x and y dimensions) required to meet factor of safety requirements. This requires that the mobilized friction angle (ϕ_{mob}) be equal to 24 deg, such that tan (ϕ) / tan (ϕ_{mob}) equals 1.3, the required factor of safety. These calcuations are provided below. The x and y dimensions must be varied in a manner consistent with the anchor line of action until the term A in the following calculations becomes equal to zero. The Mathcad computations for this are provided below. Equation 3.22 (FHWA-RD-99-065) was extremely sensitive to small variations in the failure plane angle (α) (i.e., small variations in x and y dimensions).

30-Foot High Wall External Stability July 10, 2002

Equation 3.22, FHWA-RD-98-065 File: BAA3 \ STABLE 1B

$H := 30 \cdot ft \qquad \gamma_{moist} := 115 \cdot \frac{lbf}{ft^3} \qquad \phi := 30 \cdot deg$

Mobilized δ, the interface friction angle betwwen the embedded portion of the wall and the passive zone of soil is set equal to the mobilized ϕ.

The "x" and "y" dimensions below are varied in a manner consistent with the anchor line of action until an "A" value in Equation 3.22, FHWA-RD-98-065 approximately equal to zero is obtained. Only the final values for "x" and "y" are included in the analysis.

$x := 42.5 \cdot ft \qquad X := \frac{x}{H} \qquad X = 1.417$

$y := 18.5 \cdot ft \qquad \lambda := \frac{y}{H} \qquad \lambda = 0.617$

$d := 6 \cdot ft \qquad \xi := \frac{d}{H} \qquad \xi = 0.2$

$B := \frac{H + d - y}{x} \qquad B = 0.412 \qquad \alpha := atan(B) \qquad \alpha = 22.38 \circ deg$

For soil with a friction angle equal to 30-degrees

$\phi := 30 \cdot deg \qquad \delta := 30 \cdot deg$

From Figure 27, FHWA-RD-98-065, for $-\delta / \phi$ = -1, K_p = 5.5
By Rankine with zero interface friction, K_a = 0.333

For $\phi_{mob} := 24 \cdot deg$ $\delta_{mob} := 24 \cdot deg$

From Figure 27, FHWA-RD-98-065, for mobilized $-\delta / \phi = -1$, $K_{pmob} = 4.0$
By Rankine with zero interface friction, $K_{amob} = 0.422$ (see calcs below)

$K_{pmob} := 4.0$ $K_{amob} := \tan\left(45 \cdot deg - \frac{\phi_{mob}}{2}\right)^2$ $K_{amob} = 0.422$

Equation 3.22, FHWA-RD-98-065

$$A := (1 + \xi + \lambda) \cdot X - K_{pmob} \cdot \xi^2 \cdot \sin(\delta_{mob}) + \frac{\left(K_{pmob} \cdot \xi^2 \cdot \cos(\delta_{mob}) - K_{amob} \cdot \lambda^2\right)}{\tan(\phi_{mob} - \alpha)}$$

$A = 2.006$ Approximately = 0

$FS_{STRENGTH} := \frac{\tan(\phi)}{\tan(\phi_{mob})}$ $FS_{STRENGTH} = 1.297$ Approximately = 1.3 Okay

The simplified force equilibrium procedure indicates the values selected for x and y (i.e., $x = 42.5$, $y = -18.5$), or the selected effective anchor point location, will provide a factor of safety equal to 1.3. GPSS analysis solutions, using CSLIDE and UTEXAS4, will be used to verify that the selected effective anchor point location is satisfactory.

D.3 CSLIDE

D.3.1 General

The Corps program CSLIDE can be used to assess the external stability of a tieback wall system. It is based on the equations of horizontal and vertical equilibrium applied to the soil wedges. The CSLIDE program satisfies force equilibrium only and assumes that the interface friction angle is zero ($\delta = 0$). Moment equilibrium is not considered. The use of the CSLIDE program for external stability evaluation is demonstrated below with respect to the 30-ft wall example.

D.3.2 External stability–dry conditions

The CSLIDE stability calculations are provided below for the conditions where the retained soil is "dry" (actually, "moist" unit weights are used). The depth to the effective anchor point y and the wedge angle α values in CSLIDE are based on the anchor x and y dimensions shown in Figure D.1, and are those obtained from the previous simplified force equilibrium analysis. The y and α values are highlighted in the CSLIDE input data.

Table D.1 CSLIDE Input 30-ft-High Wall "Dry" Conditions

HEADING						*Information Only*
TITL 30 FT WALL EXT. STABILITY FILE: WEL4.IN						
STRUCTURE (TIEBACK WALL) DATA FOLLOWS						*Information Only*
STRUCTURE KEYWORD	NO. OF POINTS	UNIT WEIGHT				*Information Only*
STRU	**4**	**0.150**				
X-COORDINATE	Y-COORDINATE					*Information Only*
-1.00	-36.00					
-1.00	0.00					
0.00	0.00					
0.00	-36.00					
LEFTSIDE SOIL DATA FOLLOWS						*Information Only*
KEY	LAYER NO.	NO. OF PTS	PHI (deg)	C (ksf)	UNIT WT.	TOP EL *IO*
solt	1	1	30.00	0	0.115	0.00
X-COORDINATE	Y-COORDINATE					*Information Only*
-100.00	0.00					
KEY	LAYER NO.	NO. OF PTS	PHI (deg)	C (ksf)	UNIT WT.	TOP EL *IO*
SOLT	2	1	30.00	0	0.115	-18.50
X-COORDINATE	Y-COORDINATE					*Information Only*
-100.00	-18.50					
RIGHTSIDE SOIL DATA FOLLOWS						*Information Only*
KEY	LAYER NO.	NO. OF PTS	PHI (deg)	C (ksf)	UNIT WT.	TOP EL *IO*
SORT	1	1	30.00	0	0.115	-30.00
X-COORDINATE	Y-COORDINATE					*Information Only*
150.00	-30.00					
SOIL BELOW STRUCTURE DATA FOLLOWS						*Information Only*
KEY	PHI (deg)	C (ksf)				
SOST	0	0				
WATER TABLE DATA FOLLOWS						*Information Only*
KEY	LT. SIDE EL.	RT. SIDE EL.	UNIT WEIGHT			*Information Only*
WATR	-40.00	-40.00	0.0625			
METHOD OF ANALYSIS FOLLOWS						*Information Only*
KEYWORD	METHOD DESIGNATION					*Information Only*
METH	2					*Indicates Multi-Plane Analysis*
WEDGE ANGLE SPECIFICATION FOLLOWS						*Information Only*
KEY	WEDGE NO.	ANGLE (deg)				*Information Only*
WEDG	2	-22.38				
FACTOR OF SAFETY INFORMATION FOLLOWS						*Information Only*
KEY	LOWER LIMIT	UPPER LIMIT	PASSIVE TO ACTIVE RATIO			*IO*
FACT	1.00	2.00	1.00			
END						

Notes:

1 Units are in kips and feet.

2. Elevation of lower left side wedge and the failure plane angle of the lower left side wedge were established to provide a failure plane identical to that determined in the previous simplified force equilibrium method.

CSLIDE Input

```
10010 TITL 30 FT WALL EXT STABILITY FILE: WEL4.IN
10020 STRU  4      .150
10030        -1.00       -36.00
10040        -1.00         0.00
10050         0.00         0.00
10060         0.00       -36.00
10090 SOLT  1  1       30.00      0.00      0.115       0.00
10095      -100.00      0.00
10100 SOLT  2  1       30.00      0.00      0.115       -18.50 ←" y "
10105      -100.00      -18.50
10110 SORT  1  1       30.00      0.00      0.115       -30.00
10120       150.00       -30.00
10130 SOST        0.00     0.00
10135 WATR       -40.00   -40.00     0.0625
10140 METH  2
10150 WEDG  2  -22.38 ←" Wedge Angle "
10160 FACT        1.0        2.0      1.00
10170 END
```

CSLIDE Output File: WEL400

```
------------------------------------
   PROGRAM CSLIDE - FINAL RESULTS
------------------------------------

 30 FT WALL EXT STABILITY FILE: WEL4.IN

 MULTIPLE FAILURE PLANE ANALYSIS
```

WEDGE NUMBER	FAILURE ANGLE (DEG)	TOTAL LENGTH (FT)	WEIGHT OF WEDGE (KIPS)	SUBMERGED LENGTH (FT)	UPLIFT FORCE (KIPS)
1	-56.9	22.077	12.816	.000	.000
2	-22.4	45.962	133.185	.000	.000
3	.000	1.000	5.400	.000	.000
4	33.1	10.995	3.179	.000	.000

WEDGE NUMBER	NET FORCE ON WEDGE (KIPS)
1	-8.342
2	3.458
3	.000
4	4.884

SUM OF FORCES ON SYSTEM ---- .000

FACTOR OF SAFETY ----------- **1.305**

The CSLIDE analysis indicates that the *x* and *y* dimensions obtained from the simplified force equilibrium method by CSLIDE analysis produce a factor of safety similar to that determined by the simplified force equilibrium method.

D.3.3 External stability–retained soil half submerged

The CSLIDE stability calculations for the conditions where the retained soil is half submerged are provided below. The hydrostatic water table is assumed to be in the retained soil side (only), the wall is assumed to be impervious, and the substrate into which the wall is embedded is assumed to be impervious. The anchor *x* and *y* dimensions must be increased from those used for the previous analyses in order to maintain a factor of safety equal to approximately 1.3 for partially submerged conditions. The anchor inclination (11.3 deg downward) was maintained from the previous analyses. The depth to the effective anchor point *y* and the wedge angle α values in CSLIDE are based on the anchor *x* and *y* dimensions shown in Figure D.4.

These values were varied in CSLIDE to provide a factor of safety equal to approximately 1.3. The final *y* and α values are highlighted in the CSLIDE input data. The final *x* and *y* values needed to produce the desired factor of safety (i.e., *FS* = 1.3) are shown in Figure D.4.

CSLIDE Input

```
10010 TITL 30 FT WALL EXT STABILITY HALF-SUBMERGED FILE: WEL2.IN
10020 STRU  4      .150
10030        -1.00       -36.00
10040        -1.00         0.00
10050         0.00         0.00
10060         0.00       -36.00
10090 SOLT  1  1       30.00      0.00      0.115       0.00
10095      -150.00       0.00
10100 SOLT  2  1       30.00      0.00      0.1334      -20.0 ← y
10105      -100.00       -20.0 ← y
10110 SORT  1  1       30.00      0.00      0.115       -30.00
10120       150.00       -30.00
10130 SOST          0.00     0.00
10135 WATR        -18.00   -36.00     0.0625  0
10140 METH  2
10150 WEDG  2  -18.00 ←"wedge angle"
10160 FACT         1.0        2.0      1.00
10170 END
```

Note: It was assumed that Point D on Figure D.4 approximately coincided with the water table location (on the retained soil side only) and therefore was used also as the demarcation between "dry" (i.e., "moist") and saturated soil.

CSLIDE Output File: WEL200

```
------------------------------------
   PROGRAM CSLIDE - FINAL RESULTS
------------------------------------

30 FT WALL EXT STABILITY HALF SUBMERGED FILE: WEL2.IN

 MULTIPLE FAILURE PLANE ANALYSIS

 HYDROSTATIC WATER FORCE COMPUTED FOR WEDGES

 WEDGE     FAILURE       TOTAL       WEIGHT     SUBMERGED       UPLIFT
 NUMBER     ANGLE        LENGTH     OF WEDGE     LENGTH         FORCE
            (DEG)         (FT)       (KIPS)       (FT)          (KIPS)
 ----------------------------------------------------------------------
   1        -57.1       23.813       14.865       2.381          .149
   2        -18.0       51.777      166.205      51.777        32.361
   3         .000        1.000        5.400       1.000          .563
   4         32.9       11.054        3.203        .000          .000

     WEDGE      NET  FORCE
     NUMBER      ON WEDGE
                  (KIPS)
 -------------------------
       1         -9.694
       2          4.745
       3           .000
       4          4.949

 SUM OF FORCES ON SYSTEM ----       .000

 FACTOR OF SAFETY -----------        1.284
```

The CSLIDE analysis indicates that the *x* and *y* dimensions selected (50 and 20 ft, respectively) for partial submergence of the retained side soil (see Figure D.4) produced a factor of safety approximately equal to 1.3.

D.4 UTEXAS 4

D.4.1 Dry conditions

The results from the UTEXAS4 program for "dry" conditions are presented below. The Spencer Procedure was selected for the external stability analysis. Following the UTEXAS4 procedures, the search for the critical noncircular shear surface is performed based on the procedure developed by Celestino and Duncan (1981). The failure plane was selected to pass just in back of the effective anchor location, as described in Figure D.1, except *x* and *y* dimensions equal to 40.0 ft and 18.0 ft were used rather than 42.5 and 18.5 ft.

Table D.2 UTEXAS4 Input 30-Foot High Wall "Dry" Conditions

GRAphics output activated			*Command Line*
HEAding			*Command Line*
EXTERNAL STABILITY 30-FOOT HIGH WALL FILE: TBW2.DAT			
SINGLE ANCHOR FAILURE PLANE IN BACK OF ANCHOR			
Blank Line			
PROfile			*Command Line*
PRO LINE #	MAT'L #	PROFILE LINE LABLE	*Information Only*
1	**1**	**Cohesionless retained soil**	
X-COORDINATE	Y-COORDINATE		*Information Only*
-100	**O**		
0	**O**		
Blank Line			
PRO LINE #	MAT'L #	PROFILE LINE LABLE	*Information Only*
2	**2**	**Concrete tieback wall**	
X-COORDINATE	Y-COORDINATE		*Information Only*
0	**O**		
1	**0**		
Blank Line			
PRO LINE #	MAT'L #	PROFILE LINE LABLE	*Information Only*
3	**1**	**Cohesionless material below finish grade**	
X-COORDINATE	Y-COORDINATE		*Information Only*
1	**-30**		
50	**-30**		
Blank Line			
PRO LINE #	MAT'L #	PROFILE LINE LABLE	*Information Only*
4	**1**	**Cohesionless material below wall**	
X-COORDINATE	Y-COORDINATE		*Information Only*
0	**-36**		
50	**-36**		
Blank Line			
Blank Line			
MATerial properties			*Command Line*
1	**Cohesionless Soil**		
	115 = unit weight		
	Conventional shear strengths		
	c (psf)	ϕ (deg)	*Information Only*
	0	**30.0**	
	No pore pressures		
2	**Concrete**		
	145 = unit weight		
	Very Strong		
Blank Line			

Table D.2 (Concluded) UTEXAS4 Input 30-ft-High Wall "Dry" Conditions

REInforcement				*Command Line*
REIN LINE #	REIN ROTATION ANGLE (DEG)		FORCE CODE	*Information Only*
1	**0**		**1**	
X-COORDINATE	Y-COORDINATE	LONG FORCE	TRANS FORCE	*Information Only*
-40	**-18**	**0**	**0**	
-28	**-15.6**	**24000**	**0**	
0	**-10**	**24000**	**0**	
Blank Line				
LABel				*Command Line*
EXTERNAL STABILITY – SINGLE ANCHOR				
ANAlysis				*Command Line*
Noncircular	**S**earch			*Procedure Designation*
X-COORDINATE	Y-COORDINATE	BLANK (if moveable) or FIX		*Noncircular point inf.*
-60.00	**0.00**			
-40.00	**-18.00**	**FIX**		
0.00	**-36.01**	**FIX**		
1.00	**-36.01**	**FIX**		
10.00	**-30.00**			
Blank Line				
INITIAL SHIFT DIST.	FINAL SHIFT DIST.	MAX. TOE SLOPE STEEPNESS (optional)		
5.00	**1.00**			
SAV**e**				
4		*No. of trial surfaces to be saved for graphics*		
Blank Line				
COMpute				*Command Line*

The soil profile information for the UTEXAS4 analysis is illustrated in Figure D.5.

Information on the reinforcement used to represent the tieback is illustrated in Figure D.6.

Information describing the noncircular failure plane used for the analysis is described in Figure D.7.

Input to UTEXAS4

```
GRA
HEA
EXTERNAL STABILITY 30-FOOT HIGH WALL  FILE: TBW2.DAT
SINGLE ANCHOR - FAILURE PLANE IN BACK OF ANCHOR

PRO
     1 1 Cohesionless retained soil
          -100  0
             0  0

     2 2 Concrete tieback wall
              0 0
              1 0

     3 1 Cohesionless material below finish grade
              1 -30
             50 -30

     4 1 Cohesionless material below wall
              0 -36
             50 -36


MAT
     1 Cohesionless soil
          115 = unit weight
          Conventional shear strengths
            0 30
          No pore water pressures
     2 Concrete
          145 = unit weight
          Very strong

REINFORCEMENT LINES
       1 0 1
     -40 -18.0 0     0
     -28 -15.6 24000 0
       0 -10.0 24000 0


LAB
EXTERNAL STABILITY ANALYSIS - SINGLE ANCHOR
ANA
     Noncircular Search
     -60.00   0.00
     -40.00 -18.00 FIX
       0.00 -36.01 FIX
       1.00 -36.01 FIX
      10.00 -30.00

       5.00   1.00
SAV
       4
```

```
COM
Output

**************************************
*  Critical Noncircular Shear Surface  *
**************************************

***** CRITICAL NONCIRCULAR SHEAR SURFACE *****
X:     -51.81     Y:       0.00
X:     -40.00     Y:     -18.00
X:       0.00     Y:     -36.01
X:       1.00     Y:     -36.01
X:      10.70     Y:     -30.00

Minimum factor of safety: 1.277
Side force inclination: -0.88

TABLE NO. 55
*******************************************************************
* Check of Computations by Spencer's Procedure (Results are for the *
* critical shear surface in the case of an automatic search.)       *
*******************************************************************

Summation of Horizontal Forces: 2.47988e-011

Summation of Vertical Forces: 1.20650e-011

Summation of Moments: -3.68163e-009

Mohr Coulomb Shear Force/Shear Strength Check Summation: 1.25304e-011

SPENCER'S PROCEDURE USED TO COMPUTE THE FACTOR OF SAFETY
Factor of Safety: 1.277     Side Force Inclination:  -0.88
```

The UTEXAS4 Spencer Method analysis for external stability indicates that, for the dry condition, the *x* and *y* dimensions for the effective anchor point location (as determined by the simplified force equilibrium procedure and as illustrated in Figure D.1) provide a factor of safety approximately equal to 1.3. Output plot information for the above analysis is provided in Figure D.8.

D.4.2 Partial submergence of the retained soil

The input and output results from UTEXAS4 for partial submergence of the retained soil are presented below. The hydrostatic water table is assumed to be in the retained soil side (only), the wall is assumed to be impervious, and the substrate into which the wall is embedded is assumed to be impervious. The Spencer Procedure was selected for the external stability analysis. Following the UTEXAS4 procedures, the search for the critical noncircular shear surface is performed based on the procedure developed by Celestino and Duncan (1981). The failure plane was selected to pass just in back of the effective anchor location, as described in Figure D.1.

UTEXAS4 Analysis for retained soil side half-submerged

Input

```
GRA
HEA
EXTERNAL STABILITY 30-FOOT HIGH WALL  FILE: TBW4.DAT
SINGLE ANCHOR - FAILURE PLANE IN BACK OF ANCHOR
RETAINED SOIL HALF SUBMERGED

PRO
     1 1 Cohesionless retained soil above water table
          -100  0
             0  0

     2 2 Cohesionless retained soil below water table
          -100  -18
             0  -18

     3 3 Concrete tieback wall
              0 0
              1 0

     4 4 Cohesionless material below finish grade
              1 -30
             50 -30

     5 4 Cohesionless material below wall
              0 -36
             50 -36


MAT
     1 Cohesionless soil
         115 = unit weight
         Conventional shear strengths
            0 30
         No pore water pressure
     2 Cohesionless soil
          134.2 = unit weight
          Conventional shear strengths
            0 30
          Piezometric Line
            1
     3 Concrete
          145 = unit weight
          Very strong
     4 Cohesionless soil
          115 = unit weight
          Conventional shear strengths
            0 30
       No pore water pressure


PIE
     1 PIEZOMETRIC LINE FOR RETAINED SOIL
         -100 -18
            0 -18
```

```
REINFORCEMENT LINES
       1 0 1
     -50 -20.0 0     0
     -38 -17.6 33800 0
       0 -10.0 33800 0


LAB
EXTERNAL STABILITY ANALYSIS - SINGLE ANCHOR
ANA
     Noncircular Search
     -70.00   0.00
     -50.00 -20.00 FIX
       0.00 -36.01 FIX
       1.00 -36.01 FIX
      10.00 -30.00

       5.00   1.00
SAV
       4

COM
```

Output

```
EXTERNAL STABILITY 30-FOOT HIGH WALL  FILE: TBW4.DAT
SINGLE ANCHOR - FAILURE PLANE IN BACK OF ANCHOR
RETAINED SOIL HALF SUBMERGED

TABLE NO. 41
****************************************
*  Critical Noncircular Shear Surface  *
****************************************

***** CRITICAL NONCIRCULAR SHEAR SURFACE *****
X:     -62.57     Y:       0.00
X:     -50.00     Y:     -20.00
X:       0.00     Y:     -36.01
X:       1.00     Y:     -36.01
X:       9.35     Y:     -30.00

Minimum factor of safety: 1.336
Side force inclination: 2.88

TABLE NO. 55
*******************************************************************
* Check of Computations by Spencer's Procedure (Results are for the *
* critical shear surface in the case of an automatic search.)       *
*******************************************************************

Summation of Horizontal Forces: 3.45226e-011

Summation of Vertical Forces: 1.89377e-011

Summation of Moments: -2.14459e-008

Mohr Coulomb Shear Force/Shear Strength Check Summation: 1.59170e-011
```

The UTEXAS4 Spencer Method analysis for external stability indicates that, for the condition where the soil on the retained side is half submerged, the *x* and *y* dimensions for the effective anchor point location (as determined using CSLIDE and illustrated in Figure D.4) provide a factor of safety slightly greater than 1.3. Output plot information for the above analysis is provided in Figure D.9.

D.5 Results Summary

Table D.3 Limiting Equilibrium Methods Comparison External Stability–"Dry" Conditions

Calculation Method	Anchor Location (See Figure D.1)		
	x	*y*	Factor of Safety
Simple Method	42.5	-18.5	1.30
CSLIDE	42.5	-18.5	1.30
UTEXAS4 Spencer Method	40	-18	1.27

Table D.4 Limiting Equilibrium Methods Comparison External Stability–Retained Soil Side Partially Submerged

Calculation Method	Anchor Location (See Figure D.4)		
	x	*y*	Factor of Safety
Simplified Method	50	-20	NA
CSLIDE	50	-20	1.28
UTEXAS4 Spencer Method	50	-20	1.34

The above computations illustrate various techniques that can be used to determine anchor locations that satisfy external stability factor of safety requirements.

The simplified external stability method of FHWA-RD-98-065 (identified as the Simple Method in Tables D.1 and D.2) was used for the external stability evaluation of the 30-ft-high wall under the “dry” condition. By varying the x and y dimensions (i.e., varying the failure plane angle α), Equation 3.12 of FHWA-RD-98-065 could be used to determine a minimum anchor embedment that would satisfy factor of safety requirements (i.e., safety factor equal to 1.3 for “safety with economy” designs). Equation 3.12 seemed to be sensitive to small variation in the failure plane angle α. The Corps program CSLIDE was used to verify that the Simple Method results were reasonable. Both methods produced factors of safety near 1.3 for the dry condition. In CSLIDE, the failure angle between the bottom of the wall and the effective anchor location was input. For the dry condition, it was set equal to that determined by the Simple Method. A bottom soil layer that had a top elevation at the effective anchor point and the same properties as the other soil wedges was used to set the upper boundary of the lower active failure wedge. The upper failure angle of the active failure wedge was established based on the factor of safety required to produce horizontal force equilibrium in the system.

CSLIDE can also be used in a systematic trial-and-error approach to determine the minimum anchor embedment location that would satisfy factor of safety requirements. This approach was used for the condition where the soil on the retained side was partially submerged, a condition that cannot be accommodated by the Simple Method approach. In the trial-and-error approach, various failure plane angles are assumed for the lower active failure wedge until the one that meets minimum factor of safety requirements has been determined. For the partially submerged condition, this occurred at a lower active wedge failure plane angle of 18 deg (i.e., x = 50 ft, y = 20 ft).

UTEXAS4 has unlimited capabilities for determining, for a given anchorage location, the failure plane that will produce the minimum factor of safety. In this particular evaluation, a noncircular failure plane, similar to that used in the CSLIDE analysis, was used with the lower active wedge failure plane restricted (i.e., fixed), as per the CSLIDE analysis. For the dry condition, an active wedge failure plane angle of 24.23 deg (i.e., x = 40 ft, y = 18 ft) was used. The partially submerged condition failure plane angle was identical to that used in CSLIDE. Other applications of the UTEXAS4 software with respect to external stability are shown in Appendixes E and F.

Point D
Intersection of anchor slope and failure plane slope

Soil Properties
γ_{moist} = 115 pcf
ϕ = 30 deg

For a given α, solve the following equation to get *y*:
$5y (\tan \alpha) + y = 36 + 50 (\tan \alpha)$

After "y" is determined, solve the following equation to get *x*:
$x = 5y - 50$

For α = 22.38 deg, y = 18.50 ft x = 42.50 ft

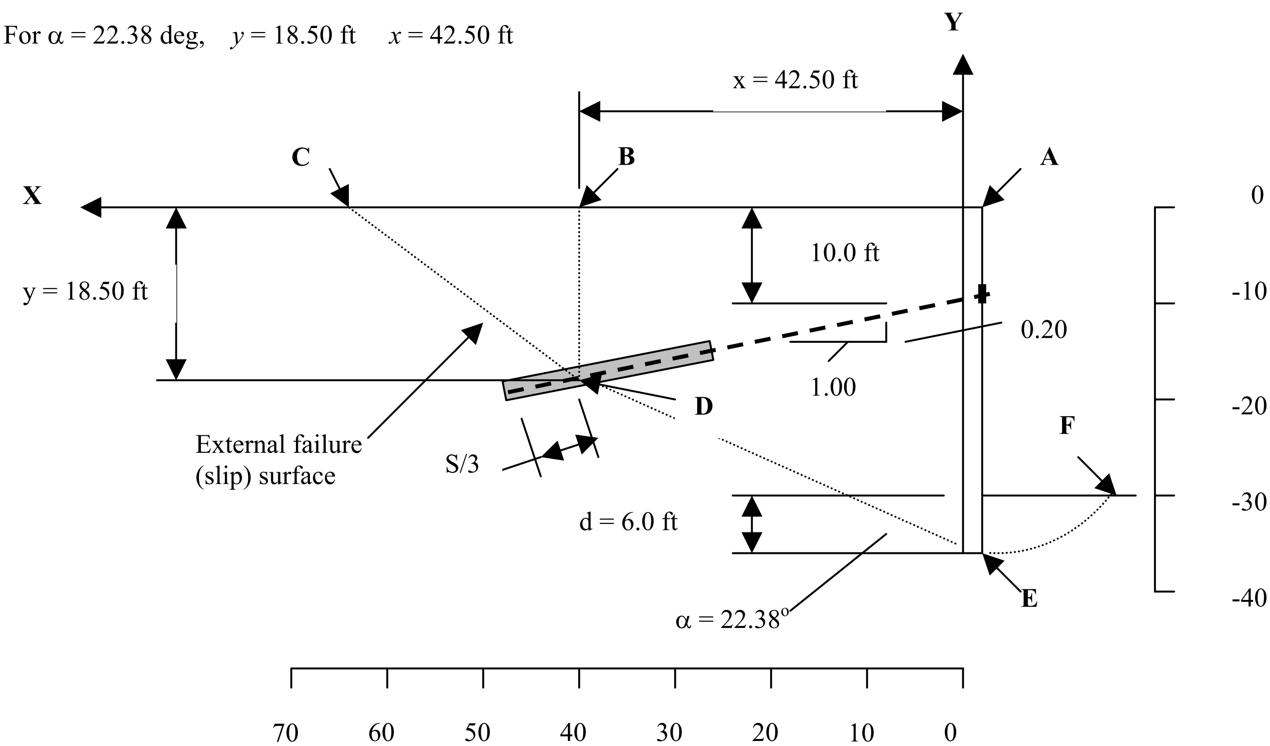


Figure D.1 Analysis for 30-ft-high wall–anchor location, retained soil "dry"

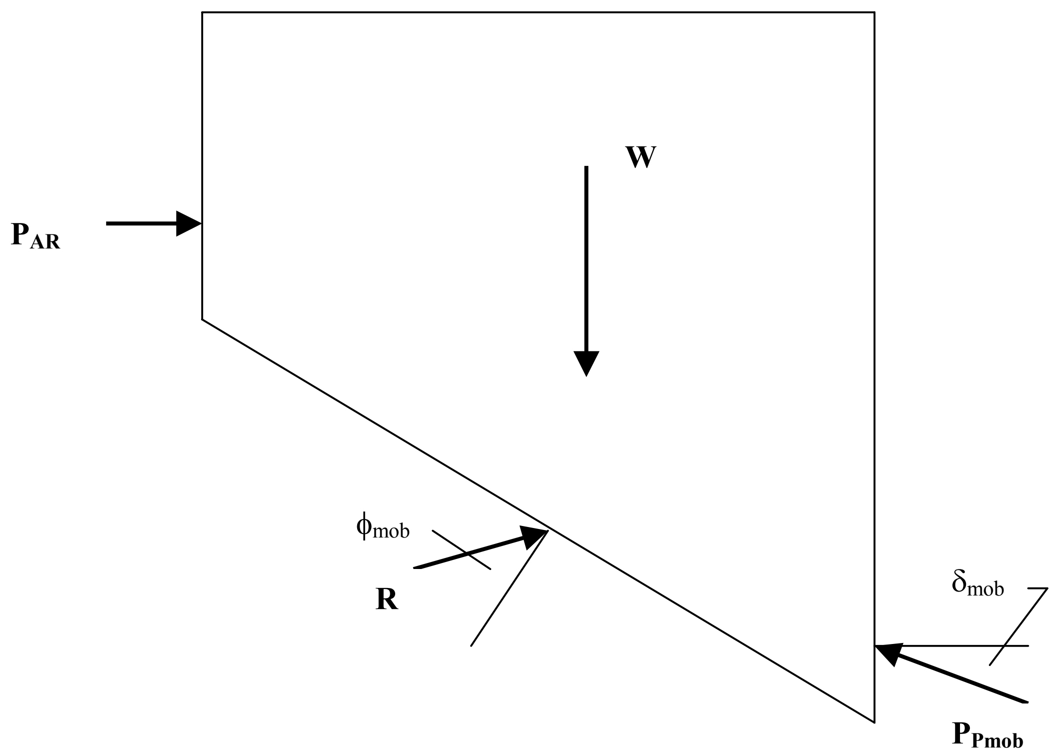


Figure D.2 Force-body diagram

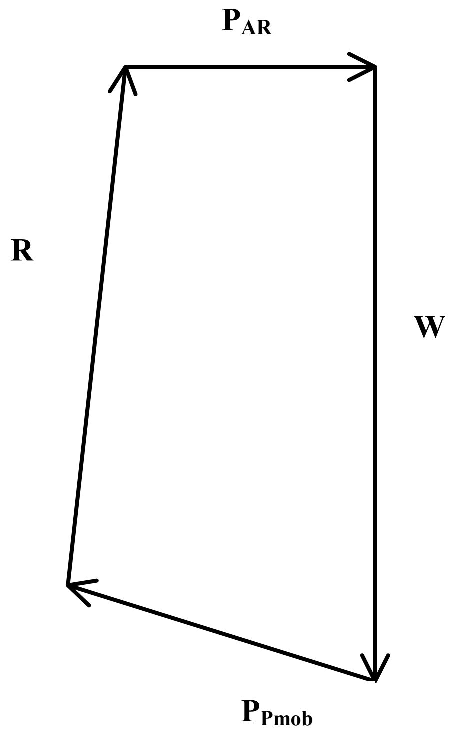


Figure D.3 Force vectors acting on area ABCDEG

Point D
Intersection of anchor slope and failure plane slope

For a given α, solve the following equation to get *y*:
$5y (\tan \alpha) + y = 36 + 50 (\tan \alpha)$

After *y* is determined, solve the following equation to get *x:*
$x = 5y - 50$

For α = 18.0 deg, y = 20.00 ft, x = 50.00 ft

Soil Properties
γ_{moist} = 115 pcf
γ_{sat} = 134.4 pcf
ϕ = 30 deg

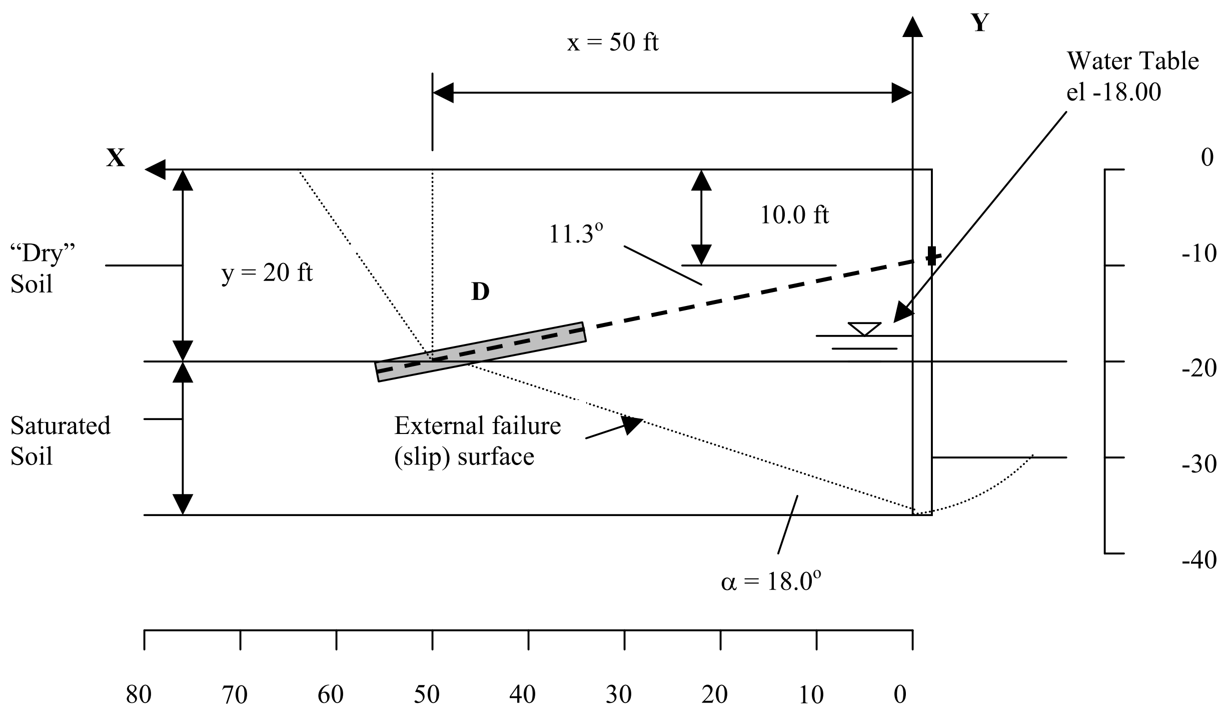


Figure D.4 CSLIDE analysis for 30-ft-high wall–anchor location, retained soil half submerged

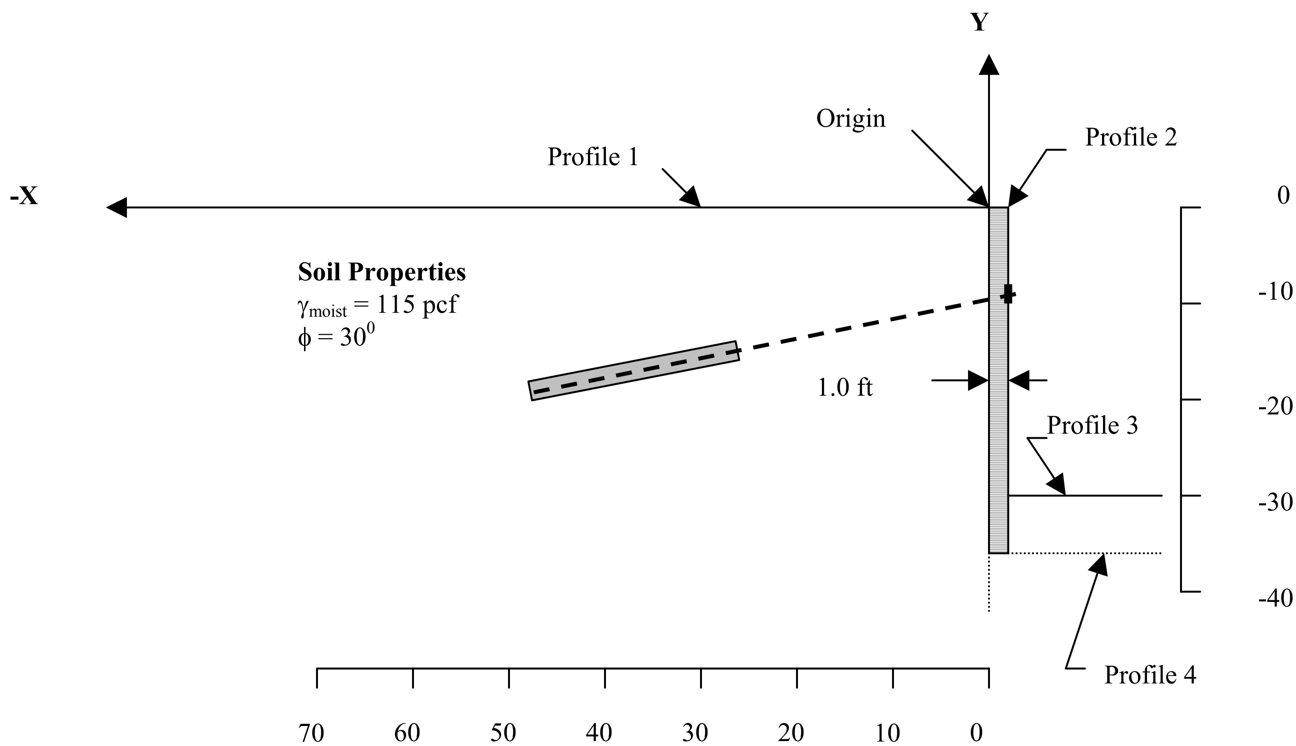


Figure D.5 UTEXAS4 analysis for 30-ft-high wall–profile information, retained soil "dry"

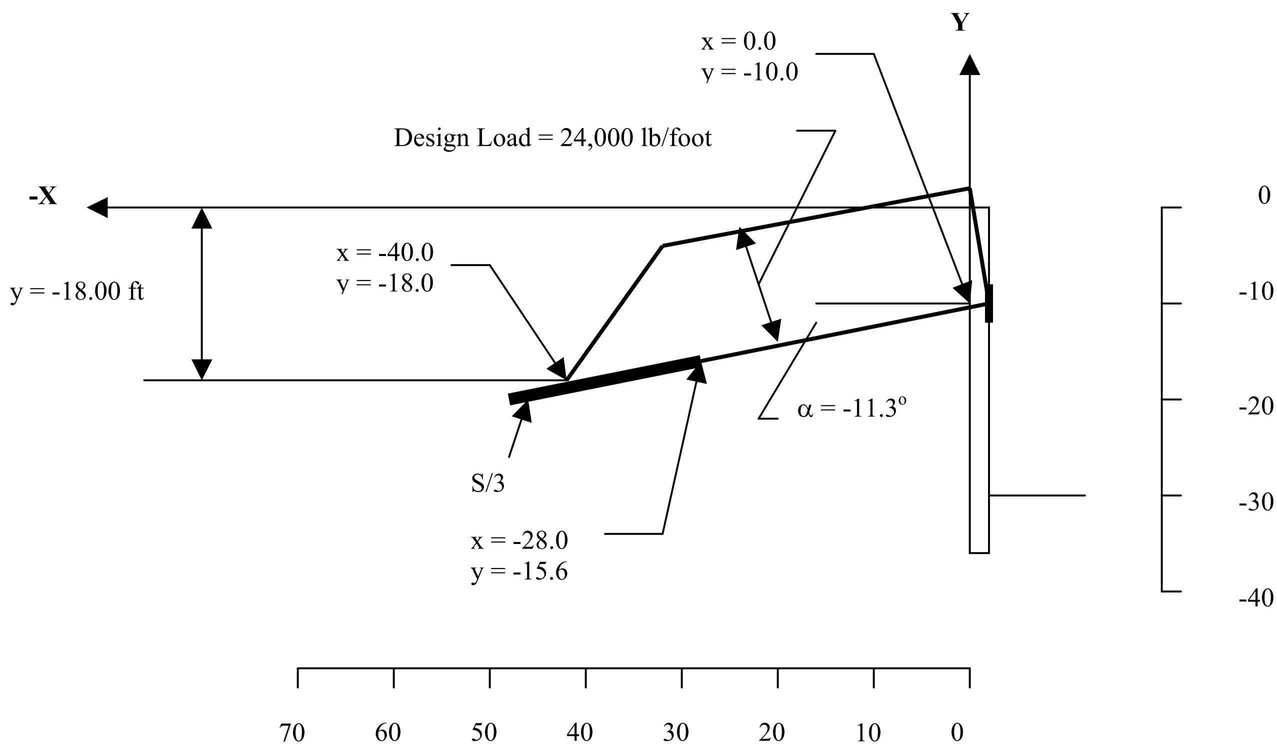


Figure D.6 Analysis for 30-ft-high wall–reinforcement information

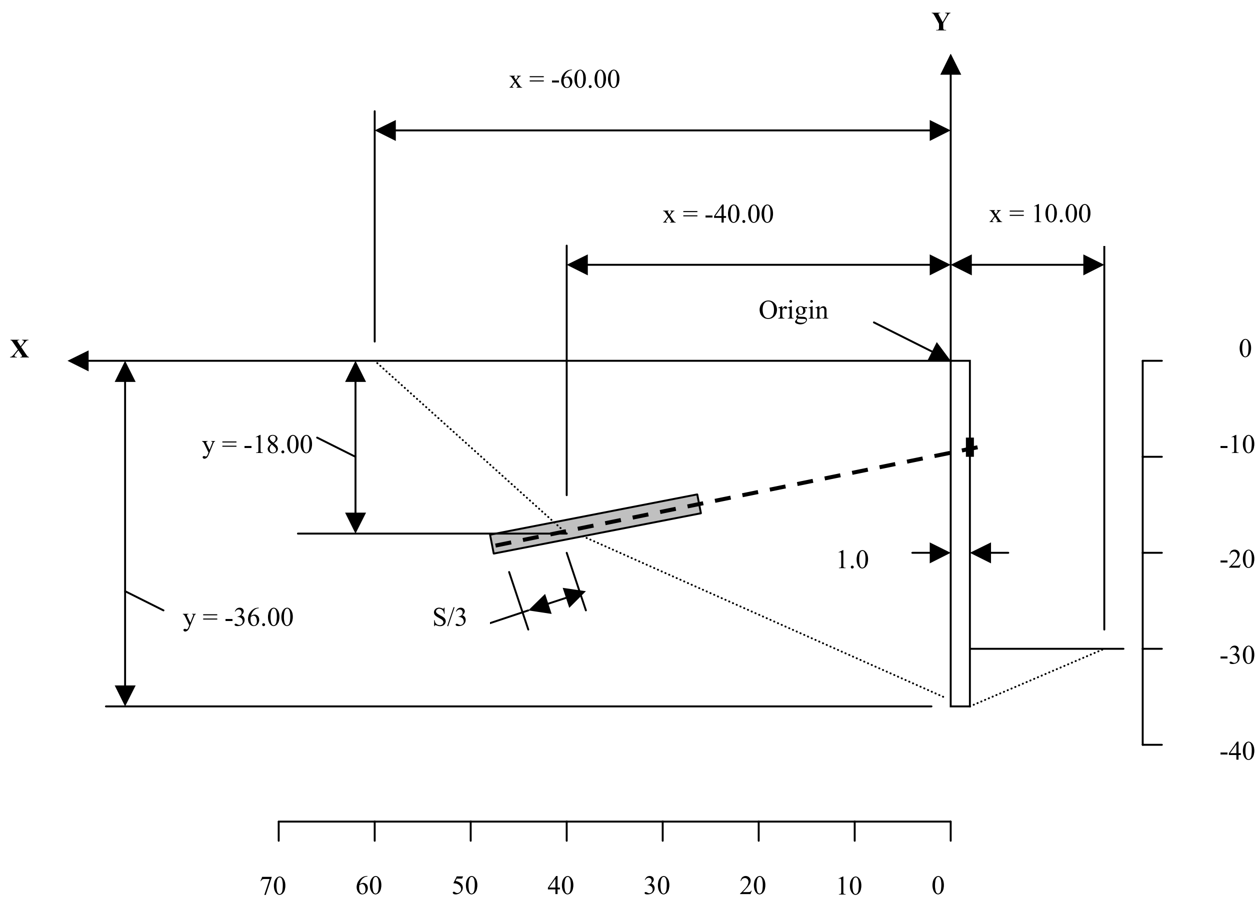


Figure D.7 Analysis for 30-ft-high wall–noncircular failure plane

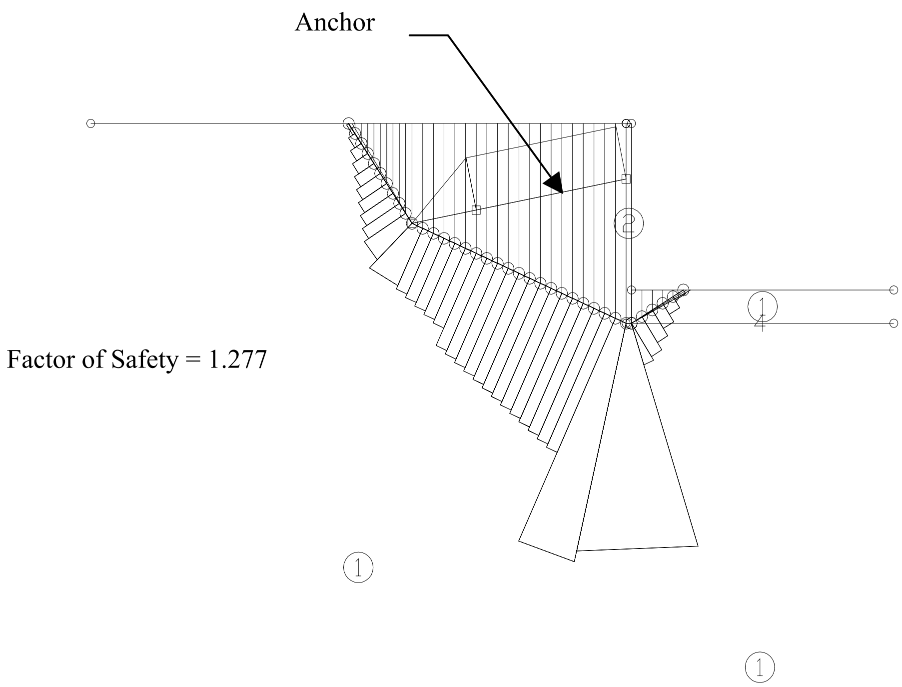


Figure D.8 **UTEXAS4 plot–external stability, "dry," single anchor, noncircular failure surface**

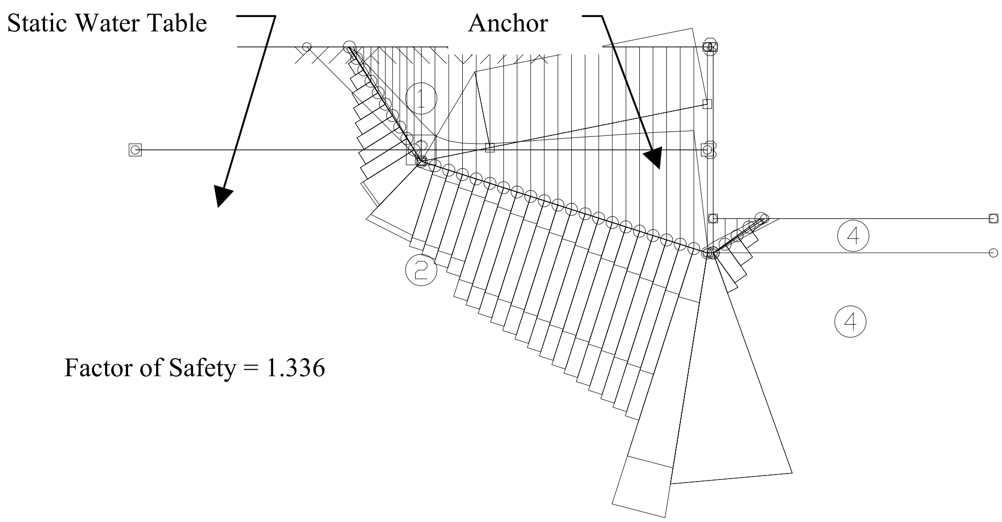


Figure D.9 UTEXAS4 plot–external stability, half submerged (retained side, only), single anchor, noncircular failure surface

Appendix E
External Stability, 30-ft-High Wall–Two Anchors

E.1 General

To illustrate the process as it applies to multi-anchored systems the external stability analysis is performed for a 30-ft wall with two ground anchors. It is assumed that “safety with economy” performance is acceptable (i.e., stringent displacement control is not important). For multi-anchored systems, various potential failure planes in back of and through the anchorage zones are considered. A factor of safety is determined for each potential failure plane, and the results are compared to established performance criteria (i.e., a factor of safety equal to 1.3 for safety with economy performance). The potential failure planes are illustrated in Figure E.1.

Noncircular and circular failure planes that pass in back of the upper and lower anchors are considered. Note that the back of the anchor is assumed at a depth equal to the depth to the end of the bond zone minus one-third the anchor horizontal spacing (in-plan). This reduction is used to account for three-dimensional effects (see FHWA-RD-98-065, paragraph 3.5.2). Assuming anchor spacing equal to 9 ft, the effective anchor depth, measured horizontally from the back of the wall is 59 ft for the upper anchor and 41 ft for the lower anchor. The half-submerged retained backfill condition of Appendix D is used for the analysis. Soil properties are also as indicated in Appendix D. The anchorage depth can be established using the CSLIDE stability approach described in Appendix D. UTEXAS4 is used herein for the final external stability evaluation. The results from the UTEXAS4 program for partially submerged conditions are presented below. The Spencer Procedure was selected for the external stability analysis. Per UTEXAS4 procedures, the search for the critical noncircular shear surfaces is performed based on the procedure developed by Celestino and Duncan (1981). The noncircular failure planes were selected to pass just in back of each effective anchor location, as described in Figure E.1. In addition, the Spencer procedure was used to evaluate the factor of safety on the critical circular failure plane.

E.2 Noncircular Failure Plane Behind Upper Anchor

E.2.1 UTEXAS4 input

```
GRA
HEA
EXTERNAL STABILITY 30-FOOT HIGH WALL  FILE: TBW22.DAT
TWO ANCHORS - FAILURE PLANE IN BACK OF TOP ANCHOR
RETAINED SOIL HALF SUBMERGED
```

```
PRO
     1 1 Cohesionless retained soil above water table
          -100  0
             0  0

     2 2 Cohesionless retained soil below water table
          -100  -18
             0  -18

     3 3 Concrete tieback wall
              0 0
              1 0


     4 1 Cohesionless material below finish grade
              1 -30
             50 -30

     5 1 Cohesionless material below wall
              0 -36
             50 -36


MAT
     1 Cohesionless soil
         115 = unit weight
         Coventional shear strengths
            0 30
         No pore water pressure
     2 Cohesionless soil
          134.2 = unit weight
          Conventional shear strengths
            0 30
          Piezometric Line
            1
     3 Concrete
          145 = unit weight
          Very strong

PIE
     1 PIEZOMETRIC LINE FOR RETAINED SOIL
         -100 -18
            0 -18


REINFORCEMENT LINES
       1 0 1
     -59.0 -19.8 0     0
     -38.0 -15.6 18000 0
       0.0  -8.0 18000 0

       2 0 1
     -51.0 -28.2 0     0
     -30.0 -24.0 18000 0
       0.0 -18.0 18000 0

LAB
```

```
INTERNAL STABILITY ANALYSIS - TWO ANCHORS
ANA
     Noncircular Search
     -80.00   0.00
     -59.00 -19.80 FIX
       0.00 -36.01 FIX
       1.00 -36.01 FIX
      15.00 -30.00

       5.00   1.00
SAV
       4

COM
```

E.2.2 UTEXAS4 output

```
Procedure of Analysis: Spencer


TABLE NO. 41
****************************************
*  Critical Noncircular Shear Surface  *
****************************************

***** CRITICAL NONCIRCULAR SHEAR SURFACE *****
X:     -72.36     Y:       0.00
X:     -59.00     Y:     -19.80
X:       0.00     Y:     -36.01
X:       1.00     Y:     -36.01
X:       8.21     Y:     -30.00

Minimum factor of safety: 1.818
Side force inclination: 3.33

TABLE NO. 55
********************************************************************
* Check of Computations by Spencer's Procedure (Results are for the *
* critical shear surface in the case of an automatic search.)       *
********************************************************************

Summation of Horizontal Forces: 3.85003e-011

Summation of Vertical Forces: 8.61888e-012

Summation of Moments: 2.44654e-010

Mohr Coulomb Shear Force/Shear Strength Check Summation: 1.31672e-011
```

Critical failure plane results for the noncircular failure plane passing in back of the upper anchor are illustrated in Figure E.2.

E.3 Noncircular Failure Plane Behind Lower Anchor

E.3.1 UTEXAS4 input

```
GRA
HEA
EXTERNAL STABILITY 30-FOOT HIGH WALL  FILE: TBW24.DAT
TWO ANCHORS - FAILURE PLANE IN BACK OF BOTTOM ANCHOR
RETAINED SOIL HALF SUBMERGED

PRO
     1 1 Cohesionless retained soil above water table
          -100  0
             0  0

     2 2 Cohesionless retained soil below water table
          -100  -18
             0  -18

     3 Concrete tieback wall
              0 0
              1 0

     4 1 Cohesionless material below finish grade
              1 -30
             50 -30

     5 1 Cohesionless material below wall
              0 -36
             50 -36


MAT
     1 Cohesionless soil
         115 = unit weight
         Coventional shear strengths
            0 30
         No pore water pressure
     2 Cohesionless soil
          134.2 = unit weight
          Conventional shear strengths
            0 30
          Piezometric Line
            1
     3 Concrete
          145 = unit weight
          Very strong

PIE
     1 PIEZOMETRIC LINE FOR RETAINED SOIL
         -100 -18
            0 -18


REINFORCEMENT LINES
       1 0 1
```

```
     -59.0 -19.8 0     0
     -38.0 -15.6 18000 0
       0.0  -8.0 18000 0

       2 0 1
     -51.0 -28.2 0     0
     -30.0 -24.0 18000 0
       0.0 -18.0 18000 0


LAB
INTERNAL STABILITY ANALYSIS - TWO ANCHORS
ANA
     Noncircular Search
     -80.00   0.00
     -51.00 -28.20 FIX
       0.00 -36.01 FIX
       1.00 -36.01 FIX
      15.00 -30.00

       5.00   1.00
SAV
       4

COM
```

E.3.2 UTEXAS4 output

```
Procedure of Analysis: Spencer


TABLE NO. 41
****************************************
*  Critical Noncircular Shear Surface  *
****************************************

***** CRITICAL NONCIRCULAR SHEAR SURFACE *****
X:     -77.72     Y:       0.00
X:     -51.00     Y:     -28.20
X:       0.00     Y:     -36.01
X:       1.00     Y:     -36.01
X:       9.62     Y:     -30.00

Minimum factor of safety: 1.783
Side force inclination: -0.87

TABLE NO. 55
*********************************************************************
* Check of Computations by Spencer's Procedure (Results are for the *
* critical shear surface in the case of an automatic search.)       *
*********************************************************************

Summation of Horizontal Forces: 3.76446e-011

Summation of Vertical Forces: 2.34248e-011
```

```
Summation of Moments: 9.85892e-010

Mohr Coulomb Shear Force/Shear Strength Check Summation: 1.55502e-011
```

Critical failure plane results for the noncircular failure plane passing in back of the lower anchor are illustrated in Figure E.3.

E.4 Circular Failure Plane

E.4.1 UTEXAS4 input

```
GRA
HEA
EXTERNAL STABILITY 30-FOOT HIGH WALL  FILE: TBW26.DAT
TWO ANCHORS - CIRCULAR FAILURE PLANE IN BACK OF ANCHORS
RETAINED SOIL HALF SUBMERGED

PRO
     1 1 Cohesionless retained soil above water table
          -100  0
             0  0

     2 2 Cohesionless retained soil below water table
          -100  -18
             0  -18

     3 3 Concrete tieback wall
              0 0
              1 0

     4 1 Cohesionless material below finish grade
              1 -30
             50 -30

     5 1 Cohesionless material below wall
              0 -36
             50 -36


MAT
     1 Cohesionless soil
         115 = unit weight
         Coventional shear strengths
            0 30
         No pore water pressure
     2 Cohesionless soil
          134.2 = unit weight
          Conventional shear strengths
            0 30
          Piezometric Line
            1
     3 Concrete
          145 = unit weight
          Very strong
```

```
PIE
     1 PIEZOMETRIC LINE FOR RETAINED SOIL
         -100 -18
            0 -18



REINFORCEMENT LINES
       1 0 1
     -59.0 -19.8 0     0
     -38.0 -15.6 18000 0
       0.0  -8.0 18000 0

       2 0 1
     -51.0 -28.2 0     0
     -30.0 -24.0 18000 0
       0.0 -18.0 18000 0



LAB
INTERNAL STABILITY ANALYSIS - TWO ANCHORS
ANA
     Circle Search 1
       0.00   42.00 1 -60
     P
       1.00 -40.00

COM
```

E.4.2 UTEXAS4 output

```
Procedure of Analysis: Spencer


TABLE NO. 33
*********************************************
* 1-STAGE FINAL CRITICAL CIRCLE INFORMATION *
*********************************************
X Coordinate of Center . . . . . . . . . . . . ..  0.00
Y Coordinate of Center . . . . . . . . . . . . ..  56.00
Radius . . . . . . . . . . . . . . . . . . . . . . 96.01
Factor of Safety . . . . . . . . . . . . . . . . . 1.903
Side Force Inclination (degrees) . . . . . . . .. -3.11
Number of Circles Tried  . . . . . . . . . . . .  76
Number of Circles F Calculated for . . . . . . .  65
Time Required for Search (seconds) . . . . . . .  1.3

TABLE NO. 55
**********************************************************************
* Check of Computations by Spencer's Procedure (Results are for the *
* critical shear surface in the case of an automatic search.)       *
**********************************************************************

Summation of Horizontal Forces: 3.87974e-011
```

```
Summation of Vertical Forces: 2.72067e-011

Summation of Moments: -2.12367e-010

Mohr Coulomb Shear Force/Shear Strength Check Summation: 2.09646e-011
```

Critical failure plane results for the circular failure plane are illustrated in Figure E.4.

Table E.1 Results Summary

Failure Plane Type	Failure Plane Location (See Figure E.1)	Analysis Method	Factor of Safety
Noncircular	Behind upper anchor	Spencer	1.818
Noncircular	Behind lower anchor	Spencer	1.783
Circular	Floating grid search	Spencer	1.903

The Spencer Procedure in UTEXAS4 was used to evaluate the external stability on two noncircular (linear) failure planes selected as shown in Figure E.1. One of the noncircular shear surface failure planes was selected to pass just behind the upper effective anchor location; the other, just behind the lower effective anchor location. Also, a floating grid search was used to find the circular shear surface failure plane producing the lowest factor of safety. The results summarized in Table E.1 indicate the anchor location selected would be capable of satisfying both "safety with economy" and "stringent displacement control" performance objectives. These have factor of safety requirements of 1.3 and 1.5, respectively.

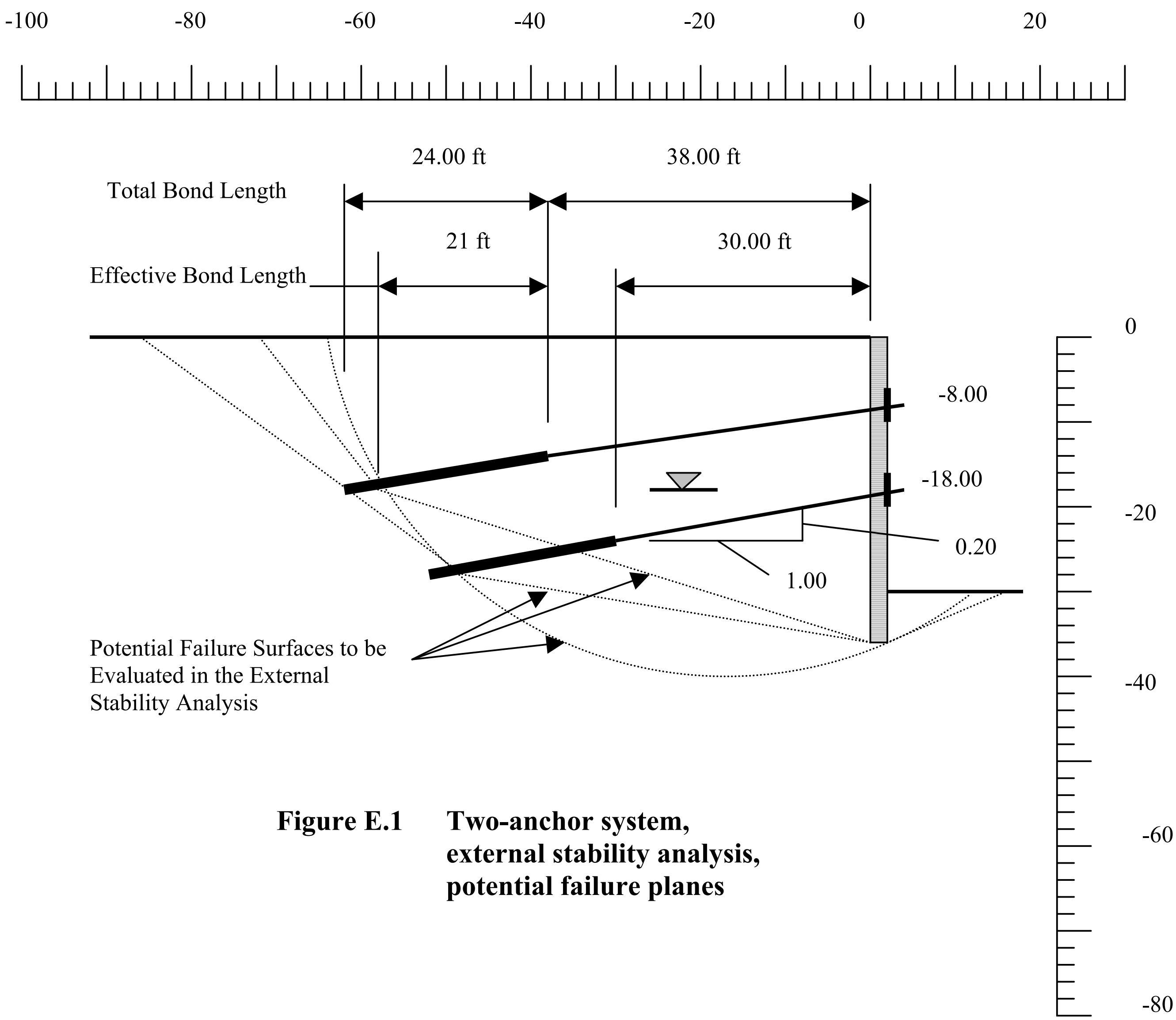


Figure E.1 Two-anchor system, external stability analysis, potential failure planes

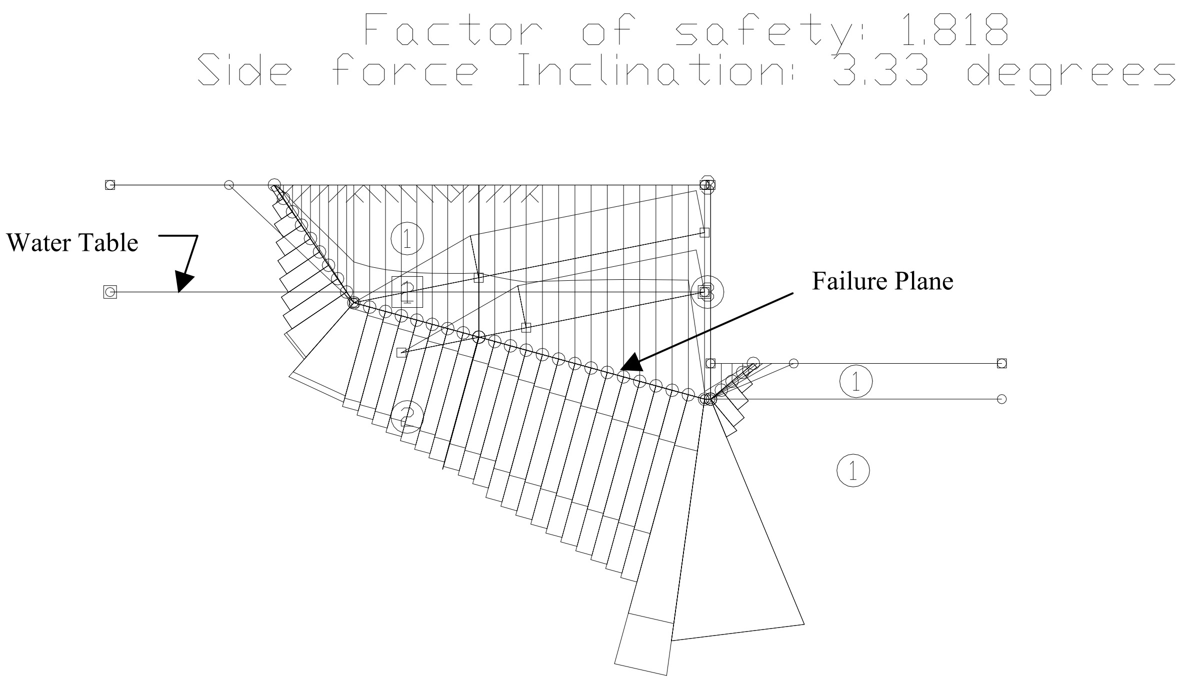


Figure E.2 External stability analysis, failure plane behind upper anchor, noncircular failure plane, Spencer Procedure

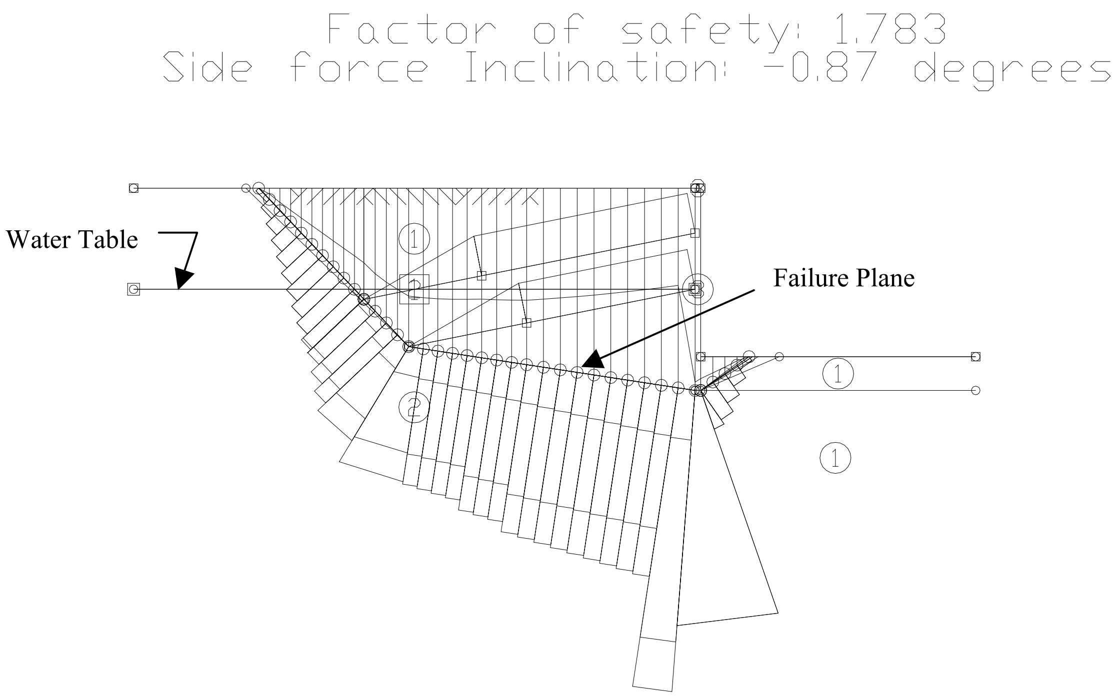


Figure E.3 External stability analysis, failure plane behind lower anchor, noncircular failure plane, Spencer procedure

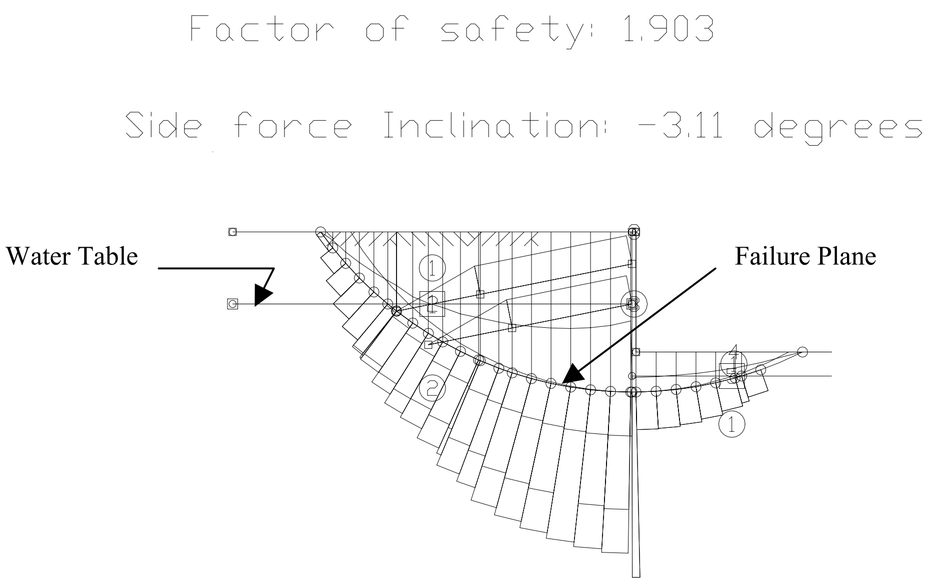


Figure E.4 External stability analysis, circular failure plane, Spencer Procedure

Appendix F
Layered Soil System–Internal and External Stability

F.1 General

This appendix illustrates the internal and external stability analysis process for a tieback wall in a layered soil system with soft to medium stiff clays below the wall and a piezometric water surface in the retained soil. The example was taken from Pockoski and Duncan (2000). The tieback wall is 44 ft high with three tiebacks located at elevations 104, 89, and 74, spaced at 4 ft on center in the horizontal direction. The tieback wall and layered soil system are illustrated in Figure F.1.

The anchorage for each tieback is 40 ft long. With an allowable pullout resistance of 4 kips per foot, each anchor therefore has an allowable load capacity of 160 kips. The force provided to stabilize the cut (P_{reqd}) is therefore equal to (160 × 3 rows)/ 4, or 120 kips per foot of wall. Information on soil properties and tiebacks is provided in Table F.1.

Since the soil is layered and has a piezometric water surface, the stability analyses will be performed using a general-purpose slope stability (GPSS) program. Information on various GPSS programs used in the stability evaluation of tieback wall systems can be found in Pockoski and Duncan (2000). The GPSS program used for these particular analyses is UTEXAS4. It has the ability to perform a Spencer Procedure analysis (Spencer 1967, Wright 1999); a Bishop's Simplified Procedure analysis (Bishop 1955); a Corps of Engineers' Modified Swedish Procedure analysis (Headquarters, Department of the Army 1970); and a Lowe and Karafaith (1960) Procedure analysis. The Spencer Procedure is used for both external and internal stability since it is the only method in UTEXAS4 that considers both force and moment equilibrium. The other methods, however, are used to check the results of the Spencer Procedure analyses. Checking is important because most of the GPSS programs, which trace their origin to the 1960s and 1970s, were not developed with tieback wall applications in mind. Checking by various other GPSS methods is always advisable since there are definitely some unusual things that can happen internally within the GPSS programs, especially with respect to tieback wall systems when searching for the critical failure plane (i.e., plane with the lowest factor of safety).

F.2 Internal Stability Analysis

The force required to provide stability to the cut (P_{reqd}) will be determined by GPSS analysis by applying a uniform surcharge to the wall face per Method 1 of FHWA-RD-97-130. The surcharge is as illustrated in Figure F.1.

The magnitude of the surcharge is varied until a factor of safety equal to 1.3 is obtained along the critical failure plane. A factor of safety equal to 1.3 is required to satisfy "safety with economy" performance requirements. The surcharge requirements are summarized in Table F.2.

The surcharge associated with a factor of safety of 1.3 is multiplied by the wall height (i.e., 44 ft) to obtain the total force (P_{reqd}) required to stabilize the cut. The input data for the UTEXAS4 analysis that produced the required factor of safety are presented below.

UTEXAS4 Input

```
GRA
HEA
INTERNAL STABILITY 44-FOOT HIGH WALL  FILE: VTLI3.DAT
CIRCULAR FAILURE PLANE BELOW BOTTOM OF WALL
VIRGINIA TECH SLOPE NO. 4 LAYERED SOIL PROBLEM - SPENCER PROCEDURE

PRO
     1 1 Concrete tieback wall
           -1  113
            0  113

     2 2 Granular fill above water table
            0  113
          200  113

     3 3 Cohesive fill
            0  109
          200  109

     4 4 Organic silt
            0  92.5
          200  92.5

     5 5 OC crust
            0  86
          200  86

     6 6 Upper marine clay
            0  82.5
          200  82.5

     7 7 Middle marine clay
         -300  69
          200  69

     8 8 Lower marine clay
         -300  39.5
          200  39.5

     9 9 Glaciomarine deposits
         -300  24
          200  24
```

```
MAT
     1 Concrete
          145 = unit weight
          Very strong
     2 Granular fill
         120.4 = unit weight
         Conventional shear strengths
            0 30
         Piezometric Line
            1
     3 Cohesive fill
          114.7 = unit weight
          Conventional shear strengths
            0 30
          Piezometric Line
            1
     4 Organic silt
         110.2 = unit weight
         Conventional shear strengths
          900 0
         Piezometric Line
            1
     5 OC crust
         117.8 = unit weight
         Conventional shear strengths
          2485 0
         Piezometric Line
           1
     6 Upper marine clay
         117.8 = unit weight
         Conventional shear strengths
          1670 0
         Piezometric Line
           1
     7 Middle marine clay
         117.8 = unit weight
         Conventional shear strengths
          960 0
         Piezometric line
           1
     8 Lower marine clay
        117.8 = unit weight
        Conventional shear strengths
         1085 0
        Piezometric Line
          1
     9 Glaciomarine deposits
       147.1 = unit weight
       Conventional shear strengths
        1500 0
        Piezometric Line
         1
```

```
PIE
     1 PIEZOMETRIC LINE FOR RETAINED SOIL
         -300  69.0
            0  69.0
            0 102.5
          200 102.5


DISTRIBUTED LOADS
    -300.00  69.00     0      0
      -1.00  69.00     0      0
      -1.00  69.00  2500   -910
      -1.00 113.00  2500   -910
      -1.00 113.00     0      0
     200.00 113.00     0      0
```

```
The surcharge pressure in "bold" was
varied until a factor of safety equal
to 1.3 was obtained.
```

```
LAB
INTERNAL STABILITY ANALYSIS
ANA
     Circular Search 1
       0.00  120.00  2  20
      T
      24.00

COM
```

The horizontal component of the surcharge is 2,500 lb per square foot of wall, meaning the force required to stabilize the cut is equal to

$$P_{reqd} = \frac{2500}{\cos 20^o}(44) = 117 \text{ kips per foot of wall} < 120 \text{ kips provided} \quad \text{OKAY}$$

The above analysis procedure represents one method for determining the force required to stabilize the cut. A tieback wall constructed under similar layered soil conditions is described in Cacoilo, Tamaro, and Edinger (1998). In that particular design, the criteria originally used to determine anchor forces were based on an effective pressure factor of 25 psf. However, the final anchor loads were determined based on a construction sequencing analysis. The types of analyses used for layered soil systems, especially for sites containing soft clay deposits, should be determined by experienced soils and structural engineers.

F.3 External Stability Analysis

An external stability analysis will now be performed to verify that the tiebacks are adequate (i.e., the critical failure plane does not pass through the anchors) and to ensure that the minimum factor of safety against external stability failure is adequate. In the external stability analysis, the tieback loads contributing to stability are determined by modeling the tiebacks as reinforcement rather than as a surcharge loading. For the multi-anchored tieback wall system (three anchors) illustrated in Figure F.2, various potential circular failure planes in back of and through the anchorage zones are investigated by GPSS analysis using the UTEXAS4 software.

In the initial analysis, a "floating grid" search is used to find the critical failure surface, and a Spencer Procedure analysis is performed. Once the minimum factor of safety has been determined, analyses by the Simplified Bishop procedure, the Simplified Janbu Procedure (i.e., Corps of Engineers' procedure), and the Lowe and Karafaith Procedure were conducted to verify the results. In addition, the results were verified by an "individual circular" shear-surface analysis. This was accomplished using the Spencer Procedure with a circular shear surface at the same location determined by the floating grid search. The individual circular shear-surface analysis produced a factor of safety identical to that determined by the floating grid analysis.

Two other individual circular shear-surface analyses were performed using a circular shear surface of the same radius as that determined by the floating grid search. These shear surfaces were located to pass through various anchorage zones, as illustrated in Figure F.3.

The purpose of the two additional individual circular shear-surface analyses was to verify that a lower factor of safety did not exist on failure planes through anchorage zones.

Input for the UTEXAS4 external stability analysis using the Spencer floating grid search procedure is presented below. The data input for the other floating grid procedures identified in Table F.3 is identical to that for the Spencer floating grid procedure, except a special analysis command is used to request the special analysis procedure desired (i.e., a procedure other than the Spencer Procedure).

```
UTEXAS4 Input


GRA
HEA
EXTERNAL STABILITY 30-FOOT HIGH WALL  FILE: VTLE2.DAT
THREE ANCHORS - CIRCULAR FAILURE PLANE IN BACK OF ANCHORS
VIRGINIA TECH SLOPE NO. 4 LAYERED SOIL PROBLEM

PRO
     1 1 Concrete tieback wall
           -1  113
            0  113

     2 2 Granular fill above water table
            0  113
          200  113
     3 3 Cohesive fill
            0  109
          200  109

     4 4 Organic silt
            0  92.5
          200  92.5
```

```
     5 5 OC crust
             0  86
           200  86

     6 6 Upper marine clay
             0  82.5
           200  82.5

     7 7 Middle marine clay
          -300  69
           200  69

     8 8 Lower marine clay
          -300  39.5
           200  39.5

     9 9 Glaciomarine deposits
          -300  24
           200  24


MAT
     1 Concrete
          145 = unit weight
          Very strong
     2 Granular fill
         120.4 = unit weight
         Conventional shear strengths
            0 30
         Piezometric Line
            1
     3 Cohesive fill
          114.7 = unit weight
          Conventional shear strengths
            0 30
          Piezometric Line
            1
     4 Organic silt
         110.2 = unit weight
         Conventional shear strengths
          900 0
         Piezometric Line
            1
     5 OC crust
         117.8 = unit weight
         Conventional shear strengths
          2485 0
         Piezometric Line
           1
     6 Upper marine clay
         117.8 = unit weight
         Conventional shear strengths
          1670 0
         Piezometric Line
           1
```

```
     7  Middle marine clay
          117.8 = unit weight
          Conventional shear strengths
           960 0
          Piezometric line
             1
     8 Lower marine clay
         117.8 = unit weight
         Conventional shear strengths
          1085 0
         Piezometric Line
           1
     9 Glaciomarine deposits
        147.1 = unit weight
        Conventional shear strengths
         1500 0
         Piezometric Line
          1

PIE
     1 PIEZOMETRIC LINE FOR RETAINED SOIL
         -300  69.0
            0  69.0
            0 102.5
          200 102.5


REINFORCEMENT LINES
       1 0 1
       0.00 104.00 40000 0
      45.11  87.58 40000 0
      82.69  73.90     0 0

       2 0 1
       0.00  89.00 40000 0
      45.11  72.58 40000 0
      82.69  58.90     0 0

       3 0 1
       0.00  74.00 40000 0
      45.11  57.58 40000 0
      82.69  43.90     0 0


LAB
EXTERNAL STABILITY ANALYSIS - THREE ANCHORS
ANA
     Circle Search 1
       8.00   147.00 1 0
     P
       0.00 24.00
```

F.4 Results Summary

The results of the various GPSS external stability analyses are presented in Table F.3.

In the internal stability evaluation, the force required to provide stability to the cut (P_{reqd}) was determined by GPSS analysis by applying a uniform surcharge to the wall face per Method 1 of FHWA-RD-97-130. The analysis indicated the anchor system provided will meet "safety with economy" performance requirements.

With respect to the external stability evaluations, the results in terms of factors of safety for the various analytical procedures used matched those determined by the Virginia Polytechnic Institute and State University analysis (Pockoski and Duncan 2000). The minimum factor of safety provided by the tieback anchor system illustrated in Figure F.2 is less than the target value (i.e., factor of safety of 1.3) required for safety with economy performance. With soft clay layers located below the tieback wall, the failure planes can be deep and located beyond the end of the tieback anchorage zones. Attempts should be made to bring the factor of safety up to the target performance level. This can sometimes be accomplished by increasing the length of the tiebacks; however, this may not significantly improve the external stability factor of safety (Cacoilo, Tamaro, and Edinger 1998).

Table F.1 Soil and Tieback Properties

Material	***γ (lb/ft)***	***c (psf)***	***ϕ (deg)***
Granular fill	120.4	0	30
Cohesive fill	114.7	0	30
Organic silt	110.2	900	0
OC crust	117.8	2485	0
Upper marine clay	117.8	1670	0
Middle marine clay	117.8	960	0
Lower marine clay	117.8	1085	0
Glaciomarine deposits	147.1	1500	0

Notes:

1. Tieback spacing at 4.0 ft OC.
2. 1.08-in.-diam 270-ksi strand
3. 4 kips/ft allowable pullout

Table F.2 INTERNAL STABILITY ANALYSES
Virginia Tech Problem – Slope No. 4
Tieback Wall in Layered Soil - Factor of Safety = 1.3

Method	File	Shear Surface Circular or Noncircular	Grid Floating or Fixed	Required Surcharge Loading (psf)
Spencer's Procedure - failure surface below bottom of wall	VTLI3	Circular	Floating	2,500

Table F.3 EXTERNAL STABILITY ANALYSES
Virginia Tech Problem – Slope No. 4
Tieback Wall in Layered Soil

Method	File	Shear Surface Circular or Noncircular	Grid Floating or Fixed	Factor of Safety
Spencer's Procedure	VTLE2	Circular	Floating	1.145
Spencer's Procedure	VTLE5	Circular	Fixed X = 8 Y = 147 R = 123	1.145
Spencer's Procedure	VTLE7	Circular	Fixed X = -4 Y = 147 R = 123	1.160
Spencer's Procedure	VTLE9	Circular	Fixed X = -16 Y = 147 R = 123	1.224
Simplified Bishop Procedure	VTLE4	Circular	Floating	1.145
Simplified Janbu Procedure	VTLE6	Circular	Floating	1.134
Lowe and Karafaith's Procedure	VTLE8	Circular	Floating	1.199

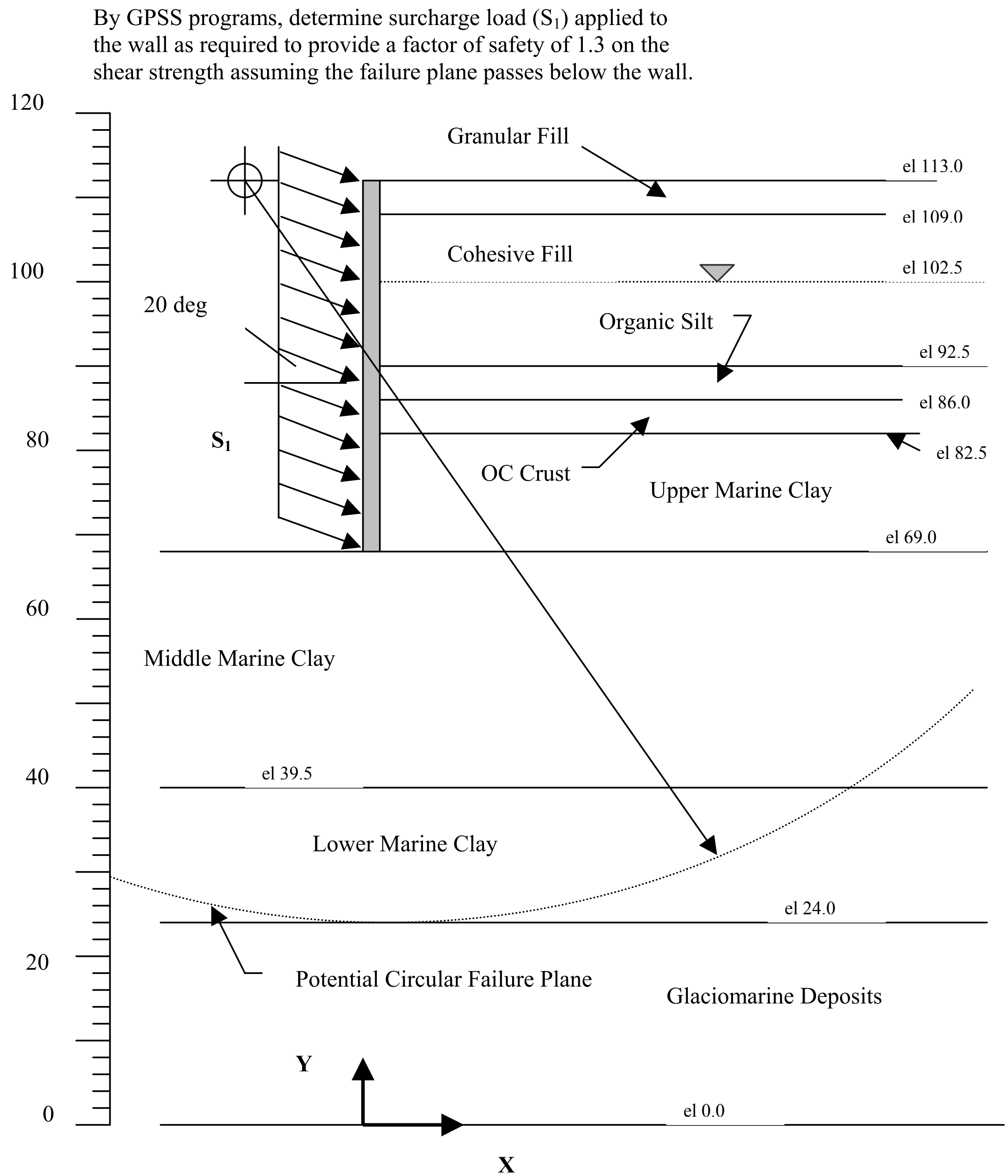


Figure F.1 Tieback wall in layered soils (after Pockoski and Duncan 2000), internal stability analysis–second analysis

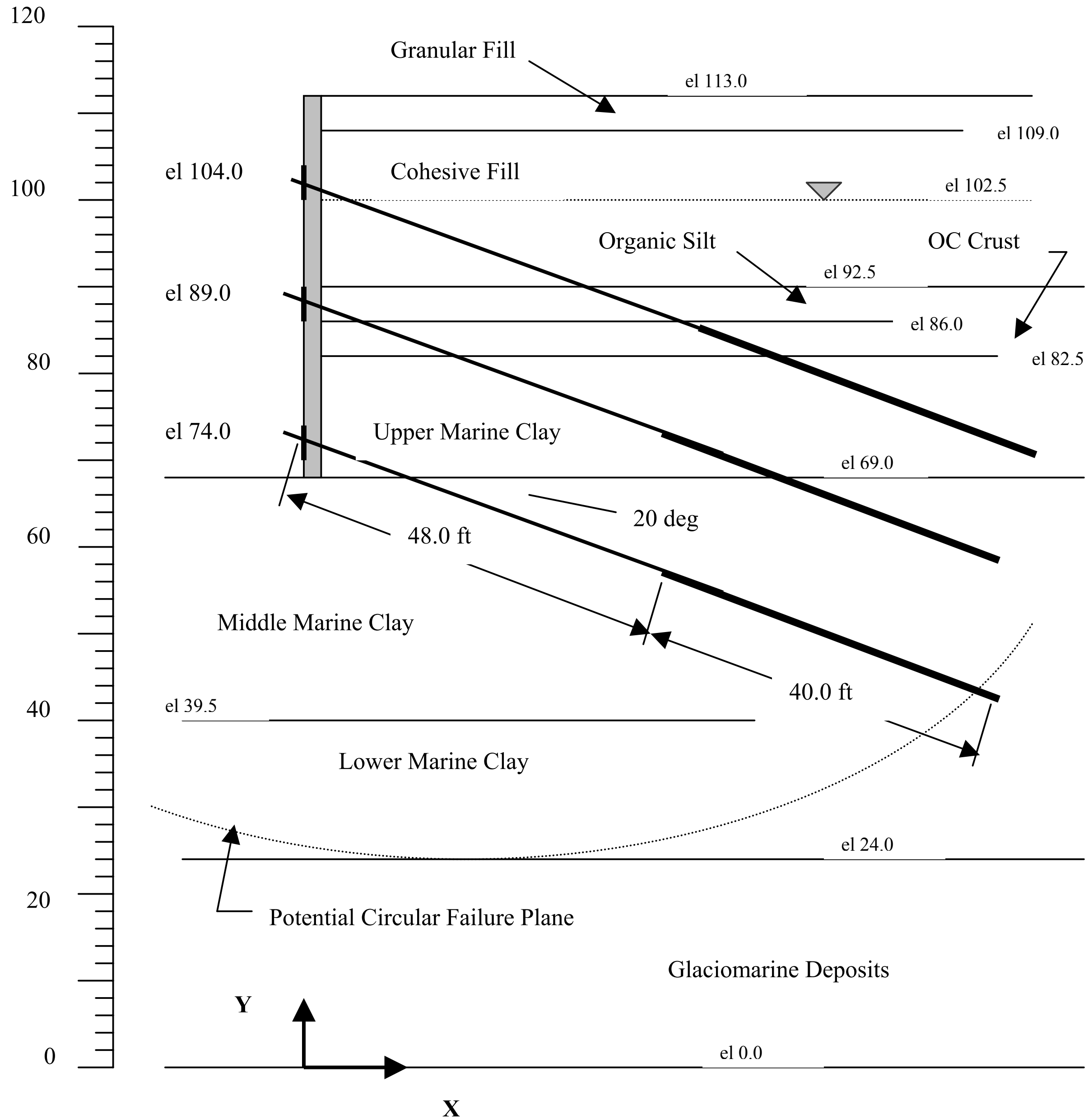


Figure F.2 Tieback wall in layered soils (after Pockoski and Duncan 2000), external stability analysis

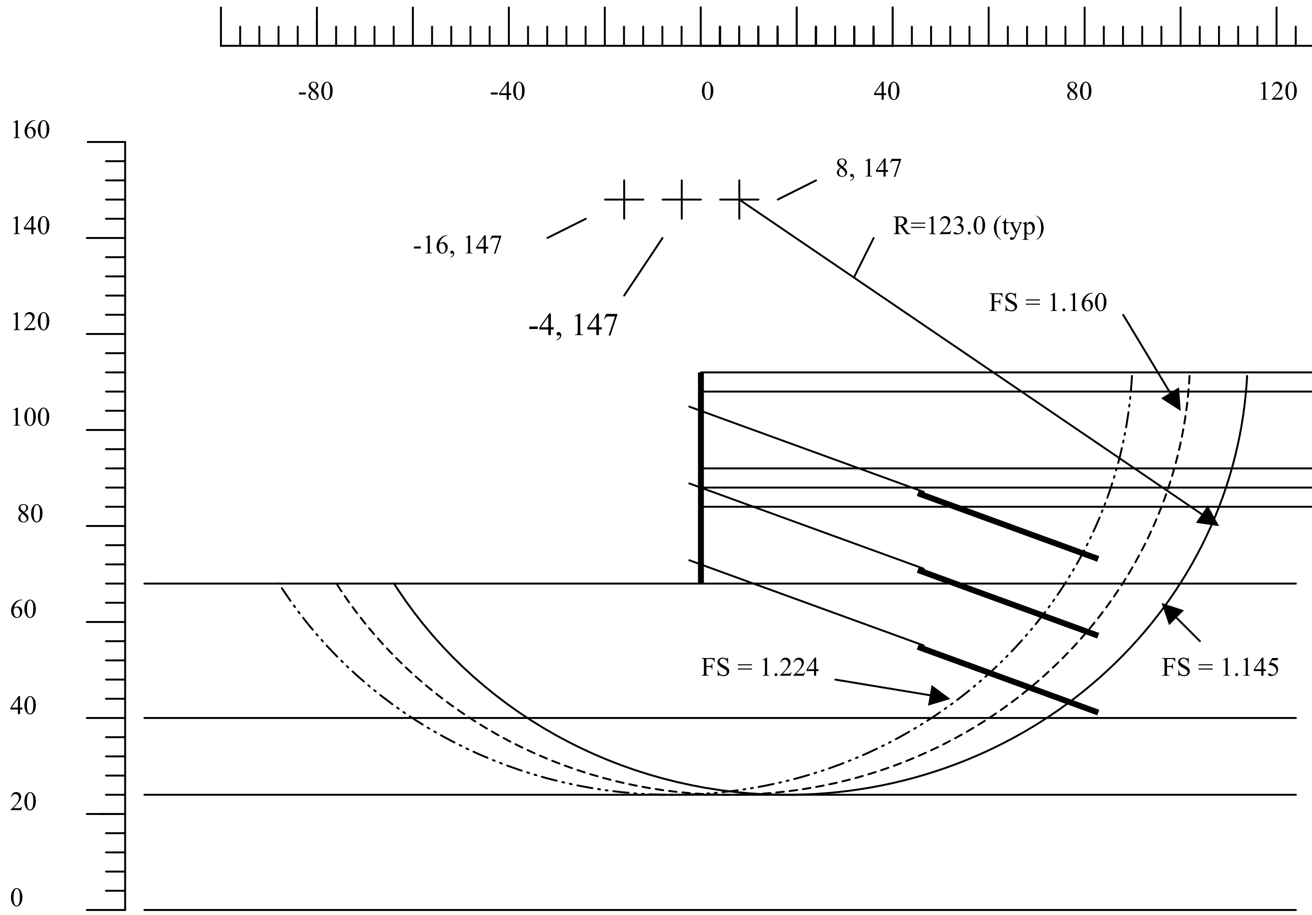


Figure F.3 Fixed circle analysis

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14. ABSTRACT

A local failure that spreads throughout a tieback wall system can result in progressive collapse. The risk of progressive collapse of tieback wall systems is inherently low because of the capacity of the soil to arch and redistribute loads to adjacent ground anchors. The current practice of the U.S. Army Corps of Engineers is to design tieback walls and ground anchorage systems with sufficient strength to prevent failure due to the loss of a single ground anchor.

Results of this investigation indicate that the risk of progressive collapse can be reduced by using performance tests, proof tests, extended creep tests, and lift-off tests to ensure that local anchor failures will not occur and to ensure the tieback wall system will meet all performance objectives; by using yield line (i.e., limit state) analysis to ensure that failure of a single anchor will not lead to progressive failure of the tieback wall system; by verifying (by limiting equilibrium analysis) that the restraint force provided by the tieback anchors provides an adequate margin of safety against an internal stability failure; and by verifying (by limiting equilibrium analysis) that the anchors are located a sufficient distance behind the wall face to provide an adequate margin of safety against external stability (ground mass) failure.

(Continued)

15. SUBJECT TERMS

Anchor failure
Anchored wall
Ground mass stability
Tieback wall

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Design measures that can be used to protect against local anchor failure are described, along with testing methods that can be used to ensure that anchor performance meets project performance objectives. Examples are given to demonstrate the yield line analysis techniques that are used to verify that the wall system under the “failed anchor” condition can safely deliver loads to adjacent anchors and to ensure that the failure of a single anchor will not lead to progressive wall failure are. Limiting equilibrium analysis procedures used for the internal and external stability of tieback wall systems are also described. Simple procedures applicable to “dry” homogeneous sites and general-purpose slope stability programs applicable to layered sites (with and without a water table) are also illustrated by example.