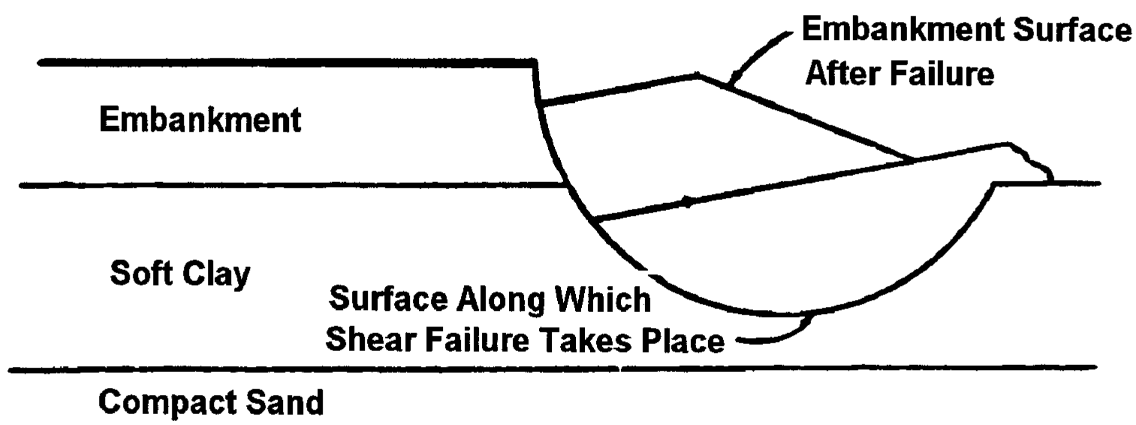
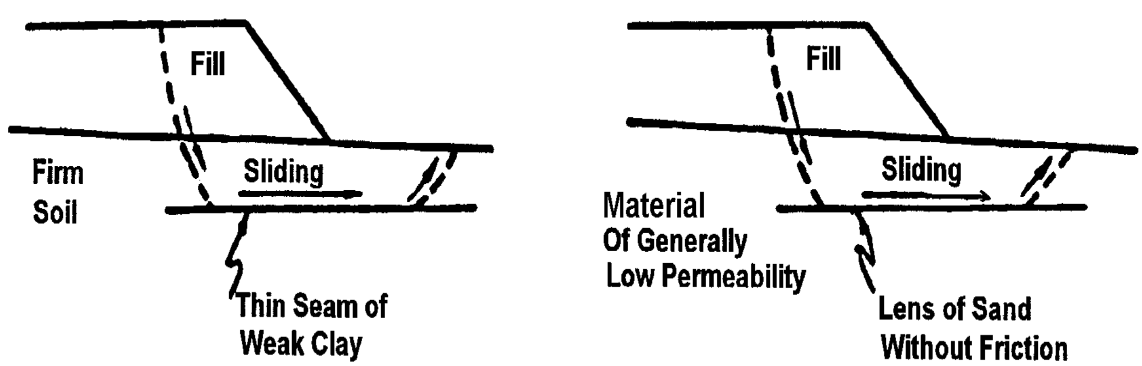
CHAPTER 5.0
SLOPE STABILITY

Ground stability must be assured prior to consideration of other foundation related items. Embankment foundation problems involve the support of the embankment by natural soil. Problems with embankments and structures occasionally occur which could be prevented by initial recognition of the problem and appropriate design. Stability problems most often occur where the embankment is to be built over soft weak soils such as low strength clays, silts, or peats. Once the soil profile, soil strengths, and depth of water table have been determined by both field explorations and field and lab testing, the stability of the embankment can be analyzed and factor of safety estimated.

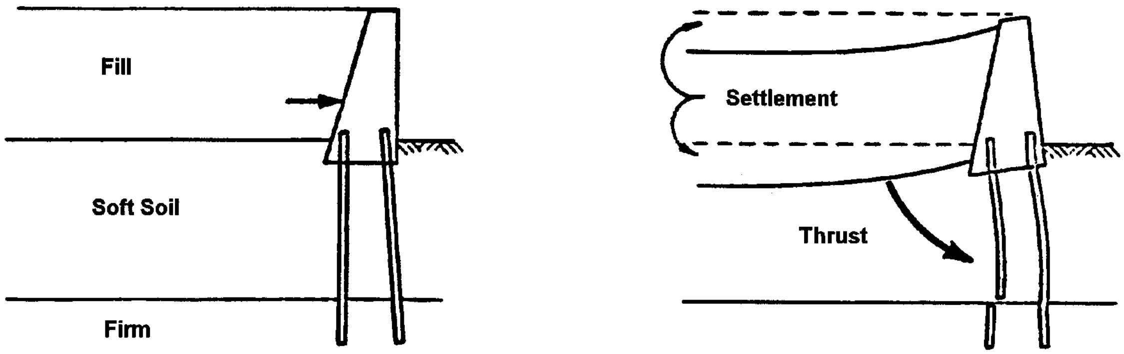
There are three major types of instability that should be considered in the design of embankments over weak foundation soils. These are illustrated in Figure 5-1.



a. Circular Arc Failure



b. Sliding Block Failure



c. Lateral Squeeze of Foundation Soil

Figure 5-1(a, b, and c): Major Types of Approach Embankment Stability Problems

Recommendations on how to recognize, analyze, and solve each of these three problems are presented in this chapter.

These stability problems as illustrated in Figure 5-1 are "external" stability problems. "Internal" embankment stability problems generally result from the selection of poor quality embankment materials and/or improper placement requirements. Internal stability may be "ordered" in project specifications by specifying granular materials with minimum gradation and compaction requirements. An example of a typical specification for approach embankment construction is shown in Chapter 6.

5.1 EFFECTS OF WATER ON SLOPE STABILITY

- **Importance of Water**

  Next to gravity, water is the most important factor in slope stability.

- **Effect of Water on Frictional Soils**

  In cohesionless soils, water does not affect the angle of internal friction (ϕ). The effect of water on cohesionless soils below the water table is to decrease the intergranular (effective) pressure between soil grains which decreases the frictional shearing resistance.

- **Effect of Water on Clays**

  Routine seasonal fluctuations in the water table do not usually influence either the amount of water in the pore spaces between soil grains or the cohesion. The attractive forces between soil particles prevent water absorption unless external forces such as pile driving, disrupt the grain structure. However, certain clay minerals do react to the presence of water and cause expansion of the clay mass.

  An increase in absorbed moisture is a major factor in the decrease in strength of expansive cohesive soils (Figure 5-2). Water is absorbed by expansive clay minerals, causing high water contents which decrease the cohesion of expansive clayey soils.

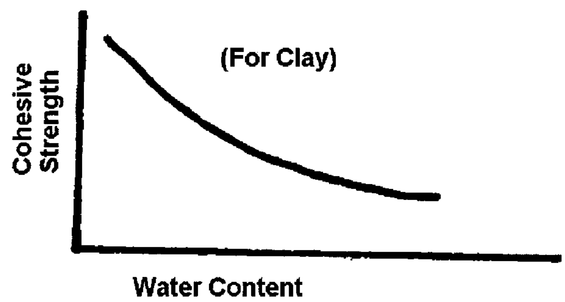


Figure 5-2: Effect of Water Content on Cohesive Strength of Clay

- **Fills on Clays**

  Excess pore pressures are created when fills are placed on clay or silt. As the pore pressure dissipates, consolidation occurs, and the clay or silt strength increases. This is the reason the factor of safety increases with time.

- **Cuts in Clay**

  As a cut is made in clay the effective stress is reduced. This will allow the clay to expand and absorb water, which will lead to a decrease in the clay strength with time. This is the reason the factor of safety of a clay cut slope decreases with time. Cut slopes in clay should be designed using effective strength parameters and the effective stress which will exist after the cut is made.

- **Slaking - Shales, Claystones, Siltstones, etc.**

  Sudden moisture increase in a dry soil can produce a pore pressure increase in trapped pore air accompanied by local soil expansion and strength decrease. The "slaking" or sudden disintegration of hard shales, claystones, and siltstones result from this mechanism. If placed as rock fill, water percolating through the fill causes these materials to disintegrate to a clay soil, which often leads to settlement and/or shear failure of the fill. Index tests such as the jar-slake test and the slake-durability test are shown in "Design and Construction of Compacted Shale Embankments," FHWA RD-78-14.

5.2 DESIGN FACTOR OF SAFETY

A minimum factor of safety of 1.25 is ordinarily used for highway embankment side slopes. This safety factor value should be increased to a minimum of 1.30 for slopes whose failure would cause significant damage such as end slopes beneath bridge abutments, major retaining structures, etc. The selection of the actual safety factor to be used on a particular project depends on:

- Stability analysis method used.
- Method of shear strength determination.
- Confidence in reliability of subsurface data.
- Consequences of failure.

5.3 CIRCULAR ARC FAILURE

Experience and observations of failures of embankments built over relatively deep deposits of soft foundation soils have shown that when failure occurs, the embankment sinks down, the adjacent ground rises and the failure surface follows a circular arc as illustrated in Figure 5-3.

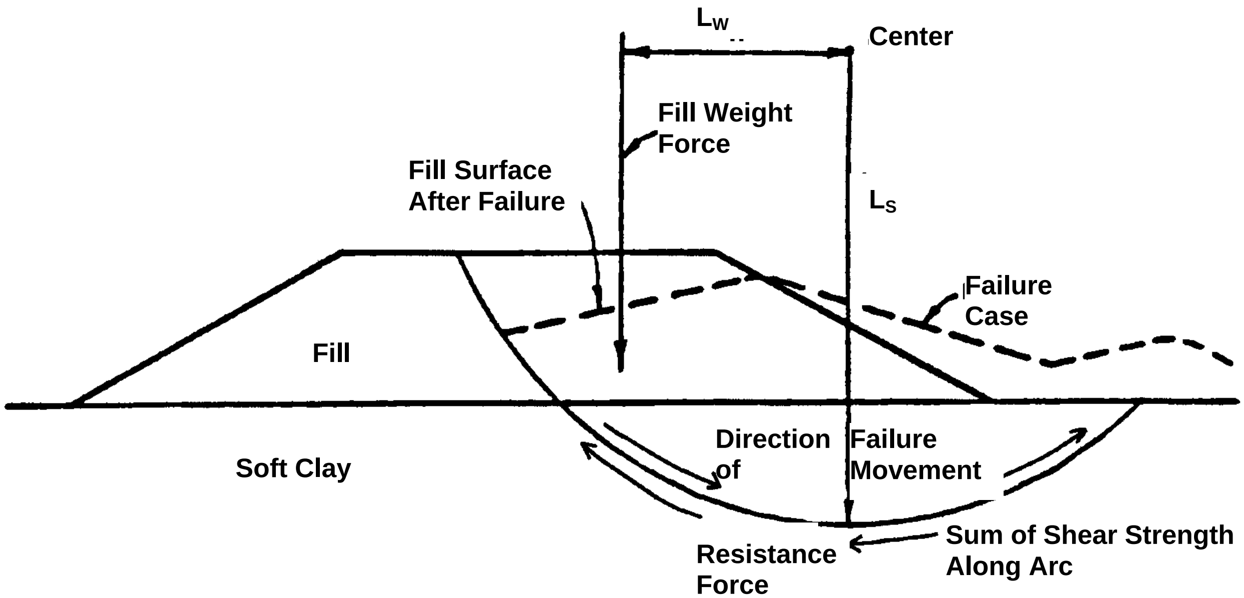


Figure 5-3: Typical Circular Arc Failure Mechanism

The failure force (driving force) consists of the weight of the embankment. The overturning moment is the product of the weight of the embankment (acting through its center of gravity) times the lever arm distance to the center of rotation (L_W).

The resisting force against movement is the sum of all soil shear strength (friction and cohesion) acting along the failure arc. The resisting moment is the product of the shear strength times the radius of the circle (L_S).

The factor of safety against overturning is equal to the ratio of the resisting moment to overturning moment.

$$\text{Factor of Safety} = \frac{\text{Total Shear Strength} \times L_S}{\text{Weight Force} \times L_W} = \frac{\text{Resisting Moment}}{\text{Overturning Moment}} \tag{5-1}$$

When the factor of safety is less than 1, failure will take place.

5.3.1 Simple Rule of Thumb for Factor of Safety

A simple rule of thumb based on simplified bearing capacity theory can be used to make a preliminary "guestimate" of the factor of safety against circular arc failure for an embankment built on a clay foundation.

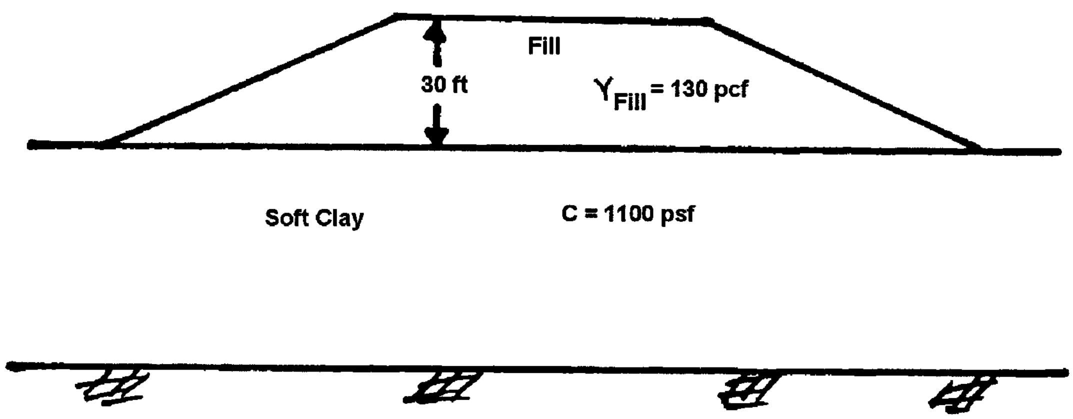
The rule of thumb is:

$$\text{Factor of Safety (F.S.)} \cong \frac{6C}{\gamma_{\text{Fill}} \times H_{\text{Fill}}} \quad (5\text{-}2)$$

Where: C = Cohesion Strength of Foundation Clay (psf)
γ_{Fill} = Fill Soil Unit Weight (pcf)
H_{Fill} = Fill Height (Feet)

For example, consider the following proposed embankment.

$$\text{F.S.} = \frac{(6)(1100\ \text{psf})}{(130\ \text{pcf})(30')} = 1.69 \quad \text{Using Rule of Thumb (Equation 5-2)}$$



The factor of safety computed using this rule of thumb should never be used for final design. The simple equation obviously does not take into account such factors as fill strength or fill slope angle and does not identify the location of a critical failure surface. If the factor of safety using the rule of thumb is less than 2.5, a more sophisticated stability analysis is required.

However, this rule of thumb can be helpful very early in the design stage to make a quick preliminary check on whether stability may be a problem and if more detailed analyses should be conducted. It can also be of use in the field while the boring and sampling is being done. For example, if in situ vane shear tests are being carried out as part of the field investigation for a proposed embankment, the vane strength can be used with the rule of thumb equation, by the soils engineer or geologist, to estimate the F.S. right in the field. This can aid in directing the drilling, sampling, and testing program while the drill crew is at the site and help insure that critical strata are adequately explored and sampled. Finally, the simple rule of thumb factor of safety can be used to check for gross errors in computer output or input.

5.3.2 Stability Analysis Methods (General)

There are several available methods that can be used to perform a circular arc stability analysis for an approach embankment over soft ground. The simplest most basic method is known as the NORMAL METHOD OF SLICES. The normal method of slices can easily be performed by a hand solution and is also a method by which the computation of driving and resisting forces is straightforward and easily demonstrated. For this method, the failure surface is assumed to be the arc of a circle as shown in Figure 5-4 and the factor of safety against sliding along the failure surface is defined as the ratio of the moment of the available soil shear strength resisting forces (friction plus cohesion) on the trial failure surface to the net moment of the driving forces (due to the embankment weight), that is:

$$\text{F.S.} = \frac{\text{Sum of Resisting Forces} \times \text{Moment Arm (R)}}{\text{Sum of Driving Forces} \times \text{Moment Arm (R)}} \qquad (5\text{-}3)$$

Note that since the method consists of computing the driving and resisting forces along (parallel) to the failure arc, the moment arm R is the same for both the driving and resisting forces, thus, R cancels out of the factor of safety equation and the equation reduces to:

$$\text{F.S.} = \frac{\text{Sum of Resisting Forces}}{\text{Sum of Driving Forces}} \qquad (5\text{-}3a)$$

The free body diagram (Figure 5-4) shows the failure surface is divided into slices and the following basic assumptions are made:

1. The available shear strength of the soil can be adequately described by the Mohr-Coulomb equation:

$$S = C + (\sigma - \mu) \tan \phi$$

Where:
- S = Total shear strength
- C = Cohesion component of shear strength
- $(\sigma - \mu) \tan \phi$ = Frictional component of shear strength
- σ = The total normal stress against the failure surface slice base due to the weight of soil and water above the failure surface
- μ = Water uplift pressure against the failure surface
- ϕ = Soil angle of internal friction
- $\tan \phi$ = Coefficient of friction along failure surface

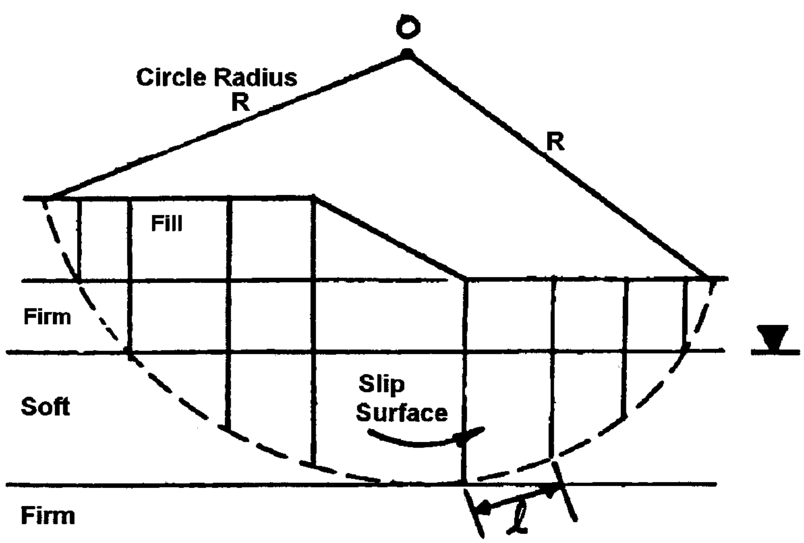


Figure 5-4: Geometry of Normal Method of Slices

2. The factor of safety is the same for all slices.

3. The factors of safety with respect to cohesion (C) and friction (tan ϕ) are equal.

4. All forces (shear and normal) on the sides of each slice are ignored.

5. The water pressure (μ) is taken into account by reducing the total weight of the slice by the water uplift force acting against the slice base.

Lastly, the convention to be used in the stability analysis should be chosen. In soil problems involving water, the engineer may compute the normal and tangential forces using either total soil weights and boundary water forces (both buoyancy and unbalanced hydrostatic forces) or submerged (buoyant) soil weights and unbalanced hydrostatic forces. The results are the same. When total weight and boundary water forces are used, the equilibrium of the entire block is considered. When submerged weights and hydrostatic forces are used, the equilibrium of the mineral skeleton is considered. The total weight notation is used herein as this method is the simplest to compute.

5.3.3 Normal Method of Slices; Step-By-Step Computation Procedure

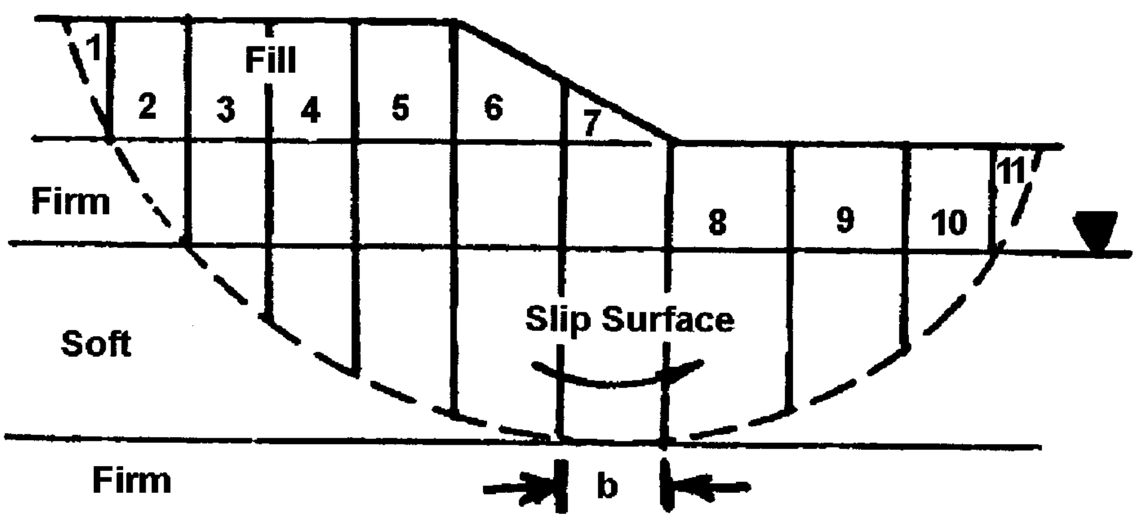
To compute the factor of safety for an embankment using the normal method of slices, the step-by-step computational procedure is as follows:

(Note: An example of the method of slices hand solution is shown for the Apple Freeway Design Example – Slope Stability)

Step 1. Draw cross-section of embankment and foundation soil profile using either 1" = 10 feet or 1" = 20 feet scale both horizontal and vertical.

Step 2. Select a circular failure surface such as shown in Figure 5-4.

Step 3. Divide the circular mass above the failure surface into 10 - 15 vertical slices as illustrated below:



To simplify computation, locate the vertical sides of the slices so that the bottom of any one slice is located entirely in a single soil layer or at the water level - circle intersection, and locate vertical slice top boundaries at breaks in the slope. The slice widths do not have to be equal. For convenience assume a one-foot thick section of embankment (this simplifies computation of driving and resisting forces).

Also as shown in Figure 5-5 and 5-6 the driving and resisting forces of each slice act at the intersection of a vertical line drawn from the center of gravity of the slice to establish a centroid point on the circle. Lines (called rays) are then drawn from the circle center to intersect the circle at the centroid point. The α angles are then measured from the vertical to each ray.

When the water table is sloping, use equation 5-4 to calculate the water pressure on slice base:

$$\mu = h_w \gamma_w \text{Cos}^2 \alpha_w \tag{5-4}$$

Where: α_w =slope of water table from horizontal in degrees

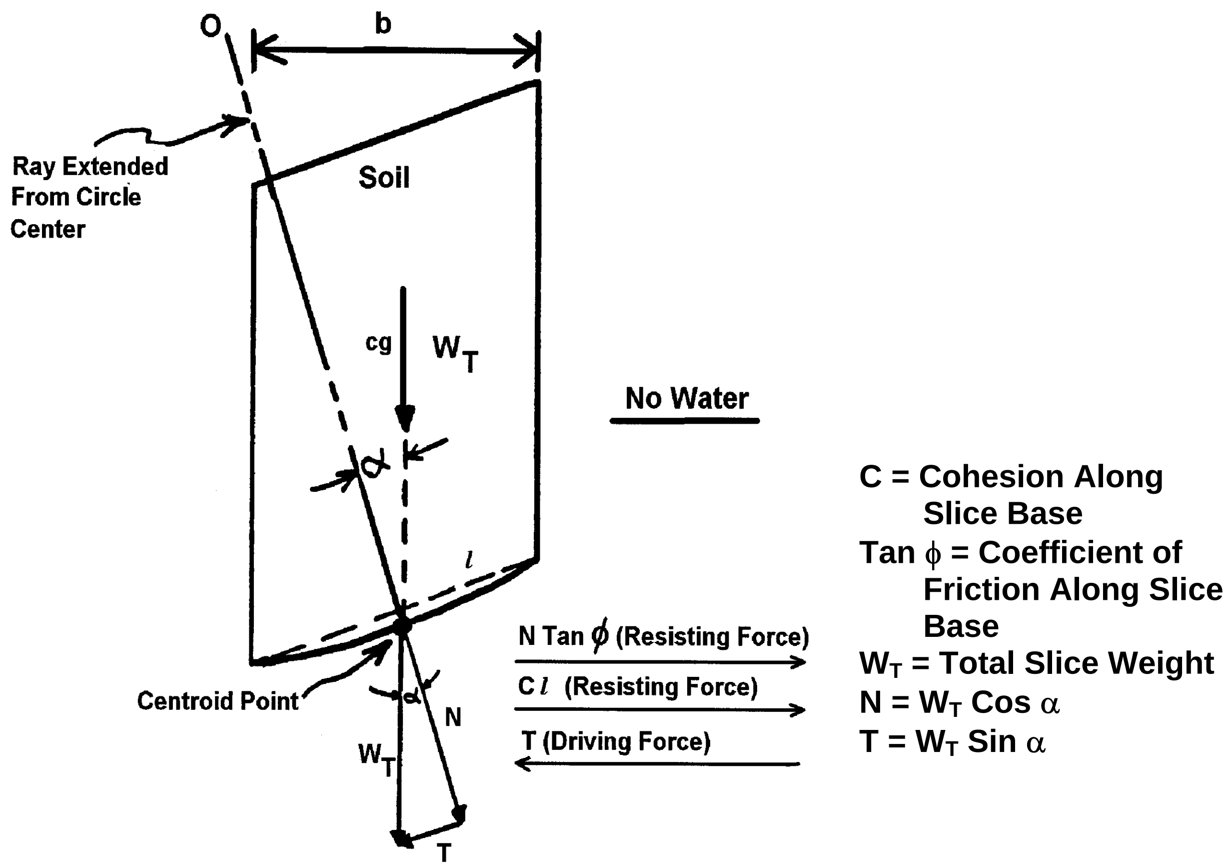


Figure 5-5: Forces on A Slice without Water Effect

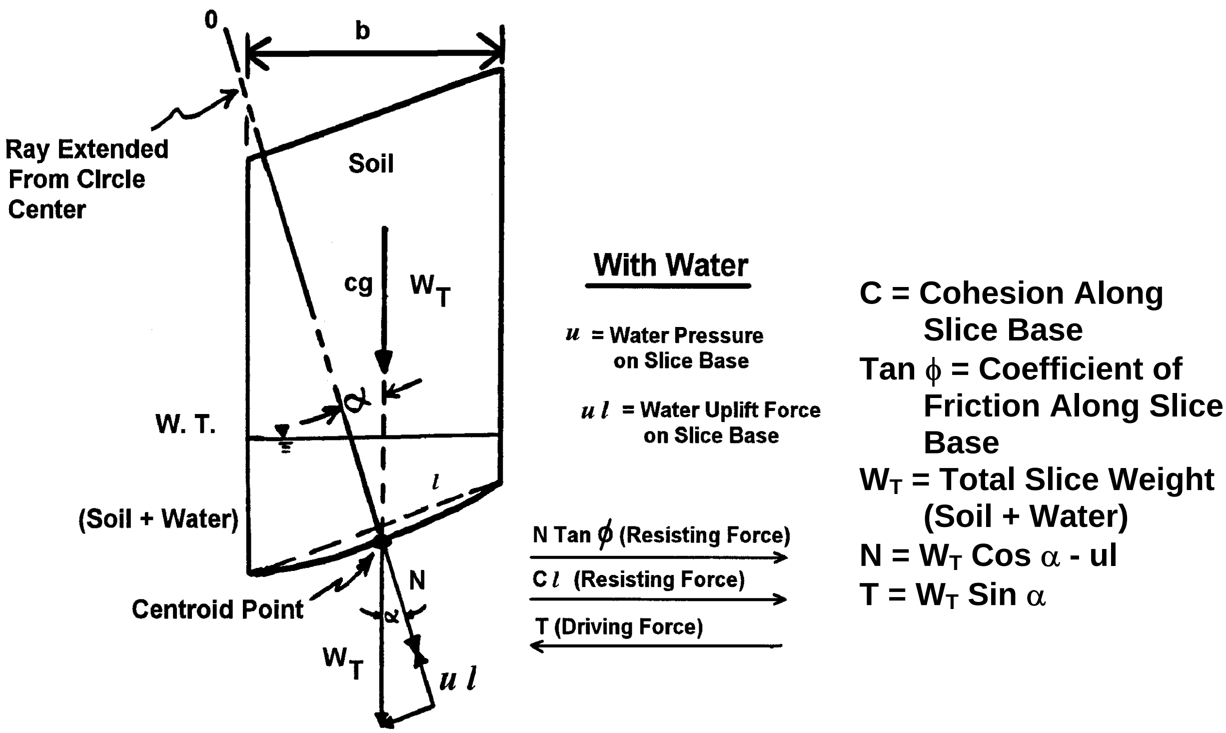


Figure 5-6: Forces on A Slice with Water

Step 4: Compute the total weight (W_T) of each slice.

For illustration, the resisting and driving forces acting on individual slices with and without water pressure are shown on Figures 5-5 and 5-6.

To compute W_T, use total soil unit weight (γ_{Total}) both above and below the water table.

$$W_T = \gamma_{Total} \times \text{Average Slice Height} \times \text{Slice Width (b)} \qquad (5\text{-}5)$$

For example: Assuming γ_{Total} = 120 pcf
Average Slice Height = 10 ft
Slice Width = 10 ft

Then W_T = (120) (10) (10) = 12,000 lbs.

Step 5: Compute N Tan ϕ (Frictional resisting force) for each slice.

$$N = W_T \cos(\alpha) - ul \qquad (5\text{-}6)$$

N = Effective normal force against the slice base (force between granular soil grains)
W_T = Total slice weight (from 4 above)
α = Angle between vertical and line drawn from circle center to midpoint of slice base (note it is also equal to angle between the horizontal and a line tangent to the slice base)
μ = Water pressure on slice base (average height of water, $h_w \times \gamma_{water}$)

l = Arc length of slice base

To simplify computations, take l as the straight-line distance along the slice base and use γ_{water} = 60 pcf.

μ l = Water uplift force against slice base
ϕ = Soil friction angle
Tan ϕ = Coefficient of friction along slice base

Note that the effect of water is to reduce the normal force against the slice base and thus reduce the frictional resisting force (N tan ϕ). To illustrate this, take the same slice used in step 4 and compute N tan ϕ for the slice with no water and then for the water table located 5 feet above the slice base.

Assume: $\phi = 25^o$
$\alpha = 20^o$
l = 11 ft

Example: If using Equation 5-6 with no water in slice:

$\mu\, l = 0$
$N = W_T \cos\alpha = (12{,}000 \text{ lbs.})(\cos 20^o) = 11{,}276$ lbs.
$N \tan\phi = (11{,}276 \text{ lbs})(\tan 25^o) = \underline{5{,}258 \text{ lbs}}$.

If with water 5 ft. above slice base:

$\mu\, l = (h_w)(\gamma_w)(l) = (5)(60)(11) = 3{,}300$ lbs.
$N = W_T \cos\alpha - \mu\, l = 11{,}276 - 3{,}300 = 7{,}976$ lbs.
$N \tan\phi = (7{,}976)(\tan 25°) = \underline{3{,}719 \text{ lbs.}}$

Step 6: Compute Cl (resisting force due to cohesion for each slice).

C = cohesive soil strength
l = length of slice base

Example: C = 200 psf
l = 11 ft
Cl = (200)(11) = 2,200 lbs.

Step 7: Compute T (tangential driving force).

$$T = W_T \text{ Sin } \alpha \tag{5-7}$$

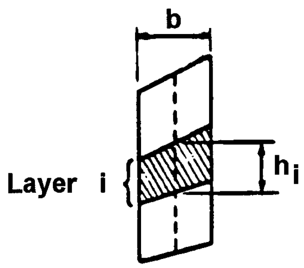
T is the component of total slice weight (W_T) acting tangent to the slice base. T is the driving force due to the weight of both soil and water in the slice.

Example: Given W_T = 12,000 lbs.
$\alpha = 20^o$
$T = W_T \sin\alpha = (12{,}000 \text{ lbs.})(\sin 20^o) = 4{,}104$ lbs.

Step 8: Sum resisting forces and driving forces for all slices and compute factor of safety.

$$F.S. = \frac{\sum \text{Resisting Forces}}{\sum \text{Driving Forces}} = \frac{\sum N \,\text{Tan}\phi + \sum C\, l}{\sum T} \qquad (5\text{-}3a)$$

Tabular computation forms for use in performing a method of slices stability analysis by hand are included on Figures 5-7 and 5-8.



γ_i = unit weight of layer i
h_i = height of layer at center of slice
W_i = partial weight = b h_i γ_i
$\sum W_i$ = total weight of slice W_T

Slice No.	b	h_I	γ_I	W_i	$\sum W_i = W_T$

Figure 5-7: Tabular Form for Computing Weights of Slices

Slice No.	W_T	ℓ	α	C	ϕ	u	$u\ell$	$W_T \cos\alpha$	$N = W_T \cos\alpha - u\ell$	$N \tan = (W_T \cos\alpha - u\ell)\tan\phi$	$c\ell$	$T = W_T \sin\alpha$
									Σ			

C = cohesion intercept
ϕ = friction angle
u = pore pressure

} at base of slice

W_T = Total Weight of Slice (Soil+Water)

$$F = \frac{\sum (W_T \cos\alpha - \mu\ell)\tan\phi + \sum c\ell}{\sum W_T \sin\alpha} = \frac{\sum N \tan\phi + \sum c\ell}{\sum T}$$

$$F = \underline{\qquad\qquad} =$$

Figure 5-8: Tabular Form for Calculating Factor of Safety by Normal Method of Slices

5.3.4 Recommended Stability Methods

There are many other stability analysis methods available besides the NORMAL method - such as Bishop method, Janbu, etc. These methods are primarily variations and refinements of the basic method of slices. The differences in the more refined methods lie in the assumption made regarding the shear and normal forces made on the sides of slices. For example, the NORMAL method assumes the vertical and horizontal slice side forces are zero. The Bishop method, by comparison, includes the horizontal slice side force and ignores the vertical slice side force. For purely cohesive clay soils the NORMAL and Bishop methods will give identical results. For soils which have frictional strength, the Bishop method should be used. The NORMAL method is more conservative and will give unrealistically lower factors of safety than the Bishop or other more refined methods. While none of the methods are 100 percent theoretically correct, currently available procedures are sufficiently accurate for practical analysis and design.

The method of analysis, which should be used to determine a factor of safety, depends on the soil type, the source of and confidence in the soil strength parameters, and the type of slope that is being designed. Soil design analyses should only be performed by qualified experienced geotechnical personnel. Design criteria recommended for analysis of Slope Stability are given in Table 5-1.

TABLE 5 -1
SLOPE STABILITY DESIGN CRITERIA

Foundation Soil Type	Type of Analysis	Source of Strength Parameters	Remarks
Cohesive	Short-term (embankments on soft clays – immediate end of construction).	UU or field vane shear test or CU triaxial test, (undrained strength parameters at P_o. $\phi = 0$ analysis).	Use Bishop method. An angle of internal friction should not be used to represent an increase of shear strength with depth. The clay profile should be broken into convenient layers and the appropriate cohesive shear strength assigned to each layer.
Cohesive	Stage construction (embankments on soft clays – build embankment in stages with waiting periods to take advantage of clay strength gain due to consolidation.	CU triaxial test. Some samples have to be consolidated to higher than existing in situ stress to determine clay strength gain due to consolidation under staged fill heights. (Undrained strength parameters at appropriate P_o for staged height	Use Bishop method at each stage of embankment height. Consider that clay shear strength will increase with consolidation under each stage. Consolidation test data needed to estimate length of waiting periods between embankment stages. Instrumentation (piezometers and settlement devices) should be used to monitor pore pressure dissipation and consolidation during construction.
Cohesive	Long-term (embankment on soft clays and clay cut slopes).	CU triaxial test with pore pressure measurements or CD triaxial test (effective strength parameters).	Use Bishop analysis with combination of cohesion and angle of internal friction (effective strength parameters from laboratory test).
Cohesive	Existing failure planes.	Direct shear or direct simple shear test. Slow strain rate and large deflection needed. Residual strength parameters.	Use Bishop, Janbu or Spencer's method to duplicate previous shear surface.
Granular	All types.	Get effective friction angle from charts of standard penetration resistance (SPT) versus friction angle or from direct shear tests.	Use Bishop Method with an effective stress analysis.

*UU= unconsolidated undrained; CU= consolidated undrained;
CD= consolidated drained;
P_o = in situ vertical effective overburden pressure

5.3.5 Stability Charts

Slope stability charts are available which are sometimes useful for preliminary analysis; such as to compare alternates which can later be examined by more detailed analyses. One of the major shortcomings is that most stability charts are for ideal, homogeneous soil conditions which are not encountered that often in practice.

The interested reader is referred to the Navy Design Manual (NAVFAC DM-7.1) or Terzaghi and Peck (1967) for examples of stability charts and their use.

5.3.6 Remarks on Safety Factor

For normal highway embankment side slopes, a minimum design safety factor of 1.25 is ordinarily used. For slopes which would cause greater damage upon failure, such as end slopes beneath bridge abutments, major retaining structures, etc., the design safety factor should be increased to at least 1.30. For cut slopes in fine-grained soils which can lose shear strength with time, a safety factor of 1.5 is desirable.

5.4 CRITICAL FAILURE SURFACE

The step-by-step procedure presented on the preceding pages shows how to compute the factor of safety for one selected circular arc failure surface. The complete analysis requires that a large number of assumed failure surfaces be checked in order to find the most critical one; the surface with the lowest factor of safety. This would obviously be a tedious and time consuming operation if done by hand.

This is where the computer becomes such a valuable design tool. The stability analysis is easily adapted to computer solution. A grid of possible circle centers is defined, and a range of radius values established for each. The computer can be directed to print out all the safety factors or just the minimum one (and its radius) for each circle center. A plot of minimum safety factor for each circle center in the form of contours can be used to define the location of the most critical circle and the minimum safety factor as shown in Figure 5-9.

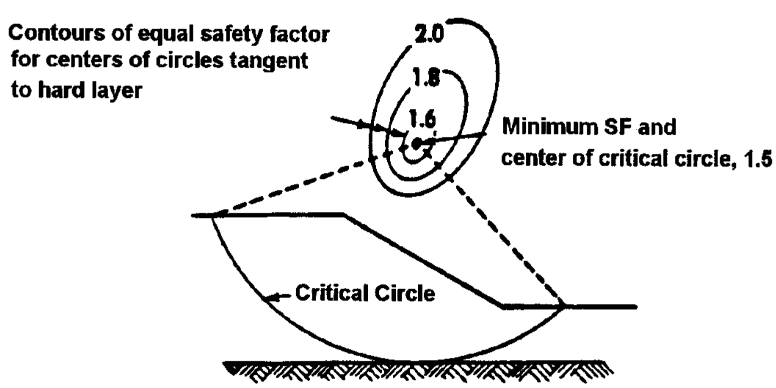


Figure 5-9: Location of Critical Circle by Plotting Contours of Minimum Safety Factors for Various Trial Circles

5.5 SLOPE STABILITY ANALYSIS - COMPUTER PROGRAMS

Slope stability procedures are well suited to computer analysis due to the interactive nature of the solution. Also, the simplified hand solution procedures do not properly account for interslice forces, irregular failure surfaces, seismic forces, and external loads such as line load surcharges or tieback forces. Several user-friendly micro-computer programs now exist to accurately analyze two dimensional slope stability problems. More complex computer programs are available for three dimensional slope stability analysis.

Highway agencies should, as a minimum, use a basic two-dimensional slope stability program. Desirable geotechnical features of such a program should include:

- Multiple analysis capability
  a. Circular arc (Modified Bishop)
  b. Non-circular (Janbu)
  c. Sliding block

- Variable Input Parameters
  a. Heterogeneous soil systems
  b. Pseudo-static seismic loads
  c. Tieback forces
  d. Piezometric levels

- Random generation of multiple failure surfaces with option to analyze a specific failure surface.

Desirable software features include:

- User-friendly input screens including a summary screen showing the cross section and soil boundaries in profile.

- Help screens and error tracking messages.

- Expanded output option of both resisting forces in friction, cohesion or tieback computations and driving forces in static or dynamic computations.

- Ordered output and plot of 5 minimum failure surface safety factors.

- Documentation of program.

A major problem for software users is technical support, maintenance and update of programs. Slope stability programs are in a continual process of improvement which can be expected to continue indefinitely. Highway agencies should only implement software which is documented and which the seller agrees to provide full technical support, maintenance and update. The web page for the FHWA Geotechnical Group, *www.fhwa.dot.gov/bridge/geo.htm*, contains links to distributors of FHWA software.

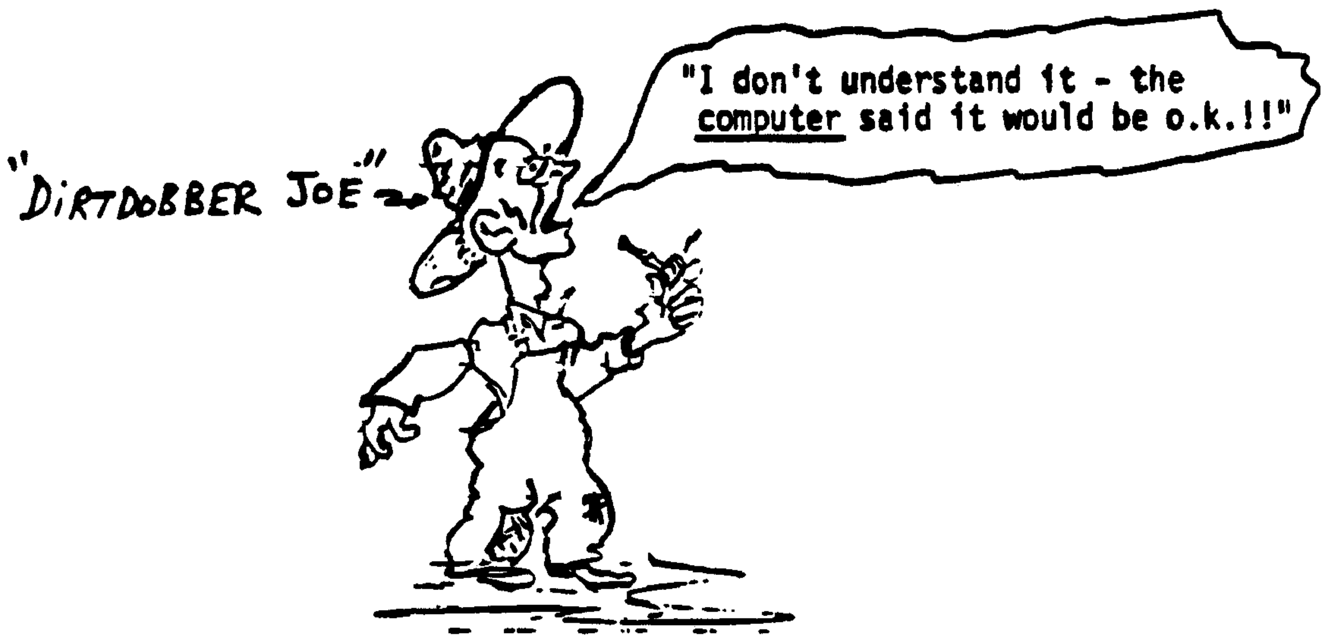
Other private firms exist which provide similar services for slope stability programs such as the STABL series, XSTABL, the UTEXAS series, etc.

IMPORTANT! IMPORTANT! IMPORTANT!

In DESIGN - Put the major emphasis where it belongs, which is on:

- Investigation
- Sampling
- Testing
- Development of Soil Profile
- Design Soil Strengths
- Water Table Location

Computer programs are only tools which aid us in the design - the answers are only as good as the input data. Don't get carried away with plugging the numbers. You may learn the "garbage in - garbage out" principle the hard way - like "Dirtdobber Joe"!



5.6 SLIDING BLOCK FAILURE

A "sliding block" type failure can occur (1) where the foundation soil contains thin seams of weak clay or organic soils, (2) where a shallow layer of weak soil exists at the ground surface and is underlain by firm soil, and (3) where the foundation soil contains thin sand or silt lenses sandwiched between more impermeable soil. The weak layer or lense provides a plane of weakness along which sliding can occur. In the case of sand or silt lenses trapped between impervious soil, the mechanism that can cause sliding is as follows: As the fill load is placed, the water pressure is increased in the sand or silt lense. Since the water cannot escape due to the impermeable soil above and below, the sand or silt loses frictional strength as a result of the intergranular effective stress between soil grains being decreased due to the water pressure. These problems are illustrated in Figure 5-15.

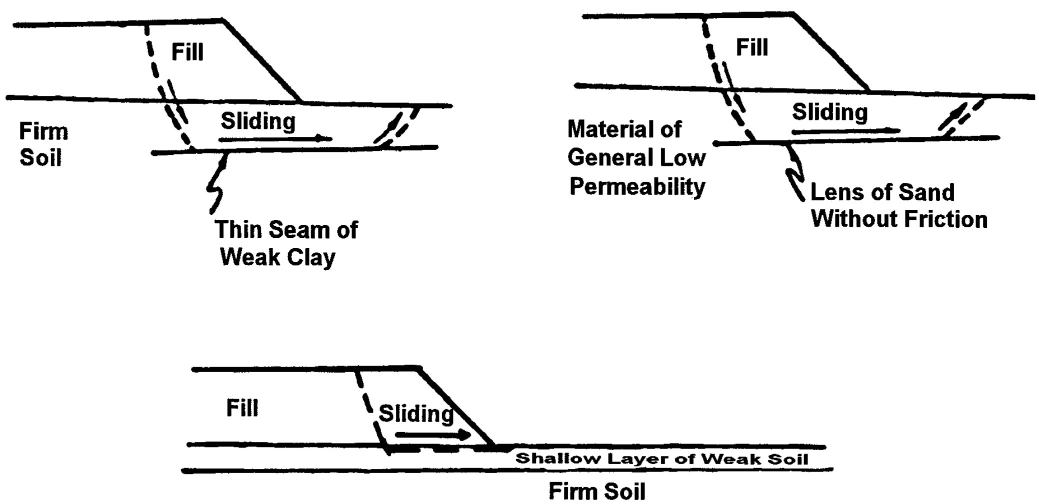


Figure 5-15: Sliding Block Failure Mechanism

When sliding occurs, an active wedge type failure occurs through the fill (similar to the active wedge that forms behind a retaining wall), and a passive wedge type failure occurs below the fill toe as soil in the toe area is pushed up out of the way. The sliding mass moves essentially as a block, thus the term "sliding block."

5.7 SLIDING BLOCK – HAND METHOD OF ANALYSIS

A simple sliding block analysis to estimate factor of safety against sliding is straightforward and can be easily and quickly performed by hand. For the analysis, the potential sliding block is divided into three parts; (1) An active wedge at the head of the slide, (2) A central block, and (3) A passive wedge at the toe. For example see figure 5-16.

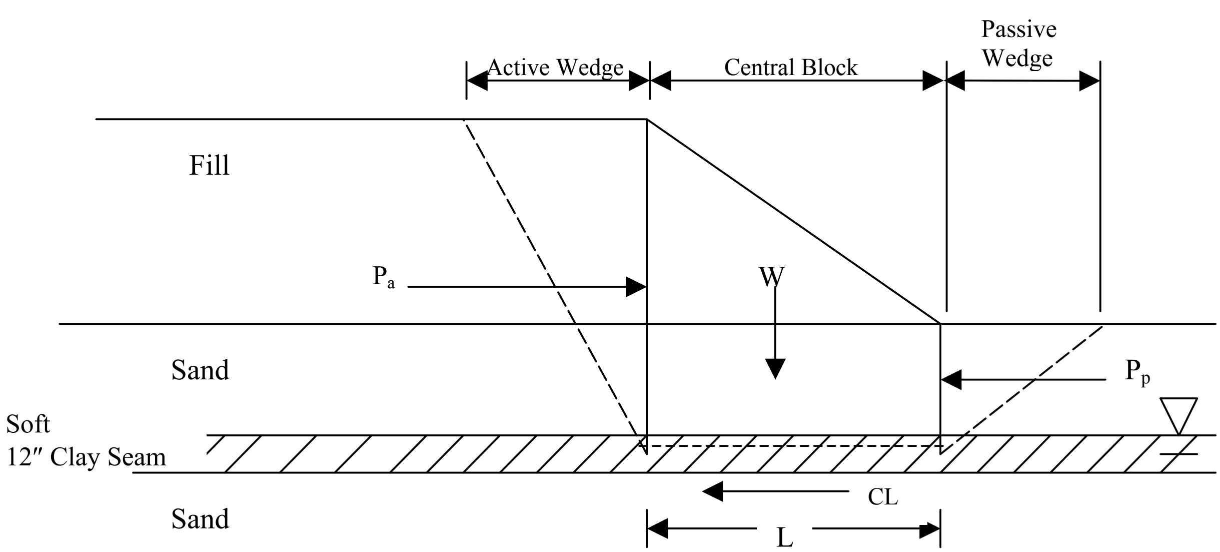


Figure 5-16: Geometry and Parameters for Sliding Block Mechanism

For the problem illustrated in Figure 5-16 above, the factor of safety would be computed by summing forces horizontally, to give:

$$F.S. = \frac{\text{Horizontal Resisting Forces}}{\text{Horizontal Driving Forces}} = \frac{P_P + CL}{P_a} \tag{5-8}$$

Where: P_a = Active Force (Driving)
P_p = Passive Force (Resisting)
CL = Resisting Force due to cohesion of clay

(For convenience of computation of 1 foot thick slice of embankment is assumed.)

Several trial locations of the active and passive wedges must be checked to determine the minimum factor of safety. Note that since wedge type failures occur at the head and toe of the slide, similar to what occurs behind retaining walls, the active and passive forces are taken as acting against vertical planes which are treated as "imaginary" retaining walls, and the active and passive forces are computed the same as for retaining wall problems.

Computation of Forces - Simple Sliding Block Analysis:

For the simple sliding block type problem illustrated on the previous page the forces used in the factor of safety computation can be calculated as follows using the Rankine approach:

Driving Force

$$P_a = 1/2\ \gamma\ H^2\ K_a \tag{5-9}$$

Where: P_a = Active force (kips)
γ = Soil unit weight (kcf)
H = Height of soil layer in active wedge (ft)
K_a = Active earth pressure coefficient for level ground surface
$K_a = \tan^2 (45^o - \phi/2)$
ϕ = Soil angle of internal friction

Resisting Force

$$P_p = 1/2\ \gamma\ H^2\ K_p \tag{5-10}$$

Where: P_p = Passive Force (kips)
γ = Soil Unit Weight (kcf)
H = Height of soil layer in passive wedge (ft)
K_p = Passive earth pressure coefficient for level ground surface
$K_p = \tan^2 (45^o + \phi/2)$

Resisting Force (CL in kips) = Clay cohesion (C in ksf) X Length of central wedge (L in feet)

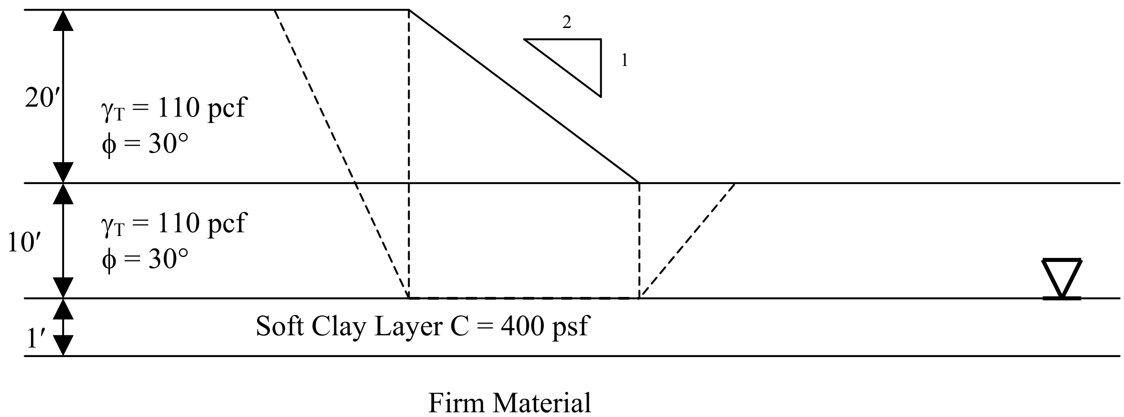
Computation Tips:

These are two important design tips that should be kept in mind when performing a sliding block analysis.

First, be aware that if the active or passive wedge passes through more than one soil type with different soil strengths or soil weights, then the active or passive pressure changes as you go from one soil layer into the next (due to change in either the soil weight and/or the earth pressure coefficient K_a or K_p). The easiest way to handle this is to first compute the active or passive pressure diagram, then compute the active or passive force from the area of the pressure diagram.

Second, when computing the active or passive pressure, remember to use buoyant (effective) soil unit weight below the water table.

Example 5.1: Find the Safety Factor For The 20′ High Embankment By The Simple Sliding Block Method Using Rankine Pressure Coefficients, for the Slope Shown Below.



Solution:

Step 1: Compute Driving Force (P_a)

- Active Driving Force (P_a) (consider a 1 ft. wide strip of the embankment)

$$P_a = \frac{1}{2}\gamma_T H^2 K_a \qquad \text{(use } \gamma_T \text{ as the water table is below the failure plane)}$$

$$K_a = \mathrm{Tan}^2(45 - \frac{\phi}{2}) = \mathrm{Tan}^2(45 - \frac{30}{2}) = 0.33$$

$$P_a = \frac{1}{2}(0.110\,kcf)(30')^2(0.33)(1') = 16.5^K$$

Step 2: Compute Resisting Force (Cl & P_p)

- Central Block Resistance (Cl)

$$Cl = (0.400\,kcf)(40')(1') = 16.0^K$$

- Passive Resisting Force (P_p)

$$P_p = \frac{1}{2}\gamma_T H^2 K_p$$

$$K_p = Tan^2(45 + \frac{\phi}{2}) = Tan^2(45 + \frac{30}{2}) = 3.0$$

$$P_p = (\frac{1}{2})(0.110kcf)(10)^2(3.0)(1') = 16.5^K$$

$$\text{Safety Factor} = \frac{Cl + P_p}{P_a} = \frac{16.0^K + 16.5^K}{16.5^K} = 1.97$$

5.8 COMPUTATION OF FORCES - COMPLICATED SLIDING BLOCK ANALYSIS

The Rankine approach is a useful tool to portray the mechanism of a planar failure condition. However a general force diagram applicable to a more difficult sliding block type problem can account for the effects of water pressure, cohesion, friction, and a sloping failure plane in the analysis. This analysis procedure, which is described in FHWA-SA-94-005, can be used both to estimate factor of safety for assumed failure surfaces in design or to "backanalyze" sliding block type landslide problems.

Computer solutions are also available for defined planar surface or non-circular surface failure modes. However most of those solutions do not use the simplified Rankine block approach but a more complex Janbu approach to the planar failure. In general a computer solution is preferred for these planar failure problems.

5.9 DESIGN SOLUTIONS - STABILITY OF EMBANKMENTS

There are usually several solutions to a stability problem. The one chosen should be the most economical considering the following factors:

1. Available materials.
2. Quantity and cost of materials.
3. Construction time schedules.
4. Line and grade requirements.
5. Right-of-way.

5.9.1 Embankment Stability Design Solutions

TABLE 5-2
PRACTICAL DESIGN SOLUTIONS TO EMBANKMENT STABILITY PROBLEMS

*1. Relocate highway alignment.	A line shift of the highway to a better soils area may be the most economical solution.
*2. Reduce grade line.	A reduction in grade line will decrease the weight of the embankment and may provide stability. (Figure 5-10)
3. Counterweight berms.	The weight of a counterweight berm as illustrated in (Figure 5-11), being on the outside of the center of rotation, provides an increased moment which resists failure. This increases the factor of safety. Berms should be built concurrently with the embankment. The embankment should never be completed prior to berm construction, since the critical time for shear failure is at the end of embankment work. The top surface of a berm should be sloped to drain water away from the embankment. Also care should be exercised in selection of materials and compaction requirements to assure the design unit weight will be achieved for berm construction.
4. Excavation of soft soil and replacement with shear key.	The strength of soft soil is often insufficient to support embankments. In such cases, soft soils are excavated and replaced with granular material (Figure 5-12).
5. Displacement of soft soil.	For deep soft deposits, excavation is difficult. The soft soil can be displaced by generating continuous shear failures along the advancing fill front until the embankment is on firm bottom. The mudwave forced up in front of the fill must be excavated to insure continuous displacement and prevent large pockets of soft soil from being trapped under the fill.
6. Slow rate or stage construction.	Many weak subsoils will tend to gain strength during the loading process as consolidation occurs and pore water pressures dissipate. For soils that consolidate relatively fast, such as some silts and silty clays, this method is practical. Proper instrumentation is desirable to monitor the state of stress in the soil during the loading period to insure that loading does not proceed so rapidly as to cause a shear failure. Typical instrumentation consists of slope inclinometers to monitor stability, piezometers to measure porewater pressure, and settlement devices to measure amount and rate of settlement. Planning of the instrumentation program and data interpretation should be done by a qualified geotechnical engineer.
7. Lightweight embankment.	In some areas of the country, lightweight blast furnace slag, shredded rubber tires, expanded polystyrene blocks, or expanded shale is available. The slag material weighs about 80 pcf. Sawdust fill weighs about 50 pcf and has friction angle of 35^o or more. Shredded tires and EPS are even lighter materials. The overturning force is decreased by the lighter embankment weight. Typical Specifications for lightweight fills used by the NYDOT and WashDOT are included in Appendix C and D.
8. Ground improvement	The use of recently developed techniques such as stone columns, soil mixing, geosynthetics, soil nailing, ground anchors, and grouting can be used to increase resisting forces. Specialty contractors should be considered for these design solutions.

*Always considers these simple solutions first to avoid more complicated, expensive solutions which follow

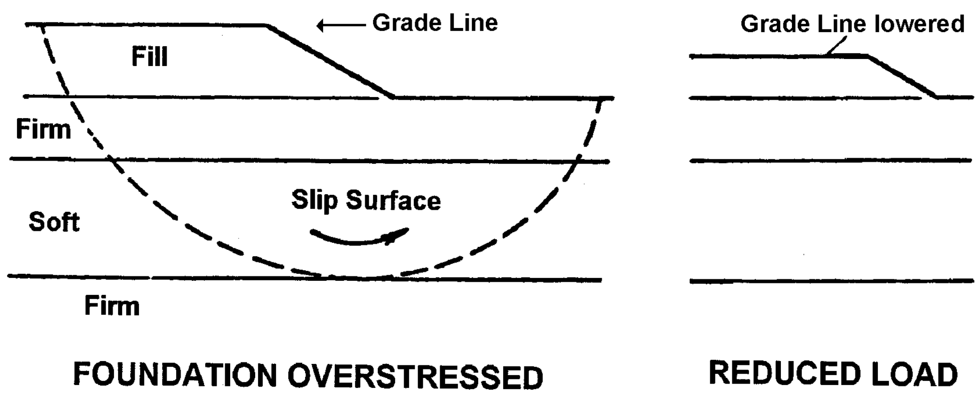


Figure 5-10 Reduction of Grade Line

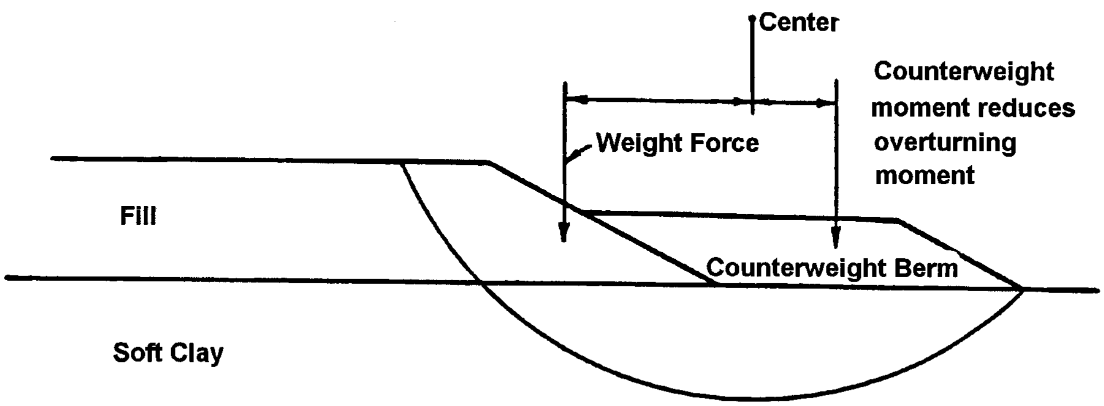


Figure 5-11 Use of Counterweight Berm to Improve Slope Stability

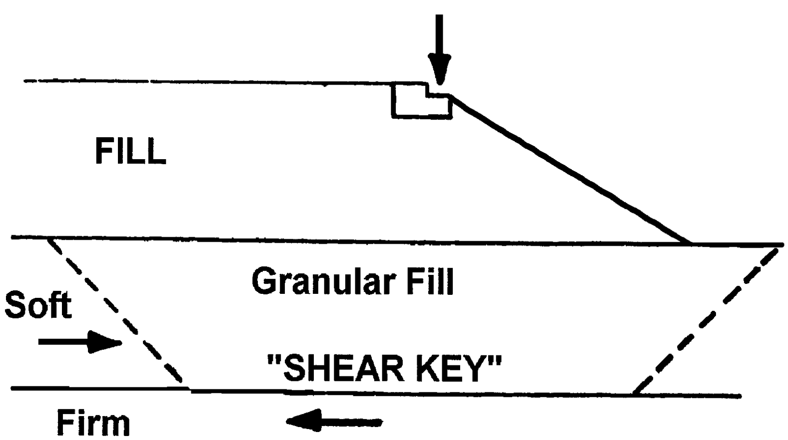


Figure 5-12 Use of Shear Key to Improve Slope Stability

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5.10 CUT SLOPE STABILITY

The two most common types of cut slope failures are deep-seated and shallow surface failures.

Type 1. Deep Seated Failure

Deep seated failure usually occurs in clay cut slopes. The clay has insufficient shearing strength to support the slope, and a circular arc shear failure occurs. If the clay has water bearing silt or sand layers, the seepage forces will also contribute to the instability. Figure 5-13 shows an example of a deep seated failure and a possible design solution.

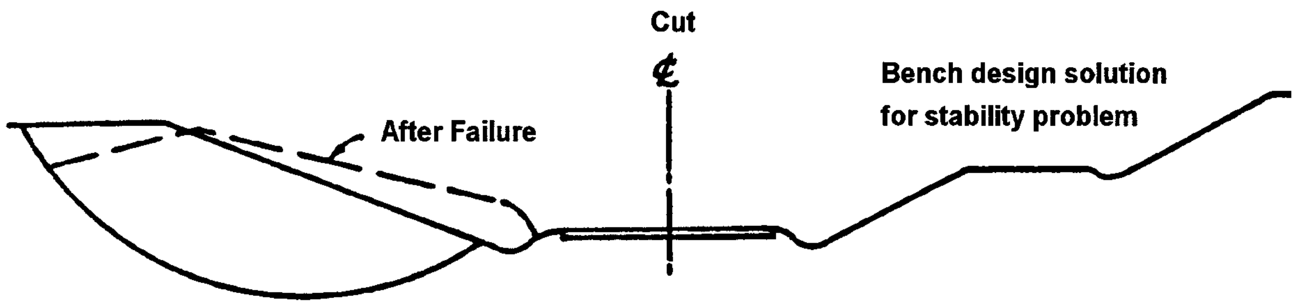


Figure 5-13: Deep Seated Slope Failure (Left) and Bench Slope Design (Right) to Prevent Slope Failure.

The following are typical design solutions to clay cut slope stability problems:

	Design Solution	Effect on Stability
a.	Flatten slope.	Reduces overturning force.
b.	Bench slope.	Reduces overturning force.
c.	Buttress toe.	Increases resisting force.
d.	Lower water table.	Reduces seepage force.

CAUTION: Design of cut slopes in clay should not be based on undrained strength of the clay from clay samples obtained before the cut is made. Designs based on undrained strength will be unconservative. The reason is that when the cut is made the effective stress is reduced because load is removed. This decrease in effective stress will allow the clay to swell and lose strength if the water is made available to the clay as illustrated as shown in Figure 5-14.

UNDRAINED CLAY IN CUT GRADUALLY WEAKENS AND MAY FAIL LONG AFTER CONSTRUCTION

Therefore, design of cut slopes in clays should be based on effective strength parameters so that the reduction in effective stress resulting from the cut excavation can be taken into account.

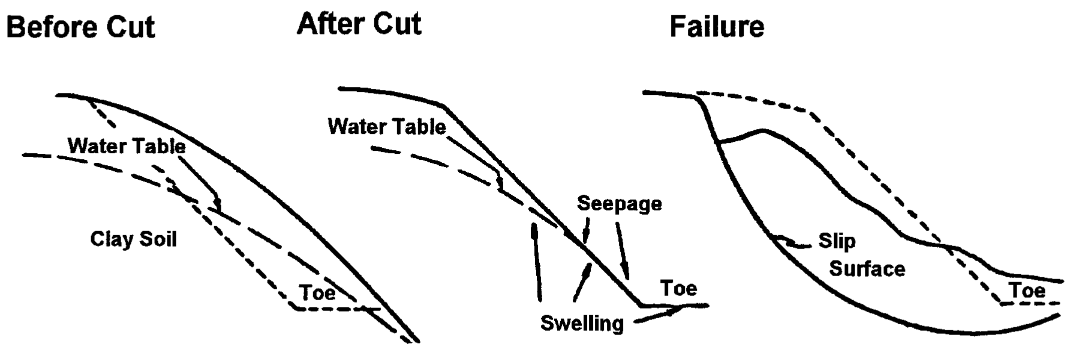


Figure 5-14: Typical Cut Slope Failure Mechanism in Clay Soils

Type 2. Surface Failures

Shallow surface failures (sloughs) are the most common clay or silt cut slope problem. These may involve either an entire slope or local areas in the slope.

The prime cause of shallow surface failures is water seepage. Water seepage reduces the strength of the surface soils, causing them to slide or flow. Soils most likely to be unstable are water bearing silts and layered clays.

Sloughing of slopes due to ground water seepage can often be remedied by placing a 2-3 foot thick rock or gravel blanket over the critical area. The blanket reduces the seepage forces, drains the water, and acts as a weight on the unstable soil. The blanket should be "keyed" into the ditch at the toe of slope. The key should extend about 4 feet below the ditch line and be about 4 feet wide. A geotextile should be placed both under the key and against the slope before blanket placement. Construction of the blanket should proceed from the toe upwards. The most effective placement is by a dozer which will track over and compact the lower blanket areas during placement of upper areas.

Factor of Safety - Cut Slopes

For stability of fine-grained cut slopes, current practice requires a minimum factor of safety against sliding of 1.50. The higher factor of safety for backslopes versus embankments is based upon the knowledge that cut slopes may deteriorate with time as a result of natural drainage conditions that embankments do not experience.

5.11 LATERAL SQUEEZE OF FOUNDATION SOIL

Field observations and measurements have shown that some bridge abutments supported on piling driven through thick deposits of soft compressible soils have tilted toward the backfill. Many of the structures have experienced large horizontal movements resulting in damage to the structure. The cause of this problem is the unbalanced fill load, which "squeezes" (consolidates) the soil laterally. This "lateral squeeze" of the soft foundation soil can transmit excessive lateral thrust which may bend or push the piles out, causing the abutment to rotate back toward the fill, as illustrated in Figure 5-18.

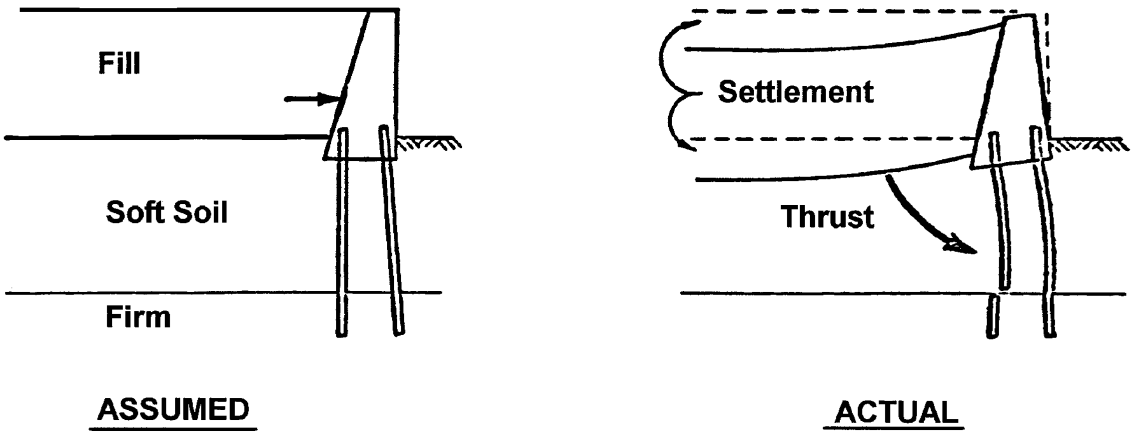


Figure 5-18: Lateral Squeeze Mechanism

5.11.1 Can Tilting Occur?

Experience has shown that if the applied surface load imposed by the fill weight exceeds 3 times the cohesive shear strength of the soft soil, i.e.,

If γ_{Fill} x $H_{Fill} > 3C$

then this lateral squeeze of the foundation soil and abutment tilting can occur.

Therefore, using the above relationship, the possibility of abutment tilting can be evaluated in design. For all practical purposes, the fill unit weight can be assumed at 125 pcf. The cohesive strength C of the soft soil must be determined either from in situ field vane shear tests or triaxial tests on high quality undisturbed Shelby tube samples.

5.11.2 Estimation of Horizontal Abutment Movement

The amount of horizontal movement the abutment may undergo toward the fill can also be estimated in design. The following table contains case history information for nine structures where measurements of abutment movements have been made:

SUMMARY OF ABUTMENT MOVEMENTS*

Foundation	Fill Settlement (Inches)	Abutment Settlement (Inches)	Abutment Tilting (Inches)	Ratio of Abutment Tilting to Fill Settlement
Steel H-piles	16	Unknown	3	0.19
Steel H-piles	30	0	3	0.10
Soil bridge	24	24	4	0.17
Cast-in-place pile	12	3.5	2.5	0.19
Soil bridge	12	12	3	0.25
Steel H-piles	48	0	2	0.06
Steel H-piles	30	0	10	0.33
Steel H-piles	5	0.4	0.5 to 1.5	0.1 to 0.3
Timber Piles	36	36	12	0.33

*Highway Research Record 334, 1971

This data provides a basis for estimating horizontal abutment movement for similar problems, providing a reasonable estimate of the post-construction fill settlement is made, using data from consolidation tests on high quality undisturbed Shelby tube samples. Note that the data for the structures listed in the previous summary showed horizontal abutment movement to range from 6 to 33 percent of the vertical fill settlement, with the average being 21 percent.

Therefore, if the fill load exceeds the 3C limit, then the horizontal abutment movement that may occur can reasonably be estimated as 25 percent of the vertical fill settlement, i.e.,

Horizontal Abutment Movement = 0.25 x Fill Settlement

5.11.3 Design Solutions to Prevent Abutment Tilting

The best way to handle the abutment-tilting problem is to get the fill settlement out before the abutment piling are driven.

If the construction time schedule or other factors do not permit the settlement to be removed before the piling can be driven, then the problems resulting from abutment tilting can be mitigated by the following design provisions:

1. Use sliding plate expansion shoes large enough to accommodate the anticipated horizontal movement.

2. Make provisions to fill in the bridge deck expansion joint over the abutment by inserting either metal plate fillers or larger neoprene joint fillers.

3. Design piles for downdrag forces due to settlement.

4. Use steel H-piles for the abutment piling since steel H-piles are capable of taking large tensile stresses without failing.

5. Use backward battered piles at the abutment and particularly the wingwalls.

Movements should also be monitored so that predicted movement can be compared to actual.

5.12 APPLE FREEWAY DESIGN EXAMPLE – SLOPE STABILITY

In this chapter the Apple Freeway Example Problem is used to illustrate the analysis and design of an embankment with respect to stability consideration. Slope stability analysis using the Normal Method by hand calculations is performed and compared to computer generated solutions. A sliding block analysis is performed and the possibility of lateral squeeze is also examined.

Site Exploration	Terrain Reconnaissance Site Inspection Subsurface Borings	
Basic Soil Properties	Visual Description Classification Tests Soil Profile	
Laboratory Testing	P_o Diagram Test Request Consolidation Results Strength Results	
	Slope Stability	Design Soil Profile Circular Arc Analysis Sliding Block Analysis Lateral Squeeze
Embankment Settlement	Design Soil Profile Settlement Time – Rate Surcharge Vertical Drains	
Spread Footing Design	Design Soil Profile Pier Bearing Capacity Pier Settlement Abutment Settlement Vertical Drains Surcharge	
Pile Design	Design Soil Profile Static Analysis – Pier Pipe Pile H – Pile Static Analysis – abutment Pipe Pile H – Pile Driving Resistance Abutment Lateral Movement	
Construction Monitoring	Wave Equation Hammer Approval Embankment Instrumentation	

Apple Freeway Design Example – Slope Stability
Exhibit A

Given: The proposed embankment geometry (Figure 2-5) and soil properties at the east approach of the Apple Freeway Bridge. Assume that the shallow (≈ 3′) surface layer of organic has been removed and replaced with select material.

Required: Compute the embankment stability with respect to circular arc failure, sliding block failure and lateral squeeze.

Solution:

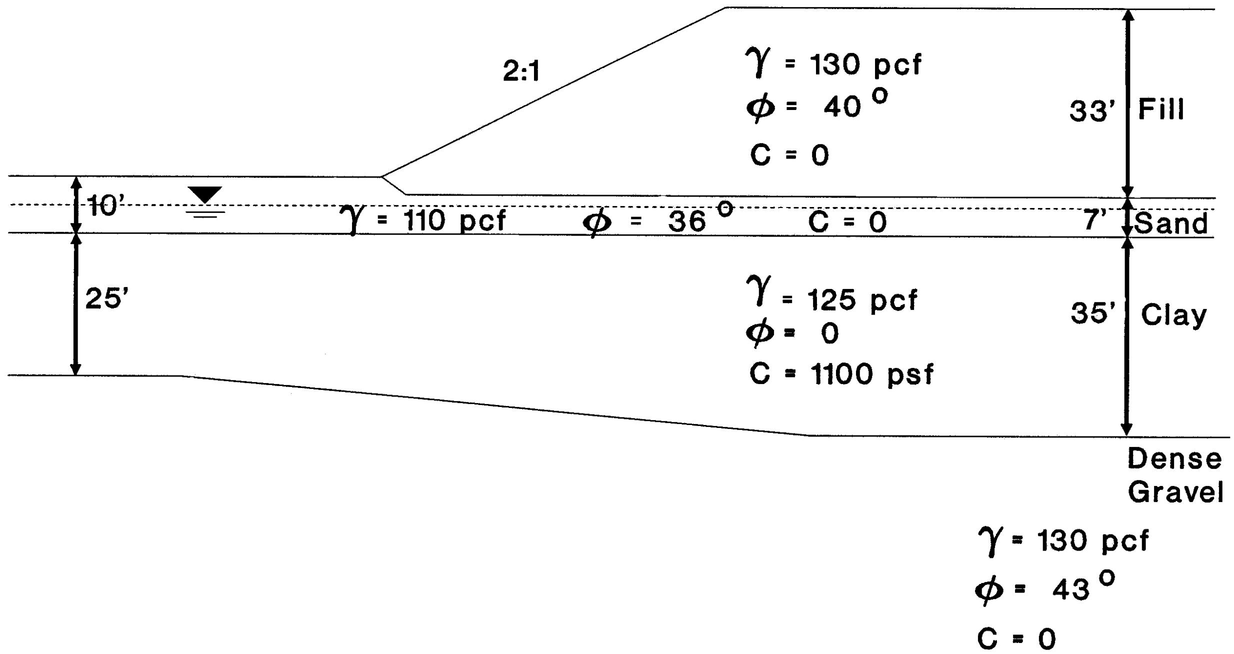
- **Compute F.S. against circular arc failure (Normal Method/ Hand Solution) and check with computer solution**

- **Compute F.S. against circular arc failure by the Bishop Simplified Method**

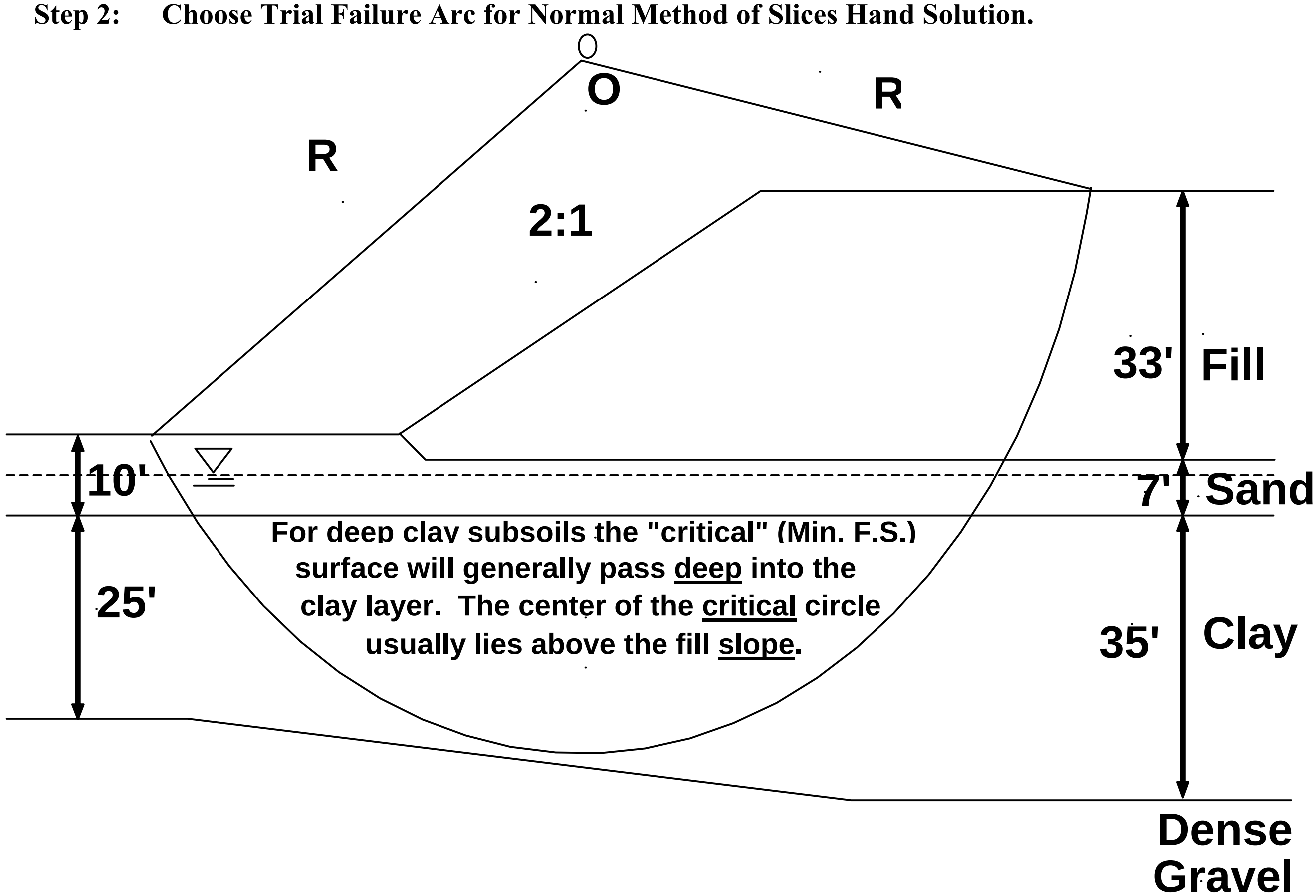
- **Compute F.S. against sliding block failure using Rankine block analysis**

- **Check if lateral squeeze is possible at this embankment location**

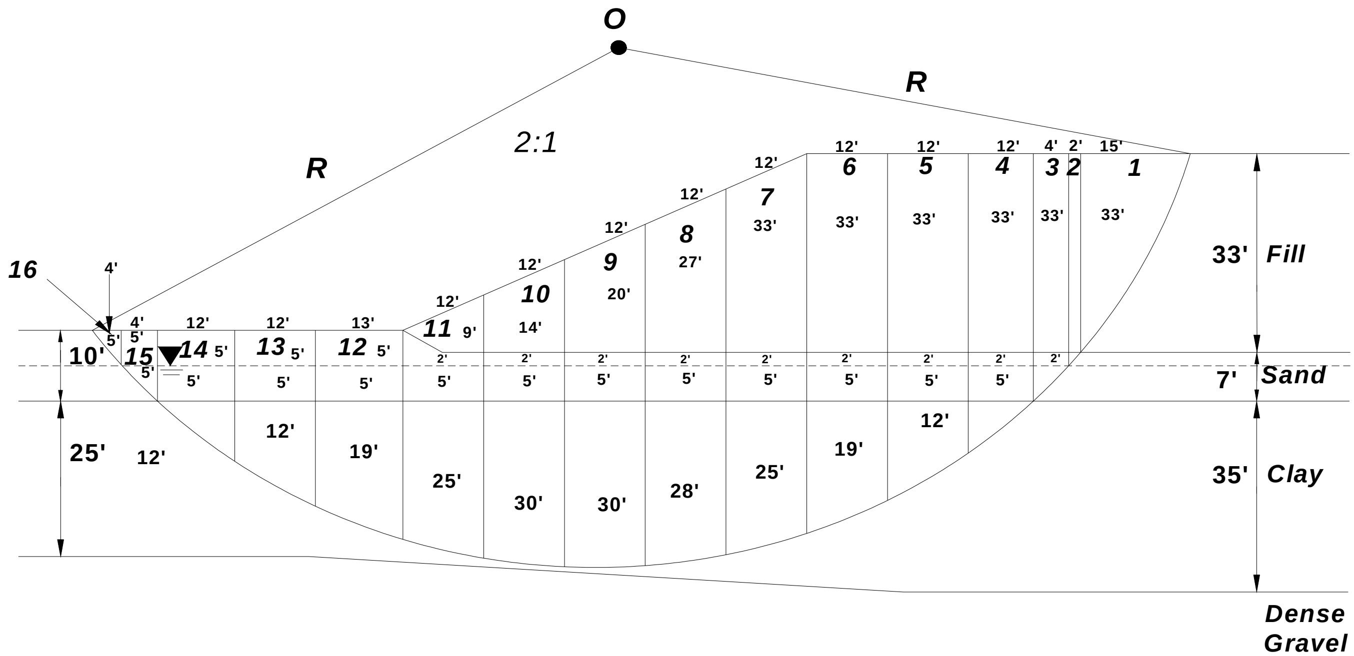
Step 1: **Obtain Soil Profile and Design Parameters**



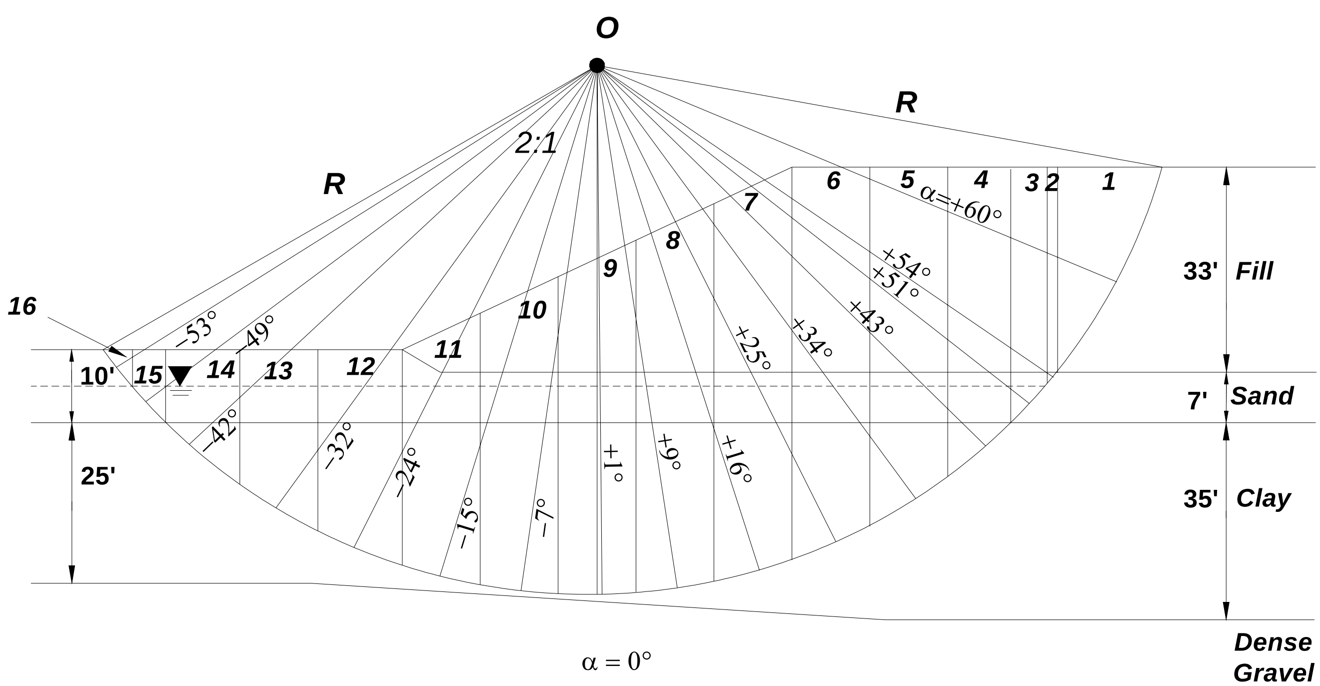
Step 2: **Choose Trial Failure Arc for Normal Method of Slices Hand Solution.**



Step 3: Circular Arc Analysis – Divide Mass Above Failure Surface into Vertical Slices.

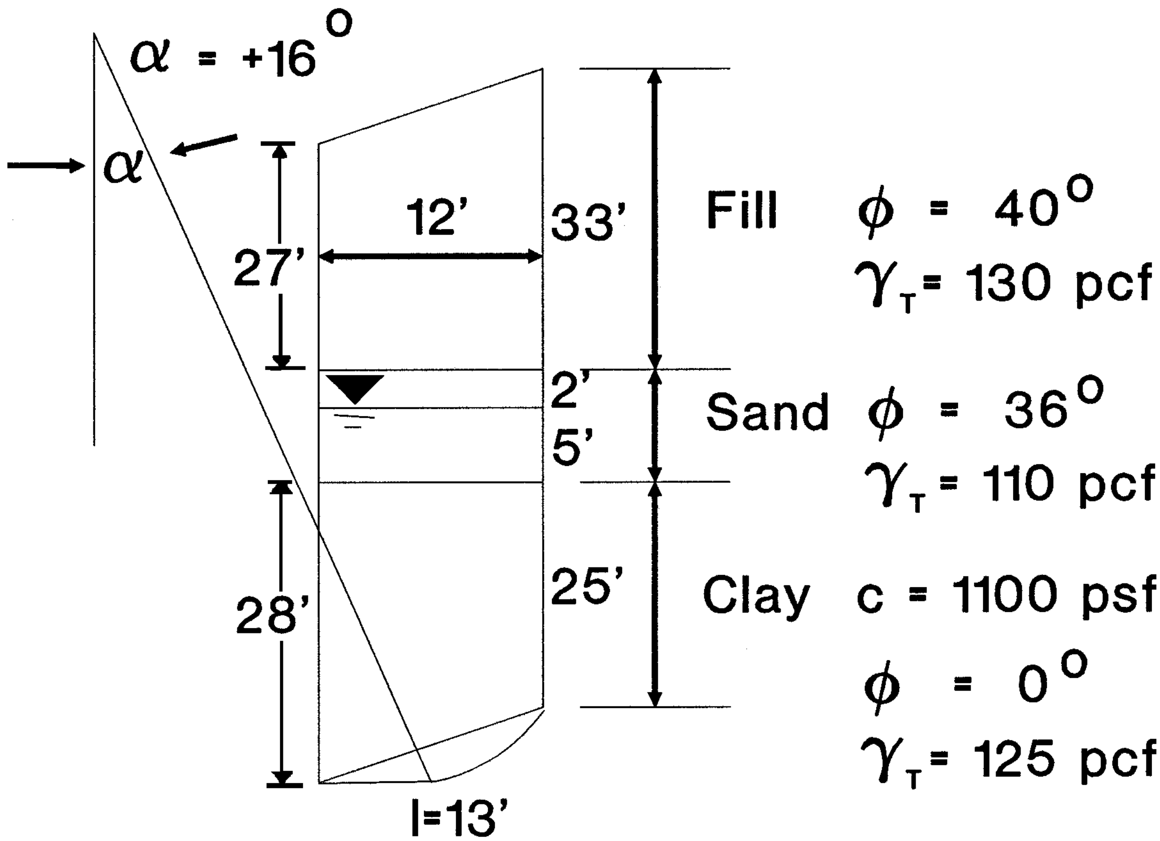


Step 4: Determine α Angles.



Step 5: Compute Resisting and Driving Forces for All Slices.

Workshop Design Problems Example Computation Slice 7



$$W_T = (12)\left(\frac{27+33}{2}\right)(130) + (12)(7)(110) + (12)\left(\frac{28+25}{2}\right)(125) = 95{,}790^{\#}$$

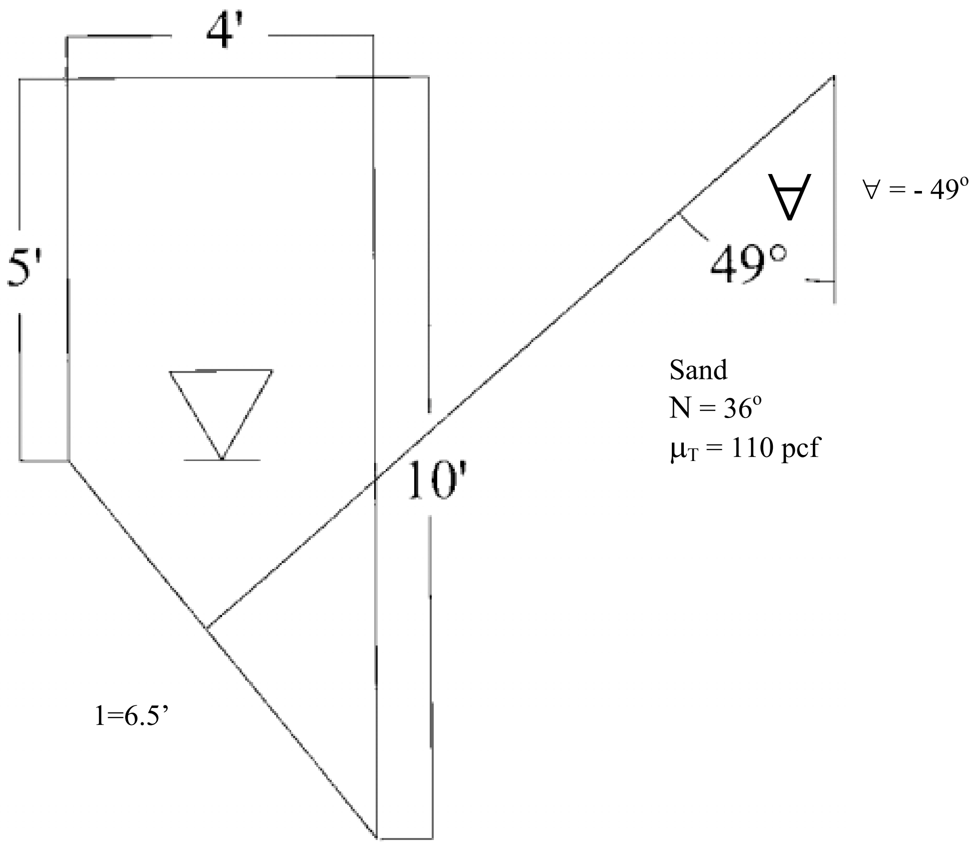
T = W_T Sin α = 95,790$^{\#}$ (Sin 16°) = 26, 403$^{\#}$

Bottom of Slice is in Clay where φ = 0 → N Tan φ = 0

c l = (1100)(13) = 14,300$^{\#}$

For slice 7: T = 26,403$^{\#}$ (Driving Force)
c l = 14,300$^{\#}$ (Resisting Force)
N Tan φ = 0, Since φ = 0

Workshop Problems Example Computation Slice 15



$$W_T = (4)\left(\frac{10+5}{2}\right)(110) = 3{,}300^{\#}$$

$$T = W_T \, Sin\alpha = 3{,}300^{\#}(Sin - 49°) = -2{,}491^{\#}$$

Note: T is negative for this slice since the weight tends to RESIST sliding.

Bottom of slice is in sand with $\phi = 36°$
$c = 0 \rightarrow cl = 0$

$$N = W_T \, Cos\alpha - \mu 1$$

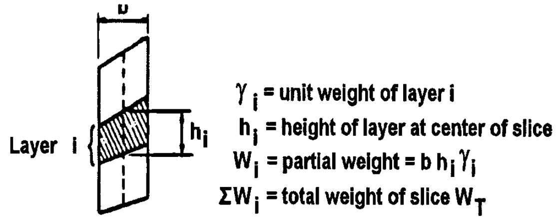
$$= (3{,}300^{\#})(Cos - 49°) - (\frac{5}{2})(60)(6.5)$$
$$= 2{,}165^{\#} - 975^{\#} = 1{,}190^{\#}$$

$N \, Tan \, \phi = 1{,}190^{\#} \, (Tan \, 36°) = 865^{\#}$

For slice 15: $T = -2{,}491^{\#}$ (Driving Force)
$N \, Tan\phi = 865^{\#}$ (Resisting Force)
$cl = 0$, Since $c = 0$

Step 6: Compute Weights for Each Slice.

Tabular Form for Computing Weights of Slices



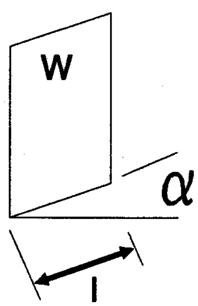
Slice No.	B	h_I	γ_i	W_i	$\sum W_i = W_T$
1	15	33/2	130	32175	32175
2	2	33	130	8580	
		2/2	110	220	8800
3	4	33	130	17160	
		(7+2)/2	110	1980	19140
4	12	33	130	51480	
		7	110	9240	
		12/27	125	9000	69720
5	12	33	130	51480	
		7	110	9240	
		(19+12)/2	125	23250	83970
6	12	33	130	51480	
		7	110	9240	
		(19+25)/2	125	33000	93720
7	12	(27+33)/2	130	46800	
		7	110	9240	
		(25+28)/2	125	39750	95790
8	12	(20+27)/2	130	36660	
		7	110	9240	
		(36+28)/2	125	43500	89400
9	12	(14+20)/2	130	26520	
		7	110	9240	
		30	125	45000	80760
10	12	(9+14)/2	130	17940	
		7	110	9240	
		(28+30)/2	125	43500	70680
11	12	(9+3)/2	130	9360	
		7	110	9240	
		(25+28)/2	125	39750	58350
12	13	10	110	14300	
		(19+25)/2	125	35750	50050
13	12	10	110	13200	
		(12+19)/2	125	23250	36450
14	12	10	110	13200	
		12/2	125	9000	22200
15	4	(5+10)/2	110	3300	3300
16	4	5/2	110	1100	1100

Workshop Design Problem

Step 7: Compute Factor of Safety.

Tabular Form for Calculating FS by Normal Method of Slices.

Slice No.	W_T (lb)	l (ft)	α (deg)	C (psf)	ϕ (deg)	μ (psf)	μ l (lb)	W_T Cos ∀ (lb)	N (lb)	N TanN (lb)	Cl (lb)	T (lb)
1	32,175	36	60	0	40	0	0	16,088	16,088	13,499	0	27,864
2	8,800	3	54	0	36	0	0	5,173	5,173	3,758	0	7,119
3	19,140	7	51	0	36	150	1050	12,045	10,995	7,988	0	14,875
4	62,720	17	43	1100	0	-	-	-	-	0	18,700	47,549
5	83,790	15	34	1100	0	-	-	-	-	0	16,500	46,955
6	93,720	15	25	1100	0	-	-	-	-	0	16,500	39,608
7	95,790	13	16	1100	0	-	-	-	-	0	14,300	26,403
8	89,400	13	9	1100	0	-	-	-	-	0	14,300	13,985
9	80,760	12	1	1100	0	-	-	-	-	0	13,200	1,409
10	70,680	12	-7	1100	0	-	-	-	-	0	13,200	-8,614
11	59,350	13	-15	1100	0	-	-	-	-	0	14,300	-15,102
12	50,050	14	-24	1100	0	-	-	-	-	0	15,400	-20,357
13	36,450	14	-32	1100	0	-	-	-	-	0	15,400	-19,316
14	22,200	16	-42	1100	0	-	-	-	-	0	17,600	-14,855
15	3,300	6.5	-49	0	36	150	975	2,165	1,190	865	0	-2,491
16	1,100	6.5	-53	0	36	0	0	662	662	481	0	-878
									$\sum$	26,591	169,400	144,154



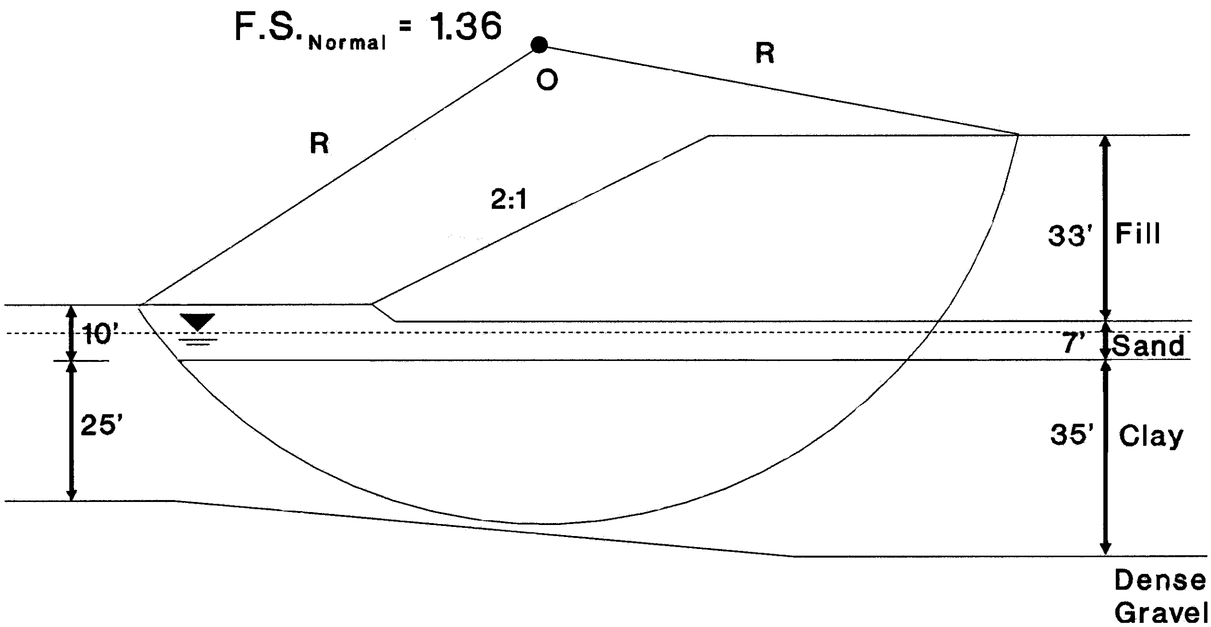
C = cohesion intercept
ϕ = friction angle
μ = pore pressure
W_T = total wt. of slice (soil + water)

$$F = \frac{\Sigma (W_T \; Cos\alpha - \mu l) Tan\,\phi + \Sigma cl}{W_T \; Sin\,\alpha}$$

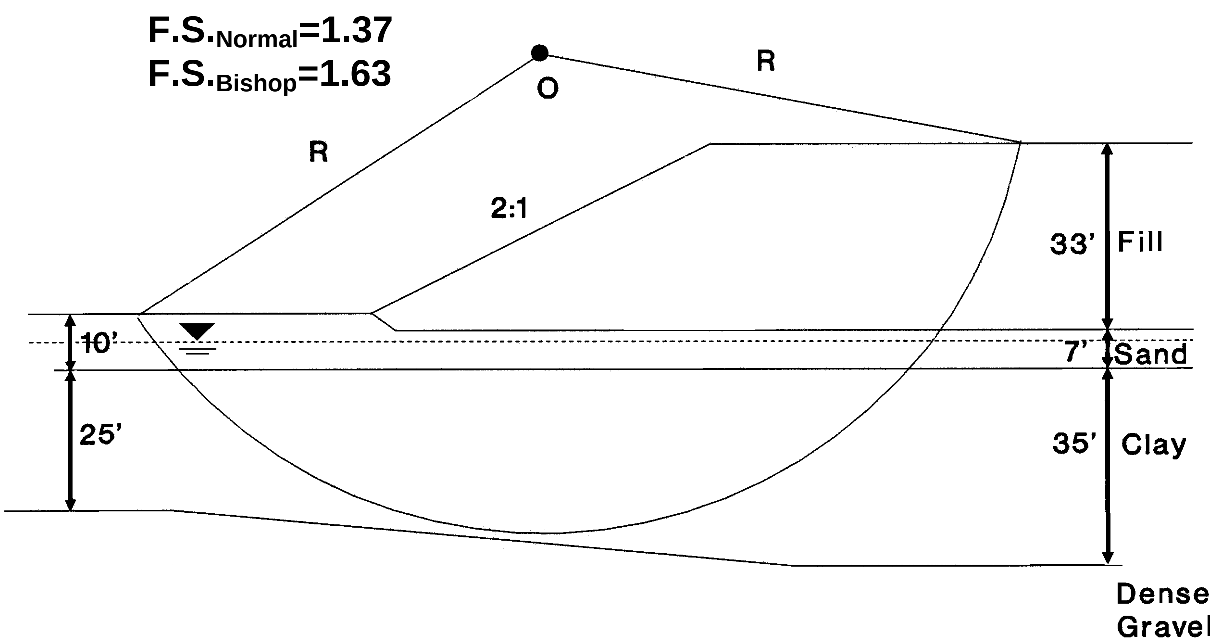
$$F = \frac{\Sigma N \; Tan\,\phi \; + \; \Sigma \, cl}{\Sigma T} = \frac{26{,}591 + 169{,}400}{144{,}154} = \underline{1.36}$$

TABULAR FORM FOR CALCULATING F.S. BY NORMAL METHOD OF SLICES

Workshop Design Problem – Hand Solution

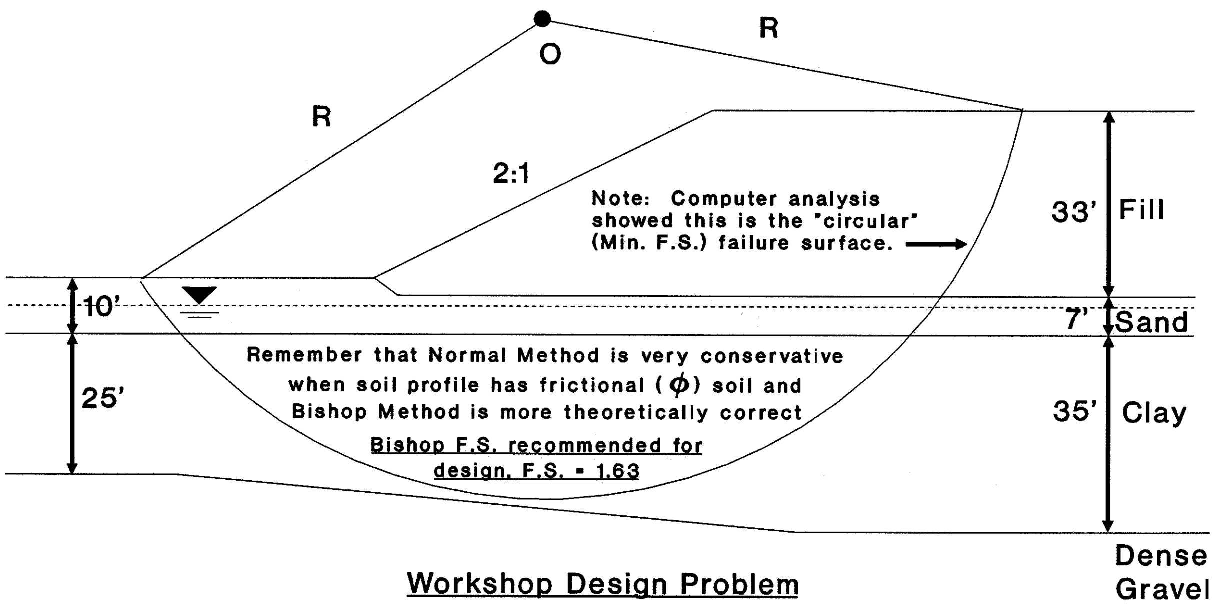


Workshop Design Problem – Computer Solution

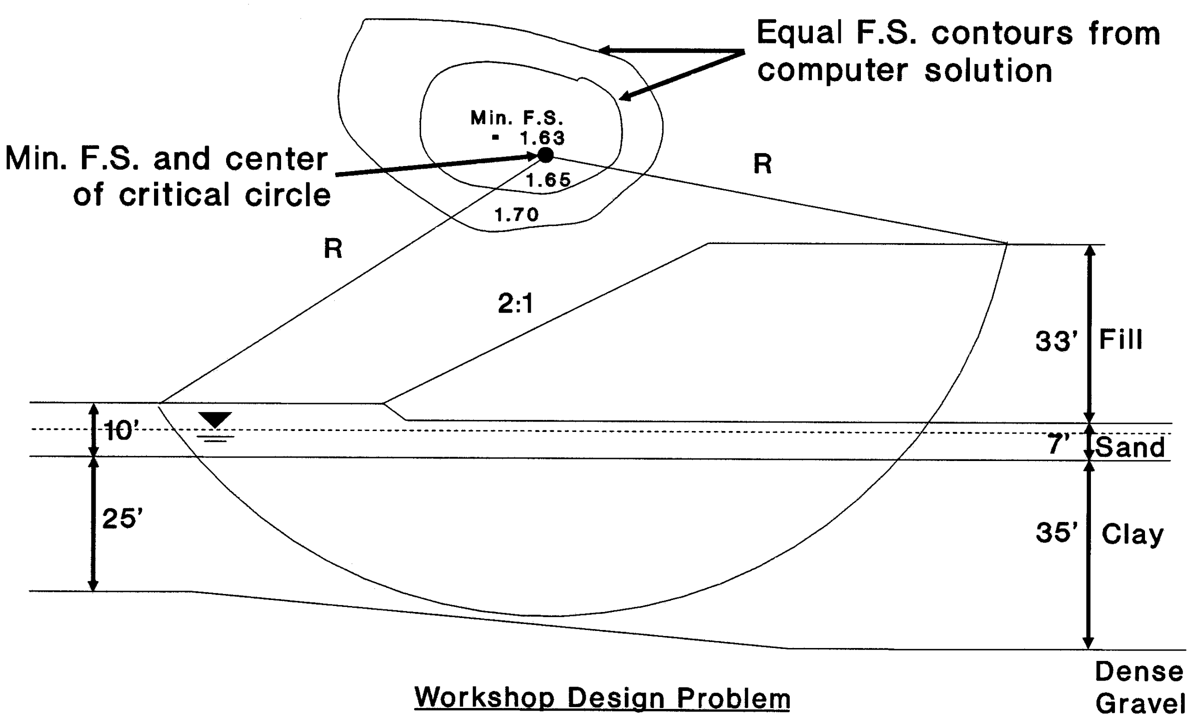


Comparison of Factors of Safety

F.S. = 1.36 Normal Method - Hand Solution
F.S. = 1.37 Normal Method - Computer Solution
F.S. = 1.63 Bishop Method - Computer Solution



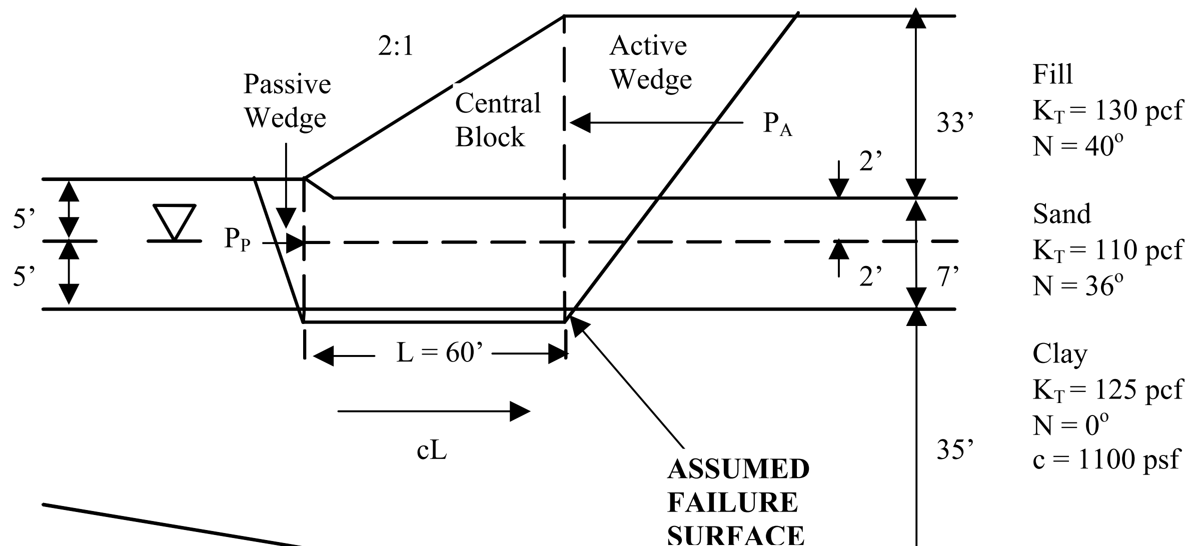
For Design use Min. F.S. (Bishop) = 1.63



WORKSHOP DESIGN PROBLEM – SLIDING BLOCK ANALYSIS

Compute Factor of Safety against sliding block type failure along top of clay layer for assumed failure surface shown.

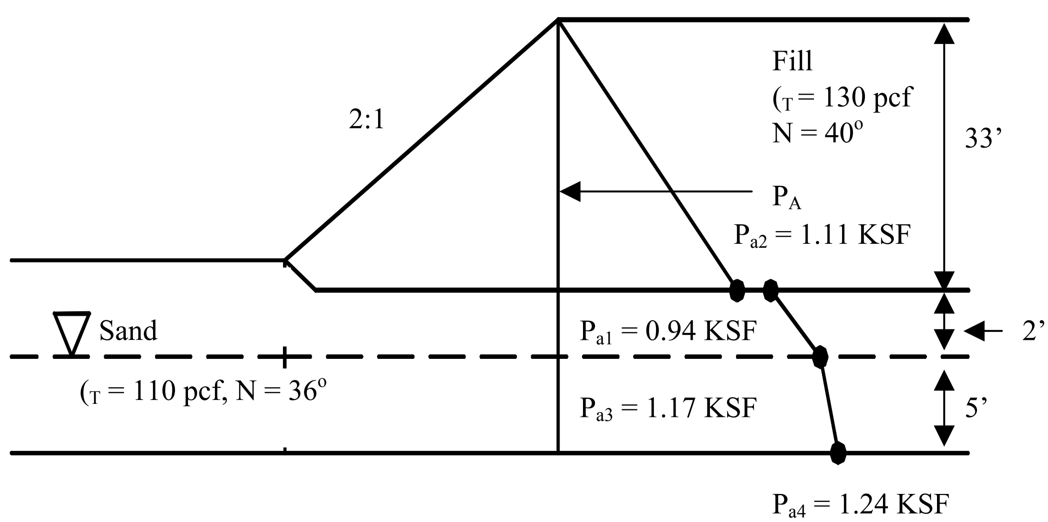
Step 1: Choose Trial Failure Surface.



Step 2: Compute Active Force (P_A)

Fill = Soil Layer 1; Fill $\phi = 40°$; $K_{A1}=\tan^2(45° - 40°/2)=\tan^2(25°)=0.22$

Soil Layer 2; Sand $\phi=36°$; $K_{A2}=\tan^2(45° - 36°/2)=\tan^2(27°)=0.26$



Step 3: Compute Active Pressure.

p_{a1} (base of fill) = $\gamma_1 h_1 K_{A1}$ = (0.130 kcf)(33′)(0.22)=0.94 ksf

p_{a2} (top of sand) = $\gamma_1 h_1 K_{A2}$ = (0.130 kcf)(33′)(0.26)=1.11 ksf

p_{a3} (2' below top of sand*) = 1.11 ksf+(0.110 kcf)(2′)(0.26)=1.17 ksf
(*Water table elevation)

p_{a4} (base of sand layer) = 1.17 ksf+(0.050 kcf*)(5′)(0.26)=1.24 ksf
(*Buoyant weight below water table)

Step 4: Plot Active Pressure Diagram & Compute Active Force.

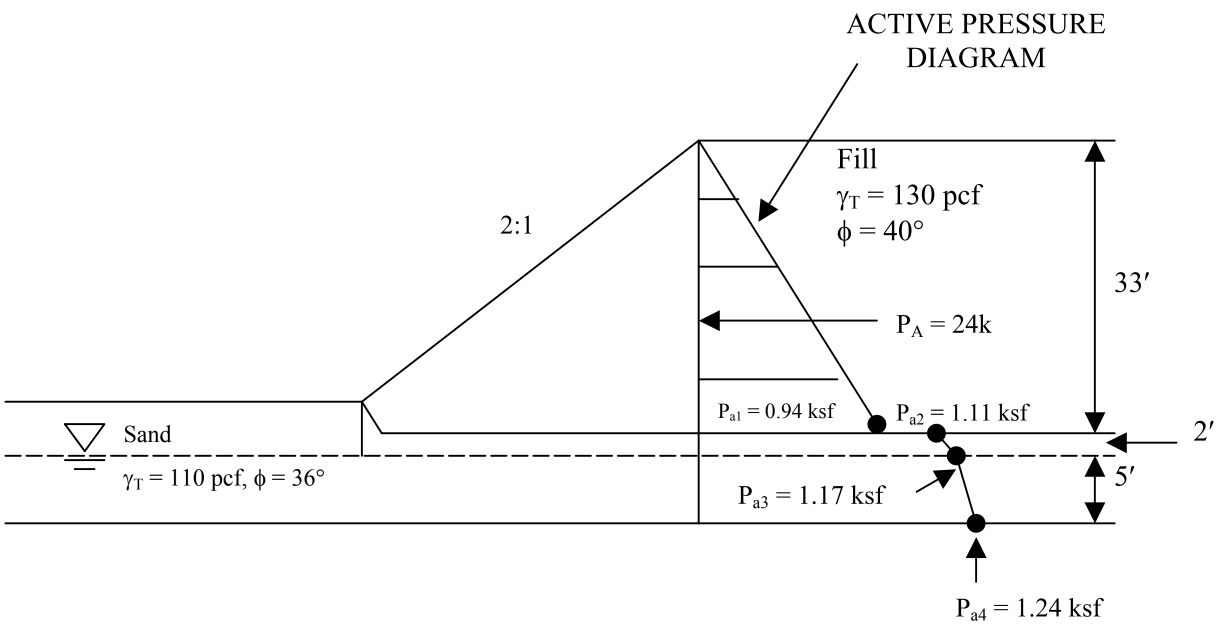
P_A = Active Force = Area of Pressure Diagram (per ft.)

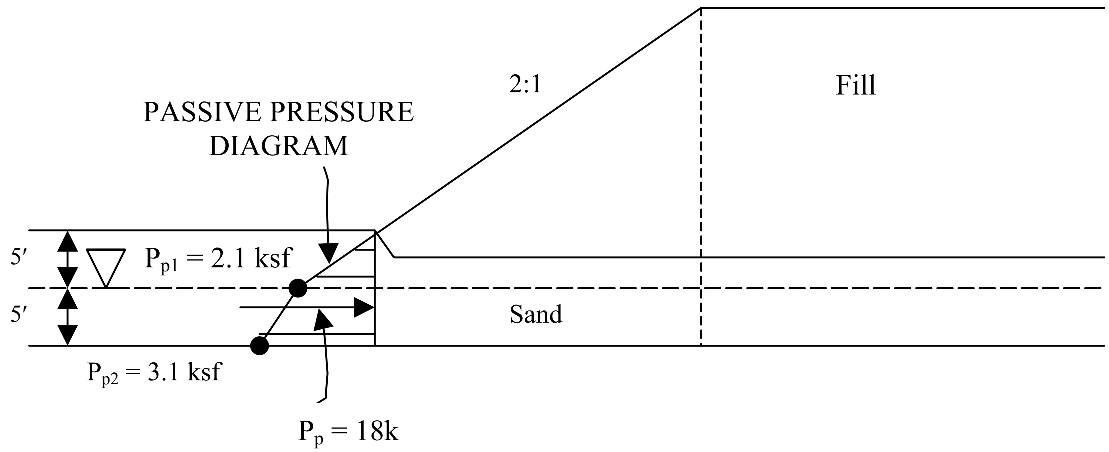
$\therefore P_A = (0.94\text{ ksf})(33')(1/2)(1')$

$+ ((1.11\text{ ksf} + 1.17\text{ ksf})/2)(2')(1')$

$+ ((1.17\text{ ksf} + 1.24\text{ ksf})/2)(5')(1')$

$= 15.5^K + 2.3^K + 6^K \therefore P_A \approx 24^K$





Step 5: **Compute Passive Force P_P.**

(a) Compute Passive Pressure

Sand $\phi = 36°$; $K_P = Tan^2(45° + \phi/2) = Tan^2(45° + 36°/2) = 3.8$

p_{p1} (5′ below top of sand*) = (0.110 kcf)(5′)(3.8)=2.1 ksf (*At water table)

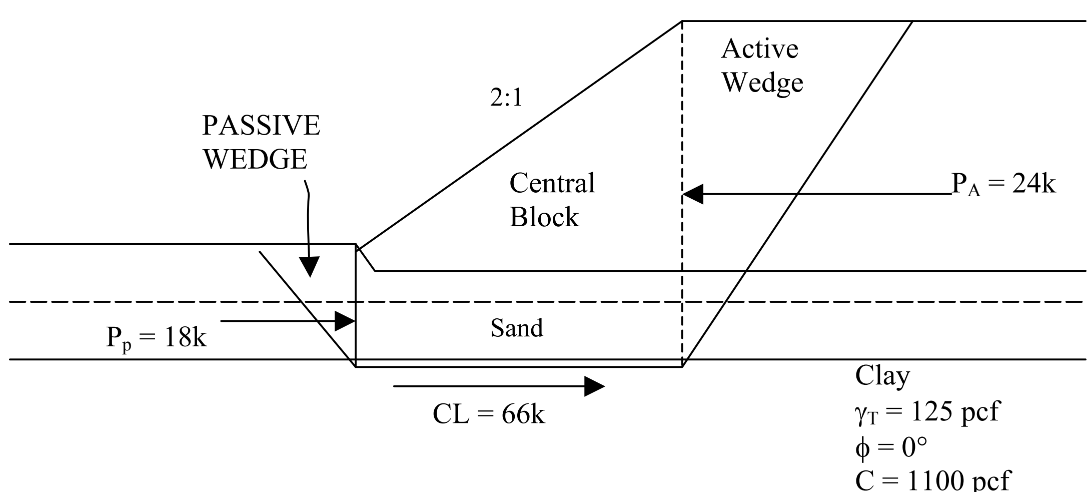
p_{p2} (base of sand layer) = 2.1 ksf+(0.050 kcf*)(5′)(3.8)=3.1 ksf (*Buoyant weight below water table)

Step 6: **Plot Passive Pressure Diagram & Compute Passive Force.**

$\therefore P_P$ (per ft)= (2.1 ksf)(5′)(1/2)(1′)

+ ((2.1 ksf+3.1 ksf)/2)(5′)(1′)

$= 5.3^K + 13^K \therefore P_P \approx 18^K$



Step 7: Compute Resisting Force of Central Block.

Assumed failure plane is along top of clay
$C = 1100$ psf = 1.1 ksf
$L = 60'$
$\therefore$ $CL = (1.1\text{ksf})(60')(1') = 66^K$ (per ft)

Step 8: Compute Factor of Safety.

$$F.S. = \frac{\text{Horizontal Resisting Forces}}{\text{Horizontal Driving Forces}} = \frac{P_p + CL}{P_A}$$

$$= \frac{18^K + 66^K}{24^K} = \frac{84^K}{24^K} = 3.5$$

F.S. = 3.5 OK $\therefore$ Circular Arc Failure More Critical

CHECK FOR - LATERAL SQUEEZE

Lateral Squeeze of Clay

Lateral squeeze causes pile supported abutments to rotate into embankment or spread footing abutments to move laterally.

Lateral Squeeze occurs if:

$$\gamma_{Fill} H_{Fill} > 3 \text{ x Cohesion}$$

For East Abutment:

130 pcf x 30' > 3 x 1100 psf
3900 psf > 3300 psf

∴ -can get lateral squeeze
-consider waiting period to dissipate settlement of fill
-do not construct abutments until settlement dissipates
(U=90%)

Summary of the Approach Embankment Stability Phase for the Apple Freeway Design Problem

- Design Soil Profile

  Soil layer unit weights and strength estimated.

- Circular Arc Analysis

  Approach embankment safety factor 1.63 against circular failure.

- Sliding & Block Analysis

  Approach embankment safety factor 3.5 against sliding failure.

- Lateral Squeeze

  Possible abutment rotation problem.