Chapter 17
EMBANKMENTS

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

June 2010

Table of Contents

List of Tables

List of Figures

CHAPTER 17

EMBANKMENTS

17.1 INTRODUCTION

This Chapter provides general guidance in stability and settlement design and analysis of earthen embankments (unreinforced slopes) and earth retaining structures (ERS) (see Chapter 18) belonging to SCDOT. This Chapter is concerned with the external stability of slopes and ERSs. The internal stability of ERSs is the responsibility of the structural designer. The settlement of earthen embankments, ERSs, foundations (shallow and deep) is discussed in this Chapter. These settlements are for the Service limit state. Settlements caused by the Extreme Event I limit state are discussed in Chapter 13. The amount of total and differential settlement and the rate of settlement shall be determined. All settlements will be determined for a twenty-year period, unless specifically directed by either the GDS or the PCS/GDS to use another time period.

Stability and settlement should be determined on the critical section. The selection of the critical section or sections is left to the geotechnical engineer-of-record. However, the following are suggested guidelines for use in the selection process:

1. Highest slope or ERS
2. Steepest slope
3. Soft underlying soils
4. Slope or ERS critical to performance of a structure (i.e. bridge, culvert, etc.)

Embankments with heights less than 5 feet and slopes 2H:1V or flatter do not require analysis, unless in the opinion of the geotechnical engineer-of-record the analysis is required.

Slopes and ERSs can be divided into two main categories; natural and man-made. Natural slopes are those slopes formed by natural processes and are composed of natural materials. Natural slopes can include river banks to the valleys passing through or parallel to mountain ridges. Man-made slopes are those slopes that are constructed by man. Man-made slopes may be subdivided into two types of slopes, fill (bottom up construction) and cut (top down construction). Fill slopes, including ERSs, are constructed by placing soil materials to elevate the grade above the natural or existing grade. Fill slopes may be unreinforced or reinforced (see Chapter 18). Cut slopes, including ERSs, are constructed by excavating material from either a natural or man-made fill slopes in order to reduce the grade. The stability and settlement procedures discussed in this Chapter exclusively apply to slopes constructed of soil and founded soil or rock materials. For the design of slopes in rock see FHWA NHI-132035 – *Rock Slopes* for design procedures.

17.2 FAILURE MECHANISMS

There are several failure mechanisms that affect slopes and ERSs. The mechanism of failure may dictate the required analysis method to be used to determine stability or instability. Further, the type of soil that the slope or ERS is comprised of will also affect the failure mechanism. The different failure mechanisms are listed below:

1. Creep
2. Flow
3. Fall and Topple
4. Slide
5. Spread
6. Deformation and settlement

17.2.1 Creep

Creep is the very slow movement of slopes, either natural or man-made, toward the toe and a more stable configuration. This movement can range up to approximately one inch per year. Slopes that creep can remain stable for extended periods of time. However, once the limit of the soil shear strength has been reached, the amount of movement may increase and the time for movement may decrease resulting in a rapid or sudden failure of the slope. Creep movements can be divided into two general types, seasonal and massive. Seasonal creep is the creep that occurs during successive seasons, such as freezing and thawing, or wetting and drying. The amount of seasonal creep can vary from year to year, but is always present. Seasonal creep extends to the depth limit of seasonal variations of moisture and temperature. Massive creep causes almost constant movement within the slope and is not affected by seasonal variations. Massive creep typically occurs in clay-rich soils. While the actual mechanism of massive creep is not fully understood, this type of creep can be attributed to exceeding some threshold shear strength that is below peak shear strength. This threshold shear strength may be a very small portion of the peak shear strength. If the stresses in the slope remain below the threshold level, then movement will not occur; however, if the stresses exceed the threshold, then movements will occur. If enough stresses accumulate to exceed the peak shear strength, then a more rapid failure is possible. In general, once creep has started it is difficult or impossible to stop. However, the rate of creep may be reduced by placement of drainage. During the geoscoping of the project site, the trees should be observed for any convex curvature with the convex part pointing down slope (see Figure 17-1).

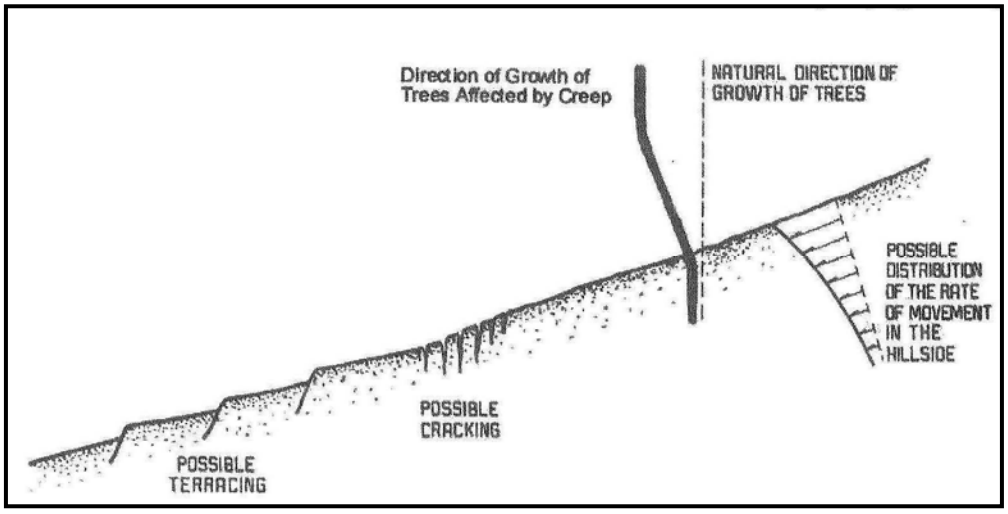


Figure 17-1, Signs of Creep
(Soil Slope and Embankment Design – September 2005)

17.2.2 Slide

Slides are downward slope movements that occur along definite slip or sliding surfaces. Slides may be translational, rotational, or a composite of rotation and translation. Translational slides are typically shallow and linear in nature. This type of slide typically occurs along thin weak layers or along the boundary between a firm overlying layer and weaker underlying layer (see Figure 17-2).

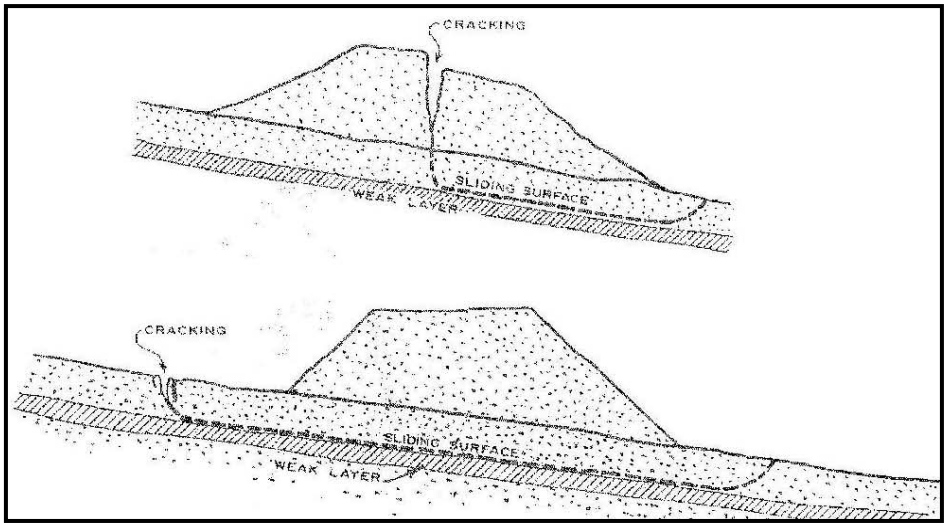


Figure 17-2, Translational Slide
(Soil Slope and Embankment Design – September 2005)

Rotational slides are slides that form an arc along the shearing surface. This is the most common type of failure analyzed. In soft, relatively homogenous cohesive materials, the rotational slide forms a deep seated arc, while in cohesionless materials; the rotational slide failure surface tends to be relatively shallow. Examples of different types of rotational slides are depicted in Figure 17-4.

Compound slides are a composite of translational and rotational slides. This type of slide tends to have a complex structure and can be difficult to analyze. Compound slides can have two general forms, retrogressive and progressive. Retrogressive compound slides continue to cut into the existing slope. After initial failure, the new slope that is formed is unstable and fails, developing another new unstable slope face that fails. This slide type may result in a series of slides that tend to converge on one extended slope. Progressive slides occur when an existing slope surface is loaded with either new fill or debris from a slope failure, resulting in failure of the slope toward the toe. Compound slide types are depicted in Figure 17-3.

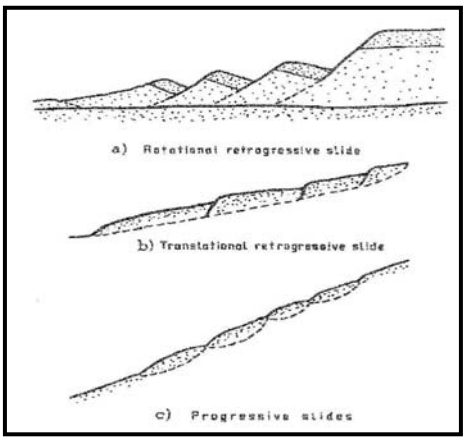


Figure 17-3, Compound Slide
(Soil Slope and Embankment Design – September 2005)

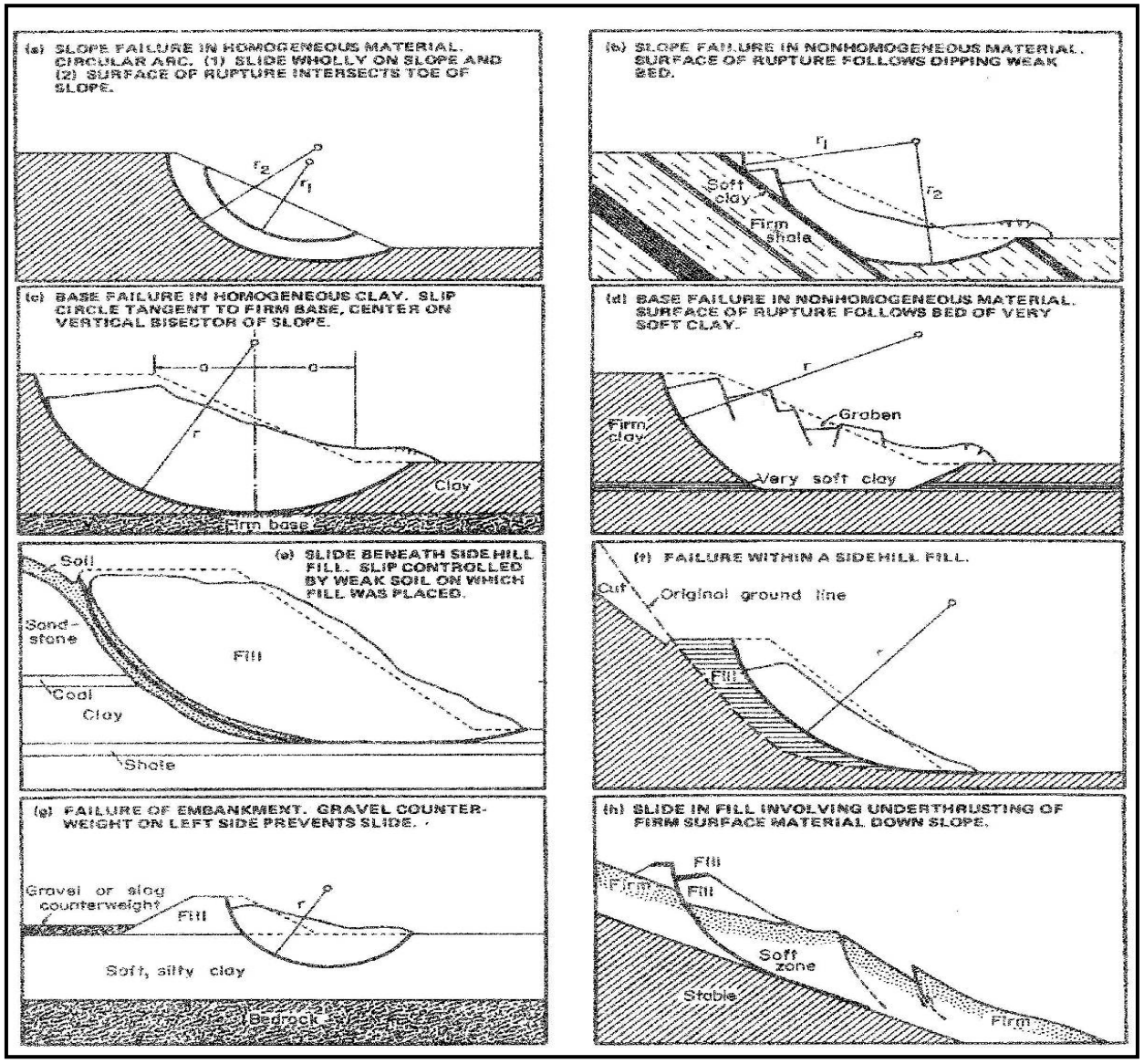


Figure 17-4, Rotational Slide
(Soil Slope and Embankment Design – September 2005)

17.2.3 Flow

Flow failures can occur in both dry, as well as wet soils, depending on the materials and the relative density. Flows are defined as mass soil movements that have greater internal deformations than slides. In a slide, the soil block will maintain some definition during sliding, whereas in flows, the definition of the block is completely lost. Flow failures, depending on the moisture condition of the soil, may behave similar to a fluid. In dry flow failures of fine-grained cohesionless soils, the movements are caused by a combination of sliding and individual particle movements. These types of failures may be caused by soils being cut on steep slopes that are stable when first constructed, but become unstable with time. Dry flow failures are also termed earthflows. Moist flows occur in soils that have higher moisture contents than the soils in a dry flow. In cohesive soils, moist flows occur when the moisture content exceeds the liquid limit of the material. In cohesionless soils, moist flows may occur when water becomes trapped in the soils by an impermeable barrier. Liquefaction is a form of moist flow that is caused by high moisture content and a seismic shock (see Chapter 13). Wet flows are also termed mudflows. See Figure 17-5 for dry and moist flow failure examples.

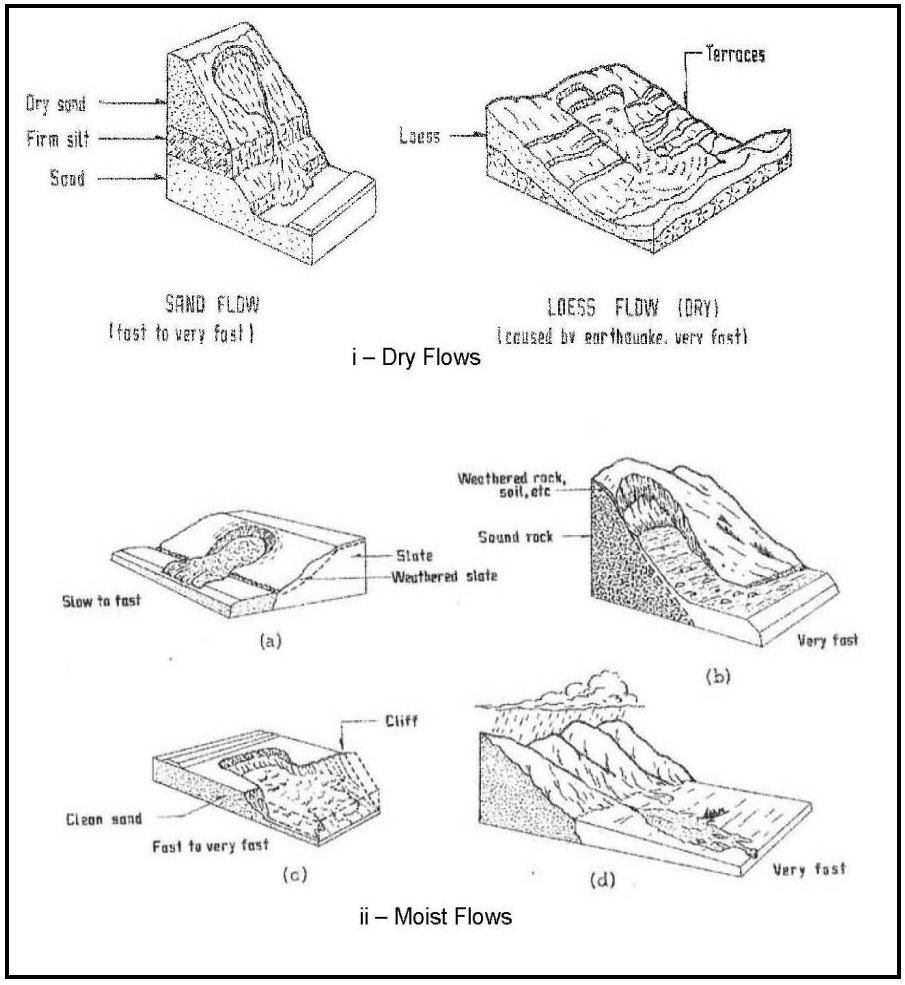


Figure 17-5, Flow Failures
(Soil Slope and Embankment Design – September 2005)

17.2.4 Spread

Spread was originally defined by Terzaghi and Peck in 1948 to describe sudden movements of water bearing seams of cohesionless materials overlain by homogeneous clays or fills. Spreads occur on very gentle slopes (< 5 percent) or flat terrain. Spreads can occur in cohesionless soils (liquefaction) or in cohesive soils (quick clays) that are externally loaded. In the case of liquefaction, the load is the seismic shock, and in quick clays, that load may be applied by the placement of fill materials. Figure 17-6 illustrates a typical soil spread.

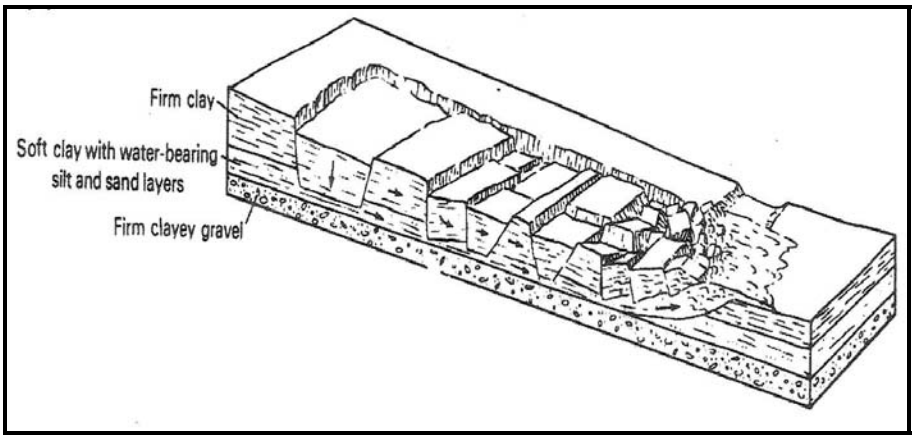


Figure 17-6, Spread Failure
(Soil Slope and Embankment Design – September 2005)

17.2.5 Fall and Topple

Fall and topple failures typically occur on rock slopes, although, topples can occur in steeply cut or constructed soil slopes. Falls are sudden movements of rocks and boulders that have become detached from steep slopes or cliffs (see Figure 17-7). Cracks can form at the top of the steep slope that may fill with water that will exert pressure on the rock mass causing it to fall. The water may freeze during colder weather exerting pressure on the rock mass as well. A topple is the forward rotation of rock or soil mass around a pivot point in the mass (see Figure 17-8). The steepness of the slope affects the formation of the topple, the slope can be constructed too steep or can be eroded to a steep configuration.

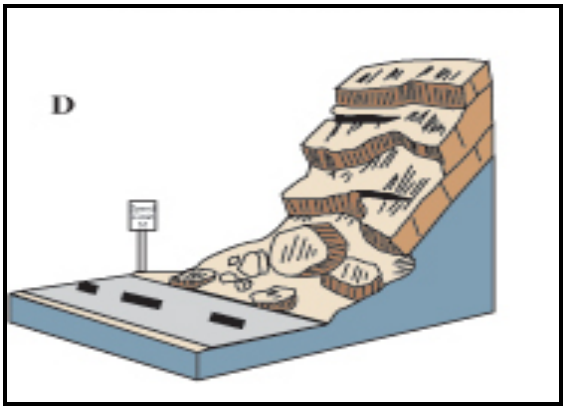


Figure 17-7, Fall Failure
(Landslide Types and Processes – July 2004)

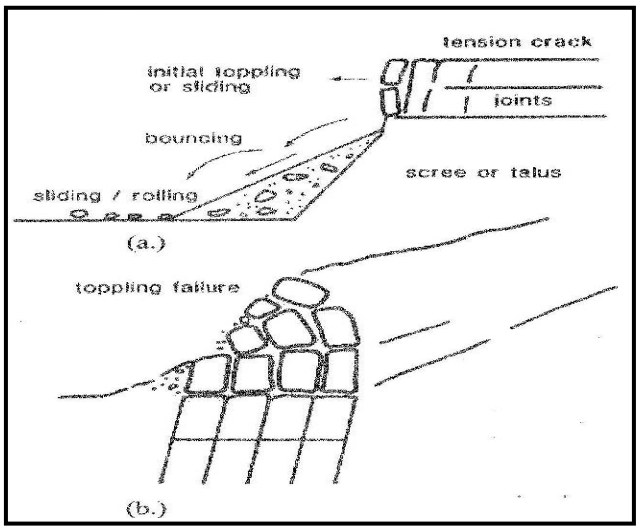


Figure 17-8, Topple Failure
(Soil Slope and Embankment Design – September 2005)

17.3 LOADING CONDITIONS

The stability of slopes and ERSs is based on the height of the slope or ERS (i.e. the load) and the resistance of the subsurface soils (i.e. shear strength) to that loading. Increasing the height and steepness of the slope increases the potential for instability. It is incumbent upon the geotechnical engineer to know and understand the loading conditions for which the stability analysis is being performed to evaluate. All of these loading conditions apply to both natural and man-made fill and cut slopes, but each condition does not have to be analyzed in every case`. These loading conditions are listed below:

1. End-of-Construction (Short-term)
2. Long-term
3. Earthquake
4. Staged Construction
5. Rapid Drawdown
6. Surcharge Loading
7. Partial Submergence

Each of these loading conditions requires the selection of the appropriate soil strength parameters. Chapter 7 provides a more detailed discussion on the selection of drained and undrained soil shear strength parameters and the differences in total and effective stress. Once the rate of loading (i.e. loading condition) is determined, the soil response should be determined (i.e. drained or undrained). The drained response of soil is determined by loading the soil slowly enough to allow for the dissipation of pore pressures ($\Delta u = 0$). Conversely, the undrained response of a soil is determined by loading the soil faster than the pore pressures can dissipate ($\Delta u \neq 0$). This change in pore pressure can be either positive or negative depending on whether the soil compresses ($\Delta u > 0$) or dilates ($\Delta u < 0$). After determining the soil response (either drained or undrained), the type of analysis is selected based on the dissipation of pore pressures and the rate of loading. If the pore pressure increases with the application of load, i.e. during fast loading on a fine-grained soil, then a total stress analysis is conducted. If the loading does not produce a change in pore pressure, i.e. during slow loading of a fine-grained

soil or the loading is placed on coarse-grained material, then an effective stress analysis is conducted.

According to Duncan and Wright (2005), "Whether slope stability analyses are performed for drained conditions or undrained conditions, the most basic requirement is that equilibrium must be satisfied in terms of total stress." In other words, all forces, including water that act on the slope or ERS, need to be accounted for in the stability analysis. The development of these forces allows for the determination of the total normal stress acting on the shear surface and the shear strength required to maintain equilibrium. Normal stresses are required to develop the soil shear strength ($\Phi > 0°$). The shear strength of cohesive, fine-grained ($\Phi = 0°$) is independent of the normal stress acting on the shear surface.

To develop effective stress shear strength parameters, the pore pressures along the shear surface need to be known and need to be subtracted from the total shear strength. For drained conditions, the pore pressures can be estimated using either hydrostatic or steady seepage boundary conditions. However, for undrained conditions, the pore pressures are a function of the response of the soil to shearing, therefore, the evaluation of the pore pressures is difficult. The development of total stress shear strength parameters does not require determination of pore pressures. Total stress analyses therefore can only be applied to undrained conditions. In total stress analyses, the pore pressures are determined as a function of the behavior of the soil during shear.

In drained soil response, the load is applied slow enough to allow for the dissipation of excess pore pressures. An effective stress analysis is performed using:

- Total unit weights
- Effective stress shear strength parameters
- Pore pressures determined from hydrostatic water levels or steady seepage analysis

Total unit weights are required in drained soil response. Since the majority of the analytical software packages account for the location of the groundwater table, it is incumbent on the engineer to know the requirements of the analytical software package and provide the correct input parameters.

In undrained soil response, the load is applied rapidly and excess pore pressures are allowed to build up. The pore pressures are controlled by the response of the soil to the application of the external load. A total stress analysis is performed using:

- Total unit weights
- Total stress shear strength parameters

The previous discussion dealt with the selection of total or effective stress strength parameters; however, these strength parameters are for peak shear strength. The use of peak shear strengths is appropriate for fill type slopes. However, the use of peak shear strength parameters in cut slopes should be considered questionable. Therefore, the use of residual shear strength shall be used in the design of cut slopes. Residual shear strength should be either determined from laboratory testing or using the procedures outlined in Chapter 7. The location of the water surface in cut slopes should be accounted for during design. The use of steady state seepage may be required, particularly, if the slopes intercepts the water table well

above the toe of the slope. In addition, surface drainage features may be required to control the flow of groundwater as it exits the slope.

17.3.1 End-of-Construction Condition

The End-of-Construction condition also termed Short-term can have either drained or undrained soil response depending on the time for excess pore pressure ($\Delta u \neq 0$) dissipation. The time for excess pore pressure dissipation shall be determined using the method described in Chapter 7 or from consolidation testing of the embankment or ERS materials. If the time for pore pressure dissipation is determined to be days or weeks (typically cohesionless soils), then drained soil response should be used. Conversely, if the time for pore pressure dissipation is months to years (typically, plastic, fine-grained soils), then undrained soil response should be used. Engineering judgment should be used for the soils that have a time for pore pressure dissipation of weeks to months. The selection on the use of drained or undrained soil response should be based on the time for the completion of construction. In addition, the slope being analyzed may consist of materials that have both drained and undrained soil responses (i.e. the slope contains cohesionless and plastic fine-grained materials). The soil response of each layer should be determined based on the time for dissipation of pore pressures in each layer. The live load caused by traffic should be included in this analysis.

For the End-of-Construction loading condition for embankments and ERSs, the weight of the pavement and live load surcharges shall be applied. The loads shall be determined as specified in Chapter 8 of this manual. The load factor (γ_i) shall be taken as 1.0.

17.3.2 Long-term Condition

The Long-term condition should use a drained soil response model. The use of the drained soil response is based on the assumption that excess pore pressures have dissipated ($\Delta u = 0$). The time for dissipation of pore pressures should be determined, if the engineer suspects that not enough time has passed to allow for the dissipation. The appropriate soil response should be selected (i.e. drained if $\Delta u = 0$ or undrained if $\Delta u \neq 0$). During Long-term analysis, the live load surcharge (see Chapter 8) and the dead load induced by the existing pavement section and any asphalt overlays (see Chapter 8 and assume overlay has a thickness of 12 inches) should be included. Similar to the End-of-Construction loading condition, the load factor (γ_i) shall be 1.0.

17.3.3 Earthquake Condition

According to Duncan and Wright (2005), the stability of slopes and ERSs is affected by earthquakes in two ways; first the earthquake subjects the soils to cyclically varying loads and secondly, cyclic strains induced by these loads may lead to a reduction in the shear strength of the soil. The soil response during cyclic loading is undrained, since the load is applied rapidly and excess pore pressures do not have time to dissipate. In soils where the shear strength reduction is less than fifteen percent, a psuedostatic analysis is normally conducted. If the soil shear strength is reduced more than fifteen percent, then a dynamic analysis should be performed. See Chapter 7 for aid in determining the reduced shear strengths that should be used. The earthquake condition is discussed in greater detail in Chapter 13. In the earthquake

analysis, include the dead load induced by the addition of asphalt to the pavement section, but do not include the live load surcharges.

17.3.4 Staged Construction Condition

The placement of embankments or ERSs over soft fine-grained soils may require vertical staging of the construction to prevent instability. The instability is caused by the low shear strength of the fine-grained soil and the increase in excess pore pressures. The increase in pore pressures lowers the drained shear strength of the soil; however, with the dissipation of the excess pore pressures, the effective strength of the soil will increase. Therefore, vertical staging can be used to increase the soil shear strength by placing a stage (thickness) of soil over the soft fine-grained soil and allowing the excess pore pressures to dissipate (waiting period). Once the excess pore pressures have dissipated, then the next stage can be placed. This process can be repeated until the final height of the embankment or ERS is achieved. The completion of placement of each stage is critical, since this is when instability is most likely to occur (i.e. the soil shear strength has not increased). Settlement of the stage will also occur as the excess pore pressures dissipate. Therefore, consolidation testing is required to determine the time rate of settlement, as well as the magnitude of total settlement, that is anticipated for each stage as well as the full embankment or ERS. If the time rate of settlement indicates that the waiting periods are too long, then wick drains may be used to increase the time to complete settlement. The increase in shear strength is a function of the Degree of Consolidation (U). Provided below are the equations relating the increase in undrained shear strength to U.

$$\Delta C_u = U_t * (\Delta\sigma * \tan(\varphi_{cu})) \qquad \text{Equation 17-1}$$

or

$$\Delta C_u = U_t * (\Delta\sigma * (\tau / \sigma'_{vo})) \qquad \text{Equation 17-2}$$

Where,

$$\tan(\varphi_{cu}) = 0.23(OCR)^{0.8} = \frac{\tau}{\sigma'_{vo}} \qquad \text{Equation 17-3}$$

Where,

- ΔC_u = Increase in undrained cohesion
- U_t = Degree of consolidation at a specific time (enter as decimal)
- $\Delta\sigma$ = Increase in applied vertical stress
- φ_{cu} = Consolidated undrained internal friction angle
- τ = Undrained Shear Strength
- σ'_{vo} = Vertical effective overburden stress
- OCR = Overconsolidation Ratio (see Chapter 7)

The soil shear strength and consolidation parameters should be determined in accordance with the procedures outlined in Chapter 7. See Chapter 19 for information concerning prefabricated vertical drain design. The stability of the slope or ERS should be monitored using the instrumentation described in Chapter 25.

17.3.5 Rapid Drawdown Condition

The Rapid Drawdown Condition occurs when a slope or ERS that is used to retain water experiences a rapid (sudden) lowering of the water level and the internal pore pressures in the slope or ERS cannot reduce fast enough. For SCDOT projects, this is not a normal condition because SCDOT slopes or ERSs are not designed to retain water. However, in some situations water may build up against a slope or ERS, where this has occurred, Rapid Drawdown should be considered. For procedures on how to conduct rapid drawdown analysis, see Duncan and Wright, 2005.

17.3.6 Surcharge Loading Condition

Surcharge loads can be either temporary or permanent. Temporary surcharge loads can include construction equipment or additional fill materials placed on an embankment to increase the rate of settlement. Temporary soil surcharges are typically used in conjunction with staged constructed embankments. Therefore, the effects of the surcharge will need to be accounted for in staged construction. Typically, for temporary surcharges like equipment, the undrained shear strength of the soil should be used. Permanent surcharges consist of asphalt overlays. These permanent surcharge loads should be included in long term analysis.

17.3.7 Partial Submergence Condition

The partial submergence condition occurs when an embankment or ERS experiences the flood stage of a river or stream. When these conditions occur water, can penetrate the embankment or ERS and affect the shear strength of the soil. The amount of water that actually penetrates the embankment or ERS is a function of the permeability of the material used in the construction of the embankment or ERS. The permeability of the embankment material or retained backfill can be estimated as indicated in Chapter 7. Further, the duration of the flood event will also determine the effect of the flood on the embankment or ERS. The longer the flood lasts, the more the potential effect on the embankment or ERS.

17.4 LRFD SLOPE STABILITY

FHWA/AASHTO has recommended that the stability of an embankment slope or an ERS be determined using the Service limit state instead of the Strength limit state. Use of Service instead of Strength limit state accounts for two design issues. The first is that current slope stability analysis software does not allow for the input of load and resistance factors. Second is that most of the strength parameters, required in stability analysis, are derived from correlations (see Chapter 7). Further, the research is incomplete for the determination of resistance factors, since relatively few embankment or ERS failures occur, where the strength of the soil can be accurately determined and applied across a broad spectrum of soils. Therefore, the basic ASD calculation methods will continue to be used. After completion of the ASD analysis, the calculated Safety Factor (SF) is inversed to convert from ASD to LRFD.

$$\textbf{Driving Force } (\Sigma q_i) \leq \textbf{Resistance Force } (\Sigma r_i) \qquad \text{Equation 17-4}$$

or

$$\mathbf{SF} = \frac{\sum \mathbf{r_i}}{\sum \mathbf{q_i}} \qquad \text{Equation 17-5}$$

As indicated in Chapter 8, the basic LRFD equation is

$$\mathbf{Q} = \sum \gamma_i \mathbf{Q_i} \leq \varphi_n \mathbf{R_n} = \mathbf{R_r} \qquad \text{Equation 17-6}$$

Where,

Q = Factored Load
Q_i = Force Effect
γ_i = Load factor
R_r = Factored Resistance
R_n = Nominal Resistance (i.e. ultimate capacity)
φ_n = Resistance Factor

In using the Service limit state versus the Strength limit state, the stability analysis reverts to the typical way of performing stability analysis since the various Service limit states (I, II, III, and IV) all use a load factor (γ_i) of 1.0. Therefore, Equation 17-5 can be rewritten:

$$\mathbf{Q} = \sum \mathbf{Q_i} \leq \varphi \sum \mathbf{R_n} = \mathbf{R_r} \qquad \text{Equation 17-7}$$

or

$$\frac{\mathbf{1}}{\varphi} = \frac{\boldsymbol{R_r}}{\boldsymbol{Q}} = \frac{\sum \boldsymbol{r_i}}{\sum \boldsymbol{q_i}} \qquad \text{Equation 17-8}$$

Equating Equation 17-5 with Equation 17-8 produces

$$\mathbf{SF} = \frac{\sum \mathbf{r_i}}{\sum \mathbf{q_i}} = \frac{\mathbf{1}}{\varphi} \qquad \text{Equation 17-9}$$

Alternately Equation 17-9 may be written

$$\varphi = \frac{\mathbf{1}}{\mathbf{SF}} \qquad \text{Equation 17-10}$$

Therefore, to obtain the required resistance factor (φ) from typical slope stability software packages, the SF obtained is simply inversed. The lower the resistance factor the higher the Safety Factor.

17.5 SLOPE STABILITY ANALYSIS

Stability analysis is based on the concept of limit equilibrium (i.e. Driving Force (Σq_i) ≤ Resistance Force (Σr_i)). The Driving Forces include the weight of the soil wedge (i.e. dead load), any live load surcharges and any other external loads (i.e. impact loads on ERSs). The Resisting Force is composed entirely of the shearing resistance of the soil. Limit equilibrium is defined as the state where the resisting force is just larger than the driving force (i.e. SF = 1.01 or φ = 0.99). According to Duncan and Wright (2005), the equilibrium can be determined for either "single free body or for individual vertical slices." Regardless of how equilibrium is determined, three static equilibrium conditions must be satisfied.

- Moment equilibrium
- Vertical equilibrium
- Horizontal equilibrium

Not all methods resolve all of the equilibrium conditions; some just resolve one condition while others solve two conditions and assume the third condition is zero. Other methods solve all three equilibrium conditions.

The single free body looks at the driving forces and the resisting forces along an assumed failure surface. These solutions tend to be relatively simple and are more conducive to the use of design charts. The use of design charts is applicable to preliminary design and should therefore be used when conducting preliminary design. The second method of solving equilibrium is dividing the slope into individual vertical slices. There are numerous procedures that resolve equilibrium using vertical slices. Listed below are some of the more common types.

- Ordinary Method of Slices
- Simplified (Modified) Bishop
- Force Equilibrium
- Spencer
- Morgenstern and Price

The Ordinary Method of Slices and Simplified Bishop assume a circular failure surface while the others assume a non-circular failure surface. The Spencer and Morgenstern and Price both provide a solution for all three equilibrium conditions. The vast majority of slope stability software packages are capable of using more than one method to determine the stability of a slope or ERS, and changing the method will affect the resistance factor (φ = 1/SF). Therefore, the method that will produce the lowest Factor of Safety (highest resistance factor) shall be used. A minimum of three methods shall be used to determine slope stability with the Simplified (Modified) Bishop Method being required. The above list is not meant to be all inclusive. If other methods not found on the list are used, the Geotechnical Report shall include a detailed discussion of the methodology, including previous projects on which the method was used by the geotechnical engineer-of-record. The use of Factor of Safety recognizes the fact that virtually all software currently in use to determine the stability of slopes utilizes Allowable Strength Design (ASD) as opposed to LRFD. Using hand calculations is recommended to check the final solution provided by a software package provided the method used to determine

stability can be readily solved by hand. A brief discussion of the methods listed above is provided in the following sections.

The use of computer generated solutions for slope stability should be limited to final designs only. The use of slope stability design charts is permitted for preliminary design during the preliminary exploration. The results of these charts are reported in the preliminary road geotechnical report. For detailed explanation and discussion on how to use the charts, see Appendix B.

17.5.1 Ordinary Method of Slices

The Ordinary Method of Slices (OMS) is one of the earliest methods for determining the stability of a slope and was developed by Fellenius in 1936. This method solves moment equilibrium conditions only and is applicable only to circular failure surfaces. This method does not solve either vertical or horizontal equilibrium conditions. This method is relatively simplistic and can be solved by hand easily. This method should not be used during final design. Its inclusion here is for completeness of the various slope stability methods.

17.5.2 Simplified Bishop

The Simplified Bishop method was developed by Bishop in 1955 and can also be called the Modified Bishop method. The Simplified Bishop method solves two of the equilibrium equations, moment and vertical. This method assumes that horizontal forces are not only perpendicular to the vertical sides of the slice, but are equal and opposite; therefore, the horizontal forces cancel out. Since horizontal equilibrium is not satisfied, the use of the Simplified Bishop method in pseudo-static seismic design is questionable and should therefore not be used. As with the OMS, Simplified Bishop can only be used on circular failure surfaces.

17.5.3 Force Equilibrium

For the Force Equilibrium method of determining slope stability, depending on the method selected (Lowe and Karafiath; Simplified Janbu; Modified Swedish), the moment equilibrium is either ignored or assumed to be zero. Duncan and Wright (2005) and the US Army Corps of Engineers (2003) contain a detailed description of each of these Force Equilibrium methods. The Force Equilibrium method solves both the horizontal and vertical forces. The main assumption required using this method is the inclination of the horizontal forces on the given slice. The inclination of the horizontal forces acting on slice may be either the slope angle or the average slope angle if multiple slopes are involved (i.e. a broken back slope). In addition, the Force Equilibrium methods solve non-circular failure surfaces and therefore, may be solved graphically.

17.5.4 Spencer's Method

Spencer's Method solves all three conditions of equilibrium and is therefore termed a complete limit equilibrium method. Spencer's Method was originally developed to determine the stability of circular failure surfaces; however, Wright (1969) determined that the Spencer Method could also be used on non-circular failure surfaces as well. This method assumes that the interslice forces are parallel and act on an angle (θ) above the horizontal. This angle is one of the

unknowns in this method. Therefore, a first approximation of the angle should be the slope angle. The other unknown is the Factor of Safety. Because the method solves for Factor of Safety, an iterative process is required; therefore, this method lends well to the use of computers.

17.5.5 Morgenstern and Price Method

The Morgenstern and Price Method is very similar to the Spencer Method. The main difference between the two methods is that Spencer solves for the interslice angle, while the Morgenstern and Price Method solves for the scaling parameter that is used on a function that describes the slice boundary conditions. The Morgenstern and Price Method provides added flexibility using the interslice angle assumptions.

17.6 SETTLEMENT – GENERAL

Regardless of the type of structure, embankments, ERSs, bridges or buildings are all placed on geomaterials (i.e. soil and rock) and will therefore potentially undergo settlement. According to FHWA-NHI-05-132, *Soil Slope and Embankment Design*, settlement is comprised of three components: immediate (elastic or instantaneous), primary consolidation and secondary compression. Settlements (strains) are caused by an increase in the overburden stress (i.e. increase in load or demand). In many cases, the amount of settlement (strain) determines the capacity (resistance) to a load (demand).

$$\Delta_V = S_t = S_i + S_c + S_s \qquad \text{Equation 17-11}$$

Where,

$S_t = \Delta_V$ = Total Settlement
S_i = Immediate Settlement
S_c = Primary Consolidation Settlement
S_s = Secondary Compression Settlement

Since immediate settlement (S_i) is typically built out during construction, the total settlement should be reported as the amount of primary consolidation (S_c) and secondary compression (S_s) settlement for use in comparing to the Performance Limits established in Chapter 10. However, the effect of the immediate settlement should be determined, (i.e., will the immediate settlement affect the existing embankment or bridge structure, or will the borrow excavation quantities need to be increased). In addition, to determining the total amount of settlement that geomaterials will undergo, the time for the settlement should also be determined. From the combination of total settlement and time for settlement to occur, a rate can be determined and compared to the Performance Limits established in Chapter 10. Another phenomenon that occurs in very soft fine-grained soils is lateral squeeze. Lateral squeeze can cause both vertical as well as lateral displacements. These displacements may induce loadings on structures that have foundations located in the layer susceptible to squeeze.

17.7 CHANGE IN STRESS

As indicated previously, in order for settlement to occur there must be an increase in stress placed on the geomaterials, especially in the case of soil. The increase in stress can be caused by placement of an embankment, shallow or deep foundation or dewatering. The effects of dewatering will not be described in this Manual; however, should dewatering be required, an expert in dewatering should be consulted. There are various methods for determining the change in stress at different depths within the soil profile. The method used in this Manual is the Boussinesq method. Discussed below is the change in stress caused by shallow foundations and by the placement of an embankment. In addition, the increased stress on buried structures caused by the placement of fill is also discussed.

17.7.1 Shallow Foundations

Shallow foundations, as indicated in Chapter 15, are used to support bridges, ERSs and other ancillary transportation facilities. Earthen embankments are theoretically supported by shallow foundations; however, the change in stress caused by embankments is discussed in the following section. According to Chapter 15, a spread footing has a length to width (L_f/B_f) ratio of less than five. Shallow foundations having a length to width ratio greater than five are termed strip or continuous footings. Figure 17-9 depicts the approximate distribution of stresses beneath spread (square) and continuous footings.

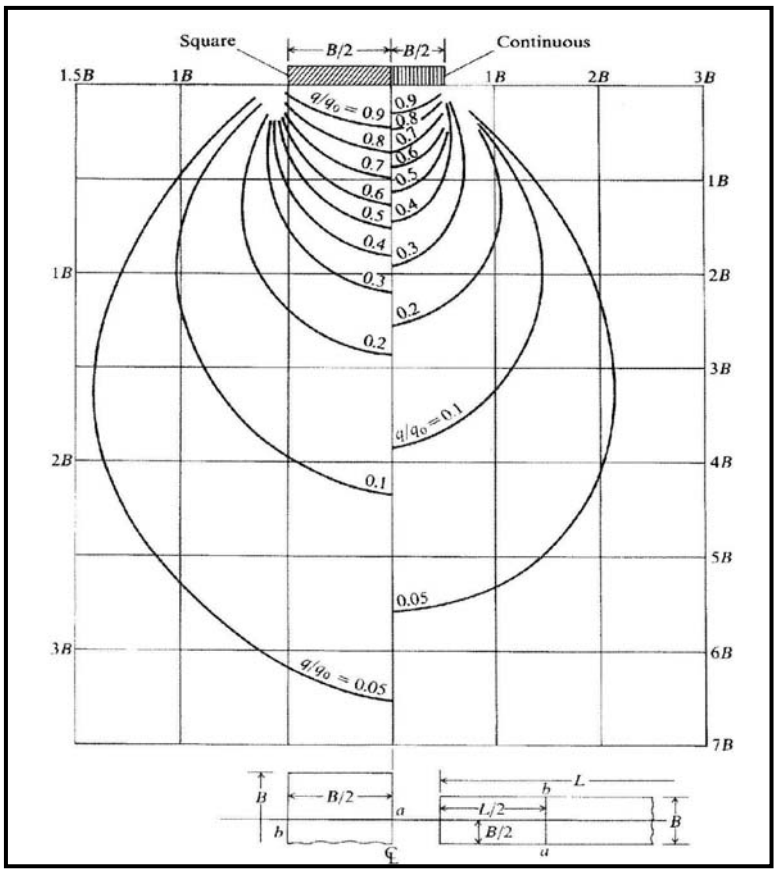


Figure 17-9, Stress Isobars
(Foundation Analysis and Design - 1986)

Where,

B = Foundation width
L = Foundation length
q = Stress at depth indicated
q_o = Applied vertical stress

Figure 17-9 should only be used as an approximate estimate of the increase in stress on a soil layer. To more accurately determine the increase in stress caused by a shallow foundation, the following equation should be used.

$$\Delta\sigma_v = \int_0^r \frac{3q_o}{2\pi Z^2} \frac{1}{\left[1+\left(\frac{r}{Z}\right)^2\right]^{\frac{5}{2}}} 2\pi dr \qquad \text{Equation 17-12}$$

Where,
q_o = Applied vertical stress
Z = Vertical depth below load
r = Horizontal distance between the load application and the point where the vertical pressure is being determined

Newmark (1935) performed the integration of this equation and derived an equation for the increase in vertical stress beneath a corner of a uniformly loaded area.

$$\Delta\sigma_v = Iq_o \qquad \text{Equation 17-13}$$

Where,
I = Influence factor which depends on m and n
m = x/z
n = y/z
x = Width of the loaded area
y = Length of loaded area
z = Depth below the loaded surface to the point of interest
q_o = Applied vertical stress

The influence factor, I, can be obtained from Figure 17-10.

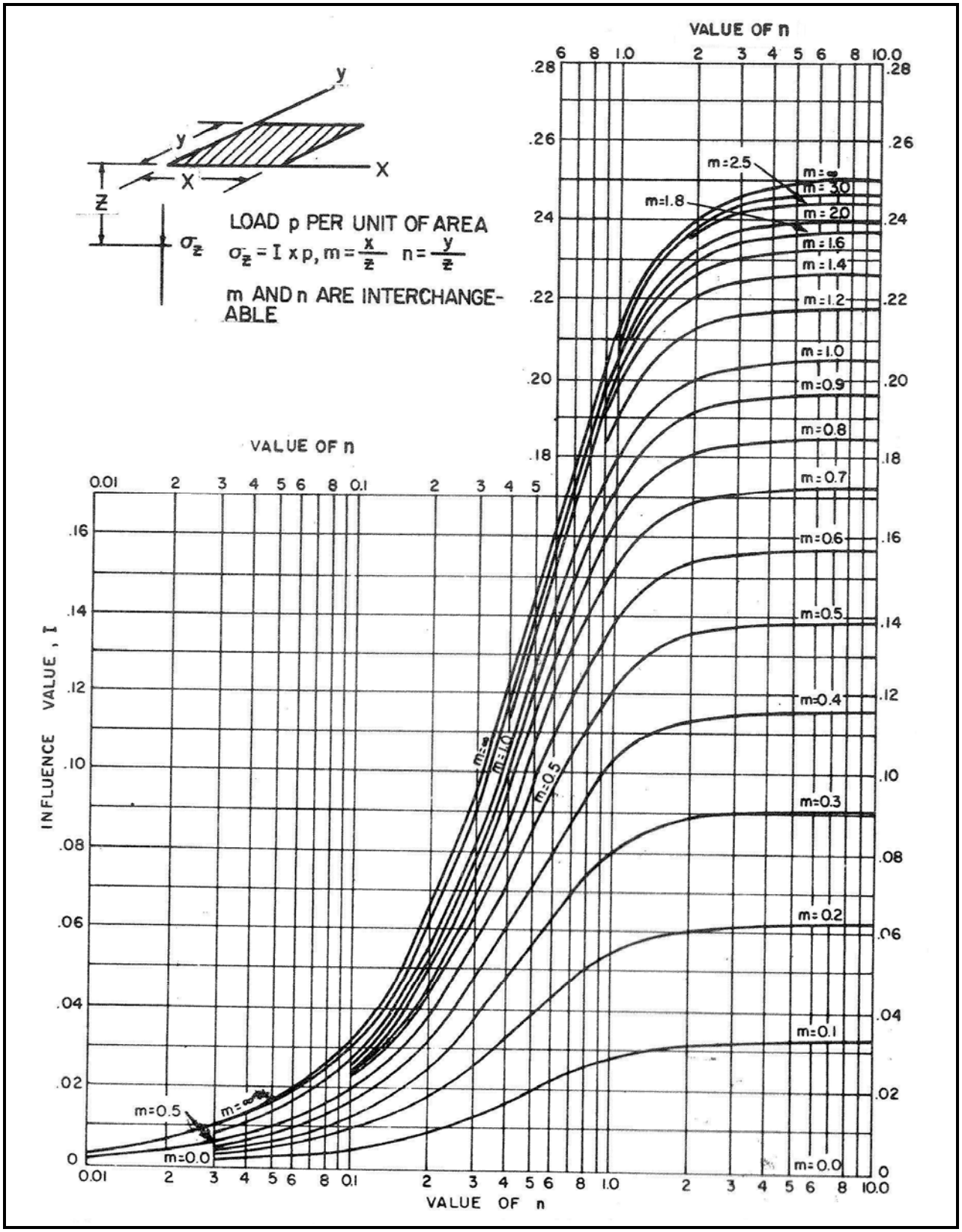


Figure 17-10, Influence Factor Chart
(NAVFAC DM 7.1 - 1982)

The change in stress determined using Equation 17-13 is for a point underneath the corner of a loaded area. Therefore, the change in stress at a depth underneath the center of the footing is

four times the amount determined from Equation 17-13. The change in stress at the mid-point of an edge of a footing is twice the amount determined from Equation 17-13. To find the change in stress at points other than the middle, middle of the edge or a corner of a footing, the Principle of Superposition is used. The Principle of Superposition states that the change in stress at any point is sum of the stresses of the corners over the point (see Figure 17-11).

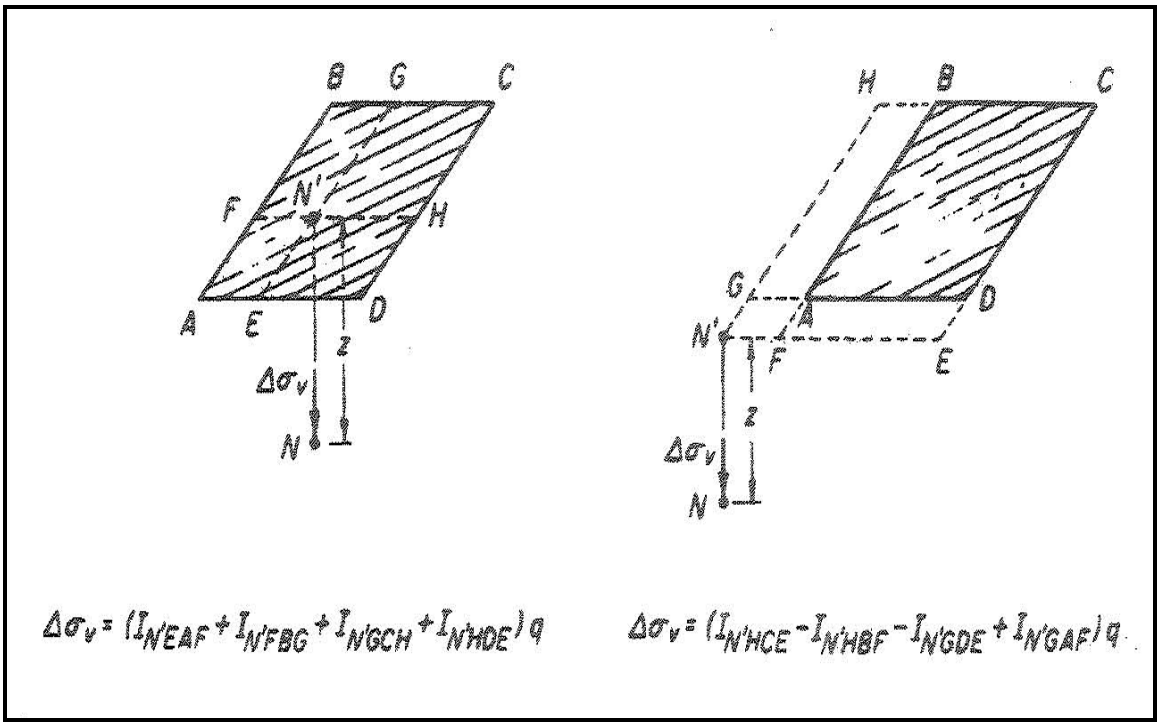


Figure 17-11, Principle of Superposition
(Soil Slope and Embankment Design – September 2005)

17.7.2 Embankments

The change in stress beneath embankments is determined differently than for shallow foundations, because the area loaded by an embankment is larger than for a shallow foundation. Further the change in stress beneath an embankment is complicated by the geometry of the embankment, i.e. the sides slope downward. Embankments comprise two groups; those with infinite length (i.e. side slopes) and those with finite length (i.e. end slopes). The first group is generally longitudinal to the direction of travel, while the other is generally transverse to the direction of travel. For infinite slope embankments, the loading can be represented as a trapezoid. The change in stress beneath an embankment is determined using Equation 17-13. Osterberg (1957) integrated the Boussinesq equations to develop the influence factors (I). Figure 17-12 provides the chart for determining the influence factors (I) for use in Equation 17-13.

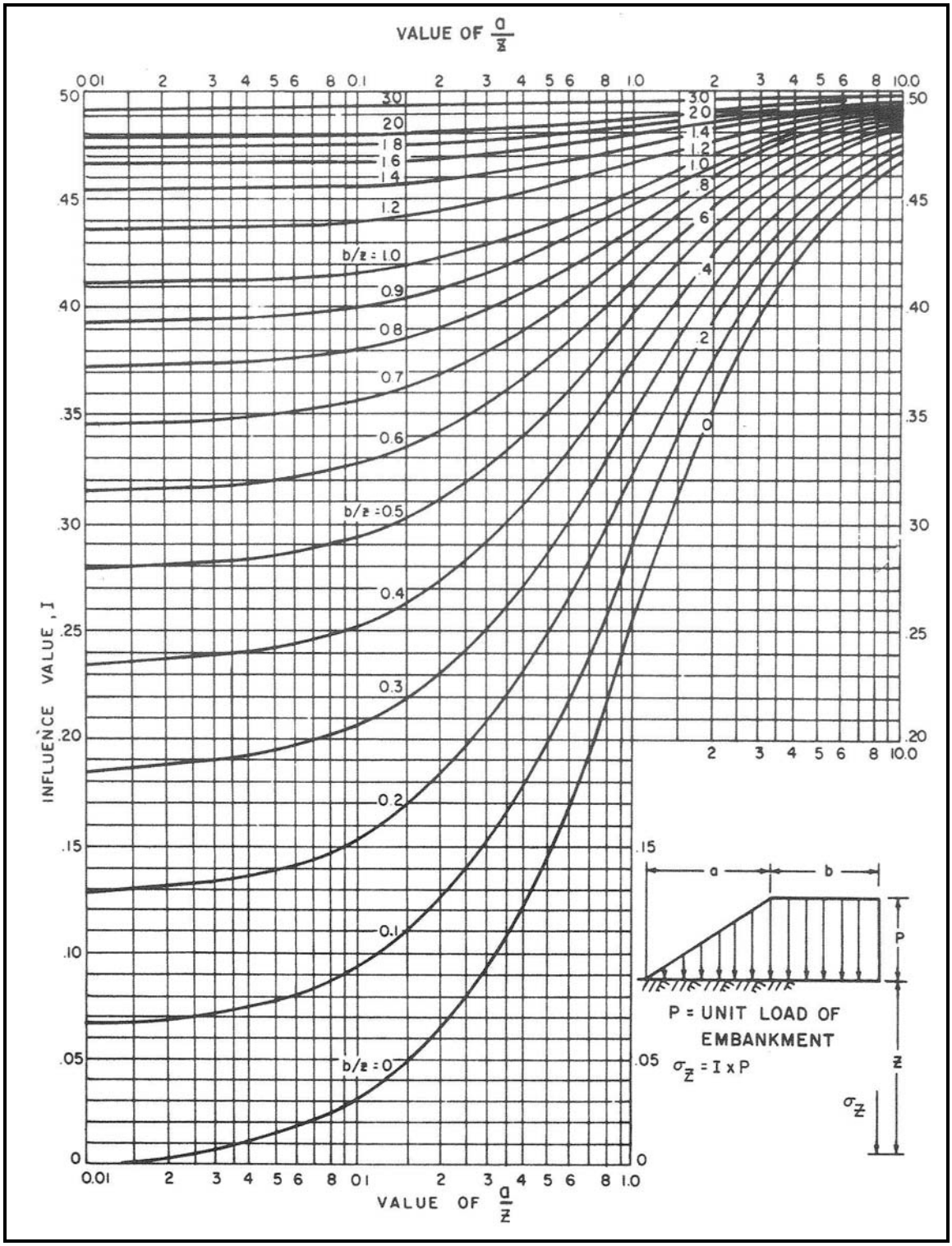


Figure 17-12, Influence Factor Chart – Infinitely Long Embankments (NAVFAC DM 7.1 - 1982)

For finite slopes, Equation 17-13 is used to determine the change in stress. However, the development of the influence factor (I) is complicated by the geometric requirements of the slope. The loading consists of two components; first the area load of the embankment and second the load of the sloping section. The influence factor for the area load is determined using Figure 17-10. Note that the stress is doubled, because the stress is determined at a

corner of the loaded area. The influence factor for the sloping portion is determined from Figures 17-13 and 17-14.

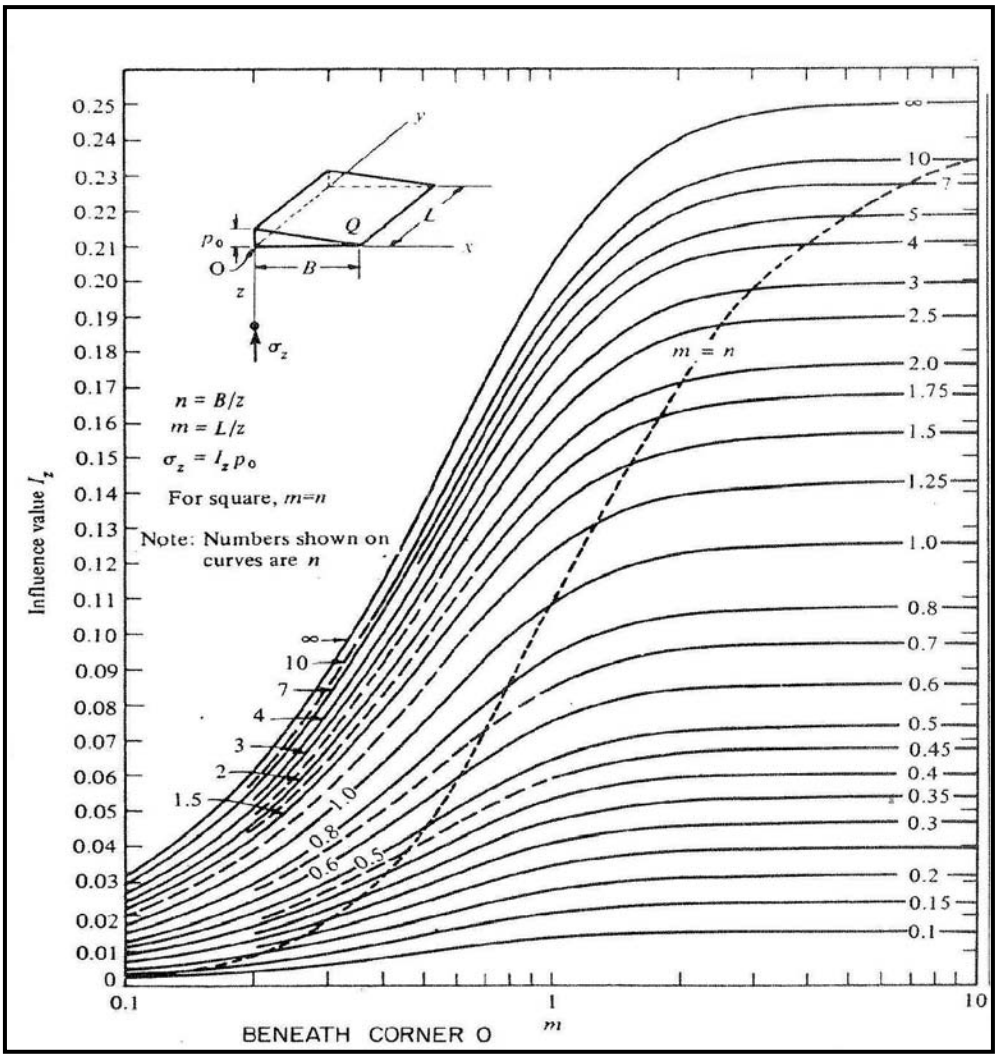


Figure 17-13, Influence Chart Beneath Crest of Slope (NAVFAC DM 7.1 - 1982)

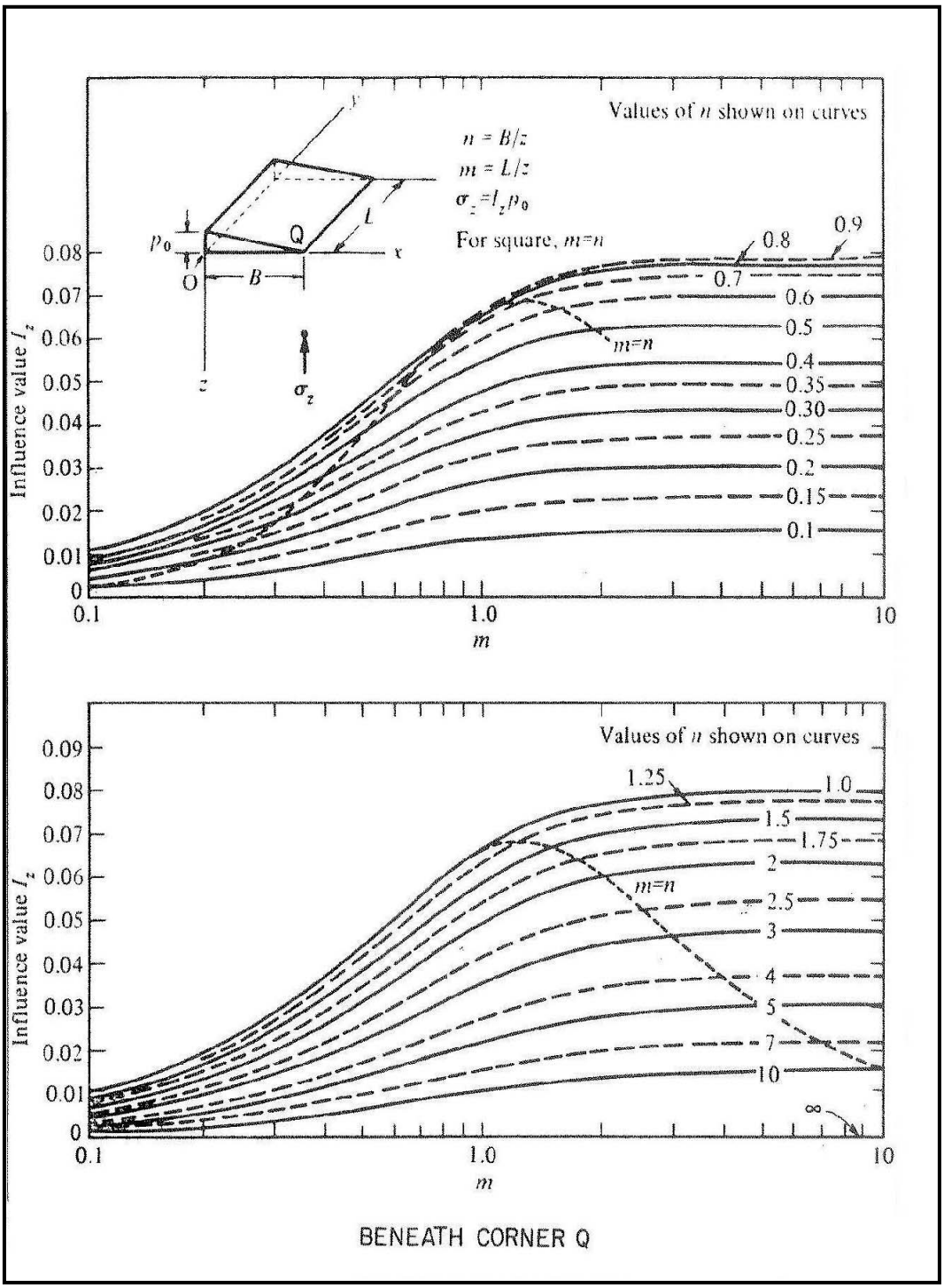


Figure 17-14, Influence Chart Beneath Toe of Slope
(NAVFAC DM 7.1 - 1982)

As alternate to the procedures indicated above for finding the change in stress between a sloped embankment, the procedure described in the *Soils and Foundations Workshop Manual* (July 2000) may be used. This method was originally developed by the New York State Department of Transportation. This method uses the following equation.

$$\rho = \boldsymbol{K}\gamma_t \boldsymbol{h} \qquad \text{Equation 17-14}$$

Where,

ρ = Change in vertical stress caused by the embankment ($\Delta\sigma_v$)

K = Influence factor from Figure 17-15

γ_t = Unit Weight of fill Material

h = Height of embankment (see Figure 17-15)

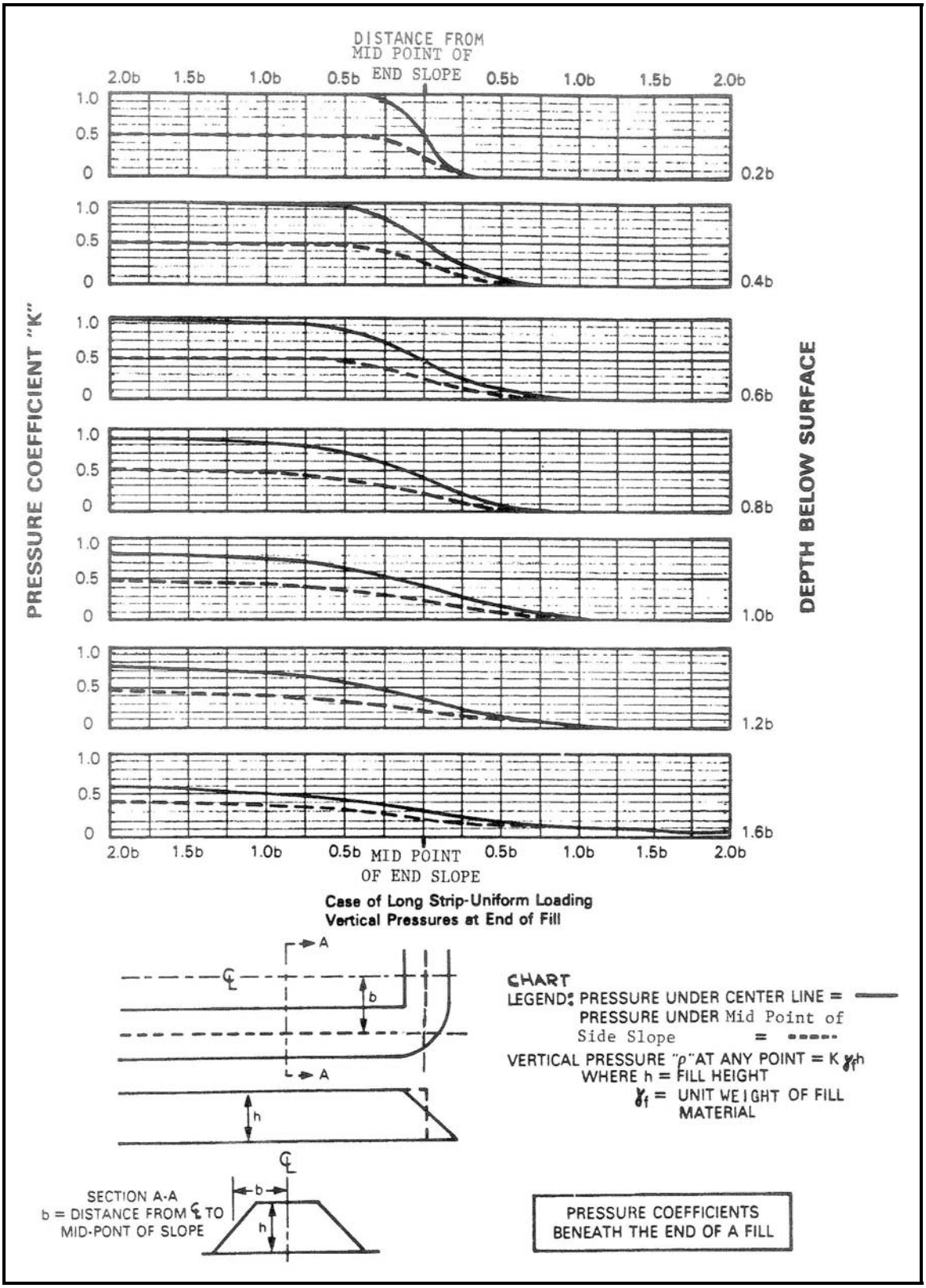


Figure 17-15, Pressure Coefficients Beneath the End of a Fill (Soil and Foundations Workshop Manual – November 1982)

17.7.3 Buried Structures

Buried structures consist of culverts, pipes, boxes, etc. and are required to be designed to handle the loads imposed by embankments. The structural design of these structures is beyond the scope of this Manual; however, the design of buried structures is handled in AASHTO LRFD Bridge Design Specifications (latest edition), Section 12 – Buried Structures and Tunnel Liners. One of the parameters required is the unit weight of the fill material above the buried structure. Therefore, the unit weight (γ_f) may be required to satisfy design requirements along with the other required fill parameters.

17.8 IMMEDIATE SETTLEMENT

Immediate settlement, also termed elastic or instantaneous, occurs upon initial loading of the subgrade soils. This type of settlement occurs in both cohesionless and cohesive soils. The amount of settlement is based on elastic compression of the soils and the time for settlement to occur typically ranges in the days to months (one to three) or typically during construction. The settlement consists of the compression of air filled voids (cohesive soils) and the rearrangement of soil particles (cohesionless soils).

17.8.1 Cohesionless Soils

Cohesionless soils as defined in Chapter 7 consist of sands, gravels, low plasticity silts and residual soils. In cohesionless soils, the immediate (elastic) settlement typically comprises the total amount of settlement anticipated. This Section provides several different methods for determining the immediate settlement of cohesionless soils. All of the methods shall be used and the largest settlement shall be used to compare to the performance limits provided in Chapter 10. Three of the methods are based on the Standard Penetration Test (SPT), one on the Electro-piezocone Test (CPT) and one on the Dilatometer Test (DMT).

17.8.1.1 SPT Methods

As indicated previously, there are three SPT methods for determining immediate settlement of cohesionless soils. The first method is the Hough (1959) method and is used in AASHTO Specifications (Section 10.6 – Spread Footings). The amount of immediate settlement (S_i) is determined using the following equation.

$$S_i = \sum_{i=1}^{n} \left(\frac{1}{C'}\right) H_i \log\left(\frac{\sigma'_{vo} + \Delta\sigma'_v}{\sigma'_{vo}}\right) \qquad \text{Equation 17-15}$$

Where,

C' = Bearing Capacity Index (from Figure 17 -16)

H_i = Thickness of the i^{th} layer

σ'_{vo} = Effective overburden pressure at the mid-point of i^{th} layer

$\Delta\sigma'_v$ = Change in effective vertical stress at the mid-point of i^{th} layer

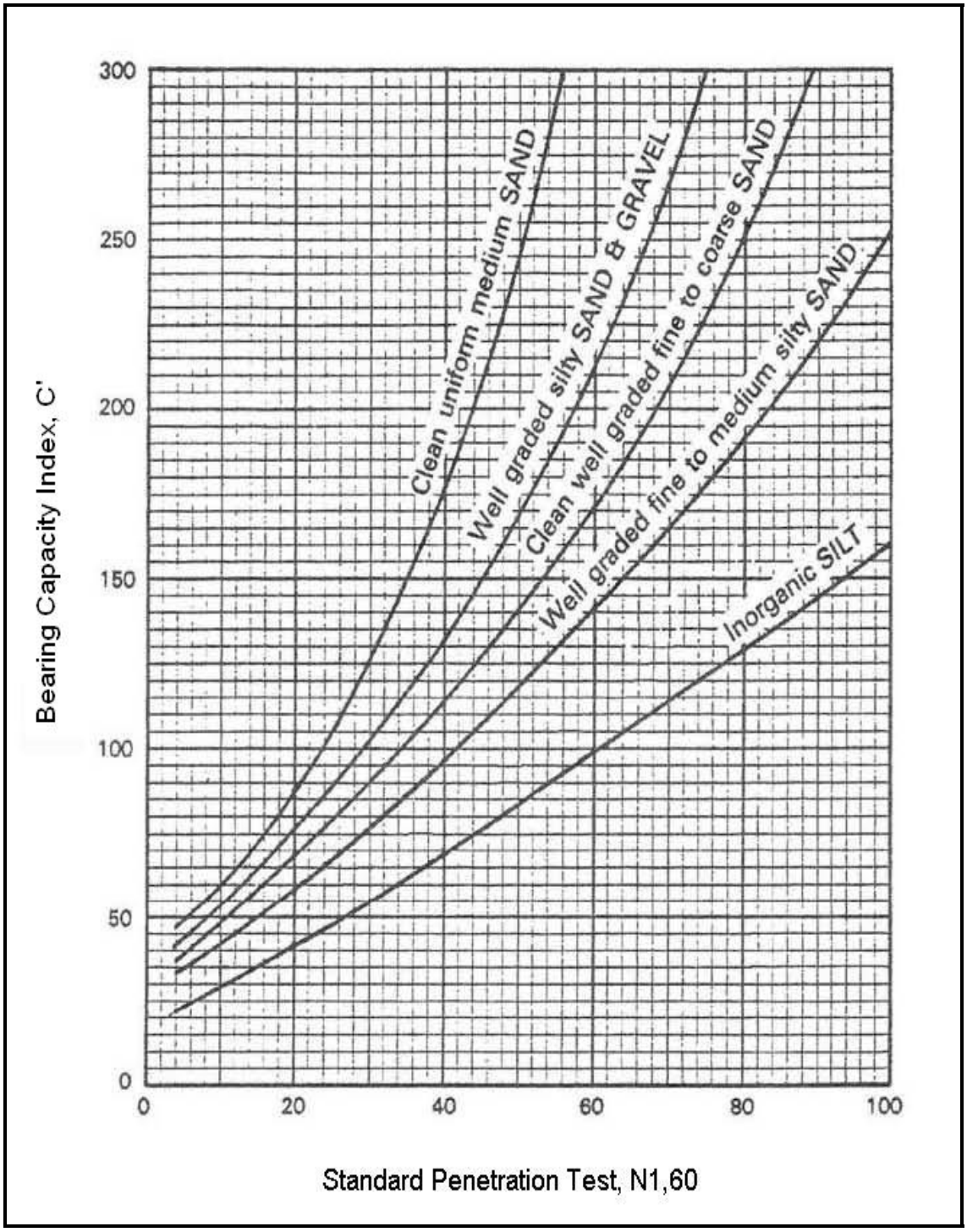


Figure 17-16, Bearing Capacity Index Chart
(Soil Slope and Embankment Design – September 2005,
Modified from Hough, 1959)

The $N_{1,60}$ is determined using the methodology discussed in Chapter 7.

The second SPT method of determining elastic settlement is the Peck and Bazaraa Method (Shallow Foundations (2001)). This method is a modification of the method described by Terzaghi and Peck (1967). It should be noted the equation used to determine the settlement is in SI units. The Peck and Bazaraa Method equation is listed below.

$$S_i = 0.265 C_w C_D \left(\frac{2q_o}{(N_{1,60})_{ave}}\right)\left(\frac{2B_f}{B_f + 0.3}\right)^2 \qquad \text{Equation 17-16}$$

$$C_w = \left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right) \qquad \text{Equation 17-17}$$

$$C_D = 1 - 0.4\left(\sqrt{\frac{\gamma D_f}{q_o}}\right) \qquad \text{Equation 17-18}$$

Where,

S_i = Immediate settlement in millimeters (mm) [1 mm = 0.03937 in]
C_w = Water table correction factor, at a depth of one-half of B_f
C_D = Embedment correction factor, use 1.0 when filling above original grade
B_f = Footing width in meters (m) [1 m = 3.28084 ft]
D_f = Depth of footing base embedment from ground surface in meters
γ = Total unit weight of soil in kiloNewtons per cubic meter (kN/m^3)
[1 kN/m^3 = 6.3656 lbs/ft^3)
q_o = Applied vertical stress or bearing pressure in kiloPascals (kPa) [1 kPa = 0.0209 ksf]
σ_{vo} = Total overburden pressure
σ'_{vo} = Effective overburden pressure
$(N_{1,60})_{ave}$ = Average $N_{1,60}$-value from base of footing to a depth of B_f below footing

The third SPT method was developed by Duncan and Buchignani (1976) based on Meyerhof (1965). The immediate settlement equation is provided below.

$$S_i = \frac{5q_o}{(N_{60} - 1.5)C_B} \qquad \text{Equation 17-19}$$

Where,

q_o = Applied vertical stress in tsf
N_{60} = Standard Penetration value corrected only for energy (see Chapter 7)
C_B = Width Correction (see Table 17-1)

The N_{60}-value is an average value from the base of the footing to a depth of B_f.

Table 17-1, Width Correction Factor, C_B
(Duncan and Buchignani, 1976)

Footing Width, B (feet)	C_B
≤ 4	1.00
6	0.95
8	0.90
10	0.85
≥ 12	0.80

Duncan and Buchignani (1976) indicate that immediate (elastic) settlement will continue to increase over time (i.e. creep). The total amount of elastic settlement over time is determined using the following equation.

$$S_{iet} = S_i C_t \qquad \text{Equation 17-20}$$

Where

S_{iet} = Elastic settlement after a period of time
S_i = Immediate or elastic settlement
C_t = Time rate factor (see Table 17-2)

Table 17-2, Time Rate Factors
(Duncan and Buchignani, 1976)

Time	C_t
1 month	1.0
4 months	1.1
1 year	1.2
3 years	1.3
10 years	1.4
30 years	1.5

For times other than those depicted in Table 17-2, the following equation may be used.

$$C_t = 0.0858 Ln(t) + 0.9907 \qquad \text{Equation 17-21}$$

Where,

C_t = Time rate factor
t = Time period over which settlement occurs in months

17.8.1.2 CPT Method

There is one CPT method available for determining the immediate settlement of cohesionless soils. It was developed by Schmertmann (1970) and is applicable only to shallow foundations (i.e., a rigid structure is required). The Schmertmann Method uses the following equations.

$$S_i = C_D C_t q_{net} \sum_{i=1}^{n} \left(\frac{H_i}{E_{si}} \right) I_{azi} \qquad \text{Equation 17-22}$$

$$C_D = 1 - 0.5\left(\frac{\sigma'_D}{q_{net}}\right) \geq 0.5 \qquad \text{Equation 17-23}$$

$$q_{net} = q_o - \sigma'_D \qquad \text{Equation 17-24}$$

$$C_t = 1 + 0.2\log\frac{t}{0.1} \qquad \text{Equation 17-25}$$

$$I_P = 0.5 + 0.1\sqrt{\frac{q_{net}}{\sigma'_{Ip}}} \qquad \text{Equation 17-26}$$

Where,

C_D = Depth correction factor
C_t = Creep correction factor (t > 0.1 years)
q_{net} = Net total average bearing pressure in kPa
H_i = Thickness of the i^{th} layer in meters
E_{si} = Modulus of Elasticity of the i^{th} layer in kPa
I_{azi} = Average vertical strain influence factor of the i^{th} layer (from Figure 17-17)
q_o = Applied bearing pressure in kPa
σ'_D = Vertical effective stress at the base of the footing in kPa
t = time in years
σ'_{Ip} = Vertical effective stress at the depth of the peak influence factor (I_P) in kPa

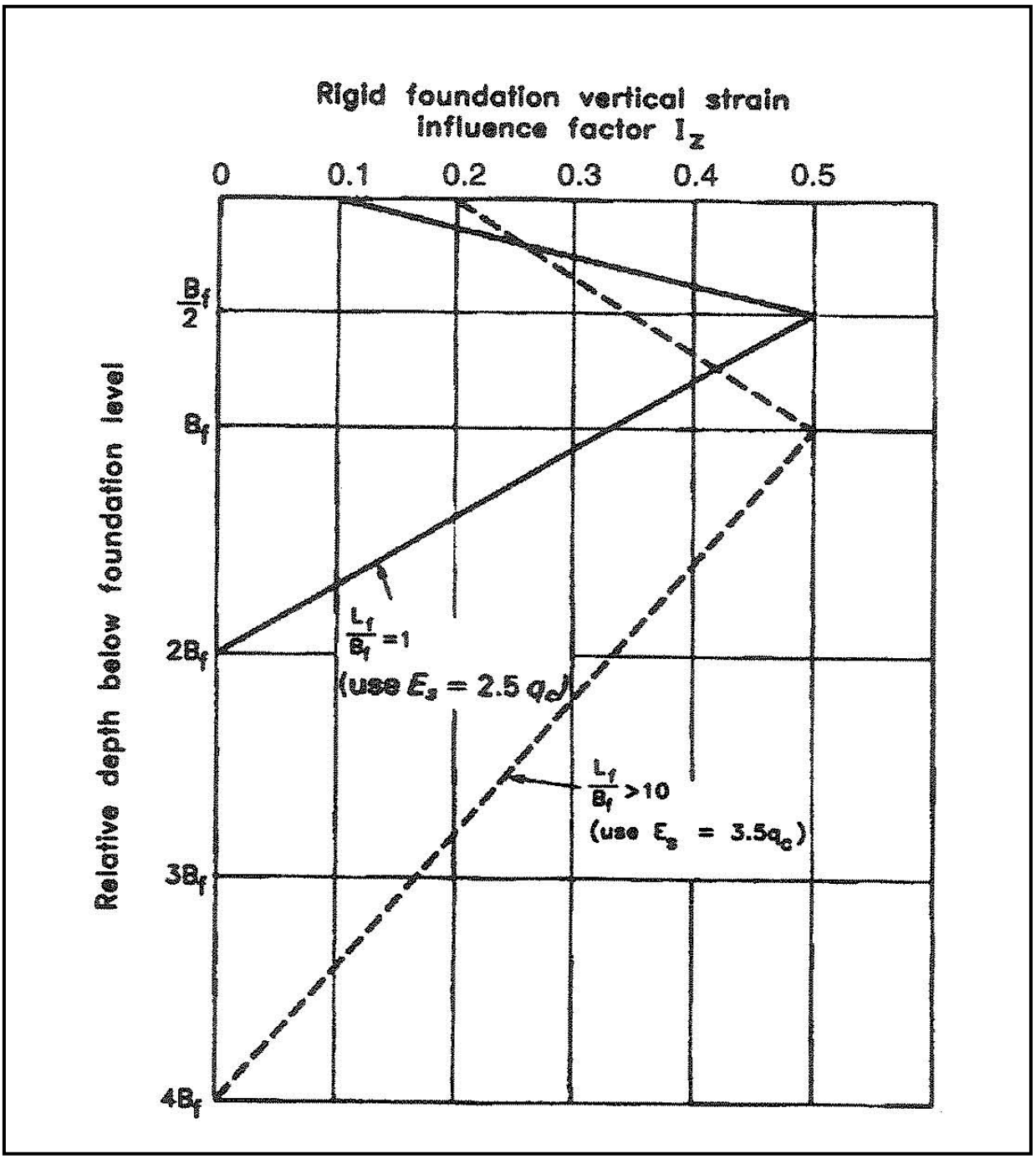


Figure 17-17, Vertical Strain Influence Factor Chart (Shallow Foundations – June 2001)

The modulus of elasticity (E_s) can be developed directly from laboratory testing, from Chapter 7 or from the following equation. This equation applies only to determining the E_s from CPT data.

$$E_s = F_s q_c$$ Equation 17-27

Where,

F_s = Correlation factor depending on cone and soil type, stress level and footing shape (see Table 17-3)

q_c = Cone penetration tip resistance in kPa

Table 17-3, Correlation Factor, F_s
(Shallow Foundations – June 2001)

Case	F_s
$L_f/B_f = 1$	2.5
$L_f/B_f > 10$	3.5

Where,

B_f = Footing width

L_f = Footing length

As an alternate to Figure 17-17, the vertical strain influence factor may be determined using the equations in Table 17-4.

Table 17-4, Vertical Strain Influence Factor Equations
(Shallow Foundations – June 2001)

Footing Shape	I Terms	H_i	I_{azi} Equations
$L_f/B_f < 1$ (square)	I_{zsq}	0 to $B_f/2$	$0.1+\left(\frac{H_i}{B_f}\right)(2I_P - 0.2)$
		$B_f/2$ to $2B_f$	$0.667I_P\left(2-\frac{H_i}{B_f}\right)$
$L_f/B_f > 10$ (continuous)	I_{zc}	0 to B_f	$0.2+\left(\frac{H_i}{B_f}\right)(I_P - 0.2)$
		B_f to $4B_f$	$0.333I_P\left(4-\frac{H_i}{B_f}\right)$
$1 < L_f/B_f < 10$ (rectangular)	I_{zr}	N/A	$I_{zsq}+0.111(I_{zc}-I_{zsq})\left(\left(\frac{L_f}{B_f}\right)-1\right)$

17.8.1.3 DMT Method

Immediate settlement can be determined from Dilatometer Test data. The method is described in detail in *The Flat Dilatometer Test* published by the Federal Highway Administration. The equation for determining immediate settlement is provided below.

$$S_i = \sum_{i=1}^{n} \left(\frac{\Delta\sigma_v}{M_i} \right) H_i \qquad \text{Equation 17-28}$$

Where,

$\Delta\sigma_v$ = Change in vertical stress at the mid-point of i^{th} layer
M_i = Averaged constrained modulus of the i^{th} layer
H_i = Thickness of the i^{th} layer

17.8.2 Cohesive Soils

There is some immediate settlement in cohesive (clays and plastic silts) soils with most settlement occurring within a relatively short period after loading is applied. Most of this immediate settlement is from distortion and compression of air filled voids. It is anticipated that very little immediate settlement would occur in saturated cohesive soils. However, for unsaturated and/or highly overconsolidated (OCR $\geq$ 4) cohesive soils immediate settlement can be a predominant portion of the total settlement (S_t). These immediate settlements can be determined using the Theory of Elasticity and are determined using the following equation.

$$S_i = \frac{\left[q_o \left(1 - \nu^2\right) \sqrt{A} \right]}{\frac{E_s}{\beta_z}} \qquad \text{Equation 17-29}$$

Where,

q_o = Applied bearing pressure at the bottom of loaded area
ν = Poisson's ratio (see Chapter 7)
A = Contact area of the load
E_s = Modulus of elasticity for the soil (see Chapter 7)
β_z = Footing shape and rigidity factor (see Table 17-5)

Table 17-5, Footing Shape and Rigidity Factors (Shallow Foundations – June 2001)

L_f/B_f	β_z Flexible	β_z Rigid
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

Where B_f and L_f are the same as in the CPT Method described previously. If L_f/B_f is between five and ten, use linear interpretation. For L_f/B_f greater than ten, use the β_z for ten. The elastic parameters Poisson's ratio (ν) and modulus of elasticity (E_s) are provided in Chapter 7 for cohesive soils.

Christian and Carter (1978) developed an improved Janbu approximation for determining immediate settlement in cohesive soils. The improved Janbu approximation is provided below and assumes that the Poisson's ratio (ν) of the soil is 0.5.

$$S_i = \mu_0 \mu_1 \left(\frac{q_o B_f}{E_s} \right) \qquad \text{Equation 17-30}$$

Where,

μ_0 = Influence factor for depth (see Figure 17-18)
μ_1 = Influence factor for foundation shape (see Figure 17-18)
q_0 = Applied vertical stress at the bottom of loaded area
B_f = Foundation width
L_f = Foundation length
D = Foundation depth below ground surface
H = Distance between bottom of foundation and a firm (non-compressible) layer
E_s = Modulus of elasticity for the soil (see Chapter 7)

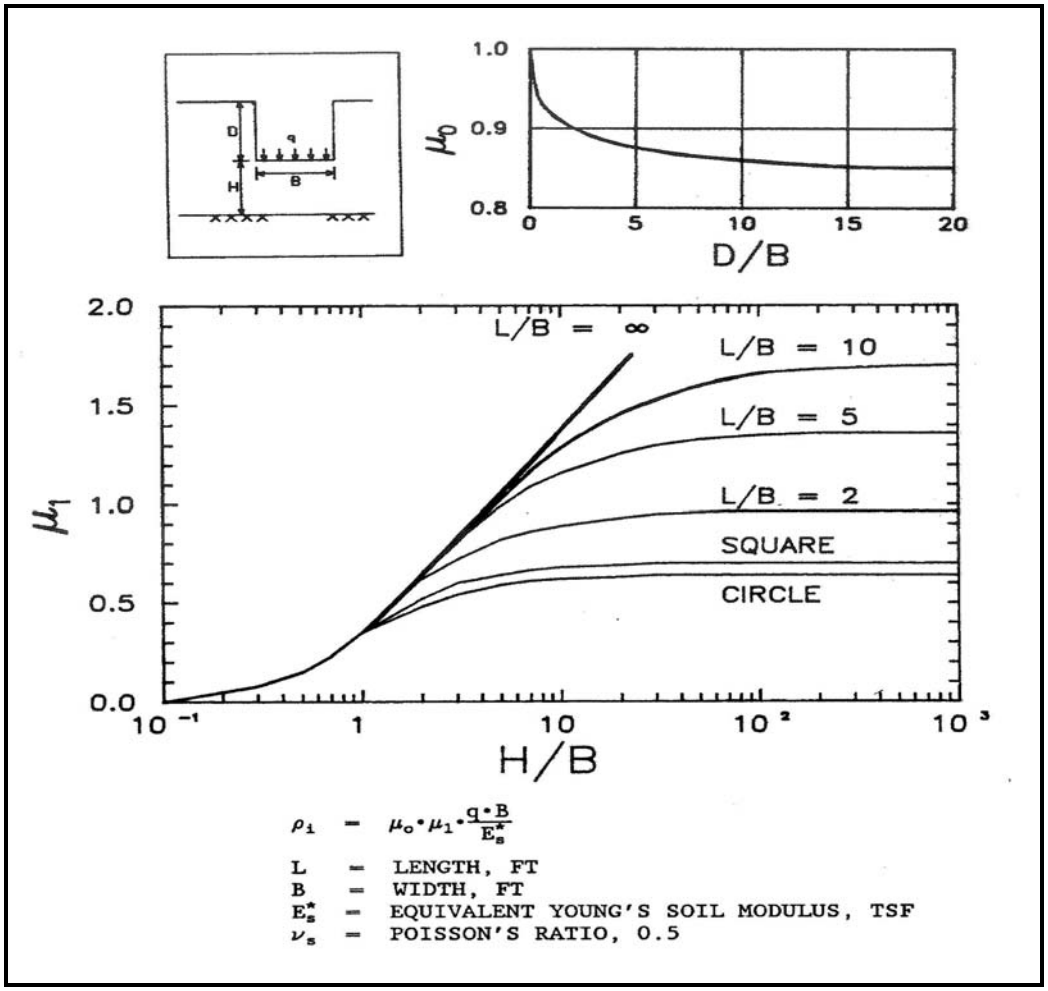


Figure 17-18, Janbu Influence Factor Chart (Settlement Analysis - 1990)

17.9 PRIMARY CONSOLIDATION SETTLEMENT

Primary consolidation settlement (S_c) occurs when the increase in load on a soil results in the generation of excess pore pressures within the soil voids. Depending on the type of soil, the time to reduce the excess pore pressures to some steady state level may be rapid (cohesionless soils) or may require long periods of time (cohesive soils). Therefore, primary consolidation settlement is comprised of two components, the amount of settlement and the time for settlement to occur. The amount of time required for cohesionless soil to settle is relatively short, typically occurring during construction, and the amount of settlement can be determined using immediate or elastic settlement theory. Therefore, the remainder of this Section will exclusively look at cohesive (clay and plastic silt) soils. Typically, normally consolidated (OCR = 1) soils undergo primary consolidation settlement. For the purpose of this Chapter, all plastic cohesive soils that have an OCR of less than four shall have the primary consolidation settlement determined.

The determination of primary consolidation settlement is based on six steps as presented in Table 17-6.

Table 17-6, Primary Consolidation Settlement Steps
(Modified from Shallow Foundations – June 2001)

Step	Item
1	Computation of the initial vertical effective stress (σ'_{vo}) [total vertical stress (σ_{vo}) and pore water pressure (u_o)] of the layer(s) midpoint
2	Determination of preconsolidation stresses (σ'_p or p'_c)
3	Computation of changes in vertical effective stress ($\Delta\sigma'_v$) [associated with changes in both total stress ($\Delta\sigma_v$) and pore water pressures (Δu)] due to the construction
4	Determination of compressibility of the clay or plastic silt
5	Computation of layer compressions (S_c)
6	Computation of time for compressions

17.9.1 Amount of Settlement

In cohesive soils, loads are carried first by the pore water located in the interstitial space between the soil grains and then by the soil grains. The pore water pressure increases proportional to the load applied at that depth. As the excess pore water pressures reduce through drainage, the load is transferred to the soil grains. This drainage causes the settlement of cohesive soils. Therefore, the settlement is directly proportional to the volume of water drained from the soil layer. Typically, the area loaded is large, resulting in the flow of water vertically (either up or down) and not horizontally. Therefore, one-dimensional consolidation theory may be use to determine settlements of cohesive soils.

The one-dimensional consolidation test is used to determine the parameters for use in one-dimensional consolidation theory. These parameters are indicated in Table 17-7.

Table 17-7, Consolidation Parameters and Symbols

Symbol	Parameter
C_c or $C_{\varepsilon c}$	Compression Index
C_r or $C_{\varepsilon r}$	Recompression Index
C_α or $C_{\varepsilon\alpha}$	Secondary Compression Index
σ'_p or p'_c	Effective Preconsolidation Stress
c_v	Coefficient of Consolidation
m_v	Coefficient of Vertical Compression

The Coefficient of Consolidation and the Coefficient of Vertical Compression are required when determining the rate of settlement and will be discussed in the next Section. The effective preconsolidation stress is extremely important because it is used to determine if a soil is normally consolidated (NC) or overconsolidated (OC). In normally consolidated soils, the effective preconsolidation stress is equal to the existing effective overburden stress (i.e. $\sigma'_{vo} = \sigma'_p$) (see Figure 17-19). Normally consolidated soils tend to have large settlements. Overconsolidated soils tend to have an effective preconsolidation stress greater than the existing effective overburden stress (i.e. $\sigma'_{vo} < \sigma'_p$) (see Figure 17-20). Overconsolidated soils do not tend to have large settlements. One-dimensional consolidation tests shall be performed in accordance with Chapter 5. In some locations within South Carolina, under consolidated soils (i.e. $\sigma'_{vo} > \sigma'_p$) (see Figure 17-21) are known to exist. These soils are still consolidating under the weight of the soil and should be anticipated to have very large amounts of settlement.

Casagrande (1936) developed a graphical procedure for determining the preconsolidation stress. The Casagrande procedure for determining preconsolidation stress is outlined in Table 17-8. While the Casagrande procedure was applicable to both e-log p and ε-log p curves, SCDOT prefers the use of the ε-log p curve for data presentation.

Table 17-8, Determination of Preconsolidation Stress (Duncan and Buchignani – 1976)

Step	Description
1	Locate the point of sharpest curvature on the e-log p or ε-log p curve
2	From this point (a) (see Figures 17-22 or 17-23), draw a horizontal line (b) and a tangent (b) to the curve
3	Bisect the angle formed by these two lines (c)
4	Extend the virgin curve (d) backward to intersect the bisector (c)
5	The point where these lines (d and c) cross determines the preconsolidation pressure (σ'_p or p'_c)

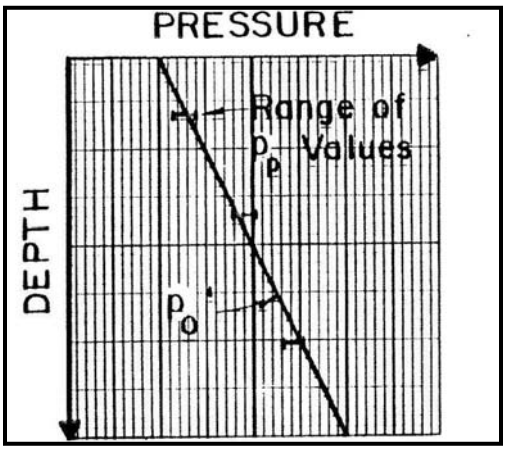


Figure 17-19, Normally Consolidated
(Duncan and Buchignani – 1976)

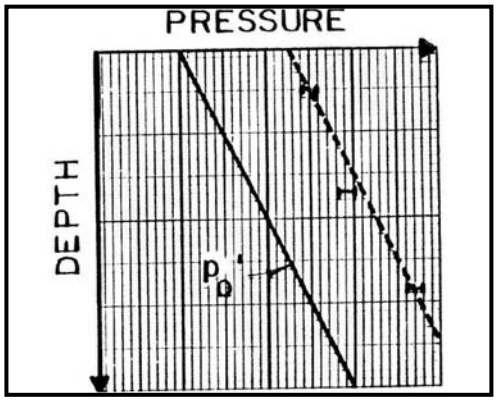


Figure 17-20, Overconsolidated
(Duncan and Buchignani – 1976)

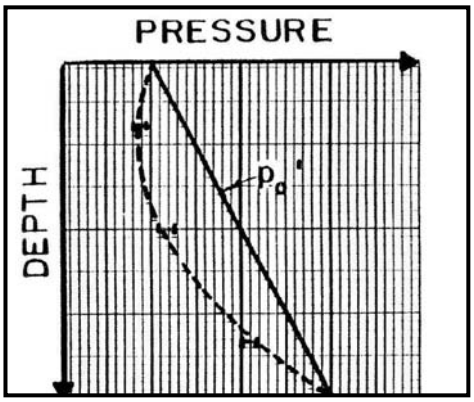


Figure 17-21, Under Consolidated
(Duncan and Buchignani – 1976)

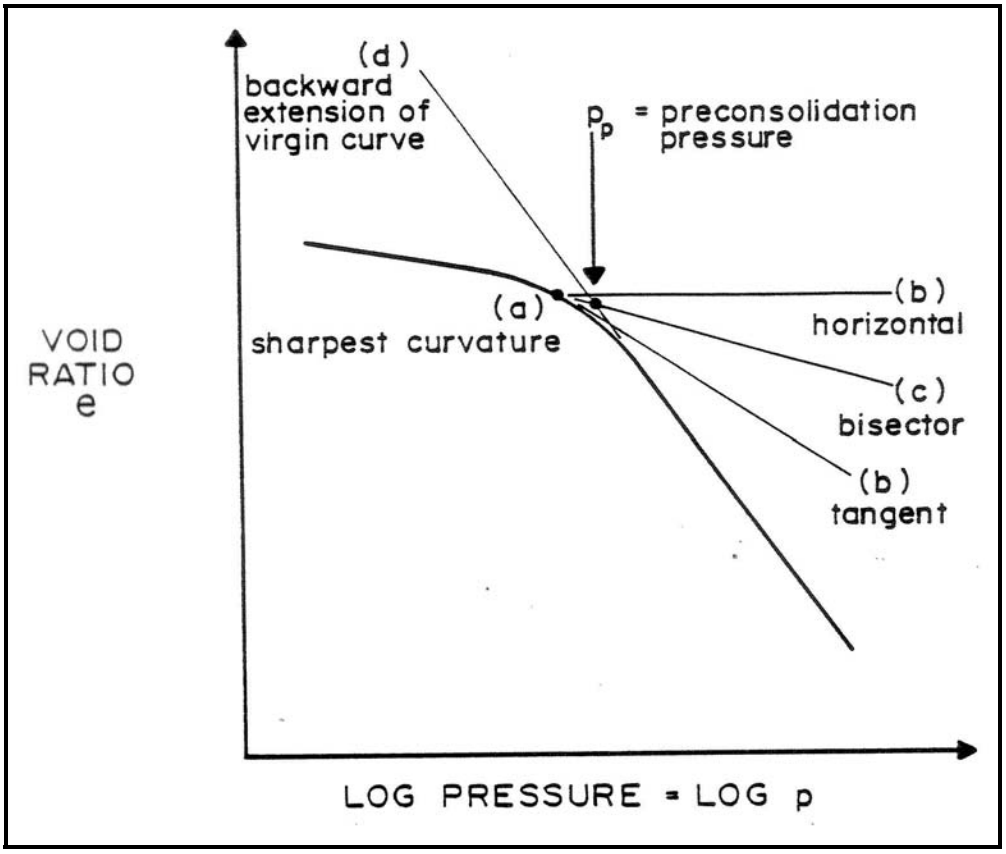


Figure 17-22, Determination of Preconsolidation Stress from e-log p (Duncan and Buchignani – 1976)

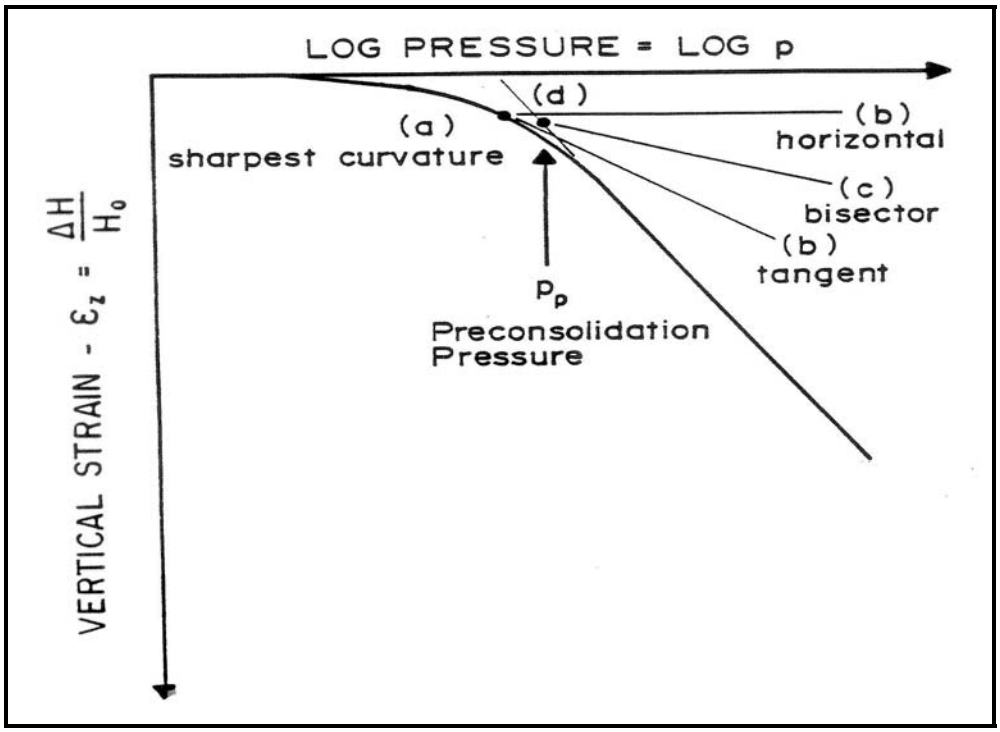


Figure 17-23, Determination of Preconsolidation Stress from ε-log p (Duncan and Buchignani – 1976)

One-dimensional consolidation tests are sensitive to sample disturbance; therefore, the results of the test must be corrected. This method is applied to test results presented as e-log p and ε-log p curves. Duncan and Buchignani (1976) provide methods for correcting both e-log p and ε-log p for both normally consolidated and overconsolidated soils. The procedures for correcting the e-log p curves (normally consolidated and overconsolidated) are presented in Table 17-9.

Table 17-9, Correction of the e-log p Curve for Disturbance (modified from Duncan and Buchignani – 1976)

Step	Description
Normally Consolidated Soil ($\sigma'_{vo} = \sigma'_p$) (Figure 17-24)	
1	Locate point A at the intersection of e_o and σ'_p (P_p)
2	Locate point B at on the virgin curve or extension where e = 0.4e_o
3	Connect points A and B with a straight line – this is the corrected virgin curve
Overconsolidated Soil ($\sigma'_{vo} < \sigma'_p$) (Figure 17-25)	
1	Locate point A at the intersection of e_o and σ'_{vo} (P_o')
2	Draw a line from point A parallel to the rebound curve and locate point B where this line intersects σ'_p (P_p)
3	Locate point C at on the virgin curve or extension where e = 0.4e_o
4	Connect points B and C with a straight line – this is the corrected virgin curve

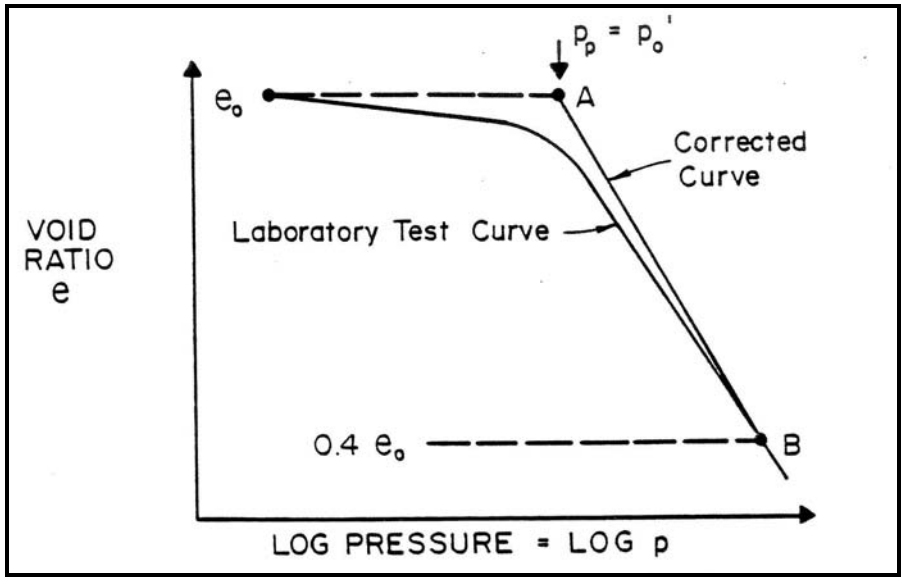


Figure 17-24, Corrected e-log p Normally Consolidated Curve (Duncan and Buchignani – 1976)

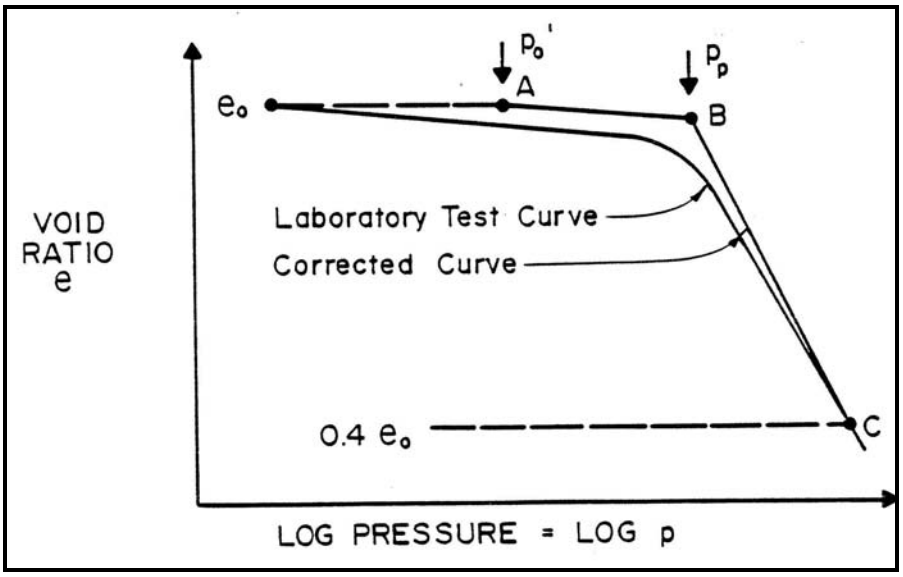


Figure 17-25, Corrected e-log p Overconsolidated Curve (Duncan and Buchignani – 1976)

The Duncan and Buchignani (1976) procedures for correcting the ε-log p curves (normally consolidated and overconsolidated) are presented in Table 17-10.

Table 17-10, Correction of the ε-log p Curve for Disturbance (modified from Duncan and Buchignani – 1976)

Step	Description
Normally Consolidated Soil ($\sigma'_{vo} = \sigma'_p$) (Figure 17-26)	
1	Locate point A at the intersection of $\varepsilon = 0$ and σ'_p (P_p)
2	Locate point B at on the virgin curve or extension where $\varepsilon = 0.4$
3	Contact points A and B with a straight line – this is the corrected virgin curve
Overconsolidated Soil ($\sigma'_{vo} < \sigma'_p$) (Figure 17-27)	
1	Locate point A at the intersection of $\varepsilon = 0$ and σ'_{vo} (P_o')
2	Draw a line from point A parallel to the rebound curve and locate point B where this line intersects σ'_p (P_p)
3	Locate point C at on the virgin curve or extension where $\varepsilon = 0.4$
4	Contact points B and C with a straight line – this is the corrected virgin curve

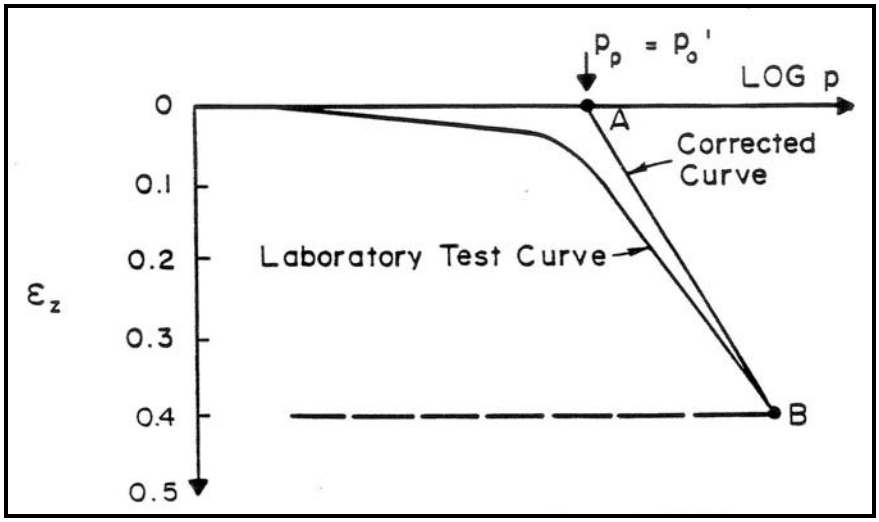


Figure 17-26, Corrected ε-log p Normally Consolidated Curve (Duncan and Buchignani – 1976)

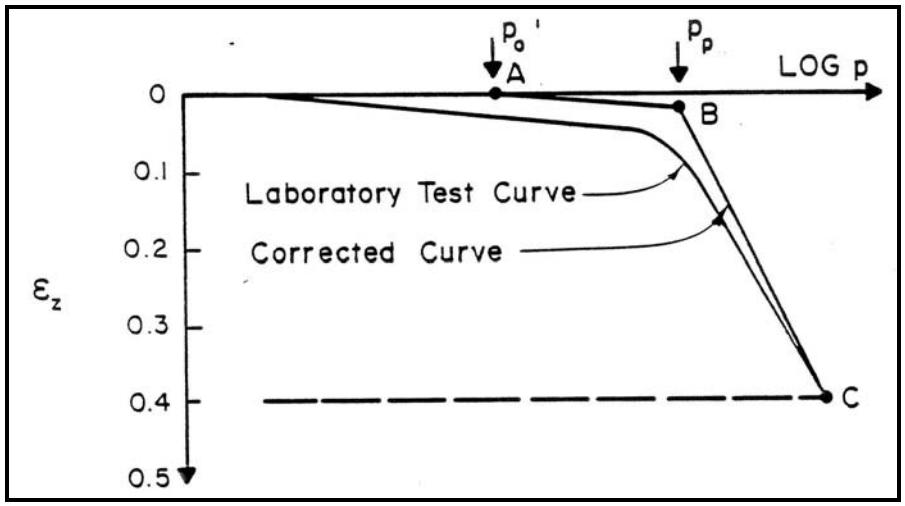


Figure 17-27, Corrected ε-log p Overconsolidated Curve (Duncan and Buchignani – 1976)

The compression (C_c or $C_{\varepsilon c}$) and recompression (C_r or $C_{\varepsilon r}$) indices are determined from the corrected curves. The compression (C_c or $C_{\varepsilon c}$) index is the slope of the virgin portion of the corrected curve, either e-log p (C_c) or ε-log p ($C_{\varepsilon c}$), over a full cycle. The recompression index is the slope of the recompression portion of the corrected curve, either e-log p (C_r) or ε-log p ($C_{\varepsilon r}$) over a full logarithmic cycle. If the slope of either portion of the curve does not extend over a full logarithmic cycle extend the line in both directions to cover a full logarithmic cycle.

The determination of the amount of settlement is dependent on whether the soil is normally consolidated, overconsolidated or a combination of both. The amount of settlement for under consolidated soils is determined the same as normally consolidated soil. In addition, the way the data is presented, either e-log p or ε-log p curves, will also determine which equation is used. Presented in Table 17-11 are the equations for determining the total primary consolidation settlement (S_c).

Table 17-11, Primary Consolidation Settlement Equations

e-log p		
$\sigma'_{vo} = \sigma'_p$	$S_c = \sum_1^i H_o \frac{C_c}{1+e_o}(\log\frac{\sigma'_f}{\sigma'_{vo}})$	Equation 17-31
$\sigma'_f < \sigma'_p$	$S_c = \sum_1^i H_o \frac{C_r}{1+e_o}(\log\frac{\sigma'_f}{\sigma'_{vo}})$	Equation 17-32
$\sigma'_{vo} < \sigma'_p < \sigma'_f$	$S_c = \sum_1^i H_o[\frac{C_c}{1+e_o}(\log\frac{\sigma'_f}{\sigma'_p}) + \frac{C_r}{1+e_o}(\log\frac{\sigma'_p}{\sigma'_{vo}})]$	Equation 17-33
ε-log p		
$\sigma'_{vo} = \sigma'_p$	$S_c = \sum_1^i H_o C_{\varepsilon c}(\log\frac{\sigma'_f}{\sigma'_{vo}})$	Equation 17-34
$\sigma'_f < \sigma'_p$	$S_c = \sum_1^i H_o C_{\varepsilon r}(\log\frac{\sigma'_f}{\sigma'_{vo}})$	Equation 17-35
$\sigma'_{vo} < \sigma'_p < \sigma'_f$	$S_c = \sum_1^i H_o[C_{\varepsilon c}(\log\frac{\sigma'_f}{\sigma'_p}) + C_{\varepsilon r}(\log\frac{\sigma'_p}{\sigma'_{vo}})]$	Equation 17-36

Where,

H_o = Thickness of i^{th} layer

e_o = Initial void ratio of i^{th} layer

σ'_f = Final pressure on the i^{th} layer

$$\sigma'_f = \sigma'_{vo} + \Delta\sigma_v \qquad \text{Equation 17-37}$$

Where,

σ'_{vo} = initial vertical effective stress on the i^{th} layer

$\Delta\sigma_v$ = change in stress on the i^{th} layer

17.9.2 Time for Settlement

As indicated previously, consolidation settlement occurs when a load is applied to a saturated cohesive soil squeezing water out from between the soil grains. The length of time for primary consolidation settlement to occur is a function of compressibility and permeability of the soil. The Coefficient of Consolidation (c_v) is related to the permeability (k) and the Coefficient of Vertical Compression (m_v) as indicated in the following equations

$$c_v = \frac{1}{\gamma_w}\frac{k}{m_v} \qquad \text{Equation 17-38}$$

$$m_v = \frac{\Delta \varepsilon_v}{\Delta \sigma'_v} \quad \text{Equation 17-39}$$

$$m_v = \frac{\Delta e}{\Delta \sigma'_v \left(1 + e_{av}\right)} \quad \text{Equation 17-40}$$

Where,

γ_w = Unit weight of water

$\Delta \varepsilon_v$ = Change in sample height

$\Delta \sigma'$ = Change in effective stress

Δe = Change in void ratio

e_{av} = Average void ratio during consolidation

The Coefficient of Consolidation is typically provided as part of the results of consolidation testing. A c_v is provided for each load increment applied during the test. The c_v used to determine the time for primary consolidation settlement should be the one at the stress (load increment) closest to the anticipated field conditions. If c_v is not provided, it should be determined using the procedures outlined in Shallow Foundations, FHWA NHI-01-023.

The time (t) for primary consolidation settlement is determined using the equation listed below.

$$t = \frac{TH_o^2}{c_v} \quad \text{Equation 17-41}$$

Where,

t = time for settlement

T = Time Factor from Equation 17-42

H_o = Maximum distance pore water must flow through

c_v = Coefficient of Consolidation

The distance the pore water must flow through is affected by the permeability of the materials above and below the cohesive material. If the cohesive material is between two cohesionless materials (i.e. highly permeable materials) then the thickness of the cohesive material is cut in half. This is also known as two-way or double drainage. If the cohesive, material is bordered by an impermeable material, then the drainage path is the full thickness of the layer. This is also called one-way or single drainage. According to Das (1990), the time factor (T) is related to the degree of consolidation (U) in the following equations.

$$T = \frac{\left(\frac{\pi}{4}\right)\left(\frac{U\%}{100}\right)^2}{\left[1 - \left(\frac{U\%}{100}\right)^{5.6}\right]^{0.357}} \leq 2.5 \quad \text{Equation 17-42}$$

for

$$0 \leq U\% < 100\% \quad \text{Equation 17-43}$$

Where,

U% = Degree of Consolidation in percent

When U equals one hundred percent, T approaches infinity (∞). The limit of T indicated in Equation 17-42 results in a U of 99.3%. If the amount of settlement at a U of 99.3% exceeds the performance limits provided in Chapter 10, contact the GDS for guidance.

17.10 SECONDARY COMPRESSION SETTLEMENT

Secondary compression settlement occurs after the completion of primary consolidation settlement (i.e. U = 99.3%). This type of compression settlement occurs when the soil continues to vertically displace despite the fact that the excess pore pressures have essentially dissipated. Secondary compression typically occurs in highly plastic (PI > 21) or organic (percent organics > 30 percent) cohesive soils. Secondary compression settlement is evident on both the e-log p and the ε-log p curves (see Figure 17-28).

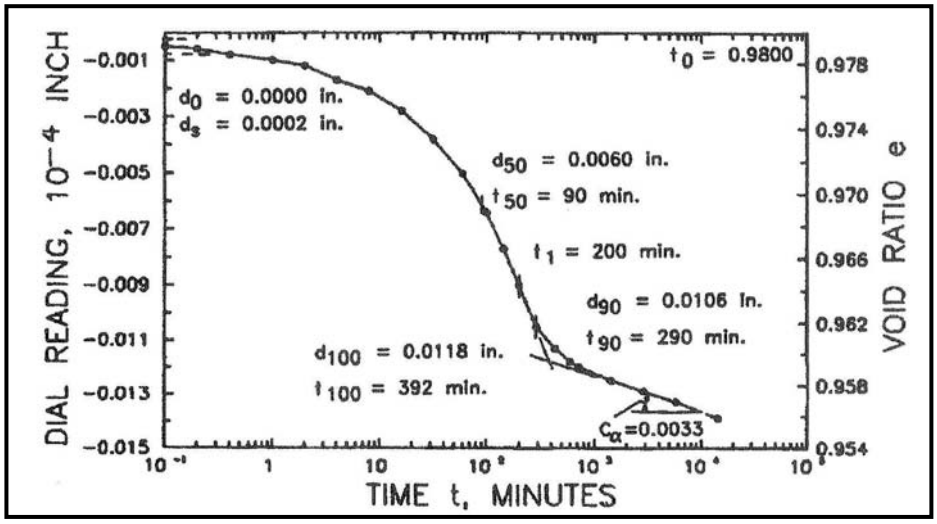


Figure 17-28, Secondary Compression
(Shallow Foundations – June 2001)

The Coefficient of Secondary Compression (C_α) can be determined using the slope of the corrected curve over one full logarithmic cycle. As with primary consolidation settlement, the amount of secondary compression settlement can be determined using either the e-log p or ε-log p curves. Presented in Table 17-12 are the equations for determining secondary compression settlement.

Table 17-12, Secondary Compression Settlement Equations

e-log p	$S_s = \sum_1^i H_o \frac{C_\alpha}{1+e_o} \log\left(\frac{t_2}{t_1}\right)$	Equation 17-44
ε-log p	$S_s = \sum_1^i H_o C_{\varepsilon\alpha} \log\left(\frac{t_2}{t_1}\right)$	Equation 17-45

Where,

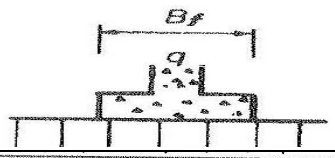
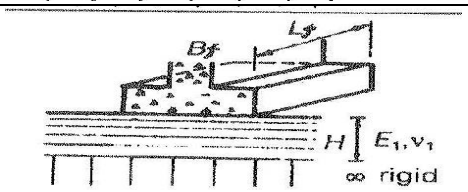
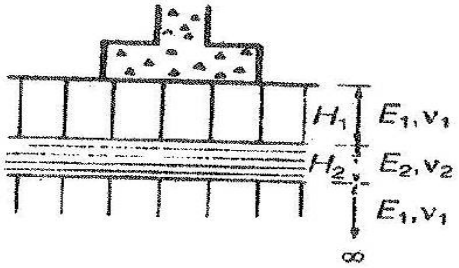
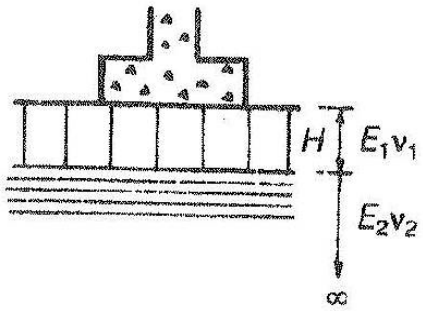
H_o = Thickness of i^{th} layer
e_o = Initial void ratio of i^{th} layer
t_1 = Time when secondary compression begins (i.e. U = 99.3%)
t_2 = Time when secondary compression is desired, usually the service life of structure

Secondary compression settlement is sometimes confused with creep. As indicated previously, secondary compression settlement occurs after the pore pressures have achieved a steady state condition and the settlement is the result of particle movement or realignment. Creep occurs after the pore pressures have achieved a steady state condition and there is no volume change. Creep is related to shear strength rather than compressibility. In many cases, it is not possible to distinguish between creep and secondary compression settlement.

17.11 SETTLEMENT IN ROCK

The settlement procedures discussed previously are for soil. Rock is normally considered incompressible; however, the potential for settlement on rock does exist. The settlement of foundations on rock can be determined using elastic theory. The settlement determinations provided in this section cover four combinations of rock (incompressible) and compressible layers (see Table 17-13).

Table 17-13, Rock Settlements on various Geological Conditions (Shallow Foundations – June 2001)

Geological Condition	Graph of Geological Condition	Settlement Calculations
Incompressible Layer		a. Determine shape factor C_d from Table 17-14; b. Calculate S_r using equation 17-46
Compressible Layer on Incompressible Layer		a. Determine ratio of H/B_f & L_f/B_f; b. Determine shape factor C_d from Table 17-14; c. Calculate S_r using equation 17-46
Compressible Layer between Incompressible Layers		a. Determine rations $(H_1 + H_2)/B_f$ & L_f/B_f; b. Calculate weighted modulus (E) for upper two layers using equation 17-47; c. Determine shape factor C'_d, for ratio $(H_1 + H_2)/B_f$ from Table 17-15; d. Calculate S_r using equation 17-46
Incompressible Layer on Compressible Layer		a. Determine ratios H/B_f & E_1/E_2; b. Determine factor α from Table 17-16; c. Determine shape factor C_d from Table 17-14; d. Calculate $S_{r\infty}$ using equation 17-46 using elastic parameters E_2 & ν_2 for overall foundation; e. Calculate S_r by reducing calculated $S_{r\infty}$ by α (see equation 17-48)

$$S_r = \frac{C_d q B_f \left(1 - \nu^2\right)}{E_m} \qquad \text{Equation 17-46}$$

$$E = \frac{\left(E_1 H_1 + E_2 H_2\right)}{\left(H_1 + H_2\right)} \qquad \text{Equation 17-47}$$

$$S_r = \alpha S_{r\infty} \qquad \text{Equation 17-48}$$

Where,

S_r = Rock Settlement

$S_{r\infty}$ = Settlement of incompressible layer underlain by a compressible layer

C_d = Shape Factor from Table 17-14

C'_d = Shape Factor from Table 17-15

q_0 = Applied vertical stress at the bottom of loaded area

B_f = Foundation width

ν = Poisson's ratio

E_m =Modulus of elasticity of rock mass (see Chapter 7)

E_1 = Modulus of elasticity of incompressible layer

E_2 = Modulus of elasticity of compressible layer

α = Elastic distortion settlement correction factor

Table 17-14, Shape Factors, C_d
(Shallow Foundations – June 2001)

Shape	Center	Corner	Middle of Short Side	Middle of Long Side	Average
Circle	1.00	0.64	0.64	0.64	0.85
Circle (rigid)	0.79	0.79	0.79	0.79	0.79
Square	1.12	0.56	0.76	0.76	0.95
Square (rigid)	0.99	0.99	0.99	0.99	0.99
Rectangle: Length/Width					
1.5	1.36	0.67	0.89	0.97	1.15
2	1.52	0.76	0.98	1.12	1.30
3	1.78	0.88	1.11	1.35	1.52
5	2.10	1.05	1.27	1.68	1.83
10	2.53	1.26	1.49	2.212	2.25
100	4.00	2.00	2.20	3.60	3.70
1000	5.47	2.75	2.94	5.03	5.15
10000	6.90	3.50	3.70	6.50	6.60

Table 17-15, Shape Factors, C'_d
(Shallow Foundations – June 2001)

H/B_f	Circle Dia. (B_f)	Rectangle Shape Foundation (L_f/B_f)						
		1	1.5	2	3	5	10	∞
0.10	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
0.25	0.24	0.24	0.23	0.23	0.23	0.23	0.23	0.23
0.50	0.48	.048	0.47	0.47	0.47	0.47	0.47	0.47
1.00	0.70	0.75	0.81	0.83	0.83	0.83	0.83	0.83
1.50	0.80	0.86	0.97	1.03	1.07	1.08	1.08	1.08
2.50	0.88	0.97	1.12	1.22	1.33	1.39	1.40	1.40
3.50	0.91	1.01	1.19	1.31	1.45	1.56	1.59	1.60
5.00	0.94	1.05	1.24	1.38	1.55	1.72	1.82	1.83
∞	1.00	1.12	1.36	1.52	1.78	2.10	2.53	∞

Table 17-16, Elastic Distortion Settlement Correction Factor, α
(Shallow Foundations – June 2001)

H/B_f	E_1/E_2				
	1	2	5	10	100
0.00	1.000	1.000	1.000	1.000	1.000
0.10	1.000	0.972	0.943	0.923	0.760
0.25	1.000	0.885	0.779	0.699	0.431
0.50	1.000	0.747	0.566	0.463	0.228
1.00	1.000	0.627	0.399	0.287	0.121
2.5	1.000	0.550	0.274	0.175	0.058
5.0	1.000	0.525	0.238	0.136	0.036
∞	1.000	0.500	0.200	0.100	0.010

17.12 LATERAL SQUEEZE

Lateral squeeze is a phenomenon that occurs when a soft cohesive soil deforms and displaces when subjected to embankment loadings and is primarily a concern at the end bents where the deep foundations may be installed through thick layers of soft cohesive soils. In addition, if the thickness of the soft cohesive soils is finite and is less then the width of the embankment (b_e) the potential for lateral squeeze is present (Figure 17-14). This phenomenon can cause rotation and horizontal displacement of the end bent and can induce excessive loadings in the deep foundations. The following equation is used to determine if the potential for lateral squeeze exists at a site.

$$\gamma_f H_f > 3\tau \qquad \text{Equation 17-49}$$

Where,

γ_f = Total unit weight of fill material
H_f = Height of fill
τ = Undrained shear strength (see Chapter 7)

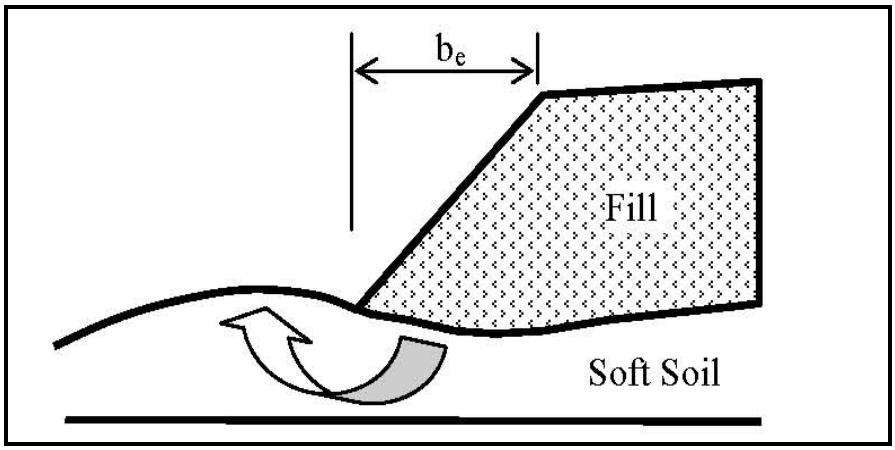


Figure 17-29, Schematic of Lateral Squeeze
(Soils and Foundations – December 2006)

If the load applied by the soil (fill height times fill unit weight) exceeds three times the undrained shear strength the potential for lateral squeeze is present; therefore, lateral movements of the soil may occur. These lateral movements may be estimated using the following equation.

$$\Delta_L = \mathbf{0.25}S_t \qquad \text{Equation 17-50}$$

Where,

Δ_L = Horizontal displacement
S_t = Total settlement of fill

17.13 REFERENCES

American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications Customary, U.S. Units, 4th Edition, dated 2007 with 2008 Interim Revisions, Washington, D.C.

Bowles, J. E., (1986), Foundation Analysis and Design, Fifth Edition, The McGraw-Hill Companies, Inc.

Casagrande, A, (1936) "The Determination of Pre-Consolidation Load and Its Practical Significance," *Proceedings of the First International Conference on Soil Mechanics*, Cambridge, Massachusetts

Christian, J. T. and W. D. Carter, III (1978), "Janbu, Bjerrum and Kjaernsli's Chart Reinterpreted," Canadian Geotechnical Journal.

Das, B. M. (1990), Principles of Foundation Engineering, Second Edition, PWS-Kent Publishing Company, Boston, Massachusetts.

Department of Defense, Department of the Army, Army Corps of Engineers, Washington D.C. (1990), *Settlement Analysis,* EM 1110-1-1904.

Department of Defense, Department of the Army, Army Corps of Engineers, Washington D.C. (2003), *Slope Stability,* EM 1110-2-1902.

Department of the Interior, US Geologic Survey, (2004), Landslide Types and Processes, Fact Sheet 2004-3072.

Department of the Navy, Naval Facilities Engineering Command, (1982), Soil Mechanics – Design Manual 7.1, Publication No. NAVFAC DM-7.1, Alexandria, Virginia.

Department of Transportation, Federal Highway Administration, Washington D.C. (1992). *The Flat Dilatometer Test*, FHWA Publication No. FHWA-SA-91-044.

Department of Transportation, Federal Highway Administration, Washington D.C. (2001). *Shallow Foundations*, FHWA Publication No. FHWA-NHI-01-023.

Department of Transportation, Federal Highway Administration, Washington D.C. (2005). *Soil Slope and Embankment Design*, FHWA Publication No. FHWA-NHI-05-132.

Department of Transportation, Federal Highway Administration, Washington D.C. (1982). *Soils and Foundations Workshop Manual*, FHWA Publication No. FHWA-HI-88-009.

Department of Transportation, Federal Highway Administration, Washington D.C. (2000). *Soils and Foundations Workshop Manual*, FHWA Publication No. FHWA-NHI-00-045.

Department of Transportation, Federal Highway Administration, Washington D.C. (2006). *Soils and Foundations Reference Manual – Volume I*, FHWA Publication No. FHWA-NHI-06-088.

Duncan, J. M. and A. L. Buchignani, (1976), *An Engineering Manual for Settlement Studies,* University of California at Berkeley, Berkeley, California.

Duncan, J. M. and S. G. Wright, (2005), Soil Strength and Slope Stability, John Wiley & Sons, Inc., New Jersey

Hough, B. K., (1959), "Compressibility as the Basis for Soil Bearing Values," ASCE Proceedings, August.

Meyerhof, G. G. (1965), "Shallow Foundations," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 91, No. SM 2.

Newmark, N. M. (1935), "Simplified Computation of Vertical Pressures in Elastic Foundations," *University of Illinois Engineering Experiment Station Circular 24*, University of Illinois, Urbana, Illinois.

Osterberg, J. O. (1957), "Influence Values for Vertical Stresses in a Semi-infinite Mass Due to an Embankment Loading," *Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering*, London, England.

Schmertmann, J. H. (1970), "Static Cone to Compute Static Settlement Over Sand," Journal of Soil Mechanics and Foundation Engineering, ASCE, Vol. 96, No. SM 3.

Terzaghi, K. and R. Peck (1967), Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley & Sons.

Timoshenko, M. J. and J. N. Goodier (1951), Theory of Elasticity, 2nd edition, McGraw-Hill.

Wright, S. G. (1969), "A Study of Slope Stability and the Undrained Shear Strength of Clay Shales," thesis presented to the University of California at Berkeley, California, in partial fulfillment of requirements for degree of Doctor of Philosophy.