EM 1110-2-1913
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US Army Corps
of Engineers

ENGINEERING AND DESIGN

Design and Construction of Levees

ENGINEER MANUAL

Chapter 3
Laboratory Testing

3-1. General

a. Reference should be made to EM 1110-1-1906 for current soil testing procedures, and to EM 1110-2-1902 for applicability of the various shear strength tests in stability analyses.

b. Laboratory testing programs for levees will vary from minimal to extensive, depending on the nature and importance of the project and on the foundation conditions, how well they are known, and whether existing experience and correlations are applicable. Since shear and other tests to determine the engineering properties of soils are expensive and time-consuming, testing programs generally consist of water content and identification tests on most samples and shear, consolidation, and compaction tests only on representative samples of foundation and borrow materials. It is imperative to use all available data such as geological and geophysical studies, when selecting representative samples for testing. Soil tests that may be included in laboratory testing programs are listed in Table 3-1 for fine-grained cohesive soils and in Table 3-2 for pervious soils, together with pertinent remarks on purposes and scope of testing.

Table 3-1
Laboratory Testing of Fine-Grained Cohesive Soils

Test	Remarks
Visual classification and water content determinations	On all samples
Atterberg limits	On representative samples of foundation deposits for correlation with shear or consolidation parameters, and borrow soils for comparison with natural water contents, or correlations with optimum water content and maximum densities
Permeability	Not required; soils can be assumed to be essentially impervious in seepage analyses
Consolidation	Generally performed on undisturbed foundation samples only where:
	a. Foundation clays are highly compressible
	b. Foundations under high levees are somewhat compressible
	c. Settlement of structures within levee systems must be accurately estimated
	Not generally performed on levee fill; instead use allowances for settlement within levees based on type of compaction. Sometimes satisfactory correlations of Atterberg limits with coefficient of consolidation can be used. Compression index can usually be estimated from water content.
Compaction	*a.* Required only for compacted or semi-compacted levees
	b. Where embankment is to be fully compacted, perform standard 25-blow compaction tests
	c. Where embankment is to be semi-compacted, perform 15-blow compaction tests
Shear strength	*a.* Unconfined compression tests on saturated foundation clays without joints or slickensides
	b. Q triaxial tests appropriate for foundation clays, as undrained strength generally governs stability
	c. R triaxial and S direct shear: Generally required only when levees are high and/or foundations are weak, or at locations where structures exist in levees
	d. Q, R, and S tests on fill materials compacted at appropriate water contents to densities resulting from the expected field compaction effort

Table 3-2
Laboratory Testing of Pervious Materials

Test	Remarks
Visual classification	Of all jar samples
In situ density determinations	Of Shelby-tube samples of foundation sands where liquefaction susceptibility must be evaluated
Relative density	Maximum and minimum density tests should be performed in seismically active areas to determine in situ relative densities of foundation sands and to establish density control of sand fills
Gradation	On representative foundation sands:
	a. For correlating grain-size parameters with permeability or shear strength
	b. For size and distribution classifications pertinent to liquefaction potential
Permeability	Not usually performed. Correlations of grain-size parameters with permeability or shear strength used. Where underseepage problems are serious, best guidance obtained by field pumping tests
Consolidation	Not usually necessary as consolidation under load is insignificant and occurs rapidly
Shear strength	For loading conditions other than dynamic, drained shear strength is appropriate. Conservative values of φ' can be assumed based on S tests on similar soils. In seismically active areas, cyclic triaxial tests may be performed

3-2. Classification and Water Content Determinations

After soil samples have been obtained in subsurface exploration of levee foundations and borrow areas, the first and essential step is to make visual classifications and water content determinations on all samples (except that water content determinations should not be made on clean sands and gravels). These samples may be jar or bag samples obtained from test pits, disturbed or undisturbed drive samples, or auger samples. Field descriptions, laboratory classifications, and water content values are used in preparing graphic representations of boring logs. After examining these data, samples of fine-grained soils are selected for Atterberg limits tests, and samples of coarse-grained soils for gradation tests.

Section I
Fine-Grained Soils

3-3. Use of Correlations

Comparisons of Atterberg limits values with natural water contents of foundation soils and use of the plasticity chart itself (Figure 3-1), together with split-spoon driving resistance, geological studies, and previous experience often will indicate potentially weak and compressible fine-grained foundation strata and thus the need for shear and perhaps consolidation tests. In some cases, in the design of low levees on familiar foundation deposits for example, correlations between Atterberg limits values and consolidation or shear strength characteristics may be all that is necessary to evaluate these characteristics. Examples of correlations among Atterberg limits values, natural water content, shear strength and consolidation characteristics are shown in Figures 3-2 and 3-3. Correlations based on local soil types and which distinguish between normally and overconsolidated conditions are preferable. Such correlations may also be used to reduce the number of tests required for design of higher levees. As optimum water content may in some cases be correlated with Atterberg limits, comparisons of Atterberg limits and natural water contents of borrow soils as shown in Figure 3-4 can indicate whether the borrow materials are suitable for obtaining adequate compaction.

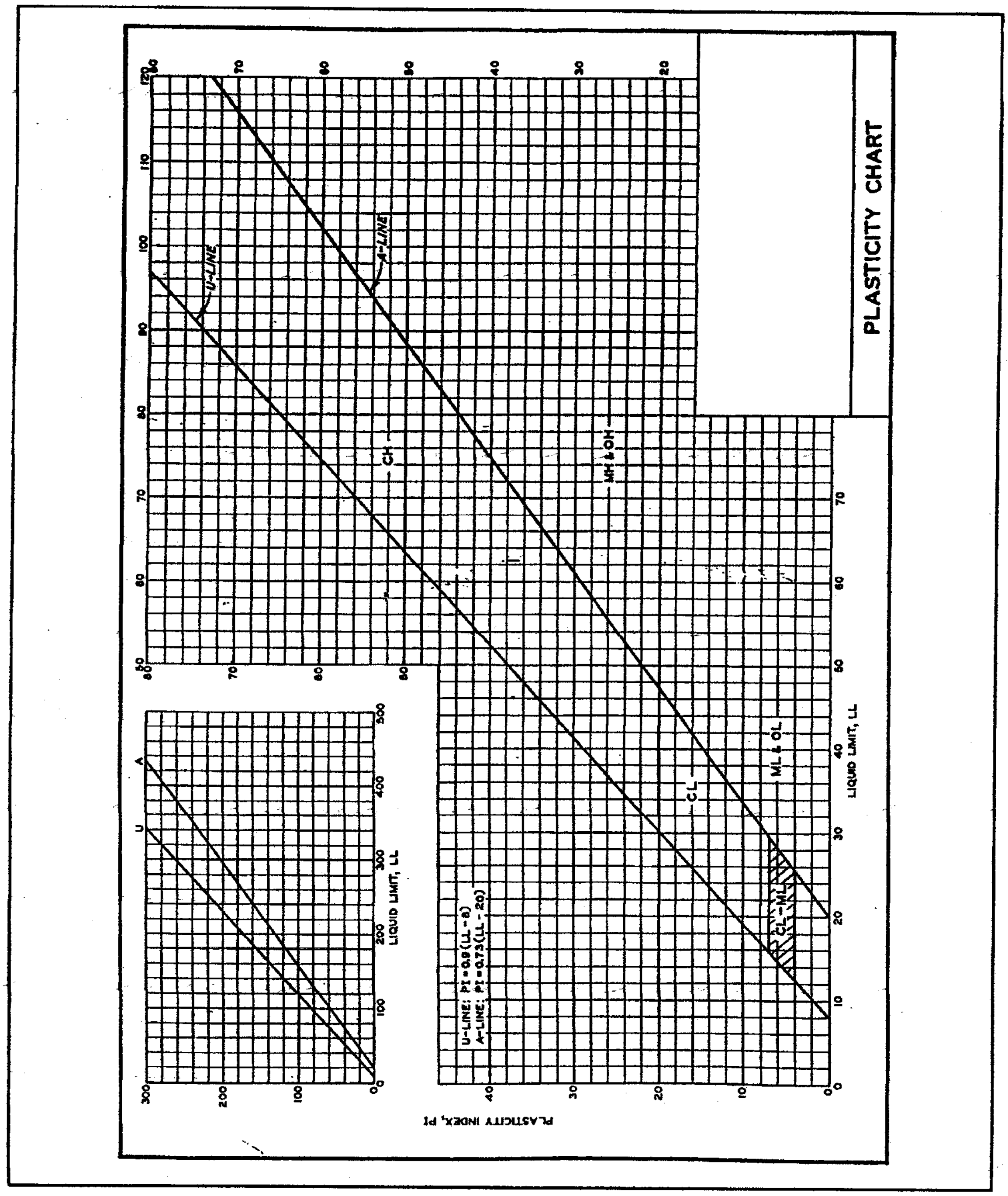
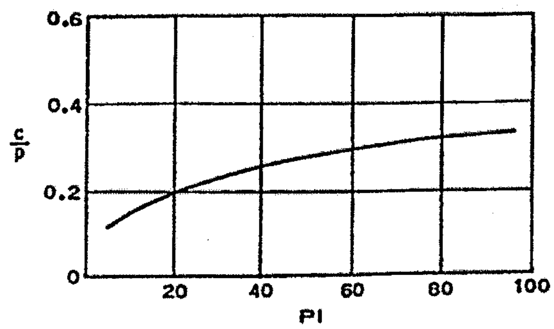
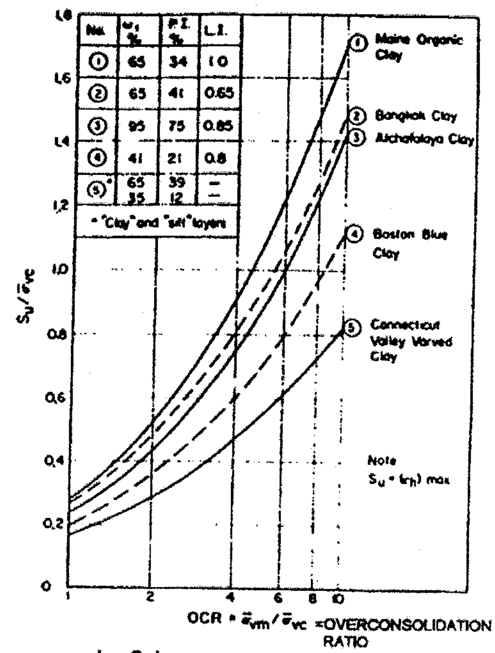


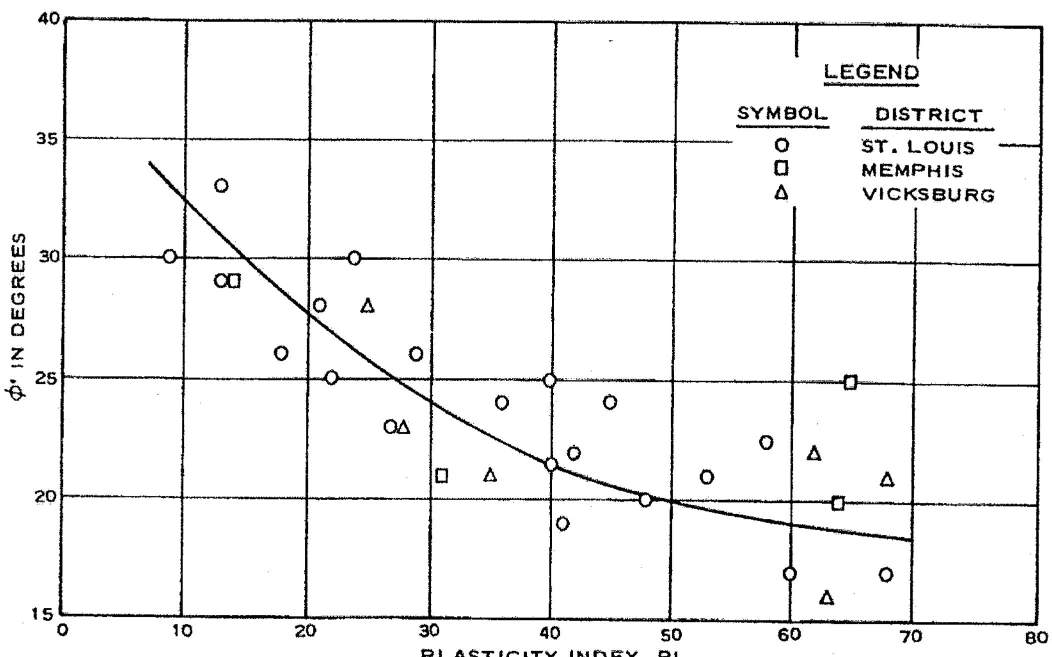
Figure 3-1. Plasticity chart (ENG Form 4334)



a. c/p versus plasticity index for normally consolidated soils (after Bjerrum, ref. A-2)

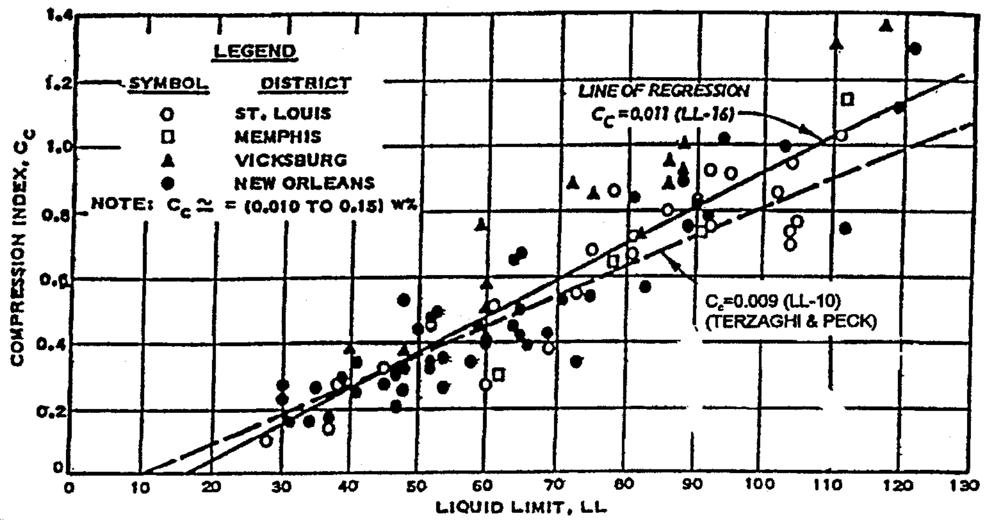


b. S_u/σ_{vc} versus overconsolidation ratio (after Ladd and Foott, 1974, Ref. A-2)

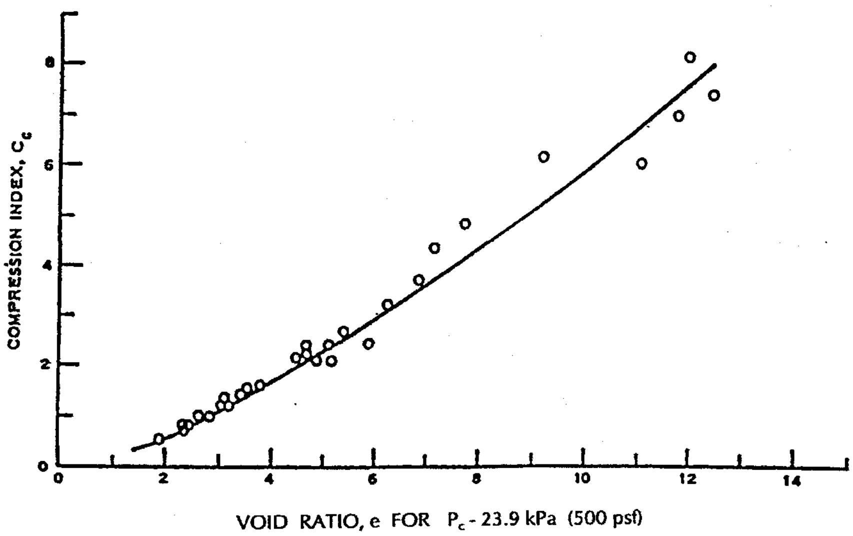


c. φ' versus plasticity index

Figure 3-2. Example correlations of strength characteristics for fine-grained soils

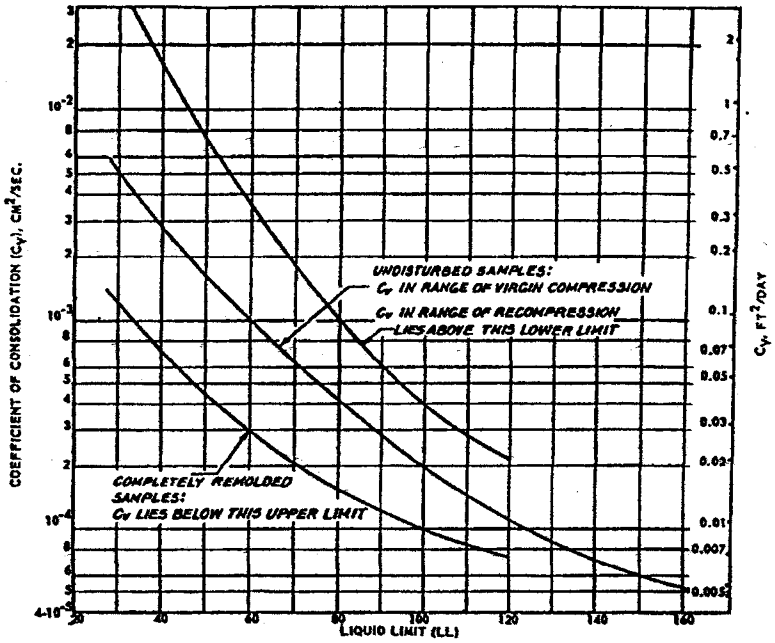


a. Compression index versus liquid limit for normally consolidated soils

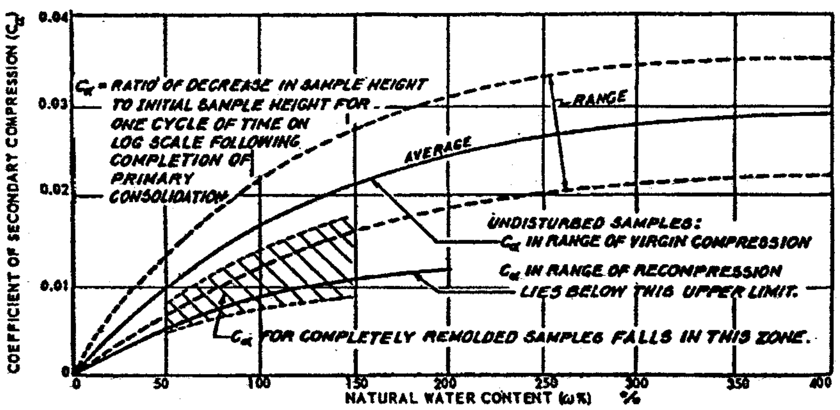


b. Compression index versus initial void ratio for tidal marsh

Figure 3-3. Example correlations for consolidation characteristics of fine-grained soils (after Kapp, ref. A-2)



c. Coefficient of consolidation versus liquid limit (from NAVFAC DM-7 ref. A-1)



d. Coefficient of secondary compression versus water content (from NAVFAC DM-7 ref. A-1)

Figure 3-3. (Concluded)

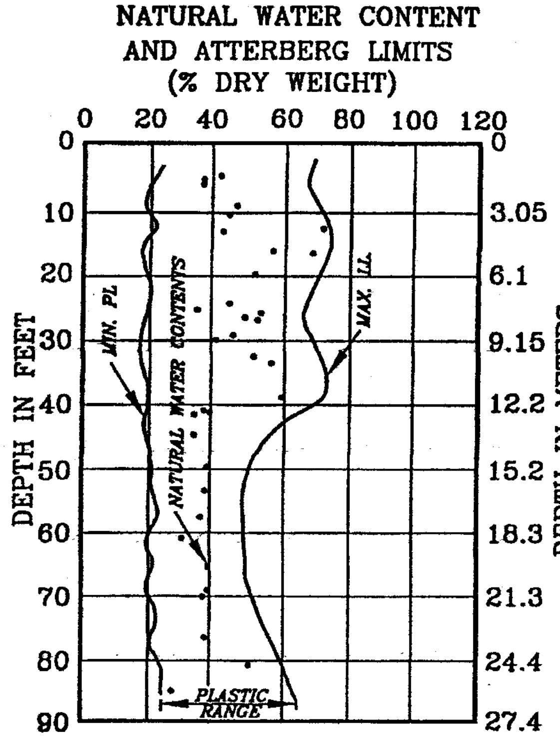


Figure 3-4. Comparisons of Atterberg limits and natural water contents

3-4. Shear Strength

Approximate shear strengths of fine-grained cohesive soils can be rapidly determined on undisturbed foundation samples, and occasionally on reasonably intact samples from disturbed drive sampling, using simple devices such as the pocket penetrometer, laboratory vane shear device, or the miniature vane shear device (Torvane). To establish the reliability of these tests, it is desirable to correlate them with unconfined compression tests. Unconfined compression tests are somewhat simpler to perform than Q triaxial compression tests, but test results exhibit more scatter. Unconfined compression tests are appropriate primarily for testing saturated clays which are not jointed or slickensided. Of the triaxial compression tests, the Q test is the one most commonly performed on foundation clays, since the in situ undrained shear strength generally controls embankment design on such soils. However, where embankments are high, stage construction is being considered, or important structures are located in a levee system, R triaxial compression tests and S direct shear tests should also be performed.

3-5. Consolidation

Consolidation tests are performed for those cases listed in Table 3-1. In some locations correlations of liquid limit and natural water content with coefficient of consolidation, compression index, and coefficient of secondary compression can be used satisfactorily for making estimates of consolidation of foundation clays under load.

3-6. Permeability

Generally there is no need for laboratory permeability tests on fine-grained fill materials, nor on surface clays overlying pervious foundation deposits. In underseepage analyses, simplifying assumptions must be made relative to thickness and soil type of fine-grained surface blankets. Furthermore, animal burrows, root channels, and other discontinuities in surface blankets can significantly affect the overall effective permeability. Therefore, an average value of the coefficient of permeability based on the dominant soil type (Appendix B) is generally of sufficient accuracy for use in underseepage analyses, and laboratory tests are not essential.

3-7. Compaction Tests

The type and number of compaction tests will be influenced by the method of construction and the variability of available borrow materials. The types of compaction tests required are summarized in Table 3-1.

Section II
Coarse-Grained Soils

3-8. Shear Strength

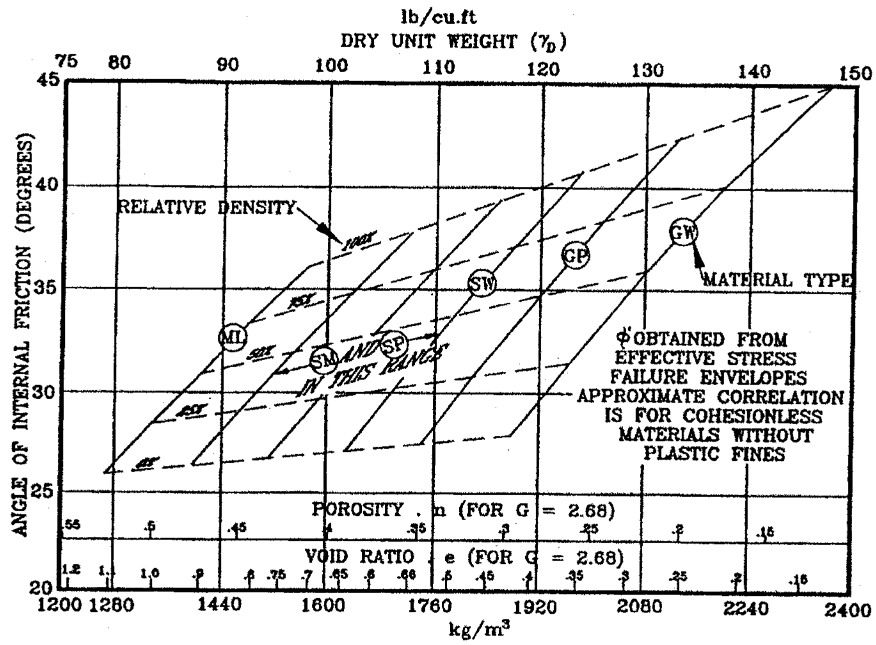
When coarse-grained soils contain few fines, the consolidated drained shear strength is appropriate for use in all types of analyses. In most cases, conservative values of the angle of internal friction (φ) can be assumed from correlations such as those shown in Figure 3-5, and no shear tests will be needed.

3-9. Permeability

To solve the problem of underseepage in levee foundations, reasonable estimates of permeability of pervious foundation deposits are required. However, because of difficulty and expense in obtaining undisturbed samples of sands and gravels, laboratory permeability tests are rarely performed on foundation sands. Instead, field pumping tests or correlations such as that of Figure 3-5 developed between a grain-size parameter (such as D_{10}) and the coefficient of permeability, k , are generally utilized.

3-10. Density Testing of Pervious Fill

Maximum density tests on available pervious borrow materials should be performed in accordance with ASTM D 4253 so that relative compaction requirements for pervious fills may be checked in the field when required by the specification. Due to the inconsistencies in duplicating minimum densities (ASTM D 4254), relative density may not be used. Factors such as (but not limited to) site specific materials, availability of testing equipment and local practice may make it more practical to utilize methods other than ASTM D 4253 and ASTM D 4254 to control the degree of compaction of cohesionless material. The other methods used include comparison of in-place density to either the maximum Proctor density or the maximum density obtained by ASTM 4253 (if vibratory table is available).



a. Angle of internal friction versus unit weight (from NAVFAC DM-7 ref. A-1)

Figure 3-5. Example correlations for properties of coarse-grained soils

Section II
Stability Analyses

6-4. Methods of Analysis

The principal methods used to analyze levee embankments for stability against shear failure assume either (a) a sliding surface having the shape of a circular arc within the foundation and/or the embankment or (b) a composite failure surface composed of a long horizontal plane in a relatively weak foundation or thin foundation stratum connecting with diagonal plane surfaces up through the foundation and embankment to the ground surface. Various methods of analysis are described in EM 1110-2-1902, and can be chosen for use where determined appropriate by the designer. Computer programs are available for these analyses, with the various loading cases described in EM 1110-2-1902, so the effort of making such analyses is greatly reduced, and primary attention can be devoted to the more important problems of defining the shear strengths, unit weights, geometry, and limits of possible sliding surfaces.

6-5. Conditions Requiring Analysis

The various loading conditions to which a levee and its foundation may be subjected and which should be considered in analyses are designated as follows: Case I, end of construction; Case II, sudden drawdown from full flood stage; Case III, steady seepage from full flood stage, fully developed phreatic surface; Case IV, earthquake. Each case is discussed briefly in the following paragraphs and the applicable type of design shear strength is given. For more detailed information on applicable shear strengths, methods of analysis, and assumptions made for each case refer to EM 1110-2-1902.

a. Case I - End of construction. This case represents undrained conditions for impervious embankment and foundation soils; i.e., excess pore water pressure is present because the soil has not had time to drain since being loaded. Results from laboratory Q (unconsolidated-undrained) tests are applicable to fine-grained soils loaded under this condition while results of S (consolidated-drained) tests can be used for pervious soils that drain fast enough during loading so that no excess pore water pressure is present at the end of construction. The end of construction condition is applicable to both the riverside and landside slopes.

b. Case II - Sudden drawdown. This case represents the condition whereby a prolonged flood stage saturates at least the major part of the upstream embankment portion and then falls faster than the soil can drain. This causes the development of excess pore water pressure which may result in the upstream slope becoming unstable. For the selection of the shear strengths see Table 6-1a.

c. Case III - Steady seepage from full flood stage (fully developed phreatic surface). This condition occurs when the water remains at or near full flood stage long enough so that the embankment becomes fully saturated and a condition of steady seepage occurs. This condition may be critical for landside slope stability. Design shear strengths should be based on Table 6-1a.

d. Case IV - Earthquake. Earthquake loadings are not normally considered in analyzing the stability of levees because of the low probability of earthquake coinciding with periods of high water. Levees constructed of loose cohesionless materials or founded on loose cohesionless materials are particularly susceptible to failure due to liquefaction during earthquakes. Depending on the severity of the expected earthquake and the importance of the levee, seismic analyses to determine liquefaction susceptibility may be required.

Table 6-1a
Summary of Design Conditions

Analysis Condition	Shear Strength[a]	Pore Water Pressure
During and End-of-Construction	Free draining soils - use effective stresses	Free draining soils - Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations for no flow or steady seepage analysis techniques (flow nets, finite element analyses or finite difference analyses).
	Low permeability soils - use undrained strengths and total stresses[b]	Low permeability soils - Total stresses are used; pore water pressures are set to zero in the slope stability computations.
Steady State Seepage Conditions	Use effective stresses. Residual strengths should be used where previous shear deformation or sliding has occurred.	Estimated from field measurements of pore water pressures, hydrostatic pressure computations for no flow conditions, or steady seepage analysis techniques (flow nets, finite element analyses or finite difference analyses).
Sudden Drawdown Conditions	Free draining soils - use effective stresses	Free draining soils - First stage computations (before drawdown) - steady-state seepage pore pressures as described for steady state seepage condition. Second and third stage computations (after drawdown) - pore water pressures estimated using same techniques as for steady seepage, except with lowered water levels.
	Low permeability soils - Three stage computations: First stage use effective stresses; second stage use undrained shear strengths and total stresses; third stage use drained strengths (effective stresses) or undrained strengths (total stresses) depending on which strength is lower - this will vary along the assumed shear surface.	Low permeability soils - First stage computations - steady-state seepage pore pressures as described for steady state seepage condition. Second stage computations - Total stresses are used pore water pressures are set to zero. Third stage computations - Use same pore pressures as free draining soils if drained strengths are being used; where undrained strengths are used pore water pressures are set to zero.

[a] Effective stress parameters can be obtained from consolidated-drained (CD, S) tests (either direct shear or triaxial) or consolidated-undrained (CU, R) triaxial tests on saturated specimens with pore water pressure measurements. Direct shear or Bromhead ring shear tests should be used to measure residual strengths. Undrained strengths can be obtained from unconsolidated-undrained (UU, Q) tests. Undrained shear strengths can also be estimated using consolidated-undrained (CU, R) tests on specimens consolidated to appropriate stress conditions representative of field conditions; however, the "R" or "total stress" envelope and associated c and ö, from CU, R tests should not be used.
[b] For saturated soils use ö = 0; total stress envelope with ö > 0 is only applicable to partially saturated soils.

6-6. Minimum Acceptable Factors of Safety

The minimum required safety factors for the preceding design conditions along with the portion of the embankment for which analyses are required and applicable shear test data are shown in Table 6-1b.

6-7. Measures to Increase Stability

Means for improving weak and compressible foundations to enable stable embankments to be constructed thereon are discussed in Chapter 7. Methods of improving embankment stability by changes in embankment section are presented in the following paragraphs.

a. Flatten embankment slopes. Flattening embankment slopes will usually increase the stability of an embankment against a shallow foundation type failure that takes place entirely within the embankment. Flattening embankment slopes reduces gravity forces tending to cause failure, and increases the length of potential failure surfaces (and therefore increases resistance to sliding).

Table 6-1b
Minimum Factors of Safety - Levee Slope Stability

	Applicable Stability Conditions and Required Factors of Safety			
Type of Slope	End-of-Construction	Long-Term (Steady Seepage)	Rapid Drawdown [a]	Earthquake[b]
New Levees	1.3	1.4	1.0 to 1.2	(see below)
Existing Levees	--	1.4[c]	1.0 to 1.2	(see below)
Other Embankments and dikes[d]	1.3[e,f]	1.4[c,f]	1.0 to 1.2[f]	(see below)

[a] Sudden drawdown analyses. F. S. = 1.0 applies to pool levels prior to drawdown for conditions where these water levels are unlikely to persist for long periods preceding drawdown. F. S. = 1.2 applies to pool level, likely to persist for long periods prior to drawdown.
[b] See ER 1110-2-1806 for guidance. An EM for seismic stability analysis is under preparation.
[c] For existing slopes where either sliding or large deformation have occurred previously and back analyses have been performed to establish design shear strengths lower factors of safety may be used. In such cases probabilistic analyses may be useful in supporting the use of lower factors of safety for design.
[d] Includes slopes which are part of cofferdams, retention dikes, stockpiles, navigation channels, breakwater, river banks, and excavation slopes.
[e] Temporary excavated slopes are sometimes designed for only short-term stability with the knowledge that long-term stability is not adequate. In such cases higher factors of safety may be required for end-of-construction to ensure stability during the time the excavation is to remain open. Special care is required in design of temporary slopes, which do not have adequate stability for the long-term (steady seepage) condition.
[f] Lower factors of safety may be appropriate when the consequences of failure in terms of safety, environmental damage and economic losses are small.

b. Stability berms. Berms essentially provide the same effect as flattening embankment slopes but are generally more effective because of concentrating additional weight where it is needed most and by forcing a substantial increase in the failure path. Thus, berms can be an effective means of stabilization not only for shallow foundation and embankment type failures but for more deep-seated foundation failures as well. Berm thickness and width should be determined from stability analyses and the length should be great enough to encompass the entire problem area, the extent of which is determined from the soil profile. Foundation failures are normally preceded by lateral displacement of material beneath the embankment toe and by noticeable heave of material just beyond the toe. When such a condition is noticed, berms are often used as an emergency measure to stabilize the embankment and prevent further movement.

6-8. Surface Slides

Experience indicates that shallow slides may occur in levee slopes after heavy rainfall. Failure generally occurs in very plastic clay slopes. They are probably the result of shrinkage during dry weather and moisture gain during wet weather with a resulting loss in shear strength due to a net increase in water content, plus additional driving force from water in cracks. These failures require maintenance and could be eliminated or reduced in frequency by using less plastic soils near the surface of the slopes or by chemical stabilization of the surface soils.

Section III
Settlement

6-9. General

Evaluation of the amount of postconstruction settlement that can occur from consolidation of both embankment and foundation may be important if the settlement would result in loss of freeboard of the levee or damage to structures in the embankment. Many districts overbuild a levee by a given percent of its height to take into account anticipated settlement both of the foundation and within the levee fill itself. Common allowances are 0 to 5 percent for compacted fill, 5 to 10 percent for semicompacted fill, 15 percent for uncompacted fill, and 5 to 10 percent for hydraulic fill. Overbuilding does however increase the severity of stability problems and may be impracticable or undesirable for some foundations.

6-10. Settlement Analyses

Settlement estimates can be made by theoretical analysis as set forth in EM 1110-1-1904. Detailed settlement analyses should be made when significant consolidation is expected, as under high embankment loads, embankments of highly compressible soil, embankments on compressible foundations, and beneath steel and concrete structures in levee systems founded on compressible soils. Where foundation and embankment soils are pervious or semipervious, most of the settlement will occur during construction. For impervious soils it is usually conservatively assumed that all the calculated settlement of a levee built by a normal sequence of construction operations will occur after construction. Where analyses indicate that more foundation settlement would occur than can be tolerated, partial or complete removal of compressible foundation material may be necessary from both stability and settlement viewpoints. When the depth of excavation required to accomplish this is too great for economical construction, other methods of control such as stage construction or vertical sand drains may have to be employed, although they seldom are justified for this purpose.