CHAPTER 3

GEOTECHNICAL CONSIDERATIONS

Section I. Subsurface Investigations

3-1. Introduction.

a. The planning, design, and construction of cofferdams should be approached as though the cofferdam is the primary structure of the project, the end result rather than the means to an end. The same degree of care, particularity, and competence should be exercised with the cofferdam as with the main structure. This necessarily involves detailed investigations because the foundation conditions, perhaps more than any other factor, impact on the cost and degree of difficulty in construction and eventual integrity of the cofferdam. Though impractical, if not impossible, to accurately determine all of the subsurface details, the major details should be determined to avoid needless delays and claims as well as possible failure resulting from inadequate subsurface investigations.

b. The investigative program should be such that the cofferdam investigations form an integral part of the overall program for the main structure. By integrating the investigative programs for the various structures, the use of resources and information is maximized. Typically, there are three main investigative stages in the development of a project: the survey investigation which is a combination reconnaissance and feasibility stage made prior to Congressional authorization to determine the most favorable site, engineering feasibility, and costs; the definite project or specifications investigation which is made after Congressional authorization to provide required geologic and foundation data for preparation of contract plans and specifications; and the construction investigation which is made as the work progresses to fill in details.

3-2. Preliminary Investigations.

a. Office Studies. Office studies of the general area of the cofferdam location should be initiated prior to any field work. These preliminary studies should include a review of all geotechnical data compiled during the survey investigation stage for the project, including reports, maps, and aerial photographs. The investigation should include a study of the topography, physiography, geologic history, stratigraphy, geologic structure, petrography, and ground-water conditions. This information should include: bedrock type, occurrence, and general structural relationship; leakage and foundation problems possible if soluble rocks are present; possible results of glaciation (buried valleys, pervious divides, lacustrine deposits); presence or absence of faults and associated earthquake problems; extent of weathering; depth and character of overburden materials; general ground-water conditions; and availability of sources of construction materials. It is essential that the regional and local geology be known and understood prior to developing and implementing a plan of subsurface investigation.

b. Field Studies. As with the office studies, field reconnaissance and, perhaps, a limited number of subsurface borings and geophysical studies are conducted for the survey investigation. The results of those initial field studies should be incorporated into investigations which are designed to reveal specific information on the cofferdam foundation conditions. The resulting information should include if possible: nature and thickness of the overburden; maps of rock outcrops denoting type and condition of rock, discontinuities, presence or absence of geologic structure; and preliminary ground-water conditions.

3-3. Development of a Boring Plan.

a. Preliminary. After a careful evaluation of all available site data, a limited number of borings should be laid out along the center line of the proposed cofferdam location.

(1) This initial exploration can ordinarily be accomplished with split-spoon standard penetration sampling of the overburden and NX-size diamond coring of the bedrock, supplemented by a number of borings drilled with non-sampling equipment such as roller rock bits. Standard penetration resistances should be obtained at least at 5-foot-depth intervals or at material changes, whichever is the lesser. The NX-size (or comparable wireline equipment) is the smallest size coring equipment that should be used, and only then if acceptable recovery is obtained. The nonsampled borings will provide information on the overburden thickness, the presence or absence of boulders, and the top of rock configuration. A number of the borings should be selected to remain open and function as piezometers to provide ground-water data. If available, downhole geophysical equipment should be used to obtain additional data from each hole. The type of probes used will be necessarily dependent on the foundation material. For example, a gamma probe would be one of the most useful tools in logging interbeds of sand and clay or limestone and shale while a caliper probe might prove invaluable in cavernous limestone. Other important geophysical instruments that might be utilized for the preliminary investigative stage are the portable seismograph and the electrical resistivity instrument. EM 1110-l-1802 covers other methods that are useful. The portable seismograph may be used to obtain information on the bedrock surface that will be invaluable in planning the detailed exploration. The electrical resistivity apparatus may be used to determine approximate depths of weathering, the extent of buried gravel deposits, and the ground-water table. In using these instruments, the investigator must keep in mind that data derived from such tools are general in nature and intended to be used as supplemental data. Care must be exercised to prevent erroneous assumptions or interpretations on unsupported or unconfirmed geophysical data.

(2) Preliminary investigations are intended to provide the general information necessary to supplement the office studies and provide the basis needed to plan a comprehensive final investigative program. Data obtained

from the preliminary program should answer the general questions as to classification of materials including index properties, consistency or relative density, overburden thickness, and ground-water conditions.

b. Final.

(1) After the preliminary investigative program has disclosed the general characteristics of the subsurface materials, a more specific program must then be designed. Economic and time limitations often control the amount of effort expended on subsurface investigations, and although there will never be enough time or money to uncover all defects and their locations, the program must be adequate to define the essential character of the subsurface materials. The program results should enable the investigators to determine the nature of the overburden and the bedrock.

(2) As with any dam, the final number, spacing, and depth of borings for foundation exploration of a cellular cofferdam are determined by several factors, principal among which is the complexity of the geologic conditions. The holes should extend to top of rock if practicable, or at least to a depth where stresses from the structure are small. Again as a general rule, the borings should extend to a depth at least equal to the designed height of the cofferdam. In applying these general rules, care must be exercised to avoid formulating a plan of borings on a predetermined pattern to predetermined depths, possibly losing available information to be gained from the flexibility afforded by a knowledgeable use of the geology. Additional borings should be located, oriented, and drilled to depths to fully and carefully explore previously disclosed trouble areas, such as layers of weak compressible clay, fault zones or zones of highly dissolved rock, or irregular rock surfaces.

(3) The program should be detailed enough to adequately cover sources of common problems in cellular cofferdam construction. Typical obstacles such as boulders in the overburden may cause difficulty in driving the cell sheets and lead to interpretations of a false top of rock. The program should also fully cover foundation features which have resulted in past cellular cofferdam failures, i.e., foundation failures precipitated by faults, slip planes, and high uplift pressures.

(4) The final plan of investigation should include continuous undisturbed sampling of the overburden to provide the necessary samples for laboratory testing. The type, number, and depth of undisturbed sample borings should be determined after an evaluation of information derived from the disturbed sample borings. Bedrock cores should be taken to adequately define the top of rock as well as the presence or absence of discontinuities in the rock. Large diameter cores for testing may not be necessary if such testing has been performed for the main structure and if there is no change in the geology.

(5) The location, orientation, and depth of core borings should be adjusted to recover as much information as possible on the more probable problem defects in the particular rock. For example, the major problem in sandstone is generally the jointing, especially if subjected to folding, whereas the

major problem in limestone is generally associated with solution cavities. Regardless of the degree of care exercised in the core drilling operations, all core may not be recovered. Particular emphasis should be accorded this "lost core," and for design purposes, this loss should be attributed to soft or weak materials unless there is incontrovertible evidence to the contrary. The possibility of such potential sliding planes should always be considered regardless of rock type, because seemingly competent rock may contain weak clay seams and adversely oriented clay-filled joints along which sliding can take place. Borings for top of rock determination should go into the rock sufficiently to determine depth of discontinuities. This depth should be adjusted to fit conditions as determined by other studies, e.g., the depth should be increased if geologic interpretations from core borings and outcrops indicate an average depth of 25 feet of cavernous rock. In addition, if there is evidence of an abrupt change in the top of rock elevation, such as an erosional scarp or severe and widespread solution activity in the limestone, a number of roller rock bit borings should be drilled on close centers to better define the condition.

(6) A reliable estimate of water inflow as well as an accurate determination of the elevation and fluctuation of the ground-water table are primary concerns in the design and construction of any hydraulic structure. One method of obtaining information is the field pumping test which may be performed to determine the permeability of the foundation materials.

(7) The investigation plan should be flexible so that information may be evaluated as soon as possible and adjustments made as needed. The program should begin as soon as possible and should carry through the design stage until adequate information is available for preparation of contract plans and specifications. The program must be refined through analysis of the geologic details to provide specific and reliable information on the character of the overburden; the depth to and configuration of the top of rock; the depth and character of bedrock weathering; the structures or discontinuities, such as faults, shear zones, folds, joints, solution channels, bedding, and schistosity; the physical properties of the foundation materials; the elevation and fluctuation limits of the ground water; and potential foundation problems and their treatment, such as leakage and stability.

(8) The final investigations should supply the necessary information to complete the interpretation or "picture" of subsurface conditions, dispel any reasonable doubts or fears on the practicability of the design, and provide adequate information for reliable estimates on foundation-related bid items for the contract.

3-4. Presentation of Data.

a. Report. Following a complete and thorough evaluation of all geotechnical data, a report should be prepared for inclusion in the design memorandum. Scheduling on a project is usually such that design exploration and actual design are done concurrently. Consequently, the data should be discussed with the design engineers as the data are evaluated. In most cases, the cellular cofferdam will be included in the feature design memorandum for the primary structure. In any case, the report should include a brief summary of the topography, of the regional and site geology including the seismic history, and of the subsurface investigations and tests that were performed. The summary of the site physiography and geology should emphasize those conditions of engineering significance, i.e., those most pertinent to the engineering structure, in this case, the cofferdam. Such conditions that should be covered are the character and thickness of the overburden with particular note of any potential trouble materials; the estimated top of rock; the type, stratigraphic sequence, and geologic structure of the rock; the nature and depth of rock weathering; and site ground-water conditions. The detailed account of the geotechnical investigations should include the number, type, and location of the explorations as well as an explanation for the particular explorations. A brief description of the various pieces of equipment used should also be provided. The account should contain a summary of the type of tests, both field and laboratory, that were performed and the results that were obtained. And finally, any search for sources of construction materials, whether specifically for the cofferdam or not, should be summarized. The report should include a detailed account of the data that were obtained, conclusions as to the subsurface conditions and their impact on the cellular cofferdam, and recommendations, particularly as to foundation treatment and construction materials. The preponderance of the information contained in this report, minus interpretations, should be presented to contractors for bidding purposes in accordance with the Unified Soil Classification System (item 89).

b. Drawings. Drawings are a necessary part of the report and should include a plan of exploration, boring logs, top of rock map, and geologic interpretations of subsurface conditions at the cofferdam site. The sections must provide the location of the borings; the character and thickness of the overburden with particular note of any potential trouble materials and an interpretation of their extent and configuration; the estimated top of rock line, both weathered and unweathered; the character of the weathered rock zone; overburden and bedrock classification; structural features such as faults, joints, and bedding planes; and ground-water conditions. The boring logs should show the designation and location, the surface elevation, and the overburden/bedrock contact, and should describe the material in terms of the Unified Soil Classification System. The boring logs should also show the depths of material change, blow counts in the overburden or areas of rapid drill penetration, defects, core loss, drill water increase or loss, water level data with dates obtained, pressure test data, the date hole was completed, percentage of core recovery, and size and type of hole. The results of any geophysical survey should be presented to support or supplement other exploratory data. The proposed cellular cofferdam location should be included on the drawings to more accurately depict the founding of the structure in relation to the subsurface conditions and to facilitate review.

3-5. Investigations During and Following Construction.

a. Construction and Postconstruction Data Acquisition. The subsurface investigations must continue throughout the construction and postconstruction period. Information on foundation conditions should be obtained and recorded whenever and wherever possible during construction and operation of the cofferdam. This information should include volume and thicknesses of any deleterious material such as a weak compressible clay bed that might necessarily be excavated and the depth or elevation of the excavation, increased sheet pile resistance and the reason for the increase such as lenses or zones of cobbles or boulders, depth of sheet pile refusals, and water inflows as evidenced by the volume of pumping required to maintain a dry working area. This information should be continuously compared with data developed for design and for preparation of plans and specifications. The information obtained during and following construction of the cofferdam may prove invaluable in the event that problems with the performance of the structure develop, resulting in remedial action and/or a contract claim.

b. Construction Foundation Report. After completion of the cofferdam construction, an as-built foundation report on construction of the cofferdam must be prepared in compliance with ER 1110-1-1801. Although this report in most cases will be included with the foundation report for the entire project, its initiation and completion should not be delayed. The report must contain all data pertinent to the foundation, including but not limited to a comparison of the foundation conditions anticipated and those actually encountered; a complete description of any materials necessarily excavated and the methods utilized in the excavation; a description, evaluation, and tabulation of the sheet pile driving including method, type, date, and depth; a description of the methods used and any problems encountered in the foundation treatment and any deviations from the design treatment (including reasons for such change); a tabulation and evaluation of the water pumping required as well as a comparison with the anticipated inflow; a detailed discussion of any solutions to problems encountered; and the results and evaluation of, and recommendations based on the instrumentation.

Section II. Field and Laboratory Testing

3-6. Estimation of Engineering Properties. Field and laboratory testing are used to estimate the engineering properties needed for the rational design of both the foundation and the structure. The foundation design requires an estimate of both the strength and seepage qualities of the foundation, The engineering properties of the cell fill can usually be estimated with sufficient accuracy from laboratory index tests.

3-7. Field Testing. During the initial phase of exploration, field tests are generally made to obtain a rough estimate of the strength of the foundation. Later stages may require similar testing to refine or extend the subsurface profile, or more sophisticated testing may be required where better estimates are needed. Field or in situ testing of rock strength is usually expensive and difficult; consequently, most such testing is reserved for projects that are large and/or have complicated or difficult foundation problems. These various in situ rock tests are listed in Table 4-4 of EM 1110-1-1804. Preliminary estimates are commonly made for soils using the methods listed in Table 3-1. Two of these methods are summarized below.

Table 3-1

Methods of Preliminary Appraisal of Foundation Strengths

Method	Remarks
Penetration resistance from standard penetration test	In clays, test provides data helpful in a relative sense; i.e., in comparing different deposits. Generally not helpful where number of blows per foot, N , is low
	In sand, N-values less than about 15 indicate low relative densities
Natural water content of disturbed or general type samples	Useful when considered with soil classification and previous experience is available
Hand examination of disturbed samples	Useful where experienced personnel are available who are skilled in estimating soil shear strengths
Position of natural water contents relative to liquid limit (LL) and plastic limit (PL)	Useful where previous experience is available
	If natural water content is close to PL, foundation shear strength should be high
	Natural water contents near LL indicate sensitive soils with low shear strengths
Torvane or pocket penetrometer tests on intact portions of general samples	Easily performed and inexpensive, but results may be excessively low; useful for preliminary strength estimates
Vane shear	
Quasi-static cone penetration	See FHWA-TS-78-209, 1977 (item 26)

a. Vane Shear Tests. The vane shear test is an in situ test and is often valuable for soft clay foundations where considerable disturbance may occur during sampling. A disturbance, especially when using conventional sampling methods, usually reduces the undrained strength of the sampled soil to a value that often would result in an uneconomical design. Because this test is performed in situ, sample disturbance is minimized. The test usually overestimates the soil's undrained strength and must be reduced by an applicable correction factor. Bjerrum (item 9) recommended a correction that is a function of the clay's plasticity index and varies as shown in Figure 3-1. Appendix D of EM 1110-2-1907 describes this test in detail. The testing procedure should be followed closely because the results can be very sensitive to the testing details. Because of the uncertainty of the results from this test, an independent method of estimating the foundation shear strength should be included in any testing program. Often unconsolidated-undrained triaxial testing of good quality undisturbed samples is a good independent check. A bracket for estimating the foundation shear strength can sometimes be established by taking corrected field vane tests as an upper bound and good quality undisturbed samples tested in undrained compression as a lower bound for shear strength.

b. Standard Penetration Test. This test is one of the most widely used methods for soil exploration in the United States. It is a means of measuring

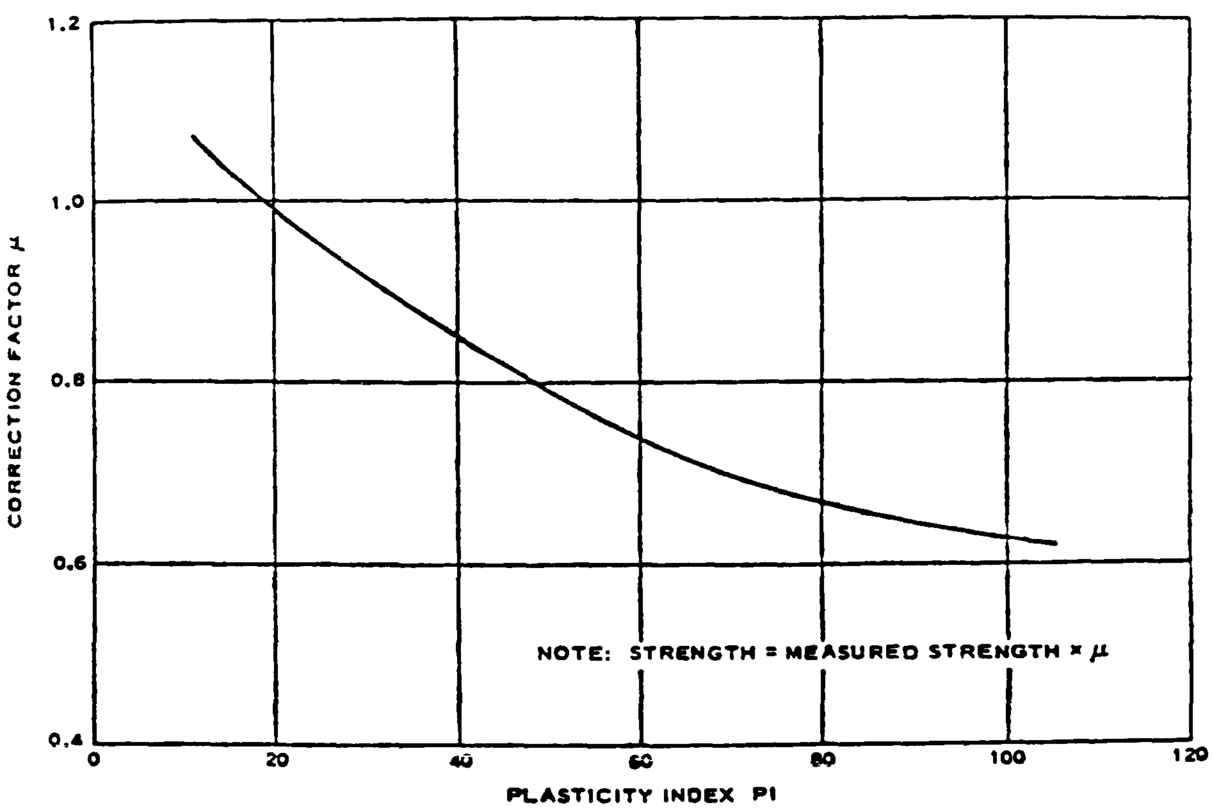


Figure 3-1. Vane shear correction chart (item 11)

the penetration resistance to the split-spoon sampler as well as obtaining a disturbed-type sample. Correlations between the penetration resistance and the consistency of cohesive soils and the relative density of granular soils have been published and are often used to estimate soil strength. These correlations are very rough and, except for the smallest of structures, should be liberally supplemented with other, better quality, strength testing. The designer should be aware of the severe limitations of using the results of this test. For example, the values obtained when testing soft clays, coarse gravels, or micaceous soils are often of little or no value. Procedures for performing the standard penetration test are given in EM 1110-2-1907.

3-8. Field Seepage Testing. The permeability of pervious foundation soils can usually be estimated with sufficient accuracy by using existing correlations with the foundation's grain-size distribution, Figure 3-2. Field pumping tests are a much more accurate means for determining the permeability of the foundation soils, especially for stratified deposits. These tests, however, are expensive and are usually justified only for unusual site

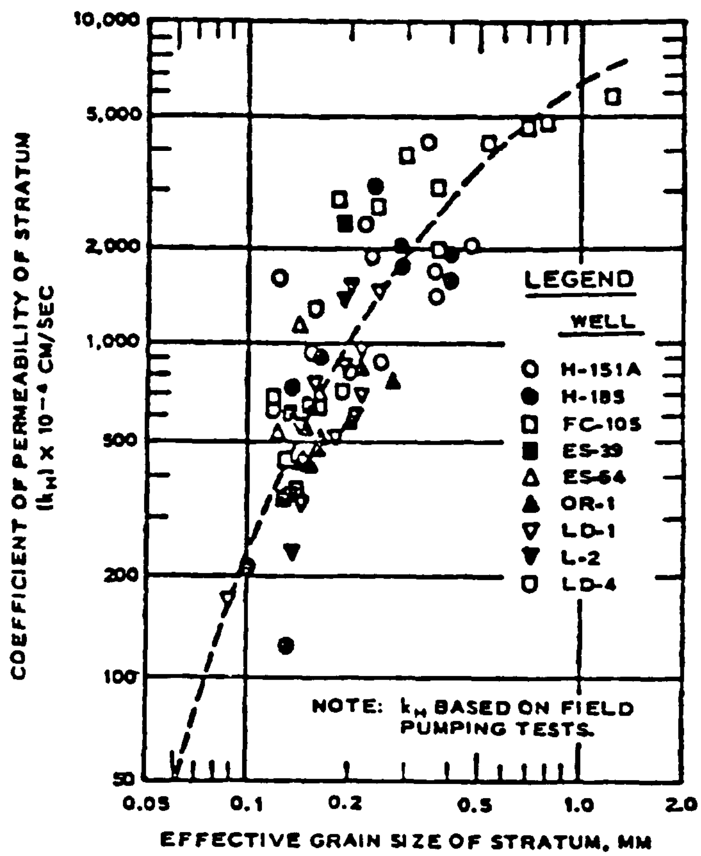


Figure 3-2. Effective grain size of stratum versus in situ coefficient of permeability. Based on data collected in the Mississippi River Valley and Arkansas River Valley (item 1)

conditions. Clay foundations are usually considered impervious for estimates of seepage quantities. However, the effects of discontinuities and thin beds of granular materials are important and should not be neglected. A number of field tests are available to measure rock mass permeability including pumping tests, tracer tests, and injection or pressure tests. The most frequently used field test is the borehole pressure test which is relatively simple and inexpensive. Among the three types of pressure tests, water, pressure drop, and air, water is the most common because it is simple to perform, is not overly time-consuming, and can be performed above or below the ground-water table. A brief description of this test is included in Paragraph 4-22 of EM 1110-1-1804. A suggested method for borehole water pressure testing is presented in the Rock Testing Handbook 381-80 (item 91).

3-9. Laboratory Testing.

a. A laboratory testing program should be designed to supplement and refine the information obtained from the subsurface investigation and field tests. The amount and type of testing depends on the type and variability of the foundation and borrow areas, the size of the structure, the consequences of failure, and the experience of the designer with local conditions. A discussion of further laboratory investigations is presented in Chapter 5 of EM 1110-1-1804.

b. Descriptions of current laboratory testing procedures are detailed in EM 1110-2-1906 and in the Rock Testing Handbook 381-80.

c. The laboratory testing program is typically performed in phases that follow the subsurface investigation program. Initially, index tests are performed on samples obtained from the exploration program. These results are then used as a basis for the selection of samples and the design of a laboratory testing program.

3-10. Index Tests.

a. Index tests are used to classify soil in accordance with the Unified Soil Classification System (Table 3-2), to develop accurate foundation soil profiles, and as an aid in correlating the results of engineering property tests to areas of similar soil conditions. Both disturbed and undisturbed soil samples should be subjected to index-type tests. Index tests should be initiated, if possible, during the course of field investigations. All samples furnished to the laboratory should be visually classified and natural water content determinations made; however, no water content tests need be run on clean sands or gravels. Mechanical analyses (gradations) of a large number of samples are not usually required for identification purposes, Atterberg limits tests should be performed on representative fine-grained samples selected after evaluation of the boring profile. For selected borings, Atterberg limits should be determined at frequent intervals on the same samples for which natural water contents are determined.

Table 3-2

Unified Soil Classification

UNIFIED SOIL CLASSIFICATION
(Including Identification and Description)

Major Divisions (1)	Major Divisions (2)		Group Symbols (3)	Typical Names (4)	Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights) (5)	Information Required for Describing Soils (6)
Coarse-grained Soils. More than half of material is larger than No. 200 sieve size. The No. 200 sieve size is about the smallest particle visible to the naked eye.	Gravels. More than half of coarse fraction is larger than No. 4 sieve size. (For visual classification, the 1/4-in. size may be used as equivalent to the No. 4 sieve size)	Clean Gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.	
		Gravels with Fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixture.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).	Give typical name; indicate approximate percentages of sand and gravel, maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).	
	Sands. More than half of coarse fraction is smaller than No. 4 sieve size.	Clean Sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain size and substantial amounts of all intermediate particle sizes.	
			SP	Poorly graded sands or gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.	
		Sands with Fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).	Example: Silty sand, gravelly; about 20% hard, angular gravel particles 1/2-in. maximum size; rounded and subangular sand grains, coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see CL below).	

Major Divisions (1)	Major Divisions (2)	Group Symbols (3)	Typical Names (4)	Identification Procedures on Fraction Smaller than No. 40 Sieve Size (5): Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)	Information Required for Describing Soils (6)
Fine-grained Soils. More than half of material is smaller than No. 200 sieve size.	Silts and Clays. Liquid limit is less than 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium	
		OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight	Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains; color in wet condition; odor, if any; local or geologic name and other pertinent descriptive information; and symbol in parentheses.
	Silts and Clays. Liquid limit is greater than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium	
		CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High	
		OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium	Example: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML).
Highly Organic Soils		Pt	Peat and other highly organic soils.	Readily identified by color, odor, spongy feel and frequently by fibrous texture.			

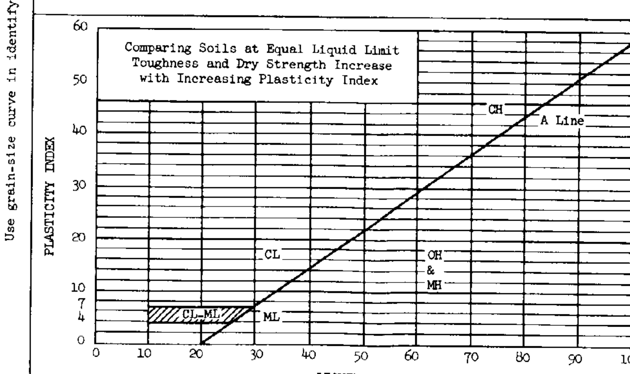
Laboratory Classification Criteria (7)

Use grain-size curve in identifying the fractions as given under field identification.

Determine percentages of gravel and sand from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows:

Less than 5% = GW, GP, SW, SP,
More than 12% = GM, GC, SM, SC.
5% to 12% = Borderline cases requiring use of dual symbols.

Group	Criteria	
GW	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
GC	Atterberg limits above "A" line with PI greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
SP	Not meeting all gradation requirements for SW	
SM	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with PI greater than 7	



PLASTICITY CHART

For laboratory classification of fine-grained soils

(1) Boundary classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder. (2) All sieve sizes on this chart are U. S. standard.

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air-drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (consistency near plastic limit)

After particles larger than the No. 40 sieve size are removed, a specimen of soil about one-half inch cube in size, is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

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b. Normally, Atterberg limits, determinations, and mechanical analyses are performed on a sufficient number of representative samples from preliminary borings to establish the general variation of these properties within the foundation, borrow, or existing fill soils. A typical boring log is shown in Figure 3-3.

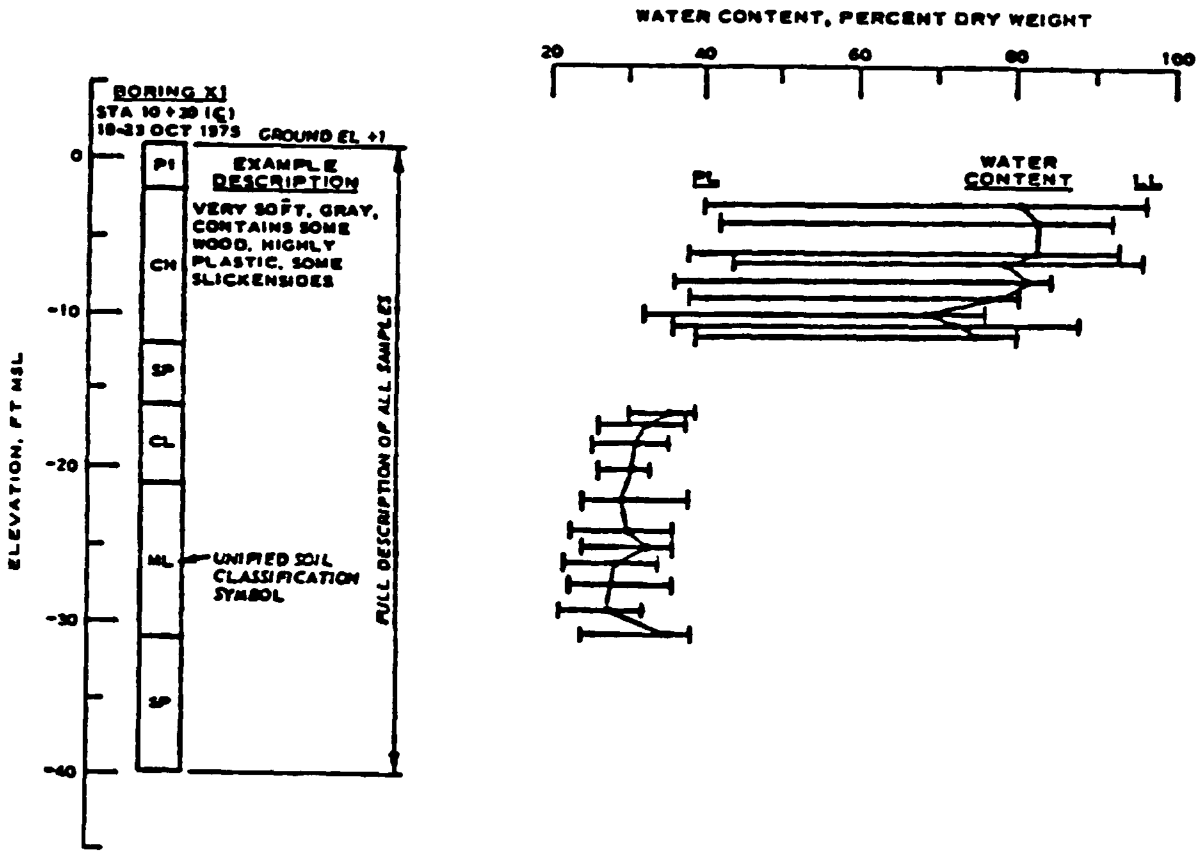


Figure 3-3. Typical boring log with results of Atterberg limits and water content tests

c. All rock cores should be logged in the field by a geologist, preferably as the cores come from the hole. A number of laboratory classification and index tests for rock are listed in Table 5-4 of EM 1110-1-1804. These tests include water content, unit weight, porosity, and unconfined compression, all of which should be performed on representative samples.

3-11. <u>Engineering Property Tests</u>. A good estimate of the strength and seepage characteristics of the foundation is necessary for an adequate foundation design. The estimate of the foundation strength is usually the most critical design parameter. Seepage characteristics are usually estimated based on the gradation of the foundation soils and an evaluation of geologic properties, especially discontinuities. The properties of the cell fill are usually estimated based on gradation analyses and the anticipated method of placement of the fill.

3-12. Permeability of Soils.

a. Fine-Grained Soils. There is generally no need for laboratory permeability tests on fine-grained fill material or clay foundation deposits. In underseepage analyses, simplifying assumptions must be made relative to thickness and soil types. Furthermore, stratification, root channels, and other discontinuities in fine-grained materials can significantly affect seepage conditions.

b. Coarse-Grained Soils. The problem of foundation underseepage requires reasonable estimates of permeability of coarse-grained pervious deposits. However, because of the difficulty and expense in obtaining undisturbed samples of sand and gravel, laboratory permeability tests are rarely performed on foundation deposits. Instead, correlations developed between grain size and coefficient of permeability, such as that shown in Figure 3-2 are generally utilized. This correlation explains the need for performing gradation tests on pervious materials where underseepage problems are indicated.

3-13. Permeability of Rock. The determination of rock mass permeability quite often depends on secondary porosity produced through fracturing and solution rather than on primary porosity of the rock. Consequently, geologic interpretations and evaluations are extremely important in determining the discontinuities that serve as ready passageways for ground-water flows.

3-14. Shear Strength--General.

a. There are three primary types of shear strength tests for soils, each representing a certain loading condition. The Q-test represents unconsolidated-undrained conditions; the R-test, consolidated-undrained conditions; and the S-test, consolidated-drained conditions. The unconsolidated-undrained strength generally governs the design of foundations on fine-grained deposits. R-tests are generally not needed for most cellular structure designs. S-tests are used where long-term stability of a fine-grained foundation is to be checked or if the soil to be tested is a granular material.

b. Q- and R-tests are performed in triaxial testing devices while S-tests are performed using direct shear and triaxial testing devices. The unconfined compression (UC) test is a special case of the Q-test in that it also represents unconsolidated-undrained conditions but is run with no confining pressure. Also, rough estimates of unconsolidated-undrained strength of clay can be obtained through the use of simple hand devices such as the pocket penetrometer or Torvane. However, these devices should be correlated with the results of Q- and UC-tests.

c. The discussion in paragraphs 3-15 and 3-16 relates the applicability of each test to the different general soil types. The applicability of the results of the different shear tests to field loading conditions and the different cases of stability are discussed in Chapter 4.

d. There are two basic types of shear strength tests utilized to obtain values of cohesion and angles of internal friction to determine strength parameters of the foundation rock: the triaxial and the direct shear. The data to determine rock strength in an undrained state under three-dimensional loading are obtained from the triaxial test. This test is performed on intact cylindrical rock samples not less than NX core size, i.e., approximately 2-1/8 inches in diameter. The direct shear test, an undrained type, is performed on core samples ranging from 2 to 6 inches in diameter. In this test, the samples are oriented such that the normal load is applied perpendicular to the feature being tested. These normal loads should be comparable to those loads anticipated in the field. Details of these tests are presented in the Rock Testing Handbook. For moisture-sensitive rocks such as indurated clays and compaction shales, soil property test procedures described in EM 1110-2-1906 should be used.

3-15. Shear Strength--Sand. Since consolidation of sand occurs simultaneously with loading, the appropriate shear strength of sands for use in design is the consolidated-drained, S-strength. However, the shear strength of sand in the foundation or cell, regardless of the method of placement, is not normally a critical or controlling factor in design. Therefore, excessive laboratory testing to determine the shear strength of sand is usually not warranted. Satisfactory approximations for most sand can also be made from correlations with standard penetration resistances and relative densities. Such correlations can be found in most standard engineering texts on soil mechanics (Figure 3-4). Seepage forces, discussed in detail in Chapter 4, can reduce the shear resistance, especially at the toe of the structure, to undesirable levels.

3-16. Shear Strength--Clay and Silt.

a. The undrained shear strength parameters should be determined for all fine-grained materials in the foundation. In areas of soft, fine-grained foundations, it is imperative that an adequate shear testing program be accomplished to establish the variation in unconsolidated-undrained shear strength with depth within the foundation (usually expressed as the ratio of undrained shear strength s_u to effective vertical stress σ'_v) as shown in Figure 3-5. A sufficient number of Q-tests, supplemented by UC tests, where appropriate, should be performed throughout the critical foundation stratum or strata. Data obtained from any field vane shear strength tests may also be helpful in establishing this variation.

b. R-tests can be helpful in estimating the variation in undrained shear strength with depth, and in determining the increase in undrained shear strength with increased effective consolidation stress. This may be necessary in estimating the gain in shear strength with time after loading.

c. The results of S-tests are used in evaluating the long-term stability of the foundation and in judging the stability of structures where pore pressure data, such as those obtained from piezometers, are available.

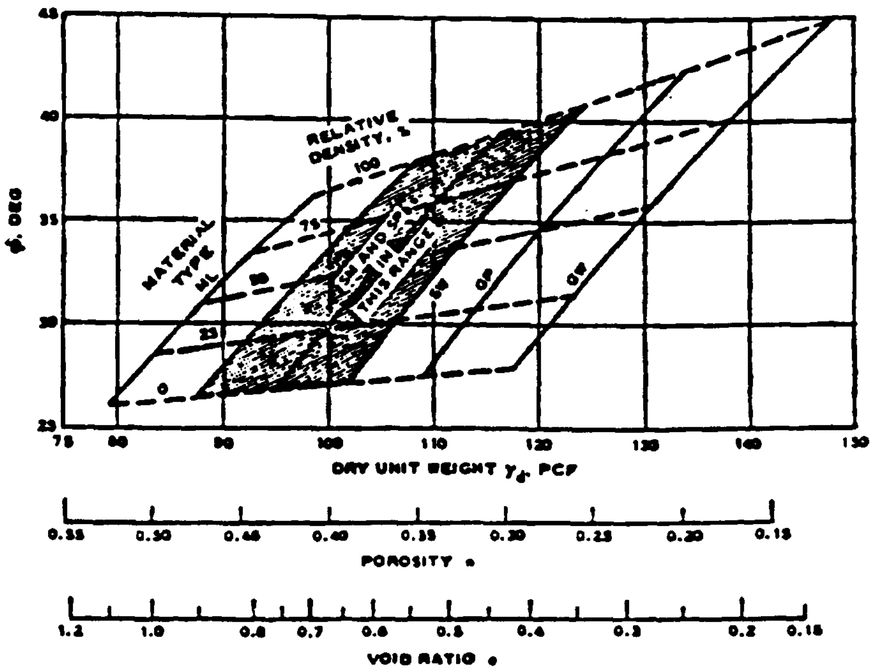


Figure 3-4. Angle of internal friction versus density for coarse-grained soils (item 50)

3-17. Procedures. Procedures for the performance of previously discussed shear tests are outlined in EM 1110-2-1906 and in Rock Testing Handbook 381-80 (item 91). In performing these tests, one should be sure that field conditions are duplicated as closely as possible. Confining pressures for triaxial tests and normal loads for direct shear tests should be chosen such that the anticipated field pressures are bracketed by the laboratory pressures based on depth and location of sample and anticipated field loadings. All samples should be sheared at a rate of loading slow enough that there will be no significant time-rate effect. The specimen size should also be chosen such that scale effects are minimized. Standard size of samples for triaxial testing of soils is 1.4 inches in diameter by 3 inches in height. However, if the sample is fissured or contains an appropriate amount of large particles such as shells, gravel, etc., then a larger size sample (2.8 inches in diameter by 6 inches in height) can be utilized in order to obtain valid results. Guidance on minimizing the effects of rate of loading, size, etc., is also contained in EM 1110-2-1906.

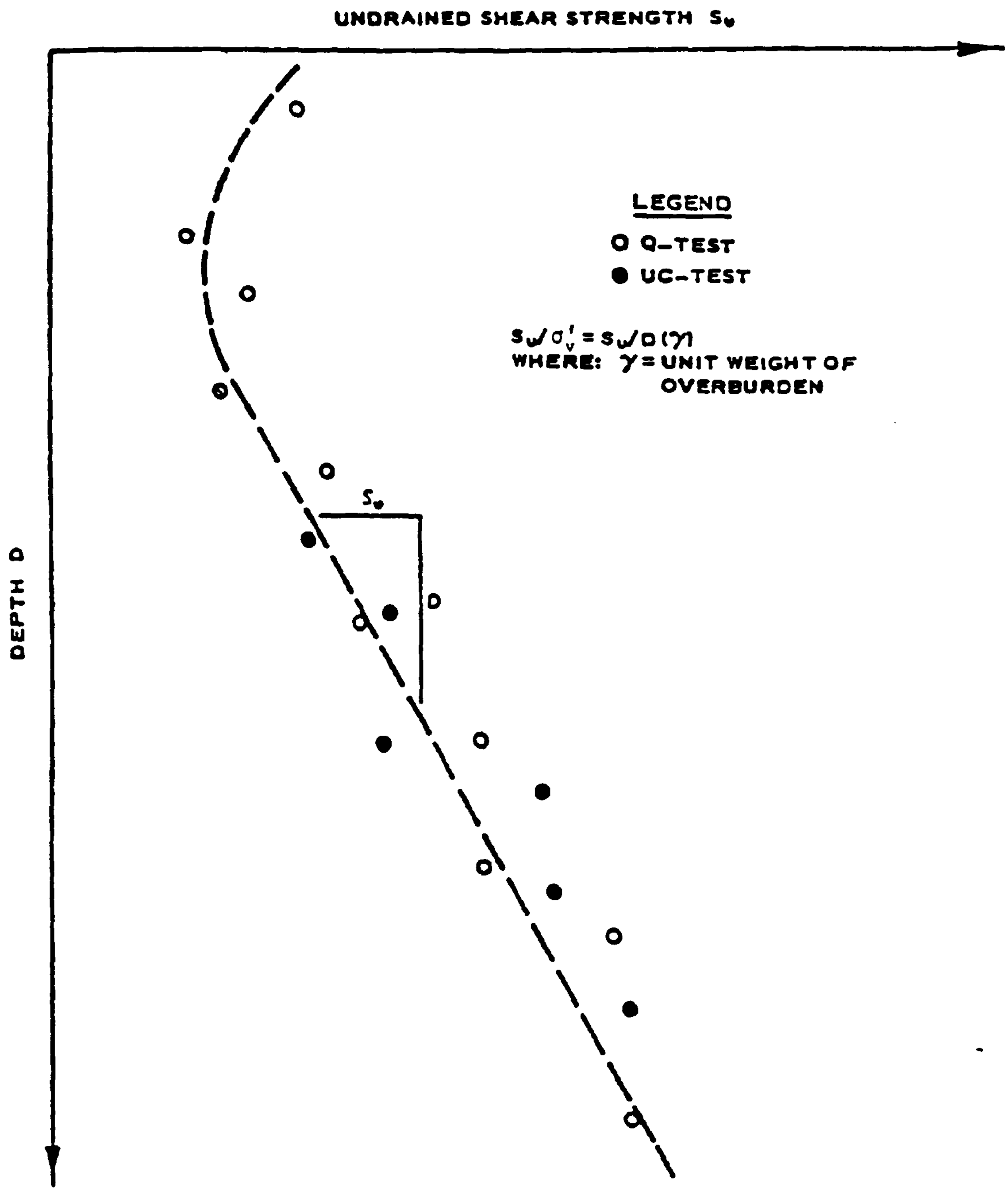


Figure 3-5. Typical plot showing variation of unconsolidated-undrained shear strength with depth

Section III. Foundation Treatment

3-18. Problem Foundations and Treatment.

a. Foundation treatment is sometimes considered for foundations with insufficient bearing capacity or problem seepage conditions. Problem seepage conditions can be the result of excessive seepage quantities or high seepage forces.

b. The following foundation treatment methods can be used to improve a deficient foundation.

(1) Removal of objectionable material. Removal may be before or after the piles are driven to form the cell.

(2) In situ compaction. Several methods are available and include vibroflotation, compaction piles, surcharge loads, and dynamic surface loads.

(3) Deep penetration of sheet piling. For design purposes, a trial penetration of two thirds of the cell height is usually considered when the cell is sited on a pervious foundation. An adjustment of this length should be based on a careful analysis of the seepage forces at the toe of the structure.

(4) Berms and blankets. Impervious blankets may be located on the outside of the cells to reduce seepage quantities and pressures. Interior berms reduce the likelihood of boiling at the toe of the structure.

(5) Consolidation. The strength of foundation material, especially fine-grained material, may be increased by consolidation. Surface preloading of the foundation and the use of sand drains are two of the methods used to accelerate consolidation of the foundation.

3-19. Grouting.

a. Correctional Methods. As for all such structures, foundation treatment should be carefully considered for cellular cofferdams. In many cases, removal of the unfavorable foundation material may be impracticable, if not impossible, and other methods of treatment must be selected. Grouting is one such method which should be considered, especially in instances where the piling of a cellular cofferdam will be driven to rock. During the evaluation of the data developed during the subsurface investigations, special note should be made of any unfavorable foundation condition that would justify at least some consideration of grouting. Such unfavorable foundation conditions might be noted as a result of evidences of solution activity such as soluble rock or drill rods dropping during drilling, open joints or bedding planes, joints or bedding planes filled with easily erodible material, faults, loss of drill fluid circulation, or unusual ground-water conditions, Generally, the problems related to such unfavorable foundation conditions can be grouped into two categories: problems related to the strength of the foundation material and problems related to the permeability of the foundation material.

b. Problems Related to Strength. Among the problems related to strength that should be anticipated are: insufficient bearing capacity, insufficient resistance to sliding failure, and general structural weaknesses due to underground caverns or solution channels, or due to voids that develop during or following construction. Problem 3 is closely related to Problems 1 and 2 and should be considered jointly. In developing parameters for allowable bearing capacity, deficiencies noted in Problem 3 must be carefully considered. All too often, rock strength parameters are used in stability analyses that are based on rock sample strengths rather than mass rock strengths. The various discontinuities that reduce the foundation rock strengths may result in consequential reductions in the ultimate bearing capacity. As mentioned above, the bedrock may contain bedding plane cavities and solution channels that can extend to considerable depth (low crossbed shear strength). In recognizing the presence of such discontinuities, the possibility must also be recognized that an unfavorable combination of these discontinuities could exist under the cellular cofferdam, thus adversely affecting the sliding stability of the structure. The presence of these weak planes must be carefully considered when doing a sliding stability analysis.

c. Problems Related to Permeability. Among the problems related to permeability that should be anticipated are: reduction in the strength of the foundation materials due to high seepage forces, high uplift forces at the base of the structure, and inability to economically maintain the coffered area in an unwatered state. In many cases, the piling of a cellular cofferdam will be driven to rock. The presumption should be that some seepage will occur not only at the piling/bedrock contact, but also through openings in the bedrock. This seepage may result in piping of materials through the bedrock openings below the cofferdam, greatly reducing the strength of the foundation. These openings along bedding planes can also result in high uplift pressures. Quite often, the vertical permeability of the rock above the open bedding plane is only a small fraction of the permeability along the plane. If such a situation exists, it is possible that the high uplift pressures will jack the foundation. The size and continuity of solution channels acting as water passageways may have a serious economic impact on the dewatering of the work area within the cofferdam. Unfortunately, there is no way to accurately estimate the dewatering problems and costs that might result from such solution channels in the foundation.

d. Selection of Treatment. Treatment of the cofferdam foundation by grouting may be used to lessen, if not eliminate, defects in the foundation, resulting in a strengthened foundation with reduced seepage; see EM 1110-2-3506. Grouting should be selected as a method of foundation treatment only after a careful and thorough evaluation of all pertinent factors. Primary factors that must be necessarily considered before selection of grouting as the method of treatment are the engineering design requirements, the subsurface conditions, and the economic aspects. Although cost is just one factor to consider, in many circumstances, cost may be the controlling factor. The cost of grouting must be weighed against such other costs as that of pumping, delays, claims, and/or failure. It may be that there is no benefit in reducing minor leakage by costly grouting.

(1) General. Information obtained and evaluated during the subsurface investigations for design of the structure should be adequate to plan the grouting program. If the grouting program is properly designed and conducted, it becomes an integral part of the ongoing subsurface investigations. A comprehensive program must necessarily take into account the type of structure, the purpose of the structure, and the intent of the grout program. As an example, foundation grouting for a cellular cofferdam is not intended to be permanent nor 100 percent effective. The program should be designed to provide the desired results as economically as possible. The program should be flexible enough to be revised during construction and performed only where there is a known need.

(2) To Strengthen. Grouting has been used on occasion to strengthen the foundation by area or consolidation grouting under the cells to increase the load-bearing capacity of the rock. This may be a viable option if the grouting is intended to increase the already acceptable factor of safety. However, if it appears that the factor of safety falls appreciably below the allowable factor of safety, total reliance should not be placed on grouting. The effectiveness of such grouting is impossible to predict or to evaluate. Certainly complete grouting is impossible because of the irregularity of the openings as well as the amount and character of any filling material.

(3) To Reduce Seepage. The principal purpose of grouting for cellular cofferdams has been in conjunction with seepage control and drainage. Curtain grouting is one method used to reduce uplift pressures and leakage under the cofferdam and thus reduce total dewatering costs. Although a single line curtain will suffice in most cases, the rock conditions may be such that it will be necessary to install a multiline curtain or a curtain with multiline segments. The exact location of the grout curtain will be influenced by a number of factors including the type of structure, the foundation conditions peculiar to the site, and the time the curtain is installed. For most cellular cofferdams, the grout curtain is located on or near the axis of the structure. However, if the curtain is not installed until the cofferdam has been constructed, it may be impracticable to drill holes through the cell fill. In this case the grout line should be moved off and just outside the cells. When installing the grout curtain, the flow of grout must be carefully controlled to prevent the grout from flowing too far, resulting in grout waste. To prevent such waste, it may be necessary to limit the quantity of grout injected, or to add a "stopper" line grouted at low pressure. The orientation and inclination of the holes should be adjusted to intercept the principal water passageways. Occasionally, however, conditions may render this impracticable and it may be that vertical holes on closer centers are more feasible.

(4) Development of Program. Following an evaluation of the foundation conditions and the selection of grouting as a method of foundation treatment, the evaluations, conclusions, and recommendations should be included in the

report of the subsurface investigations. Using data developed from the investigations, the pertinent reference manuals, and especially past experience, plans and specifications should be prepared for the grouting program. After having reviewed all available and pertinent data and having decided on the particular grouting program to be implemented, a number of basic factors must be decided: the area selected for grouting, the selection of the grout, the selection of the type of grouting, and the need for special instructions, provisions, or restrictions.

(a) Selection of Location. The area indicated for grouting should be a zone large enough to include any anticipated treatment. This is especially important in installing a grout curtain for a cellular cofferdam. This should be coincidental with provisions to provide for grouting anytime within the contract period without additional mobilization and demobilization costs to the Government. The drawings rightfully should show a grout curtain to be installed beneath the cofferdam along an approximate alignment and to definite limits. However, because of the numerous unknowns inherent in a grouting program, the plans and specifications should provide that the area of grouting extend some distance beyond the limits shown.

(b) Selection of Grout. The selection of the grout should be made only after a careful evaluation of the foundation conditions or materials being tested. The type of grout used in reducing or stopping high velocity flows would be different from that used for slow seeps, or the grout used to fill large cavities might be different from that used to fill small voids. A factor to be considered in sealing high velocity flow would be the time of set; the large quantities and costs would necessarily be considered in filling large cavities; while in filling small voids, the size of the void and the particle size of the grout are necessary considerations.

(c) Selection of Type of Grouting. Grouting may be done before, during, and/or after installation of the cofferdam or other construction activities in any given area. In the installation of a grout curtain, all or portions of the curtain may be constructed from the original ground surface and/or from floating plant in the river. If done from floating plant, in general, stop-grouting methods should be used because it is not practical to stage drill and grout from floating plant. Drilling and grouting from floating plant by the stop-grouting method should be considerably less costly than stage grouting, the holes being drilled and grouted to the bottom of the curtain in one setup.

(d) Special Instructions. In drilling from floating plant, it should be expressly understood that the depth of water penetrated will not be credited to the drilling footage for payment. If drilling and grouting are performed from the cofferdam, only drilling that is required below the original ground surface should be paid for. To effectively grout water-bearing openings associated with cavernous rock, the following general procedure should be followed: the grout holes should be drilled through the overburden and the casing should be seated a minimum of 1 foot in rock; the hole should be drilled at least 5 feet into rock, if the top of rock is lower than anticipated; if stop-grouting methods are used, grouting of the rock should be performed through a packer set just below the bottom of the casing; should a special feature be encountered in the hole, the packer setting may be varied to isolate and treat this feature. Grouting of the overburden, if necessary, can then be done immediately following the rock grouting. The specifications should provide that if, as the work progresses, supplemental grouting is required at any area within specified limits at any time, such additional grouting will be at the established contract unit prices for the items of work involved. Although pressure testing should be provided for in the specifications, the condition of the foundation may be such that all grout holes should be grouted, in which case, pressure testing would not be necessary. If at all possible, the initial dewatering of the cofferdam should be performed at the lowest possible river stage or other measures should be taken to ensure a stable cofferdam capable of being unwatered until the foundation and the adequacy of the foundation treatment can be checked.

Section IV. Sources and Properties of Cell Fill

3-20. Borrow Area. Borrow-related problems occur frequently in earth-work-related construction, and sometimes result in costly design changes and contract modifications. Special diligence during the exploration and characterization of borrow fill will be beneficial during both the design and construction of the project.

3-21. Location. Borrow areas are generally located as close to the project site as possible to reduce hauling costs. The final selection of the borrow site, however, is governed by several additional considerations.

a. Cell Fill Properties. When the most desirable cell fill is not locally available, the cost of processing or designing the structure around marginal cell fill should be compared with the increase in cost due to longer haul distances.

b. Land Use. Although cell fill is often dredged from river channels, it is sometimes desirable to locate the borrow areas outside of the river. When this occurs, special consideration and planning should be initiated to provide proper reclamation of the area.

c. Environmental Aspects. Environmental considerations may restrict the use of certain potential borrow sites. An early review of the probable borrow sites for any detrimental environmental consequences should be considered. These consequences are sometimes mitigated by placing restrictions on the use of the borrow area and by special reclamation of the site. For example, wildlife habitats or recreational areas can sometimes be created at these sites with a small additional cost.

3-22. Selection of Cell Fill.

a. Almost all modern cellular sheet pile structures are designed based on the assumption that a free-draining granular fill will be available near the construction site. Soils with less than about 5 percent of the particles by weight passing the No. 200 sieve and 15 percent passing the No. 100 sieve are usually termed free draining. Granular fills with many fines and even fine-grained fills have occasionally been used in the past; however, the poor performance of these fills usually favors use of better quality fill.

b. The performance of the sheet pile structure is directly related to the drainage characteristics of the cell fill. Free-draining fill will have a lower seepage line within the fill than less pervious material. The lower seepage line improves the cell performance by:

(1) Reducing the sheet pile interlock force. (Reducing this force is especially beneficial for high cells or where marginal material is used. However, a reduction in the interlock force may reduce the stiffness of the structure, with slightly larger structural movements.)

(2) Increasing the effective stress at the base of the cell, increasing lateral sliding resistance.

(3) Increasing the internal shear resistance.

Section V. Seepage Control

3-23. <u>Seepage Through Cell</u>.

a. The location of the free water surface in a cell is usually estimated using empirical relationships based on the type of cell fill. The recommendations in Figure 3-6 serve as a guide and starting point for estimating the location of the seepage line. These recommendations are conservative for most applications; however, each design should be evaluated for conditions that would tend to raise the seepage line. If both the quality of the cell fill and the assurance of proper inspection cannot be guaranteed during the design of the project, full saturation of the cell should be considered for design purposes. Some conditions that require evaluation are:

(1) Possible leakage from pipelines crossing the cells.

(2) Waves overtopping the outboard piles.

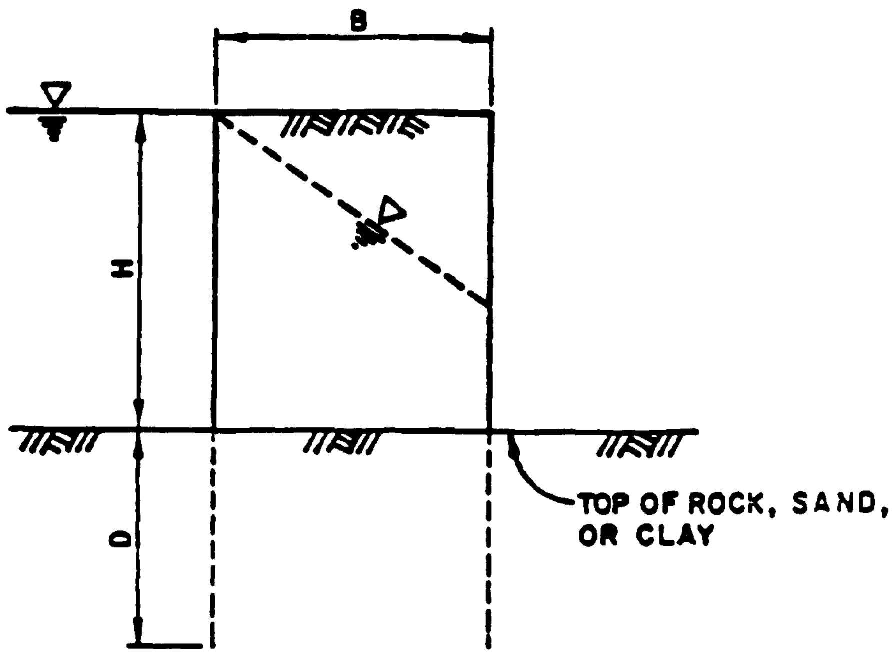
(3) Excessive leakage through the outboard piles.

(4) Poor drainage through the inboard piles.

(5) Lower permeability than expected of the cell fill.

(6) Hydraulic filling of cell fill.

b. The quantity of seepage through the cell is a function of both the tightness and integrity of the outboard piles and the type of cell fill, the chief barrier being the outboard piling. The tightness of the outboard piling



SLOPE OF FREE-WATER SURFACE IN CELLS DEPENDS ON PERMEABILITY OF CELL FILL UNLESS SPECIAL DRAINAGE IS PROVIDED AND SLOPE IS CONTROLLED, ASSUME THE FOLLOWING:

- FREE-DRAINING COARSE-GRAINED FILL (GW, GP, SW, SP): SLOPE 1 VERTICAL TO 1 HORIZONTAL
- SILTY COARSE-GRAINED FILL (GM, GC, SM, SC): SLOPE 1 VERTICAL TO 2 HORIZONTAL
- FINE-GRAINED FILL: SLOPE 1 VERTICAL TO 3 HORIZONTAL

Figure 3-6. Estimate of free water location in fill

depends on the physical condition of the piling and the piling interlock force. An increase in seepage through the cell can generally be expected when:

(1) Second-hand piling is used. New piling in good condition should be considered for major structures. For other structures, used piling may be considered when either seepage conditions are slight or pose little threat to the safety to the structure.

(2) Rough driving is experienced during construction. The foundation exploration program should investigate conditions that lead to rough driving. Contract specifications, discussed in Chapter 7, should restrict hard driving.

(3) The interlock forces are small. The increase in seepage due to this condition is usually small, and is usually not considered.

3-24. Foundation Underseepage.

a. Foundation underseepage is generally not a problem for structures built on clay or good quality rock foundations. Problems almost always are confined to coarse-grained soil such as gravel and sand and sometimes silty materials. The most treacherous conditions occur where undetected pervious seams exist in the foundation.

b. Cofferdams on sand are often designed using a trial sheet pile penetration of two thirds of the height of the structure above-the dredgeline. A flow net is most often used to estimate the seepage forces. If the exit gradient at the toe of the structure is large, a loaded filter or a wide-base berm should be considered.

c. Depending on the site conditions, up to 50 percent of the passive resistance, even with 2/3H penetration, at the toe can be lost due to seepage forces. This loss increases the possibility of excessive penetration of the inboard piles. Methods and criteria for seepage control are discussed in Chapter 5.

Section VI. Seismic Considerations

3-25. Structure-Foundation Interaction. The susceptibility of cellular structures to damage due to earthquake loadings depends on the complex interaction of the structure and the foundation. Structural design for dynamic loading is reviewed in Chapter 4. In addition to these loads, a reduction in strength of the foundation, cell fill, or backfill behind a cellular bulkhead can also simultaneously occur during an earthquake. Structures founded on saturated, cohesionless materials or cohesive soils that contain lenses of saturated, cohesionless soil can lose practically all of their foundation support when subjected to a vibratory loading, such as an earthquake. Similarly, the cell fill or the backfill can also liquefy, increasing the lateral loading against the cell.

3-26. Liquefaction Potential.

a. The significant factors influencing the liquefaction potential of the foundation or fill include: soil type, relative density or void ratio, initial confining pressure, intensity of ground shaking, and duration of ground shaking. The vulnerability of liquefaction-susceptible foundations can be initially estimated using simplified methods and charts that incorporate the most important variables that contribute to liquefaction.

b. Seed and Idriss (item 67) and Christian and Swiger (item 17) discuss these methods. Figures 3-7 and 3-8 define conditions where liquefaction is: very likely to occur, not very likely to occur, or a marginal condition exists where additional factors or further analysis should be considered. Charts of this nature are frequently updated and improved. For this reason, more recent material should be consulted for marginal or complex conditions. An estimate of the degree of seismic activity in the region can be obtained from ER 1110-2-1806.

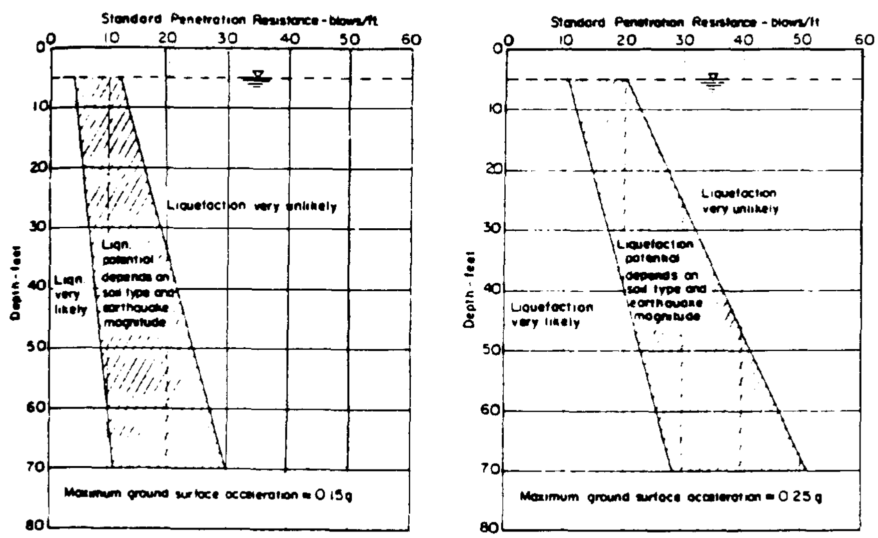


Figure 3-7. Liquefaction potential evaluation charts for sands with water table at depth of about 5 feet (item 67)

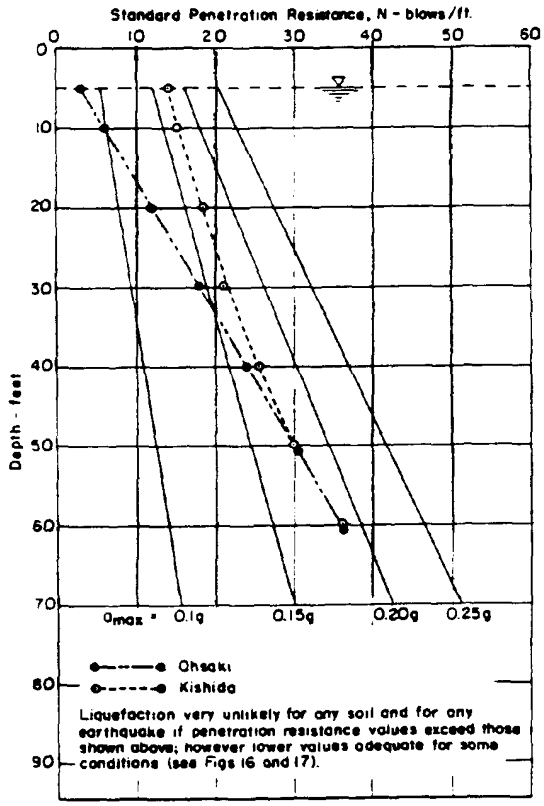


Figure 3-8. Penetration resistance values for which liquefaction is unlikely to occur under any conditions (item 67)