CHAPTER XIII

GEOTECHNICAL

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1. **PURPOSE.** The purpose of the following criteria is to provide information that will clarify and supplement the standard criteria and design guidance for geotechnical investigations and for the development and presentation of Foundation Design Analyses and Pavement Design Analyses.

1.1 **Metrication.** The metric units used are the International System of Units(SI)adopted by the U.S. Government as described in Chapter I, Paragraphs 3 and 4.2.1.

2. **REFERENCES.**

NOTE: Army Technical Manuals, Engineer Manuals, Engineer Regulations, and Engineer Technical Letters are available from Headquarters, U.S. Army Corps of Engineers on the Internet at http://www.hnd.usace.army.mil/techinfo/index.htm.

2.1 **Army Technical Manuals.**

2.1.1 TM 5-809-12, Concrete Floor Slabs on Grade Subjected to Heavy Loads, 25 Aug 1987.

2.1.2 TM 5-818-1, Soils and Geology Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), 21 Oct 1983.

2.1.3 TM 5-818-6, Grouting Methods and Equipment, 27 Feb 1970.

2.1.4 TM 5-818-7, Foundations in Expansive Soils, 1 Sep 1983.

2.1.5 TM 5-818-8, Engineering Use of Geotextiles, 20 Jul 1995.

2.1.6 TM 5-822-5, Pavement Design for Roads, Streets, Walks, and Other Open Storage Areas, 12 Jun 1992.

2.1.7 TM 5-822-7, Standard Practice for Concrete Pavements, 16 Aug 1987.

2.1.8 TM 5-822-8, Bituminous Pavements - Standard Practice, 30 Jul 1987.

2.1.9 TM 5-822-9, Repair of Rigid Pavements Using Epoxy Resin Grouts, Mortars, and Concretes, 20 Jan 89.

2.1.10 TM 5-822-10, Standard Practice for Pavement Recycling, 26 Aug 88.

2.1.11 TM 5-822-11, Standard Practice for Sealing Joints and Cracks in Rigid and Flexible Pavements, 16 Jun 93.

2.1.12 TM 5-822-13, Pavement Design for Roads, Streets, and Open Storage, 24 Oct 1994.

2.1.13 TM 5-822-14, Soil Stabilization for Pavements, 25 Oct 1994.

2.1.14 TM 5-825-1, General Provisions for Airfield/Heliport Pavements Design, 9 Mar 1994.

2.1.15 TM 5-825-2-1, Flexible Pavement Design for Airfields (Elastic Layered Method), 27 Nov 1989.

2.1.16 TM 5-825-3, Rigid Pavements for Airfields, 11 Aug 1988.

2.2 **Engineer Manuals**.

2.2.1 EM 1110-1-1802, Geophysical Exploration for Engineering and Environmental Investigations, 31 Aug 1995.

2.2.2 EM 1110-1-1804, Geotechnical Investigations, 29 Feb 1984.

2.2.3 EM 1110-1-1904, Settlement Analysis, 30 Sep 1990.

2.2.4 EM 1110-1-1905, Bearing Capacity of Soils, 30 Oct 1992.

2.2.5 EM 1110-1-1906, Soil Sampling, 30 Sep 1996.

2.2.6 EM 1110-1-2908, Rock Foundations, 30 Nov 1994.

2.2.7 EM 1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads, 30 Jun 95.

2.2.8 EM 1110-2-1810, Coastal Geology, 31 Jan 95.

2.2.9 EM 1110-2-1902, Stability of Earth and Rock-fill Dams, 1 Apr 1970.

2.2.10 EM 1110-2-1909, Calibration of Laboratory Soils Testing Equipment, 1 Dec 1970.

2.2.11 EM 1110-2-1913, Design and Construction of Levees, 31 Mar 96.

2.2.12 EM 1110-2-2006, Roller Compacted Concrete, 01 Feb 92.

2.2.13 EM 1110-2-2502, Retaining and Flood Walls, 29 Sep 89.

2.2.14 EM 1110-2-2504, Design of Sheet Pile Walls, 31 Mar 94.

2.2.15 EM 1110-2-2906, Design of Pile Foundations, 15 Jan 1991.

2.2.16 EM 1110-2-3301, Design of Beach Fills, 31 Mar 95.

2.2.17 EM 1110-2-3506, Grouting Technology, 20 Jan 1984.

2.2.18 EM 1110-2-3800, Systematic Drilling and Blasting for Surface Excavation, 01 Mar 72.

2.3 **Engineer Regulations.**

2.3.1 ER 1110-1-1807, Procedures for Drilling in Earth Embankments, 30 Sep 1997.

2.3.2 ER 1110-1-8100, Laboratory Investigations and Testing, 31 Dec 1997.

2.3.3 ER 1110-2-8152, Planning and Design of Temporary Cofferdams and Braced Excavations, 31 Aug 94.

2.3.4 ER 1110-3-104, Family Housing Design, 30 Jun 94.

2.3.5 ER 1110-34-1, Transportation Systems Mandatory Center of Expertise, 10 Jan 1990.

2.4 **Engineer Technical Letters.**

2.4.1 ETL 1110-1-125, Guidance for Fuel Resistant Sealers for Pavement, 4 May 1984.

2.4.2 ETL 1110-1-129, Use of Engineering Fabrics and Asphalt Rubber Interlayer to Minimize Reflective Cracking in Pavements, 15 Dec 1985.

2.4.3 ETL 1110-1-138, Standard Penetration Test, 31 Mar 1988.

2.4.4 ETL 1110-1-139, Selecting Asphalt Cements, 22 Jun 1990.

2.4.5 ETL 1110-1-141, Thickness Design of Roller-Compacted Concrete Pavements for Airfields, Roads, Streets, and Parking Areas, 29 Jan 1988.

2.4.6 ETL 1110-2-282, Rock Mass Classification Data Requirements for Rippability, 30 Jun 1983.

2.4.7 ETL 1110-2-300, Characterization and Measurement of Discontinuities in Rock Slopes, 31 Oct 1983.

2.4.8 ETL 1110-3-393, Design of Surfaced Areas, 28 Oct 1988.

2.4.9 ETL 1110-3-394, Aircraft Characterizations for Airfield/Heliport Design and Evaluation, 27 Sep 1991.

2.4.10 ETL 1110-3-435, Drainage Layers for Pavements, 1 May 1992.

2.4.11 ETL 1110-3-471, Design and Construction of Conventionally Reinforced Ribbed Mat Slabs (RRMS), 25 Sep 1995.

2.4.12 ETL 1110-3-474, Cathodic Protection, 14 Jul 1995.

2.2.13 ETL 1110-3-475, Roller Compacted Concrete Pavement Design and Construction, 10 Oct 1995.

2.4.14 ETL 1110-3-486, Army Airfield/Heliport Pavement Design, 3 Nov 1997.

2.4.15 ETL 1110-3-487, Use of Petroleum Contaminated Soil in Cold Mix Asphalt Stabilized Base Course, 1 Mar 1998.

2.4.16 ETL 1110-3-488, Design and Construction Management Practices for Concrete Pavements, 01 Mar 98.

2.4.17 ETL 1110-9-10(FR), Cathodic Protection System Using Ceramic Anodes, 05 Jan 91.

2.5 **Miscellaneous.**

2.5.1 Annual Book of ASTM Standards, American Society of Testing and Materials.

2.5.2 Munsell Soil Color Charts (standard), Part No. 50216; Supplementary (tropical and subtropical), Part No. 50021; GretagMacbeth, New Windsor, NY, (914)565-7660 or (800)622-2384.

3. **GEOTECHNICAL INVESTIGATIONS.**

3.1 **Scope of Investigations.**

3.1.1 Preconcept and Site Selection Studies. Geotechnical investigations during preconcept and site selection studies should be performed to a level that insures adequate information on general subsurface conditions and any special treatment or foundation requirements such as deep foundations. This information should be sufficiently complete to permit selection of the most favorable site within the study area, determine the general type of structure that would be best suited to the site conditions, assess the geotechnical aspects of environmental impact, and ascertain the costs of the project. The scope of the investigations should not be greater than that scope necessary to accomplish these goals. For projects on existing military installations much of the geotechnical information needed for preconcept and site selection studies will be available and additional investigations will be minimal. Results of geotechnical investigations should be compiled in summary reports.

3.1.2 Concept Studies. Geotechnical investigations for concept studies should advance the information to that required for design and budget development that would constitute approximately 35 percent of total design. Reporting of the results of geotechnical investigations presents additional emphasis on selection of foundation types and the influences of subsurface conditions.

3.1.3 Final Design Studies. Geotechnical investigations for final design should provide additional information to the preconcept and concept investigations for a complete design. Final design studies provide a complete set of working drawings, technical specifications, design analyses, and detailed cost estimate for the project. Reporting of the geotechnical investigations will place further emphasis on analyses for selection of foundation types and details of the foundation design.

3.2 **Survey of Available Information.** Information obtained from previous geotechnical investigations may be available and pertinent to the proposed project, especially if the proposed project is located on a military installation. District archives contain boring logs, laboratory test data, and foundation and paving design analyses from previous investigations. The supervising District can provide access to this information.

3.3 **Field Investigations.**

3.3.1 Location and Protection of Underground Utilities.

3.3.1.1 General. The location of underground utilities must be determined and those utilities, and all other utilities, protected from possible damage during drilling and excavating activities.

3.3.1.2 Drilling Permit. A permit is required prior to drilling or excavating on any military installation. This permit is available from the Base Civil Engineer (Air Force) or from the Department of Public Works (Army). Two weeks should be allowed for processing to obtain this permit. Coordination for utility clearances will accompany approval of the permit; electrical (both overhead and underground), gas, steam, water, storm sewer, wastewater (sanitary) sewer, and cable TV will usual be located upon receipt of permit. Telephone lines are the responsibility of the Signal Corps (Army) and may require separate notification. Fuel lines near flight lines (Air Force) may not be located during processing of the permit and may require assistance from flight line personnel.

3.3.1.3 Utility Clearances. Clearances must be obtained from individual utilities prior to drilling or excavating at sites not on military installations. The project site must also be checked for interstate high-pressure gas lines and communication cables.

3.3.1.4 Protection of the Environment. After the locations for proposed borings have been determined, route of access to the area and specific boring locations should be selected with care in order to minimize damage to the environment. For military projects, environmental clearances, including archeology clearances, may be obtained from the BCE or the DPW.

3.3.2 Borings.

3.3.2.1 Location and Spacing.

Borings spaced in a rigid pattern often do not disclose unfavorable subsurface conditions; therefore, boring locations should be selected to define geological units and subsurface non-conformities. Borings may have to be spaced at 40 feet or less when erratic subsurface conditions are encountered, in order to delineate lenses, boulders, and other irregularities. When localized building foundation areas are explored, initial borings should be located near building corners, but locations should allow some final shifting on the site. The number of borings should never be less than three and preferably five: one at each corner and one at the center, unless subsurface conditions are known to be uniform and the foundation area is small. These preliminary borings must be supplemented by intermediate borings as required by the extent of the area, location of critical loaded areas, subsurface conditions, and local practice.

3.3.2.2 Depth of Borings. The required depth of exploration may be only 1.5 to 3 meters (5 to 10 feet) below grade for residential construction and lightly loaded warehouses and office buildings, provided highly compressible soils are known to not occur at greater depths. For important or heavily loaded foundations, borings must extend into strata of adequate bearing capacity and should penetrate all soft or loose deposits even if overlain by strata of stiff or dense soils. The borings should be of sufficient depth to establish if groundwater will affect construction, cause uplift, or decrease bearing capacity. When pumping quantities must be estimated, at least two borings should extend to a depth that will define the aquifer depth and thickness. Borings may generally be stopped when rock is encountered or after a penetration of 1.5 to 6 meters (5 to 20 feet) into a strata of exceptional stiffness. To assure that boulders are not mistaken for bedrock, rock coring for 1.5 to 3 meters (5 to 10 feet) is required. When an important structure is to be founded on rock, core borings should penetrate the rock sufficiently to determine quality and character and the depth and the thickness of the weathered zone. Rock coring is expensive and slow, and the minimum size standard core diameter should be used that will provide good cores. NX or larger core sizes may be required in some rock strata. Core barrels can remove cores in standard 1.5-, 3-, 6-meters (5-, 10-, and 20-feet)lengths; actual core may be much fractured, however. Detailed exploration should be carried to a depth that encompasses all soil strata likely to be significantly affected by the structure loading. If the structure is not founded on piles, the significant depth is about 1½ to 2 times the width of the loaded area.

3.4 **Sampling**.

3.4.1 General. The sampling program may depend on drilling equipment available and on the laboratory facilities where the tests will be performed.

3.4.2 Recommended Undisturbed Sample Diameters.

TEST	Minimum Sample Diameter, mm (in)
Unit weight	76 (3.0)
Permeability	76 (3.0)
Consolidation, 2.75-inch	76 (3.0)
Consolidation, 4-inch	127 (5.0)
Unconfined compression	76 (3.0)
Triaxial compression *	127 (5.0)
Direct shear	127 (5.0)

* Triaxial test specimens are prepared by cutting a short section of 127-mm (5-inch) sample axially into four quadrants and trimming each quadrant to the proper size so that all specimens represent the same depth.

3.4.3 Recommended Minimum Sample Quantity.

TEST	Minimum Sample Dry Weight kg (lb)*
Water content	0.2 (0.5)
Atterberg limits	0.2 (0.5)
Shrinkage limits	0.2 (0.5)
Specific gravity	0.1 (0.2)
Grain-size analysis	0.2 (0.5)
Proctor Compaction	13.5 (30.0)
Permeability	0.9 (2.0)
Direct shear	0.9 (2.0)
Consolidation, 70-mm (2.75-in)	0.7 (1.5)
Consolidation, 102-mm (4-in)	0.9 (2.0)
Triaxial, 36-mm (1.4-inch) (4 points)	0.9 (2.0)
Triaxial, 72-mm (2.8-inch) (4 points)	4.5 (10.0)

* Fine grained material, all minus No.4 sieve.

3.5 **Field Tests.**

3.5.1 Standard Penetration Test. The standard penetration test (SPT) is literally a standard of the industry for soil sampling. Reference for this test is ASTM D 1586.

3.5.2 Cone Penetrometer. The cone penetrometer is less popular than the standard penetration test, but is an acceptable method of testing in situ materials. Use of the cone penetrometer should be expected to require 2 holes per boring location, especially if undisturbed samples are obtained.

3.5.3 Pocket Penetrometer. The pocket penetrometer should be used to estimate the relative consistency of cohesive soils from a specific boring in order to provide an accurate description of the soil. Readings from pocket penetrometers should not be used for design.

3.5.4 Soil Resistivity Test. Soil resistivity tests are performed to provide an estimate of the corrosive nature of the soils at a site in order to design cathodic protection. The soil resistivity test should be performed in accordance with ASTM G 57 and the instructions of the equipment manufacturer.

3.6 **Groundwater Observations.**

3.6.1 Borehole Observations. Water levels during and immediately after drilling should be measured and recorded on the field log of the boring with the date and time of the water level measurements and the date of the boring. The water level after 24 hours should also be measured and recorded on the log. Water level observations made in a borehole during or shortly after drilling may be misleading.

3.6.2 Observation Wells. Observation wells provide an accurate means for determining the groundwater level over a period of time. A temporary observation well could be constructed of 38-mm (1½-inch) diameter plastic pipe with slotted end placed in the borehole. The top few meters of the borehole should be sealed with tamped backfill to seal the borehole from surface infiltration.

3.7 **Inspection.**

A field inspector will be present during drilling and should be an experienced engineering geologist or geotechnical engineer. The duties shall include observing, classifying, and describing geologic materials; selecting and preserving samples; logging and disposition of core samples; completing the boring logs; and recording information and data from field tests.

3.8 **Boring Logs**.

3.8.1 Field Logs.

A field log for each boring can provide an accurate and comprehensive record of the stratigraphy and lithology of soil and rock encountered with other relevant information obtained during drilling, sampling, and field testing. A field log will be prepared for each boring. A field log will be prepared for each excavation, which has the purpose of characterizing subsurface materials and geologic conditions. All field boring logs will be prepared in the inspector's own handwriting. All logs will provide the pertinent data for the borings including, but not limited to, name of project, boring location, drilling organization, boring number, name of drilling organization, name of driller, inclination of boring, size and type of drilling bit, date boring was started and date completed, elevation of top of hole, type and manufacturer's designation of drill, and number of samples and core boxes obtained.

3.8.2 Reproducible Boring Logs. Final logs for inclusion in design documents and in plans and specifications will be composed in the Computer-Aided Design and Drafting (CADD) System specified in the contract for services. Forms, symbols, and other graphic aids for preparation of the reproducible (CADD) boring logs are contained within the geotechnical cell library of the A/E/C CADD Standards Manual. Chapter VIII - Drawings, gives additional guidance.

3.9 **Geophysical Explorations.**

Geophysical explorations are not prohibited but should not be the main investigative technique and must be correlated with drilling and sampling.

3.10 **Investigations along Proposed Utility Routes.**

3.10.1 General. The primary purpose of investigations along proposed utility routes is to delineate common (soil) excavation from rock excavation for contract bidding. Visual logs prepared during excavations can provide considerable information. Samples of the excavated soil or rock can be obtained and submitted for classification tests.

3.10.2 Equipment. Conventional drilling equipment may be too costly and cumbersome to provide borings at selected locations along routes of the proposed utilities. Less expensive and more adaptive equipment and methods for obtaining shallow excavations in soil and soft rock include small locally available backhoes for the excavation of test pits and small locally available bulldozers and trenching machines for the excavation of shallow trenches. Power and manual augers and posthole diggers can be carried onto any location and can obtain samples from small shallow borings.

3.11 **End of Field Investigations.**

3.11.1 General. At the close of field investigations and related activities, the site will be restored to its initial condition. All boreholes, test pits, trenches, and other excavations must be backfilled.

3.11.2 Soil Backfill. Boreholes or excavations may be backfilled with random soil from borehole cuttings or excavation material, or from an offsite borrow source. The quality of the backfill material must be sufficient to prevent water movement or collapse. The soil backfill should be tamped to minimize additional settlement.

3.11.3 Grouting. To grout boreholes, the borehole should be grouted by injection through a grout pipe inserted to the bottom of the borehole, which will displace the water or drilling mud and fill the borehole with a continuous column of grout. The grout should contain bentonite or similar swelling material to inhibit shrinkage and ensure a good seal. A grout mixture of about 4 to 7 percent bentonite and 93 to 96 percent Portland cement is suitable for sealing boreholes. Sand may be added to the grout as filler if the proper mixing and pumping equipment is available.

3.11.4 Concrete. Concrete may be used for backfilling boreholes if a shrinkage inhibitor is added. Concrete should be placed in the bottom of the borehole by the tremie method to prevent segregation of the mixture and to ensure that water or drilling mud is displaced and the borehole is filled with a continuous column of concrete.

3.11.5 Special Considerations. Boreholes located near dams or levees, and boreholes located in areas of hazardous pollutants or in environmentally sensitive areas require special considerations for backfilling. For these sensitive locations, special instructions will be provided by the supervising District.

3.12 **Disposition of Samples**.

3.12.1 Care and Handling of Samples. All samples of soil and rock shall be properly sealed and stored on the project site until transport to the testing laboratory. Special provisions are required during winter operations to prevent the samples from freezing. Undisturbed samples shall be transported in carriers in such manner as to prevent disturbance. Special cushioned racks are required to transport unopened samples from Denison barrel and other soil-coring samplers and to transport Shelby-tube or other thin-walled push samples. These samples must be transported vertically and with the top of the sample up. Undisturbed samples for classification and index tests must be sealed to preserve the natural moisture content. Upon arrival at the testing laboratory and after being logged in to the laboratory records, all samples will be stored in a moist room until time for preparation prior to testing.

3.12.2 Disposition of Soil Samples. Soil samples may be discarded once the testing program for which they were taken is complete. Soil samples are not normally retained for long periods of time because even the most careful sealing and storing procedures cannot prevent the physical and chemical changes that, in time, would invalidate any subsequent test results. This does not pertain to soil samples taken for other than traditional geotechnical purposes; soil samples taken for chemical content for environmental testing will require special considerations and instructions from the supervising District.

3.12.3 Disposition of Rock Samples. In general, rock cores will be retained until the detailed logs, photographs, and test data have been made a matter of permanent record.

4. **LABORATORY TESTING**

4.1 **General Considerations**.

4.1.1 Classification. Laboratory testing determines index values for identification and correlation by means of classification tests. Laboratory testing further defines the engineering properties in parameters usable for design of foundations. The Unified Soil Classification System, based on identification of soils according to grain-size distribution, plasticity characteristics, and grouping with respect to behavior, will be used to classify soils in connection with foundation and pavement design. The geological classification of rock is complex, and for most engineering applications, a simplified system of classification, as presented in TM 5-818-1 (Reference 2.1.2), is adequate.

4.1.2 Guidance for Assigning Laboratory Tests. Guidance for assigning laboratory tests for developing foundation design parameters for buildings, other structures, and pavements is available in TM 5-818-1, Soils and Geology Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), 21 Oct 1983.
(Reference 2.1.2).

4.1.3 Reference Standards. Procedural methods for laboratory testing of geotechnical samples shall be as outlined in the specifications of the respective standard of the American Society for Testing and Materials (ASTM).

4.2 **Index and Classification Tests**.

TEST	REMARKS
Water content	Required for every sample except clean sands and gravels.
Atterberg limits	Required for every stratum of cohesive soil; always have associated natural water content of soil tested and compute liquidity index.
Grain-size analysis	Generally performed on sands and gravels with occasional tests on cohesive soils. Correlate with Atterberg limits for cohesive soils.
Slaking test	Performed on highly preconsolidated clays and clay shales where deep excavations are to be made or foundations will be near-surface. Wet and Dry cycles should be used.
Penetrometer	Performed on cohesive soils, undisturbed samples and intact chunks of disturbed samples. Regard results with caution; use mainly for consistency classification and as guide for assigning shear tests.

4.3 **Engineering Property Tests - Soils**.

4.3.1 Shear Strength.

4.3.1.1 Unconfined Compression Tests. Unconfined compression tests are performed on samples of cohesive soils, cemented soils (i.e., cement-stabilized soil), and (soft) rock. The test specimen is usually cut directly from a length of extruded sample from a thin-walled sampler or from a core barrel. Although test results may indicate a broad scatter, unconfined compression tests are the most common laboratory test to determine the strength of cohesive soils.

4.3.1.2 Triaxial Compression Tests. Triaxial compression tests are performed under three conditions of test specimen drainage. Tests corresponding to these drainage conditions are: unconsolidated-undrained triaxial (UU or Q) tests in which the water content is kept constant during the test; consolidated-undrained triaxial (CU or R) tests in which consolidation or swelling is allowed under initial stress conditions, but the water content is kept constant during application of shearing stresses; and consolidated-drained triaxial (CD or S) tests in which full consolidation or swelling is permitted under the initial stress conditions and also for each increment of loading during shear. The appropriate triaxial test should be selected to reflect the various prototype loading cases and drainage conditions. Normally, fine-grained soils are not subjected to consolidated-drained triaxial (CD or S) tests, but instead are subjected to direct shear tests.

4.3.1.3 Direct Shear Tests. Direct shear tests are performed on fine-grained soils instead of consolidated-drained triaxial (CD or S) tests. The value from the direct shear test is set as the angle of internal friction and the cohesion intercept is assumed to be zero.

4.3.1.4 Selection of Design Shear Strengths. Where the results from shear tests on undisturbed foundation soils and compacted soils do not show a significant drop in shear or deviator stresses after peak stresses are reached, the design shear strength can be chosen as the peak shear stress in direct shear tests, the peak deviator stress, or the deviator stress at 15 percent strain where the shear resistance increases with strain. For each soil layer, design shear strengths should be selected such that two-thirds of the test values exceed the assigned design values.

4.3.2 Consolidation and Swell. The parameters required to perform settlement and rebound analyses are obtained from consolidation tests on highly compressible clays or on compressible soils subjected to high stresses. The sequence and magnitude of test loading should approximate the various loading cases for which settlement and rebound analyses are to be performed. For expansive soils, the standard consolidation test or a modification of this test may be used to estimate both settlement and swell. Consolidometer swell tests tend to predict minimal levels of heave. Soil suction tests can be used to estimate swell, but tend to overestimate heave compared to field observations.

4.3.3 Compaction Tests.

4.3.3.1 Cohesive Soils. The modified Proctor compaction test (ASTM D 1557) will be the laboratory test to evaluate the compaction characteristics of cohesive borrow material or cohesive material from required excavations to be used as borrow. Traditionally, the modified Proctor compaction test has been used in military construction to correlate the relative compaction of fills and backfills for site grading, structural backfill, and pavement subgrades and bases.

4.3.3.2 Cohesionless Soils. The modified Proctor compaction test (ASTM D 1557) will be the laboratory test to evaluate the compaction characteristics of cohesionless borrow material or cohesionless material from required excavations to be used as borrow. The relative density tests for cohesionless soils (ASTM D 4253 and ASTM D 4254) have fallen into disfavor because of the inability to consistently reproduce the minimum density (ASTM D 4254).

4.4 **Engineering Property Tests - Rock**.

4.4.1 Unconfined Compression Test. For building foundation evaluation, the unconfined, uniaxial compression test is performed primarily to provide the unconfined compressive strength of a rock sample. The unconfined compressive strength can be used to provide allowable bearing capacity and to provide rippability for excavation.

4.4.2 Shales and Moisture-Sensitive Rocks.

4.4.2.1 General. Most moisture-sensitive rocks are sedimentary in origin or are their metamorphic equivalents. These rocks range from undurated clays to compaction shales, poorly to moderately cemented sandstones, and the earthy rock types such as marl. As these rocks have soil-like characteristics, the index properties of these rocks should be determined prior to more comprehensive testing. The results of the index testing will usually indicate the engineering sensitivity of the rocks, and should be used as a guide to further testing.

4.4.2.2 Triaxial and Direct Shear Tests. Most triaxial and direct shear tests conducted on hard, brittle rock samples are of the undrained type. For hard, brittle rock, pore pressures do not play a dominant role, and strength values are in terms of total stress. However, as softer rock types are encountered with correspondingly higher absorption values (e.g., greater than 5 percent), the role of pore pressure buildup during the rock shearing process begins to become more important. The same condition is true for many clay shales and other similar weak and weathered rock types. For moisture-sensitive rocks, soil property tests should be used when possible. Critical pore pressures that may substantially reduce the net rock strength can be monitored throughout the entire testing cycle.

4.4.2.3 Test Data Interpretation. Laboratory test data on shales and moisture-sensitive rocks should be interpreted with caution. The laboratory undrained strength of intact specimens is rarely representative of in-place field shear strengths. Frequently, shales, clay shales, and highly overconsolidated clays are reduced to their residual shear strength with minor displacements. The geotechnical explorations, laboratory testing, and review of field experiences must establish whether residual or higher shear strengths are appropriate for design. Results of laboratory tests should be confirmed by analysis of the field behavior of the material from prior construction experience in the area, analysis of existing slopes or structures, and correlation with similar geologic formations at sites where the field performance is known.

5. FOUNDATION DESIGN ANALYSIS

5.1 Engineering Evaluation.

5.1.1 Bearing Capacity Analysis.

Reference 2.2.4, EM 1110-1-1905, Bearing Capacity of Soils

5.1.1.1 General. The shearing strength of soil, s_u, is a function of cohesion, c, of the soil, the angle of internal friction, ϕ, and confining pressure, p. Estimation of the shearing strength is usually as: $s_u = c + p \tan \phi$. For cohesionless soils and for cohesive soils in long-term analyses, neither of which are affected by pore pressures, the effective angle of internal friction, ϕ', should be used (effective stress). For cohesive soils in short-term analyses, which are affected by pore pressures, the angle of internal friction, ϕ, should be used (total stress).

5.1.1.2 Preliminary Analyses. For cohesionless soils, estimate ϕ' from standard penetration tests (N-values) or cone penetration resistance. For cohesive soils and for short-term analysis, estimate s_u from standard penetration tests. For cohesive soils and long-term loading, estimate ϕ' from correlation with index properties for normally consolidated soils.

5.1.1.3 Detailed Design Analyses. For cohesionless soils, estimate ϕ' from standard penetration tests (N-values) or cone penetration resistance. For cohesive soils and for short-term analysis, determine s_u from unconsolidated-undrained (UU or Q) triaxial tests on undisturbed samples with confining pressure, σ_3, equal to overburden pressure. For long-term analysis, obtain ϕ' from drained direct shear tests on undisturbed samples. For transient loadings after consolidation obtain ϕ and c parameters from consolidated-undrained (CU or R) triaxial tests with pore pressure measurements on undisturbed samples.

5.1.1.4 Clay-Shale. The allowable bearing capacity of clay-shale and other soft, moisture-sensitive rock should be developed using the same procedures as for cohesive soils.

5.1.1.5 Factors of Safety. Factors of safety for design of structures on soils depend on the extent and detail of subsurface information. Typical factors of safety for design are presented in Table XIII-1.

TABLE XIII-1 Typical Factors of Safety

Structure	FS
Public buildings	3.5
Light industrial building	3.5
Apartments, offices	3
Warehouses (superflat floors)	>3
Warehouses (typical)	2.5
Footings	3
Mats	>3
Deep foundations	
With load tests	2
Driven piles (dynamic analysis)	2.5
Without load tests	3
Multilayer soils	4
Groups	3
Retaining walls	3
Temporary braced excavation	>2

5.1.1.6 Rock. The allowable bearing capacity of hard, massive rock should be developed from the results of unconfined compression tests on core samples. For estimating bearing capacity of the rock, a factor of safety of at least 10 is traditionally used.

5.1.2 Settlement or Consolidation.

Reference 2.2.3, EM 1110-1-1904, Settlement Analysis

5.1.2.1 Standard Analyses. For cohesionless soils, use design charts relating Standard Penetration Test (SPT) results or cone resistance with soil pressure and settlement. For cohesive soils, estimate the virgin compression index, C_c, from lab test data for the liquid limit, LL, natural water content, W_n, and initial void ratio e_o.

5.1.2.2 Detailed Analyses. For cohesionless soils, use the Schmertmann Approximation method with Standard Penetration Test (SPT) results or cone resistance. For cohesive soils, develop consolidation parameters from the results of consolidation tests on selected samples.

5.1.3 Slope Stability.

Reference 2.2.9, EM 1110-2-1902, Stability of Earth and Rock-fill Dams, 1 Apr 1970.

5.1.3.1 General. Stability Analyses are required on excavation slopes and embankment slopes. Guidance in this segment is for slopes in the soils routinely encountered within Southwestern Division. Slopes in soils that present special problems, such as stiff-fissured clays and shales, hydraulic fills, dredged material, and loess, and special loading conditions, such as earthquakes, are outside of the scope of this guidance.

5.1.3.2 Cohesionless Slopes on Firm Soil or Rock. The stability of slopes consisting of cohesionless soils depends on the angle of internal friction, ϕ, of the soil, the slope angle, the unit weight of the soil, and pore pressures. Slope failure normally occurs by surface raveling or shallow sliding. Where consequences of failure may be important, required slopes can be determined using simple infinite slope analysis. Values of ϕ' for stability analyses are determined from laboratory tests or estimated from the density of the sand. Correlation with SPT values can provide reasonable strength values. Values of $\phi = 25$ degrees for loose sands and $\phi = 35$ degrees for dense sands are conservative for most cases of static loading. If higher values are used these higher values should be justified by the results from R or S tests. Pore pressure due to seepage reduces slope stability, but static water pressure, with the same water level inside and outside the slope, has no effect. Benches, paved ditches, and planting on slopes can be used to reduce runoff velocities and to retard erosion. Saturated slopes in cohesionless soils may be susceptible to liquefaction and flow slides during earthquakes, while dry slopes are subject to settlement and raveling.

5.1.3.3 Cohesive Slopes Resting on Firm Soil or Rock. The stability of slopes consisting of cohesive soils depends on the strength of soil, the unit weight of the soil, the slope height, the slope angle, and pore pressures. Failure usually occurs by sliding on a deep surface tangent to the top of firm materials. For relatively high slopes that drain slowly, it may be necessary to analyze the stability for three limiting conditions.

5.1.3.3.1 Short-Term or End-of-Construction Condition. Analyze this condition using total stress methods, with shear strengths determined from unconsolidated-undrained (UU or Q) tests on undisturbed samples. Shear strengths from unconfined compression tests may be used but generally may show more scatter. This condition is often the only one analyzed for stability of excavated slopes. The possibility of progressive failure or large creep deformations exists for safety factors less than about 1.25 to 1.50.

5.1.3.3.2 Long-Term Condition. If the excavation is open for several months or years, it may be necessary to analyze this condition using effective stress methods, with strength parameters determined from consolidated-undrained (CU or R) tests or consolidated-drained (CD or S) tests on undisturbed samples. Pore pressures are governed by seepage conditions and can be determined using flow nets or other types of seepage analysis. Both internal pore pressures and external water pressures should be included in the analysis.

5.1.3.3.3 Sudden Drawdown Condition. Analyze this condition using total stress methods, with shear strengths measured in R and S tests. Shear strength shall be based on the minimum of the combined R and S envelopes. This case is not normally encountered in excavation slope stability.

5.1.3.4 Effect of Soft Foundation Strata. The critical failure mechanism is usually sliding on a deep surface tangent to the top of an underlying firm layer. Short-term stability of an embankment over soft foundation strata is usually more critical than long-term stability. The strength of soft clay foundation strata should be expressed in terms of total stresses and determined using unconsolidated-undrained (UU or Q) tests on undisturbed specimens.

5.1.3.5 Methods of Stability Analysis. For simple slopes of excavations or embankments, the use of slope stability charts will provide adequate estimates of factors of safety. For complex slope geometry and complex layering of materials, the use of limit-equilibrium methods or finite element methods are required.

5.1.4 Dewatering and Groundwater Control. The evaluation and design of dewatering and groundwater control should be based on appropriate references and guidance in technical literature, field investigations, pump tests and seepage analysis as are appropriate.

5.2 **Selection of Recommended Foundation Type.**

5.2.1 General Considerations. Selection of an appropriate foundation depends upon the function of the structure, soil and groundwater conditions, construction schedules, construction economy, and other factors. Preliminary information concerning the purpose of the structure, loads, and subsurface conditions can be used to evaluate alternative types of foundations. Estimates of the total and differential foundation movements should be developed and their effect on the proposed structure should be evaluated.

5.2.2 Spread Footings.

5.2.2.1 Adequate Depth of Footings. The footing should be placed below the frost line because of volume changes that occur during freezing and thawing, and also below the depth where seasonal volume changes occur. The minimum depth below which seasonal volume changes do not occur is usually 1.2 meters (4 feet), but varies with location. On sloping ground, the footings should be placed at a depth such that they will not be affected by possible erosion.

5.2.2.2 Allowable Bearing Capacity. The allowable bearing capacity should be estimated from the strength of the foundation material and the appropriate factor of safety (Table XIII-1). In some instances, the allowable bearing capacity will be governed by the allowable settlement.

5.2.2.3 Settlement of Footings on Cohesive Soils. If the settlement is expected to occur in strata beneath the footings to a depth equal to the distance between the footings, a settlement analysis should be made assuming the footings are independent of each other. Compute settlements for the maximum bearing pressure and for lesser values. If significant settlements can occur in strata below a depth equal to the distance between footings, the settlement analysis should consider all footings to determine the settlement at selected footings. Depending on the nature of subsurface conditions, it may or may not be possible to proportion footings to equalize settlements. The possibility of proportioning footing areas can be determined only on the basis of successive settlement analyses. If the differential settlements between footings are excessive, change the layout of the footings, use a mat foundation, or use piles or drilled piers. If foundation soils are nonuniform horizontally, the settlement analysis should be made for the largest footing, assuming that it will be founded on the most unfavorable soils disclosed by the field investigations, and for the smallest adjacent footing. The results of these settlement analyses should be presented in charts, which relate settlement, footing size, bearing pressures, and column loads. Proper footing sizes can readily be determined from such charts when the allowable settlement is known. After a footing size has been selected, compute the factor of safety with respect to bearing capacity for dead load plus maximum live load.

5.2.2.4 Settlement of Footings on Cohesionless Soils. The settlement of footings on cohesionless soils is generally small and will take place mostly during construction. Consideration should be given to the potential for saturation of the cohesionless foundation soils at some future time. Saturation of cohesionless foundation soils will cause, at that time, additional settlement that will be in excess of the initial settlement.

5.2.3 Drilled Piers.

5.2.3.1 Bearing Depth. Drilled piers must be founded on firm, relatively incompressible material. This material varies dramatically within Southwestern Division. Selection of the bearing depth should be based on the results of field investigations and lab testing and from investigation and evaluation of the performance of existing structures founded on drilled piers. Often, the shallow bearing depth of drilled piers and their exaggerated bells require bearing capacity analysis as spread footings. These shallow drilled piers are actually spread footings constructed in auger holes rather than wooden formwork. Special considerations are required to establish the bearing depth of drilled piers in areas of expansive foundation soils.

5.2.3.2 Allowable Bearing Capacity. The allowable end bearing capacity should be estimated from the strength of the foundation material at the bearing depth, and the appropriate factor of safety (Table XIII-1). The allowable shaft resistance should be estimated if the foundation material along the pier shaft will provide a continuous resistance to the pier load. If the foundation material along the pier shaft is compressible, an additional load should be expected on the pier as that foundation material consolidates. Often the pier depth is sufficiently shallow relative to the pier width that the analysis should be as for a spread footing; the bearing capacity factor, N_C, may be less than 9.

5.2.3.3 Expansive Foundation Soils. In areas of expansive foundation soils, drilled piers must bear on strata below the depth of the active zone and on firm, relatively incompressible materials that have relatively stable moisture contents. The depth of this active zone in central and north Texas and in central Oklahoma is as much as 15 feet. The depth can be estimated locally by observing the relationship of moisture content to plastic limit of the foundation soils. Conversely, the selected bearing depth can be too great in areas of expansive foundation soils, particularly in areas of deep, soft clay-shales such as the San Antonio Area and western Oklahoma. At depth, these expansive clay-shales are moisture-deficient. Drilled piers at great depths provide a conduit for moisture into the deep clay-shale and heave of the drilled piers can be expected. In San Antonio, the bearing depth of drilled piers is usually to a dense basal gravel layer immediately overlying the clay-shale. The potential uplift force due to shaft adhesion from the expansive soils in the active zone should be computed and provided for foundation design. Recommendations should also be provided on anchoring the drilled piers by socketing into underlying rock or by pier weight and supported loading. Structurally the pier shaft must have enough reinforcing steel to resist the potential tension that may come from the expansive soils in the active zone. Either the minimum shaft tension steel area required or the maximum potential heave from the expansive soils should be provided for foundation design.

5.2.3.4 Structurally Supported Floors. Buildings in areas of expansive foundation soils and on drilled piers should have supported structural slabs for interior floors. The structural slab may be a cast-in-place slab on carton forms. Grade beams between drilled piers should also be constructed on carton forms to provide voids below the grade beams. Precast planks/tees and bar joists can be used to support the floor slab. Precast planks/tees can be used with or without a crawl space, but bar joists must have a crawl space for ventilation to prevent corrosion of the steel bar joists. However, in areas of expansive foundation soils, a crawl space would be required to provide space for heave of the active zone. If appropriate, these items should be addressed in the geotechnical report (Foundation Design Analysis).

5.2.3.5 Construction Considerations. Special considerations should be provided for anticipated situations which could develop during construction: use of casing, tremie placement of concrete, inspection of pier, obstacles to underreaming, and increase in reinforcing steel to compensate for heave.

5.2.4 Pile Foundations. Bearing piles are deep foundations used to transmit foundation loads to rock or soil layers having adequate bearing capacity to support the structure and to preclude settlement resulting from consolidation of soil above these layers. When the bearing strata are below the groundwater table, and when off-shore structures are being built, piles may be the most economical type of deep foundation available because they do not require dewatering of the site. Piles also may be used to compact cohesionless soils and to serve as anchorages against lateral thrust and vertical uplift. The selection, design, and placement of pile foundations are presented in detail in EM 1110-2-2906, Design of Pile Foundations (reference 2.3.15).

5.2.5 Ribbed Mat Slabs. Ribbed mat slabs have been used extensively to provide a cost-effective foundation for a variety of structural and architectural systems. While competent structural performance has been achieved, many ribbed mat projects have experienced significant cosmetic cracking of floor slabs. This is typically due to volumetric shrinkage of slabs with large lateral dimensions and restraint created by the stiffening beams during curing of the concrete. (see the structural chapter of this AEIM for a description of design measures to control shrinkage cracking.) The selection of the foundation type should include aesthetic considerations. In general, ribbed mat slabs should not be the preferred foundation system for sites with low or non-expansive soils, or buildings with exposed concrete floors where noticeable shrinkage cracking will be objectionable. A ribbed mat slab is often the most economical foundation for sites with expansive soils. However, if the building has exposed concrete floors, a ribbed mat slab may not be appropriate because of the potential for visible cracking in the floor. Administration and barracks buildings typically have carpet or vinyl covering on the floor and the tight shrinkage cracks typically do not result in aesthetic or structural problems. In tactical maintenance shops, warehouses, fire stations, and other similar buildings, ribbed mat slabs may be an acceptable design choice, even if the slabs contain a limited amount of visible, tight cracks on the exposed concrete floors. Detailed guidance has been developed and published for the development and presentation of geotechnical parameters for design of ribbed mat slabs. That detailed guidance is presented in SWD Engineer Technical Letter dated 16 April 1987, Criteria for Developing Geotechnical Design Parameter for CESWD Ribbed Mat Design Methodology. Access to this ETL may not be universal so the ETL has been incorporated into this AEIM as Section 5.3, below.

5.3 **Geotechnical Parameters for Ribbed Mat Foundations.**

5.3.1 Soil-Structure Interaction Modes. Two heave-induced deformation conditions appropriate for ribbed mat slab structural analysis is center lift and edge lift.

5.3.1.1 Center Lift. Center lift refers to doming of the foundation in the interior area of a slab-on-grade with heave differential to the perimeter area as depicted in Figure 1. This may be caused either by drying of the expansive subgrade around the perimeter beam or by wetting of the expansive subgrade in the interior. Loss of support along perimeter and first interior transverse stiffener beams results if the magnitude of center-lift heave is large enough and the beams are sufficiently rigid to cantilever from the supported interior region.

5.3.1.2 Edge Lift. Edge lift involves more complex soil-structure interactions than does center lift. In edge lift, the structure is supported by heaving subgrade at the perimeter and in the relatively moisture-stable interior. Loss of support develops when the edge-lift deformation is large enough and the spanning beam is sufficiently rigid. Edge lift is depicted in Figure 2.

5.3.1.3 Analyses. Soil-structure interaction within the interior-supported region is reasonably represented as a beam on non-linear subgrade. Soil-structure interaction at the perimeter is more complex because the soil deflects under the structural load as a bean on non-linear subgrade, but also the swelling soil either loads and/or deflects the beam upward. To further complicate matters, the amount of edge-lift heave and the soil-beam interface pressure are interrelated and unique for each specific site. Structural analyses are particularly sensitive to edge-lift parameters (edge-lift heave magnitude and limiting beam-soil interface pressure). For example, large values for these may cause the solution to either fail to converge, or indicate that the beam must be very deep and/or very heavily reinforced. Analyses of site conditions may sometimes dictate massive, very rigid stiffener beams, which are not generally necessary. Estimates of edge-lift heave of less than 25 mm to 40 mm (1.0 to 1.5 inches) during design analysis produce reasonable and constructible beams.

5.3.2 Center Lift Parameters for Structural Design.

5.3.2.1 General. Center lift parameters to be provided in the foundation design analysis include (1) modulus of subgrade reaction (K_1), (2) design allowable bearing for beams (q_{all}), (3) magnitude of center lift (Y_{MCL}),and (4) loss of support distance around the perimeter (L_{MCL}).

5.3.2.2 Modulus of Subgrade Reaction. The modulus of subgrade reaction should be taken as K_1 = 200 pci for beams up to 12 inches wide and bearing on compacted, nonexpansive fills consisting of gravel, crushed rock, or limestone screenings, or on cement-stabilized materials if these materials extend significantly ($D \geq 3B$) below the stiffener beam of width B. The foundation design analysis should direct that K_1 values be factored to account for width effects such that $K_{DESIGN} = K_1/B$, where B is the effective beam width in feet for soil-structure interaction. Note that the resultant effective beam width may include a significant width of the slab and is therefore significantly greater than actual beam width. Structural design calculations are not sensitive to variations in K values.

5.3.2.3 Design Allowable Bearing. A design allowable bearing value (q_{all}) has historically been assigned for sizing of stiffener beams, perimeter beams, and enlarged beam intersections beneath columns. Bearing values typically consider the beam to be a continuous strip footing or the beam intersection to be a spot footing and carrying either line or concentrated loads, respectively. The allowable bearing value is typically developed based on the average strength of engineered fill at shallow depth with a factor of safety of not less than 3.0. Design loads typically include full dead load plus one-half live load. The purpose in sizing the beams and beam intersections for this design allowable is to provide uniform contact pressures at the beam-soil interface therefore limiting differential settlement. The assumptions of minimal load sharing between the slab and beams, ample safety factor on the fill strength, and minimum beam widths specified in Chapter IV (Structural) of this AEIM combine to limit the mobilized soil strains to low levels. This leads to very small structurally induced deflections given uniform, nominal fill depths. Actual values assigned for design bearing capacities have seldom exceeded q_{all} = 95 kPa (2.0 KSF) although values as high as 145 kPa (3.0 KSF) have been assigned in limited cases where required and justifiable. Seldom are there structural requirements for larger allowable bearing values since specified minimum beam widths generally govern.

5.3.2.4 Magnitude of Center Lift. The magnitude of center lift heave potential (Y_{MCL}) given in the foundation design analysis should be the residual heave potential at the site. The value of Y_{MCL} should include effects due to subgrade removal and replacement, any effects due to fill above original subgrade, and the weight of the proposed structure. Maximum design value for center-lift potential should not exceed 40 mm (1.5 inches). Where attainable with reasonable removal/replacement depths $\leq$ 1 meter (36 inches), Y_{MCL} should be limited to not more than 25 mm (1.0 inch), which is well within the "tolerable" deformation range of most structures. The minimum remove/replace depth should be taken to the bottom elevation of the ribbed mat slab beams. The heave potential is determined by three soil parameters: the coefficient of swell (C_s), depth of active zone (X_a), and expansion pressure (P_{exp}).

5.3.2.4.1 Coefficient of Swell. Caution should be used in selecting coefficient of swell (C_s) values for heave analyses since swell pressure test results significantly underestimate (C_s) values compared to controlled expansion-consolidation-rebound tests. Additionally, both test methods tend to give low (C_s) values since most rebound time curves are terminated well before primary swell is completed.

5.3.2.4.2 Depth of Active Zone. An appropriate design value of the depth of the active zone (X_a) typically lies between the present depth to the stable relative moisture content (estimated by observing the relationship of moisture content to the plastic limit) and the maximum depth observed, such as the maximum depth of weathering. Typical (X_a) values for the central and north Texas regions and for the central Oklahoma region appear to vary from about 3 to 4.5 meters (10 to 15 feet). These values have been estimated for regression heave analyses for distressed structures and for depth of moisture variation versus approximate return/duration interval studies. Values smaller than 4.25 meters (14 feet) may be applicable in specific cases such as where the active zone is the distance between the structural foundation element or slab on grade and a perched water table, a condition common in these regions. Center lift analyses should consider "saturated" conditions to a depth of X_a. If a nominal remove/replace depth and saturated subgrade assumptions indicate unreasonable residual heave potential, consider increasing the depth of remove/replace and/or recommending a more defensive design to prevent saturation of the subgrade.

5.3.2.4.3 Expansion Pressure. Expansion pressures should be developed versus depth using small depth intervals. These should be developed from laboratory data for the site. Additionally, these data may be supplemented using proper correlations with nearby, and preferably adjacent, sites.

5.3.2.5 Edge Moisture Variation Distance. The edge moisture variation distance (L_{MCL}) may control the design of the interior stiffener beams that are adjacent to the perimeter. The maximum moments and shear are induced in the transverse beams when these elements cantilever free of foundation support from the interior supported region to the outside of the perimeter beam. The length of cantilever is largely controlled by the value of L_{MCL}. This concept was adopted from Post-Tensioning Institute (PTI) guidelines, originally developed for lightly loaded flexible mats. Standard practice in the San Antonio area has been to assign upper or near upper bound values from the Thornthwaite Moisture Index (TMI) for design limit L_{MCL} values. The Thornthwaite Moisture Index for Southwestern Division is presented on Figure 3. The Thornthwaite Moisture Index (TMI) versus Edge Moisture Variation Distance (L_{MCL}) is presented as Figure 4. The actual edge moisture variation distance is moderated by relatively deep perimeter beams which act as physical barriers and by the non-expansive fill which tends to make changes in moisture content (and therefore any resultant heave or shrinkage) more uniform and provide a surcharge effect as well. The very short return interval of edge moisture variation events presented in TMI, and reported by some sources to range from 1 to 2 years, may not provide an adequate estimate of the return interval for project design. The typical project design life exceeds 20 to 30 years, and may well exceed 50 years. Estimated edge moisture variation values considering a 100 percent probability of experiencing a 20 to 30-year return interval event may well be twice typical TMI values. Based on a subjective combination of all factors, it is suggested that L_{MCL} be taken as the edge moisture variation distance determined using Figures 3 and 4. These values should be modified, either up or down, based on site specific geotechnical investigations and engineering judgment.

5.3.3 Edge Lift Parameters for Structural Design.

5.3.3.1 General. Edge lift parameters to be provided in the foundation design analysis include (1) modulus of subgrade reaction (K_1), (2) magnitude of edge lift heave (Y_{MEL}), (3) limiting soil-beam interface pressure (P_{sw}) for that portion of the beam being acted on by the heaving subgrade, and (4) a value for edge moisture variation distance (L_{MEL}).

5.3.3.2 Modulus of Subgrade Reaction. Values of modulus of subgrade reaction given for center lift are considered appropriate for edge lift.

5.3.3.3 Soil-Beam Interface Pressure and Magnitude of Edge Lift. The limiting soil-beam interface pressure (P_{sw}) and magnitude of edge lift potential (Y_{MEL}) are related, and the analysis for solution determines both simultaneously. As edge lift develops and loss of support occurs between the perimeter and interior regions, the heaving soil may well exert a pressure on the stiffener beams well in excess of typical design interface pressures (q_{all}). As the soil column swells and lifts the overlying beam, the soil-beam contact area increases toward the interior region to accommodate the greater structural reaction. The soil-structure interaction in the edge lift region can be visualized as a three-component system: (1) a structural element (a beam or mat strip), (2) an element of nonexpansive fill beneath the structural element plus that piece of the expansive subgrade restrained against heave by the weight of the overlying fill, and (3) the heaving column of soil to a depth of X_a beneath the bottom of the nonexpansive fill blanket (Figure 5). The load-deformation relationship of element 1 interacting with element 2 can be represented by a P-Y curve as shown in Figure 6. The load-deformation relationship of element 3 interacts with elements 1 and 2 in the column immediately below the beam as shown on Figure 7. The plot consists of the net heave potential of the swelling soil column versus those forces resisting the tendency to swell, taken at the base of the structural beam. These relationships can be added algebraically to produce a composite p-y curve that can be easily utilized by available soil-structure interaction programs for structural analysis. Since such analysis is within the purview of the structural engineer, the geotechnical engineer need only furnish the pressure heave relationship in useable form in the foundation design analysis. This information should be provided in a tabulated format giving coordinates for at least three points. These minimum three points should be the P_{sw} and Y_{MEL} coordinates for (1) pressure equal to P_{ult}, (2) pressure equal to P_{all}, and (3) pressure equal to zero.

5.3.3.4 Edge Moisture Variation Distance. Edge moisture variation distance (L_{MEL}) for edge lift analysis may be taken from the TMI chart given in Figure 8. The TMI values represent approximate environmentally induced events. As a result, upper bound values should be selected for design. It is recommended, however, that average values be used for all SWD projects. Additionally, recommendations should be made in the foundation design analysis to limit the potential for developing "hot spots" due to long term sources of free water around the building perimeter.

5.3.3.5 Excepted Structures. The analysis of certain structure-site situations may warrant deleting edge-lift analyses:

* Pre-engineered metal building without interior masonry walls or heavy interior dead or permanent live loads.
* Structures in which defensive design efforts have been incorporated and reasonable confidence exists that these will be constructed and maintained as intended.
* Structures in which minor architectural distress (such as cracking of masonry walls, plaster walls, tiled surfaces) is not likely to cause undue user concern or raise maintenance requirements significantly.

5.3.3.6 Structural Design of Ribbed Mat Slabs. Guidance on use of Geotechnical parameters for structural design of the ribbed mat slab is in Chapter IV, Structural.

6. PAVEMENT DESIGN ANALYSIS

6.1 Airfields and Heliports.

Military transportation systems designs for all airfields, railroads, ports, and special vehicle guideways and roadways will be performed through the Transportation Systems Mandatory Center of Expertise. Design criteria and special instructions will be provided by TSMCX

6.2 Roads, Streets, and Open Storage Areas.

6.2.1 Design. The design of roads, streets, and open storage areas will be in accordance with the applicable technical manuals or special instructions. New concepts and materials, such as roller-compacted concrete, paving blocks, and asphalt additives, are encouraged when the benefits have been documented and cost reductions can be shown. New concepts and materials should be applied only after a thorough review and approval by HQUSACE (CEMP-E). Roads and streets must be approached as individual problems. The pavement design will be based on the maximum loads and traffic anticipated for each individual segment or general use, or both, in the road and street system. In addition to pneumatic-tired vehicles, some roads and streets will be required to sustain traffic of half- or full-track vehicles having variable weights. Flexible pavements for roads and streets for tracked vehicles will be based on current criteria for high-pressure tires. The design of rigid pavements will require particular attention to joint types and spacing, and reinforcement due to a variety of conditions.

6.2.2 Computer Aided Design. Software for computer aided design has been developed by Waterways Experiment Station. The software is based on the guidance given in TM 5-822-5, Pavement Design for Roads, Streets, Walks, and Other Open Storage Areas (reference 2.1.16).

6.2.3 Type of Pavement. The type of pavement to be considered for vehicular traffic will be determined by the intended use and by the initial and maintenance costs. Rigid pavements are required in certain critical areas including: (1) aprons adjacent to maintenance shops; (2) fueling aprons; (3) maintenance areas; (4) open storage areas using heavy duty loaders; (5) tracked vehicle parking and turning areas; and (6) wash racks.

6.2.4 Curbs and Gutters. Curbs and gutters, when required, will be of Portland cement concrete.

6.3 **Parking Areas**.

6.3.1 Nonorganizational Vehicles. Pavement design will be based on the maximum loads anticipated for each area, but in no case will pavements be designed for less than a 1,814.4-kg (4,000-pound) wheel load and 275 kPa (40 psi) tire pressure, or Design Index 1 from TM 5-822-5 (reference 2.1.15).

6.3.2 Organizational Vehicles. Parking lots for organizational vehicles must be approached as individual design problems. Parking for cars and light trucks should be similar to nonorganizational parking. Heavy trucks, specialized vehicles, and tanks will require special designs. All organizational vehicle parking will be rigid pavement. If identified in the project DD Form 1391 by using service, paved areas for organizational vehicles will be designed for the heaviest vehicle at the installation.

6.4 **Soil Stabilization.**

Stabilization of subgrade soils may be required to provide an adequate pavement structure. Guidance for soil stabilization is provided in TM 5-822-14, Soil Stabilization for Pavements (reference 2.1.13).

6.5 **Review by TSMCX.**

Unless specifically requested, military transportation systems designs for roads, streets, organizational vehicle parking, and all facilities directly in support of transportation vehicles will not be reviewed by the TSMCX.

7. **REPORT OF SUBSURFACE AND DESIGN INVESTIGATIONS.**

Within the Corps of Engineers, the geotechnical report for structural foundations is referred to as the Foundation Design Analysis and the geotechnical report for paving is referred to as the Pavement Design Analysis. Either report should contain sufficient descriptions of field and laboratory investigation, subsurface conditions, typical test data, basic assumptions, and analytical procedures to permit detailed review of the conclusions, recommendations, and final design. The amount and type of information to be presented shall be consistent with the scope of the investigation. For some structures, a cursory review of foundation conditions may be adequate. For major structures, the following outline should be used as a guide:

7.1 A general description of the proposed project should be presented including purpose, size of structure(s), and any special requirements. The traffic loading should be presented for paving projects.

7.2 A general description of the site, indicating areal extent, principal topographic features, ground cover, and presence of existing structures should be presented. A plan view that shows the surface contours, the location of the proposed project, and the location of all borings should be included.

7.3 The regional geology and the site geology should be described in general terms to provide a background for the geotechnical data obtained during field investigations.

7.4 The results of field investigations should be presented, including graphic logs of all foundation and borrow borings, locations of and pertinent data from piezometers, if any, and a general description of subsurface materials, based on the borings. Groundwater conditions should be included, with information on seasonal variations in groundwater level and results of field pumping tests, if performed.

7.5 A general description of the laboratory tests that were performed should be presented with a range of test values and detailed test data on representative samples. Atterberg limits should be plotted on a plasticity chart and typical grain-size curves should be plotted on a grain-size distribution chart. Laboratory test data should be summarized in tables and figures as appropriate. If laboratory tests were not performed, the basis for determining soil or rock properties should be presented, such as correlations or reference to pertinent publications.

7.6 A generalized geologic profile should be presented, showing properties of subsurface materials and design values of shear strength for each critical stratum. Geologic profiles and sections for inclusion in design documents and in plans and specifications should be prepared in the specified Computer-Aided Design and Drafting (CADD) System. Forms, symbols, and other graphic aids are contained within the geotechnical cell library of the A/E/C CADD Standards.

7.7 A discussion of the foundation considered, or alternative foundations considered, should be presented. Foundations for existing structures in the project vicinity and the performance of those existing foundations should also be discussed. Selection of type of foundation must be coordinated with the design structural engineers.

7.8 A table or sketch should be provided that shows the final size and depth of footings or mats, or final size and lengths of piles or drilled piers, if used. Pertinent geotechnical data should be presented for design.

7.9 Basic assumptions for loadings, basis for selecting design strengths, and the computed factors of safety for bearing-capacity calculations should be presented. Basic assumptions, loadings, and results of settlement analyses should also be presented. The estimated heave of subgrade soils, if appropriate, should be presented. The effects of computed differential settlements, and also the effects of swell, on the structure should be discussed. Basic assumptions and the results of other analyses, as pertinent, should also be provided.

7.10 For paving projects, the assumed traffic loading should be presented and the development of the recommended pavement discussed. A discussion of existing pavement in the project vicinity and the performance of that pavement should also be discussed.

7.11 The groundwater conditions at the site should be discussed along with the potential impact on construction. An estimate of dewatering requirements should be provided, if necessary

7.12 Special precautions relative to construction of the foundations should be presented. Possible sources for fill and backfill, if required, should also be given. Compaction requirements should be described.

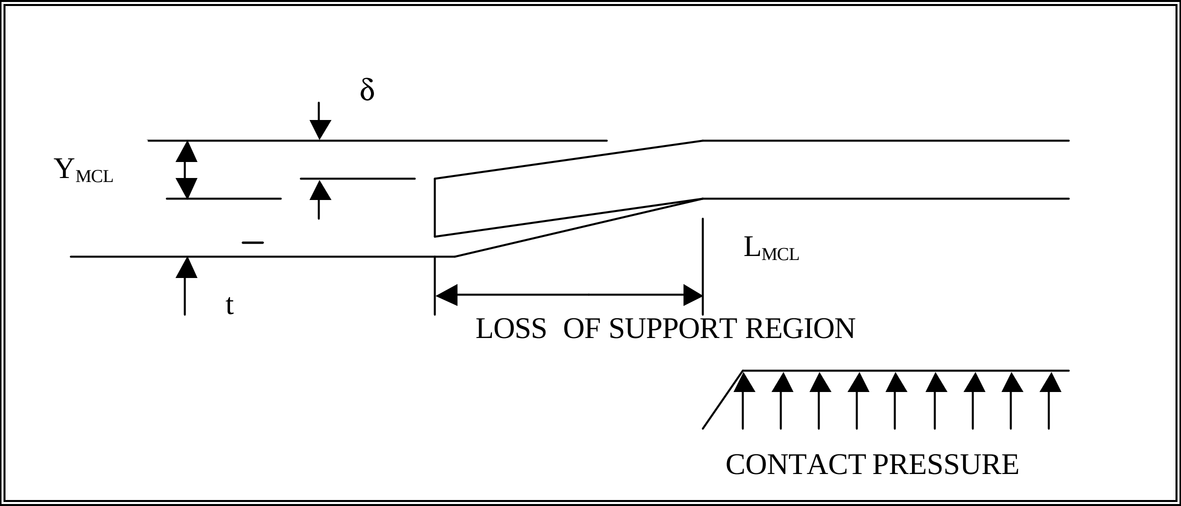


FIGURE 1. CENTER LIFT

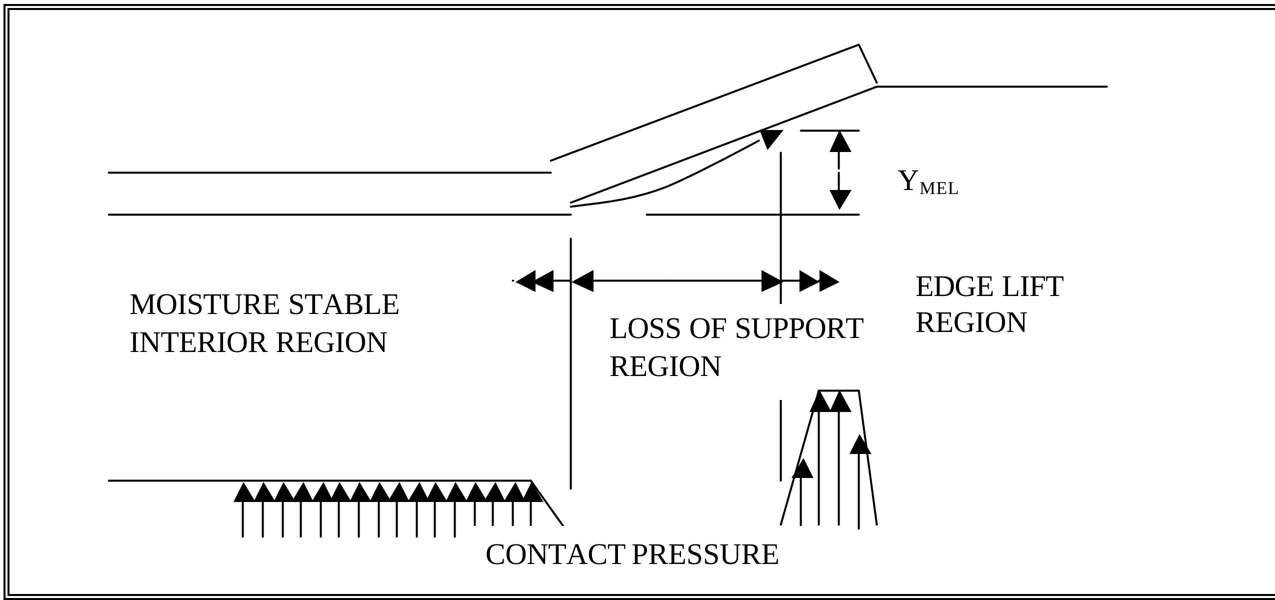
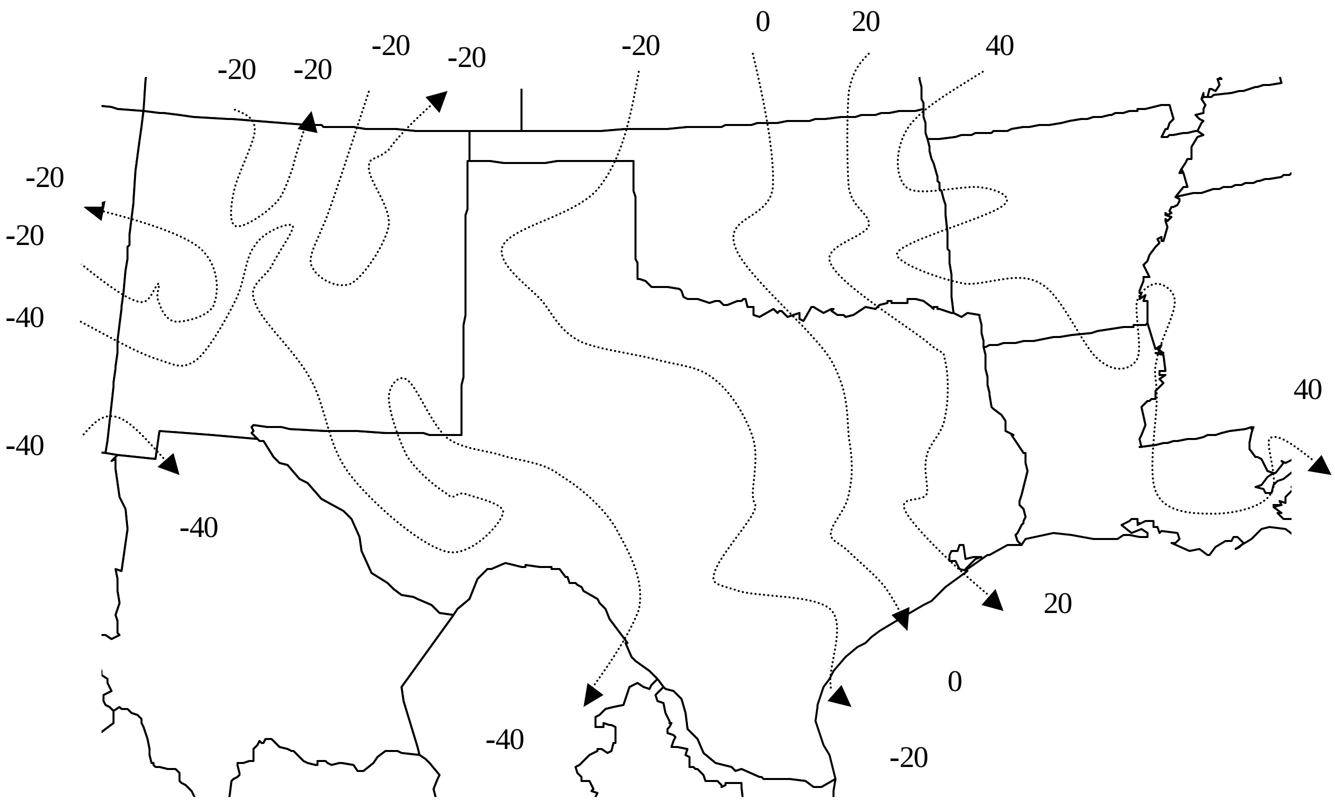


FIGURE 2. EDGE LIFT.



LOCATION	TMI	LOCATION	TMI	LOCATION	TMI	LOCATION	TMI
ARKANSAS:		**NEW MEXICO:**		**TEXAS**:		**TEXAS**	
Blytheville AFB	43	Fort Wingate	-26	Abilene	-24	Karnack	23
Little Rock AFB	42	Gallup	-26	Austin	- 3	Killeen	-2
Pine Bluff	42	Holloman AFB	-41	Bergstrom AFB	- 3	Laughlin AFB	-35
		Kirtland AFB	-19	Big Spring	-33	Lonestar AAP	28
LOUISIANA:		Las Cruces	-43	Carswell AFB	- 3	Longhorn AAP	24
Fort Polk	32	Santa Fe	-16	Corpus Christi	-22	Lubbock	-22
Leesville	31	White Sands MR	-43	Dallas	2	Red River AD	28
New Orleans	40			Del Rio	-35	Reese AFB	-23
Louisiana AAP	31	**OKLAHOMA**		Dyess AFB	-25	San Antonio	-21
Shreveport	30	Altus AFB	- 7	Ellington AFB	16	San Angelo	-32
		McAlester AFB	17	El Paso	-44	Sheppard AFB	-10
NEW MEXICO:		Oklahoma City	- 1	Fort Bliss	-44	Texarkana	29
Alburquerque	-19	Tinker AFB	- 1	Fort Hood	- 3	Wichita Falls	-10
Alamogordo	-40			Fort Worth	- 2		
Cannon AFB	-26			Goodfellow AFB	-32		
Clovis	-26			Houston	16		

FIGURE 3. Thornwaite Moisture Indices for Southwestern Division. From Thornwaite, C.W., "An Approach Toward a Rational Classification of Climate," Geographical Review, Vol. 38, No. 1, 1948, pp. 55-94.

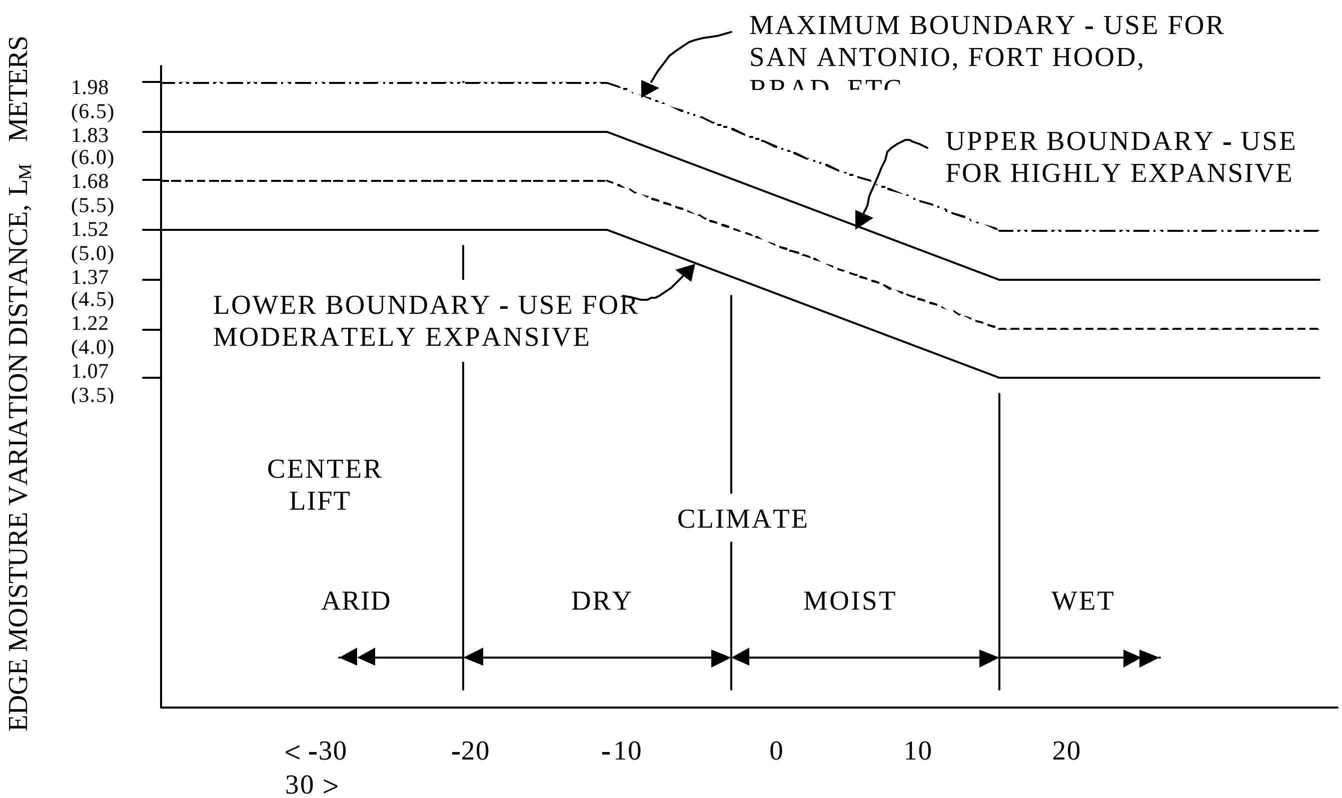


FIGURE 4. Approximate Relationship Between Thornthwaite Index and Moisture Variation Distance

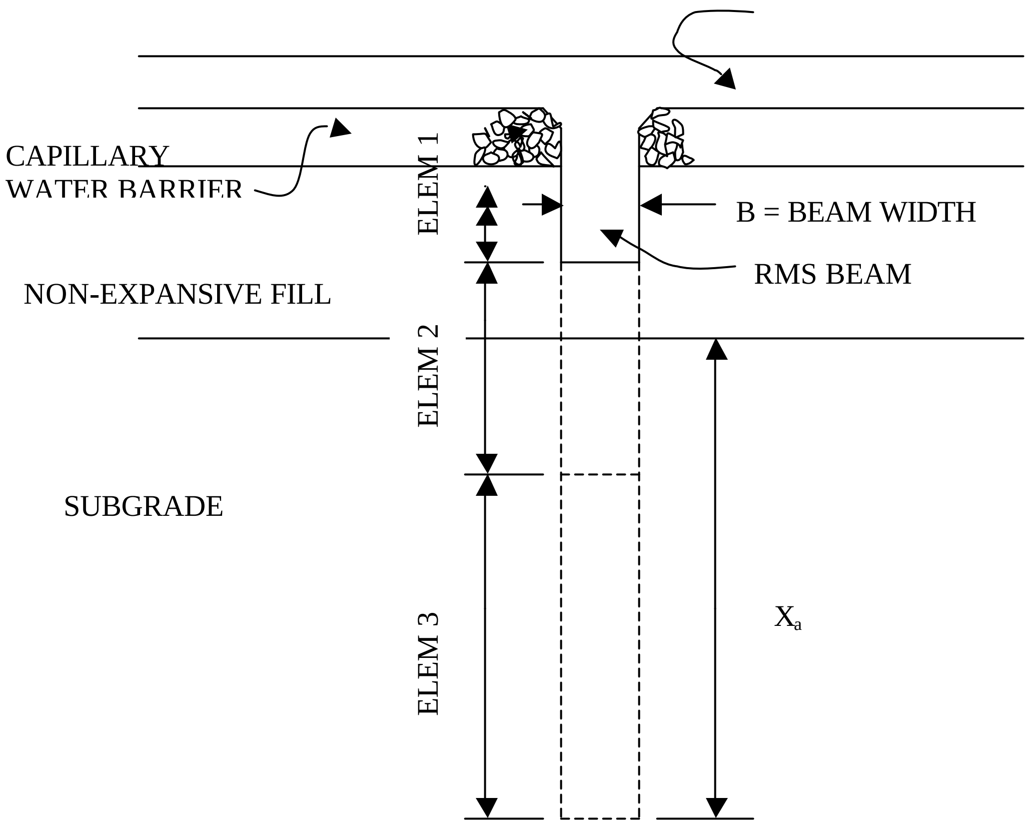

CAPILLARY
WATER BARRIER
ELEM 1
B = BEAM WIDTH
RMS BEAM
NON-EXPANSIVE FILL
ELEM 2
SUBGRADE
X
a
ELEM 3

FIGURE 5.

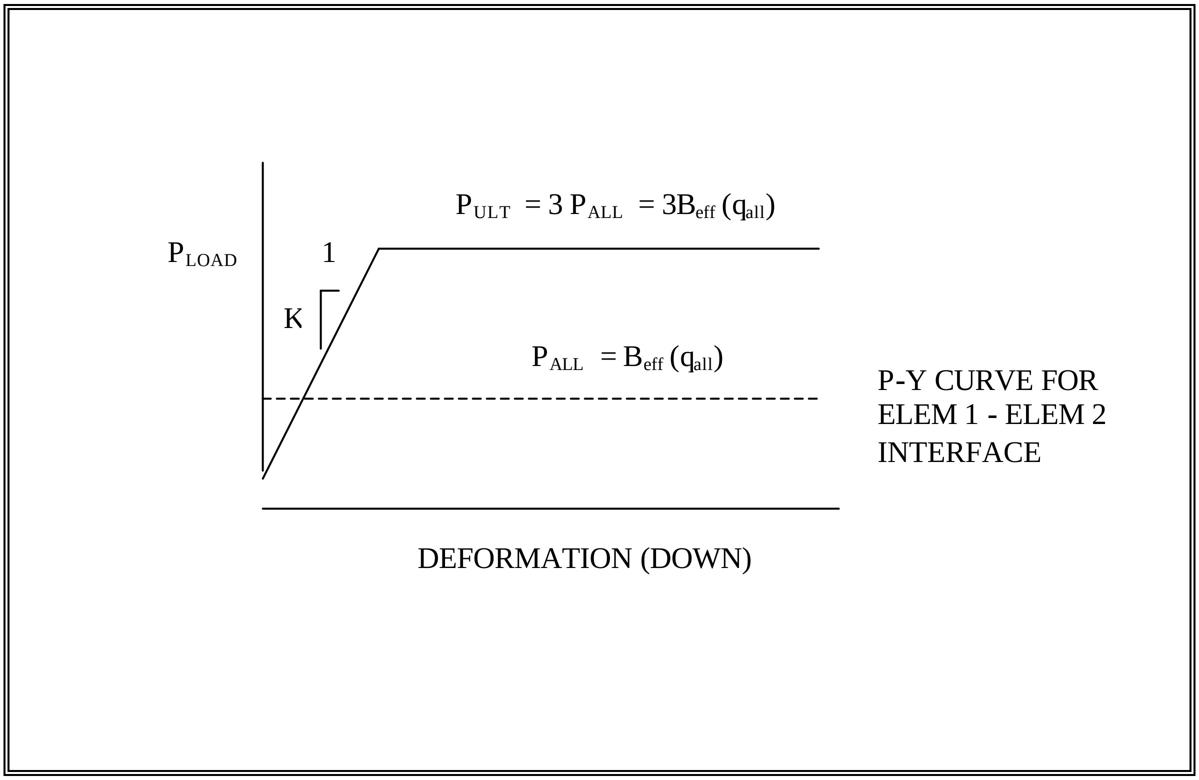


FIGURE 6.

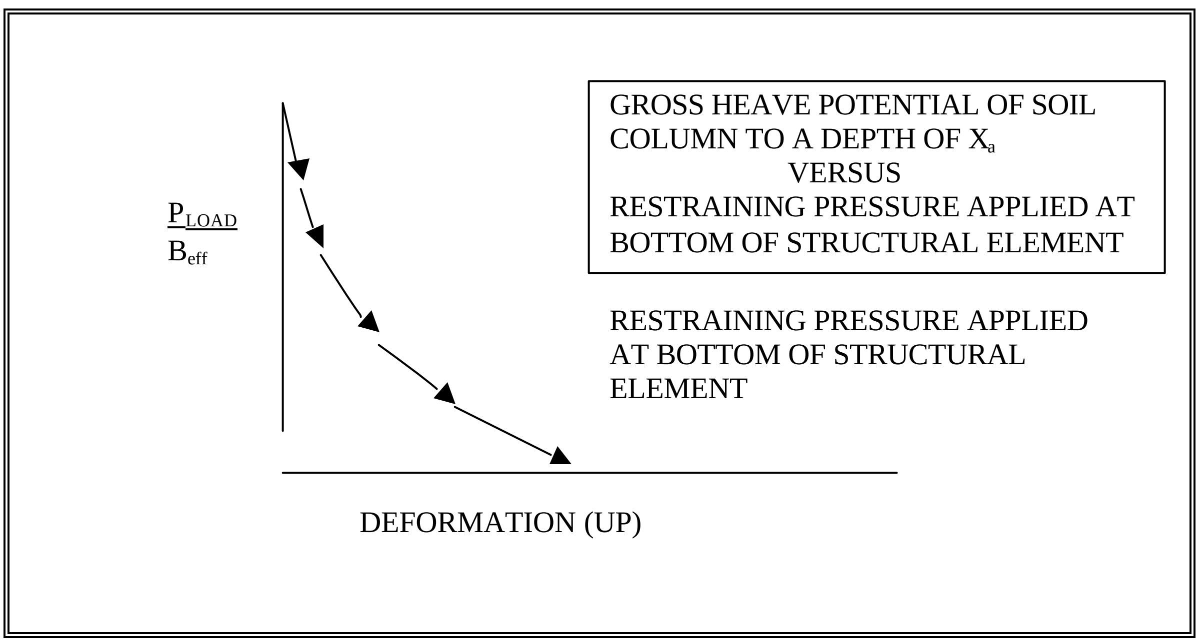
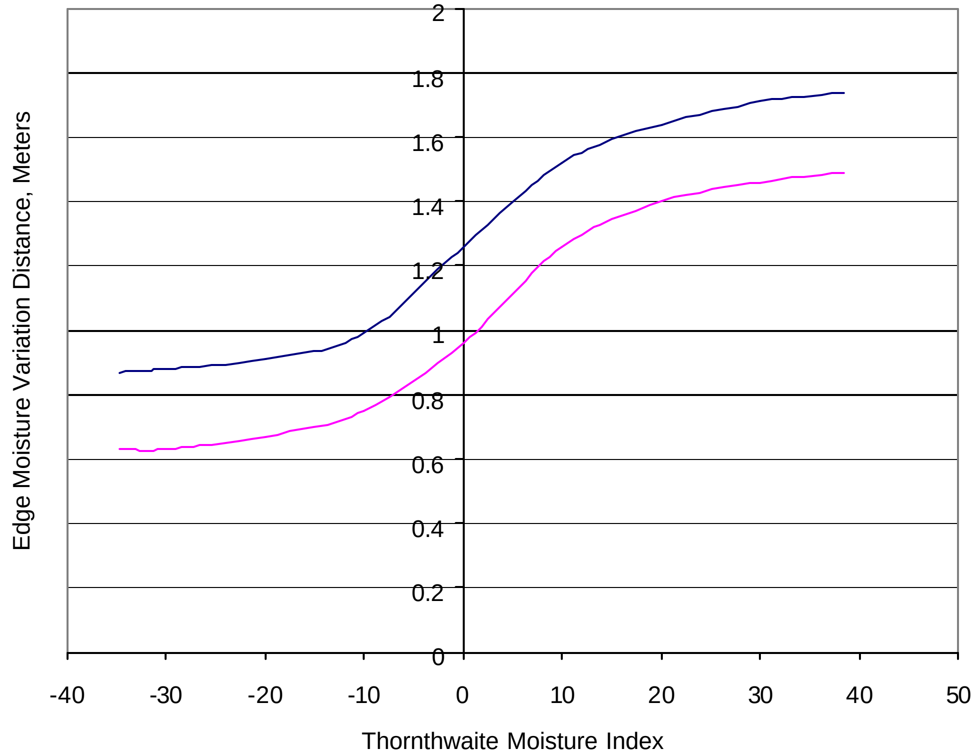


FIGURE 7.



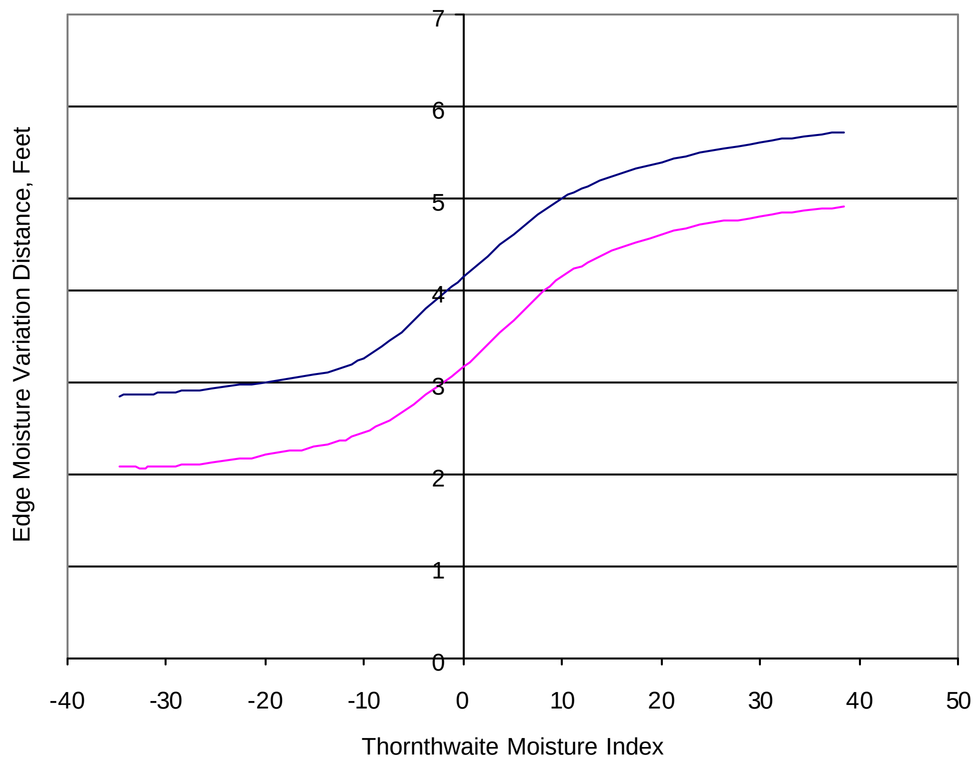


Figure 8. Approximate relationship between the Thornwaite Moisture Index (TMI) and the edge lift distance.

APPENDIX A

CHAPTER XIII
GEOTECHNICAL DESIGN/REVIEW CHECKLIST

PROJECT: ____

PROJECT LOCATION: ____

PROJECT GEOTECHNICAL ENGINEER: ________________________________

___ All geotechnical explorations, lab testing, evaluation, and engineering have been completed.

___ Objectives of geotechnical explorations and scope of work were met.

___ Geotechnical explorations were adequate.

___ Boring logs and subsurface profiles were completed and included as appropriate. Plates prepared as requested.

___ Laboratory tests were appropriate and adequate.

___ Laboratory test data were included on logs or profiles as appropriate.

___ Groundwater information has been presented.

___ Classification of soil and/or rock accurate based on boring and laboratory data.

___ Engineering properties of soil and rock were adequately defined. (Density, compaction characteristics, permeability, consolidation characteristics, shear strengths, elastic properties, shrink-swell characteristics, earth pressure coefficients)

___ Engineering analyses, as pertinent, were performed: settlement, bearing capacity, slope stability, seepage, swell pressures.

___ Selections of structural foundations, if pertinent, were made and foundation recommendations were prepared: shallow footings and/or mat foundations, drilled piers, pile foundations.

GEOTECHNICAL DESIGN/REVIEW CHECKLIST

PROJECT: ____

___ Paving analyses: vehicle and traffic considerations, subgrade preparation/stabilization, base course, pavement design.

___ Consideration was made for site improvement through soil stabilization.

___ Evaluations were performed, if pertinent, for equipment vibrations and seismic activity.

___ Surface drainage, landscape plantings, and sprinkler systems in consideration of foundations on expansive soils.

___ Specifications (site preparation): care of water, dewatering, unwatering, site drainage, clearing, grubbing, site preparation.

___ Specifications (earthworks): earthfill/fill placement, backfill for structures, excavation, backfill for utilities.

___ Specifications (structural foundations): drilled piers, piles.

___ Paving specifications: subgrade preparation/ stabilization, soil cement, base course, bituminous pavement, Portland cement concrete pavement.

___ Quantities prepared for Cost Estimating.

___ Technical coordination with others: Civil Design, Hydraulics, Hydrology, Structural.

___ Funding is adequate for the scope of work with adherence to budget through each phase of geotechnical input to the project.