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TECHNICAL MANUAL

FOUNDATIONS
IN
EXPANSIVE SOILS

HEADQUARTERS, DEPARTMENT OF THE ARMY
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CHAPTER 1

INTRODUCTION

1-1. Purpose

This manual presents guidance and information for the geotechnical investigation necessary for the selection and design of foundations for heavy and light military-type buildings constructed in expansive clay soil areas. The information in this manual is generally applicable to many types of structures such as residences, warehouses, and multistory buildings. Emphasis is given to the maintenance of an environment that encourages constant moisture conditions in the foundation soils during and following construction. Special attention must always be given to specific requirements of the structure such as limitations on allowable differential movement.

a. The guidance and information provided in this manual can significantly reduce the risk of undesirable and severe damages to many structures for numerous expansive soil conditions. However, complete solutions for some expansive soil problems are not yet available; e.g., the depth and amount of future soil moisture changes may be difficult to predict.

b. This manual presents guidance for selecting economical foundations on expansive soil to minimize structural distress to within tolerable levels and guidance for minimizing problems that may occur in structures on expansive soils.

1-2. Scope

a. Guidelines of the geotechnical investigation and analysis necessary for selection and design of military-type buildings constructed in expansive clay soil areas, as outlined in chapters 2 to 5, consist of methods for the recognition of the relative magnitude of the swelling soil problem at the construction site, field exploration, laboratory investigations, and application of methodology for prediction of volume changes in swelling foundation soils. Chapter 6 presents guidance for selection of the type of foundation with structural details of design procedures provided for reference. Chapters 7 to 9 discuss methods of minimizing foundation movement, construction techniques and inspection, and considerations for remedial repair of damaged structures.

b. Guidance is not specifically provided for design of highways, canal or reservoir linings, retaining walls, and hydraulic structures. However, much of the basic information presented is broadly applicable to the investigation and analysis of volume changes in soils supporting these structures and methods for minimizing potential soil volume changes. Guidance is also not specifically provided for the design of structures in areas susceptible to soil volume changes from frost heave and chemical reactions in the soil (e.g., oxidation of iron pyrite), although much of the information presented can be useful toward these designs.

1-3. Background

This manual is concerned with heave or settlement caused by change in soil moisture in nonfrozen soils. Foundation materials that exhibit volume change from change in soil moisture are referred to as expansive or swelling clay soils. Characteristic expansive or swelling materials are highly plastic clays and clay shales that often contain colloidal clay minerals such as the montmorillonites. Expansive soils as used in this manual also include marls, clayey siltstones, sandstones, and saprolites.

a. Damages from differential movement. The differential movement caused by swell or shrinkage of expansive soils can increase the probability of damage to the foundation and superstructure. Differential rather than total movements of the foundation soils are generally responsible for the major structural damage. Differential movements redistribute the structural loads causing concentration of loads on portions of the foundation and large changes in moments and shear forces in the structure not previously accounted for in standard design practice.

b. Occurrence of damages. Damages can occur within a few months following construction, may develop slowly over a period of about 5 years, or may not appear for many years until some activity occurs to disturb the soil moisture. The probability of damages increases for structures on swelling foundation soils if the climate and other field environment, effects of construction, and effects of occupancy tend to promote moisture changes in the soil.

c. Structures susceptible to damages. Types of structures most often damaged from swelling soil include foundations and walls of residential and light (one- or two-story) buildings, highways, canal and reservoir linings, and retaining walls. Lightly loaded

one- or two-story buildings, warehouses, residences, and pavements are especially vulnerable to damage because these structures are less able to suppress the differential heave of the swelling foundation soil than heavy, multistory structures.

(1) *Type of damages.* Damages sustained by these structures include: distortion and cracking of pavements and on-grade floor slabs; cracks in grade beams, walls, and drilled shafts; jammed or misaligned doors and windows; and failure of steel or concrete plinths (or blocks) supporting grade beams. Lateral forces may lead to buckling of basement and retaining walls, particularly in overconsolidated and nonfissured soils. The magnitude of damages to structures can be extensive, impair the usefulness of the structure, and detract aesthetically from the environment. Maintenance and repair requirements can be extensive, and the expenses can grossly exceed the original cost of the foundation.

(2) *Example of damages. Figure* 1-1 illustrates damages to a building constructed on expansive soil with a deep water table in the wet, humid climate of Clinton, Mississippi. These damages are typical of buildings on expansive soils. The foundation consists of grade beams on deep drilled shafts. Voids were not provided beneath the grade beams above the expansive foundation soil, and joints were not made in the walls and grade beams. The floor slab was poured on-grade with no provision to accommodate differential movement between the slab and grade beams. The heave of the floor slab exceeded 6 inches. The differential soil movement and lack of construction joints in the structure aggravated cracking.

14. Causes and patterns of heave

a. Causes. The leading cause of foundation heave or settlement in susceptible soils is change in soil moisture, which is attributed to changes in the field environment from natural conditions, changes related to construction, and usage effects on the moisture under the structure (table 1-1). Differential heave may be caused by nonuniform changes in soil moisture, variations in thickness and composition of the expansive foundation soil, nonuniform structural loads, and the geometry of the structure. Nonuniform moisture changes occur from most of the items given in table 1-1.

b. Patterns of heave.

(1) *Doming heave.* Heave of foundations, although often erratic, can occur with an upward, long-term, dome-shaped movement that develops over many years. Movement that follows a reduction of natural evapotranspiration is commonly associated with a doming pattern of greatest heave toward the center of the structure. Evapotranspiration refers to the evaporation of moisture from the ground surface and trans-

piration of moisture from heavy vegetation into the atmosphere. Figure 1-2 schematically illustrates some commonly observed exterior cracks in brick walls from doming or edgedown patterns of heave. The pattern of heave generally causes the external walls in the superstructure to lean outward, resulting in horizontal, vertical, and diagonal fractures with larger cracks near the top. The roof tends to restrain the rotation from vertical differential movements leading to additional horizontal fractures near the roofline at the top of the wall. Semiarid, hot, and dry climates and deep water tables can be more conducive to severe and progressive foundation soil heaves if water become available.

(2) *Cyclic heave.* A cyclic expansion-contraction related to drainage and the frequency and amount of rainfall and evapotranspiration may be superimposed on long-term heave near the perimeter of the structure. Localized heaving may occur near water leaks or ponded areas. Downwarping from soil shrinkage (fig. 1-2) may develop beneath the perimeter during hot, dry periods or from the desiccating influence of trees and vegetation located adjacent to the structure. These edge effects may extend inward as much as 8 to 10 feet. They become less significant on well-drained land. Heavy rain periods may cause pending adjacent to the structure with edge lift (fig. 1-3) and reversal of the downwarping.

(3) *Edge heave.* Damaging edge or dish-shaped heaving (fig. 1-3) of portions of the perimeter maybe observed relatively soon after construction, particularly in semiarid climates on construction sites with preconstruction vegetation and lack of topographic relief. The removal of vegetation leads to an increase in soil moisture, while the absence of topographic relief leads to ponding (table 1-1). A dish-shaped pattern can also occur beneath foundations because of consolidation, drying out of surface soil from heat sources, or sometimes lowering of the water table. Changes in the water table level in uniform soils beneath uniformly loaded structures may not contribute to differential heave. However, structures on a deep foundation, such as drilled shafts with a slab-on-grade, can be adversely affected by a changing water table or changes in soil moisture if the slab is not isolated from the perimeter grade beams and if internal walls and equipment are not designed to accommodate the slab movement.

(4) *Lateral movement.* Lateral movement may affect the integrity of the structure.

(a) Lateral thrust of expansive soil with a horizontal force up to the passive earth pressure can cause bulging and fracture of basement walls. Basement walls and walls supporting buildings usually cannot tolerate the same amount of movement as free-standing retaining walls. Consequently, such walls must be designed to a higher degree of stability.

a. Vertical cracks

b. Diagonal and vertical cracks

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Figure 1-1. Examples of cracks in an exterior wall.

(b) The walls and foundations of structures constructed on slopes greater than 5 degrees (9 percent) may experience damage from downhill soil creep of cohesive expansive soils. Downhill creep can also shear shaft foundations. The mechanism of creep may be such that the soil alternately expands and contracts aided by gravity. The depth of creeping soil varies from a few inches to several feet.

Table 1-1. Examples of Causes of Foundation Heave from Changes in Soil Moisture

Changes in field environment from natural conditions	1.	Significant variations in climate, such as long droughts and heavy rains, cause cyclic moisture changes resulting in edge movement of structures.
	2.	Changes in depth to the water table lead to changes in soil moisture.
	3.	Frost heave and chemical reactions in the soil, such as oxidation of iron pyrite, noted.
Changes related to construction	1.	Covered areas reduce natural evaporation of moisture from the ground increasing soil moisture.
	2.	Covered areas reduce transpiration of moisture from vegetation increasing soil moisture.
	3.	Construction on a site where large trees were removed may lead to an increase of moisture because of prior depletion of soil moisture by the extensive root system.
	4.	Inadequate drainage of surface water from the structure leads to ponding and localized increases in soil moisture. Defective rain gutters and downspouts contribute to localized increases in soil moisture.
	5.	Seepage into foundation subsoils at soil/foundation interfaces and through excavations made for basements or shaft foundations leads to increased soil moisture beneath the foundation.
	6.	Drying of exposed foundation soils in excavations and reduction in soil surcharge weight increase the potential for heave.
	7.	Aquifers tapped.
Usage effects	1.	Watering of lawns leads to increased soil moisture.
	2.	Planting and growth of heavy vegetation, such as trees, at distances from the structure less than 1 to 1.5 times the height of mature trees aggravate cyclic edge heave.
	3.	Drying of soil beneath heated areas of the foundation, such as furnace rooms, leads to soil shrinkage.
	4.	Leaking underground water and sewer lines can cause foundation heave and differential movement.

1-5. Elements of design

The foundation should be constructed or taken to a depth to protect the structure against damage by swelling or shrinking soil. Furthermore, the foundation should transmit the combined dead and imposed loads to the ground without causing settlements or other movements that are large enough to impair or damage the structure or reduce its overall usefulness. Finally, the foundation should provide protection from the freeze-thaw cycle of soil in cold climates and adequately resist any chemical or deleterious attack such as by sulfates and other harmful material in the soil.

*a. Decision process of design.*

(1) Figure 1-4 shows steps in the decision process, during the predesign and design phases, to properly select the foundation and superstructure. These steps include: site and soil investigations; a study of topography, drainage, and soil stabilization; and the selection of the foundation and superstructure.

(2) A foundation report for future reference should be made after construction.

b. Economics of the foundation. A thorough geotechnical study and an investigation of the foundation system during the predesign and preliminary design phases are normally essential.

(1) The features of the design should be kept simple to minimize costs and future maintenance expenses. Irregular geometries should be avoided. Construction of independently supported rectangular sections of the structure separated by joints, for example, may be appropriate if differential movement and separation between the independent sections does not significantly detract from the aesthetics or present a safety hazard. External parts of the structure, such as porches, terraces, breezeways, and garages, should be supported by part of the engineered foundation or isolated from the main structure. If the external parts of
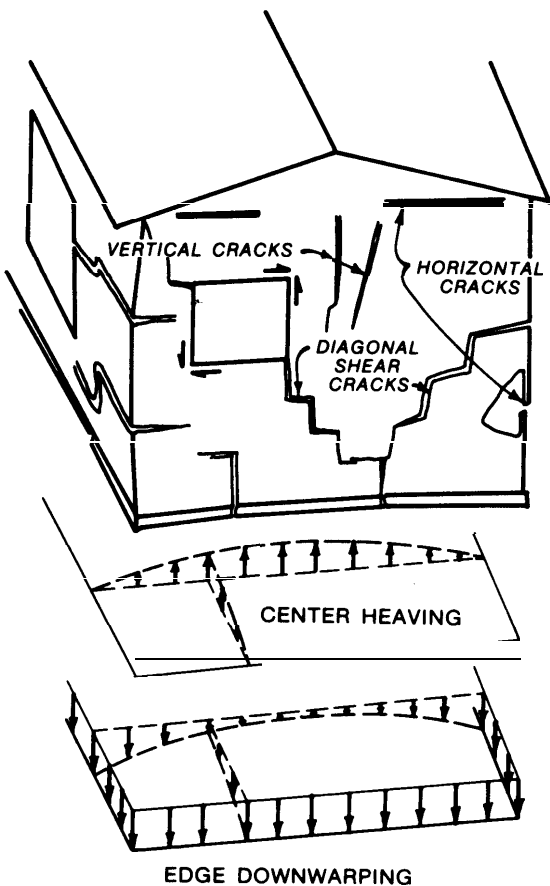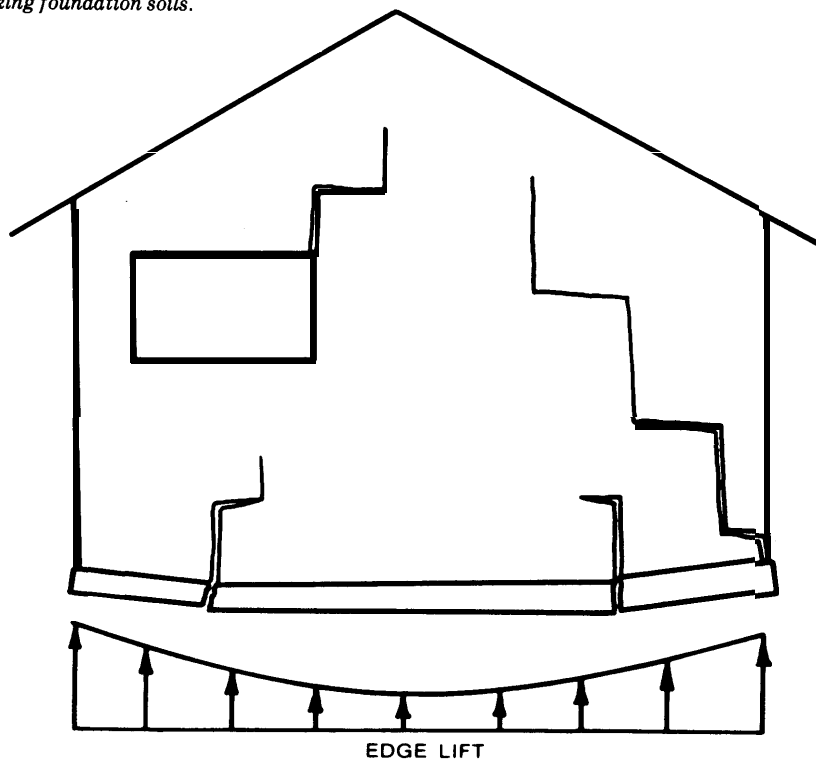
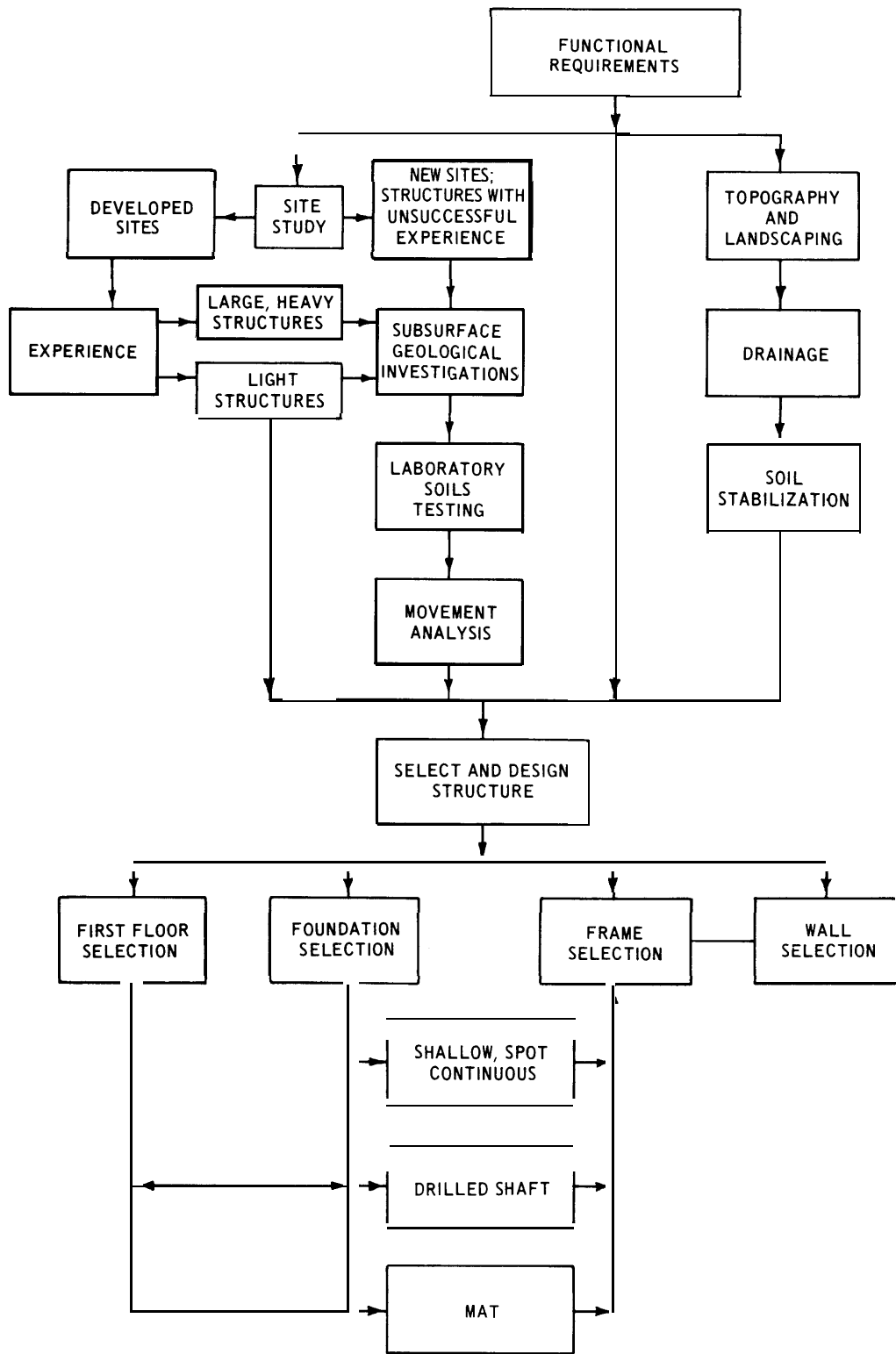*Figure 1-2. Examples of wall fractures from doming heave of swelling and shrinking foundation soils.*

the structure are simply supported on-grade or attached to the structure, they can contribute to future maintenance problems.

(2) Potential problems that could eventually affect the performance of the structure are best determined during the predesign and preliminary design phases when compromises can be made between the structural, architectural, mechanical, and other aspects of the design without disrupting the design process. Changes during the detailed design phase or during construction will probably delay construction and pose economic disadvantages.

Figure 1-3. Examples of fractures from dish-shaped lift on swelling foundation soils.

Figure 1-4. Decision process of design.

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CHAPTER 2

RECOGNITION OF PROBLEM AREAS

2-1. Site selection

The choice of the construction site is often limited. It is important to recognize the existence of swelling soils on potential sites and to understand the problems that can occur with these soils as early as possible. A surface examination of the potential site as discussed in paragraph 3-2 should be conducted and available soil data studied during the site selection.

a. Avoidance of potential problems. If practical, the foundation should be located on uniform soils subject to the least swelling or volume change. Discontinuities or significant lateral variations in the soil strata should be avoided. Swampy areas, backfilled ponds, and areas near trees and other heavy vegetation should be avoided, Special attention should be given to adequate compaction of filled areas, types of fill, and leveling of sloped sites (para 7-1).

(1) *Undeveloped sites.* Undeveloped sites generally have little or no subsurface soil information available and require subsurface exploration (para 3-3).

(a) Substantial differential heave may occur beneath structures constructed on previously undeveloped sites where trees and other heavy vegetation had been removed prior to construction, Soil moisture will tend to increase since loss of heavy vegetation reduces the transpiration of moisture. Construction of the foundation over the soil will tend to further increase soil moisture because of reduced evaporation of moisture from the ground surface.

(b) Swampy or ponded areas may contain greater quantities of plastic fine particles with a greater tendency to swell than other areas on the site.

(c) Future irrigation of landscaped areas and leakage from future sewer and other water utility lines following development of the site may substantially increase soil moisture and cause a water table to rise or to develop if one had not previously existed. Filled areas may also settle if not properly compacted.

(2) *Developed sites.* Subsurface exploration should be conducted if sufficient soil data from earlier borings are not available for the site selection and/or problems had occurred with previous structures. Some subsurface exploration is always necessary for site selection of any structure of economic significance, particularly multistory buildings and structures with special requirements of limited differential distortion.

(a) An advantage of construction on developed sites is the experience gained from previous construction and observation of successful or unsuccessful past performance. Local builders should be consulted to obtain their experience in areas near the site. Existing structures should be observed to provide hints of problem soil areas.

(b) The soil moisture may tend to be much closer to an equilibrium profile than that of an undeveloped site. Differential movement may not be a problem because previous irrigation, leaking underground water lines, and previous foundations on the site may have stabilized the soil moisture toward an equilibrium profile. Significant differential movement, however, is still possible if new construction leads to changes in soil moisture. For example, trees or shrubs planted too close to the structure or trees removed from the site, change in the previous irrigation pattern following construction, lack of adequate drainage from the structure, and improper maintenance of drainage provisions may lead to localized changes in soil moisture and differential heave. Edge movement of slab-on-grade foundations from seasonal changes in climate may continue to be a problem and should be minimized as discussed in chapter 7.

(3) *Sidehill or sloped sites.* Structures constructed on sites in which the topography relief is greater than 5 degrees (9 percent gradient) may sustain damage from downhill creep of expansive clay surface soil. Sidehill sites and sites requiring split-level construction can, therefore, be expected to complicate the design. See chapter 7 for details on minimization of foundation soil movement.

b. Soil surveys, Among the best methods available for qualitatively recognizing the extent of the swelling soil problem for the selected site is a careful examination of all available documented evidence on soil conditions near the vicinity of the site. Local geological records and publications and federal, state, and institutional surveys provide good sources of information on subsurface soil features. Hazard maps described in paragraph 2-2 document surveys available for estimating the extent of swelling soil problem areas.

2-2. Hazard maps

Hazard maps provide a useful first-order approximation of and guide to the distribution and relative expansiveness of problem soils. These maps should be

used in conjunction with local experience and locally available soil surveys and boring data. The maps discussed in a and b below are generally consistent with each other and tend to delineate similar areas of moderately or highly expansive soil.

a. *Waterways Experiment Station (WES) Map.* This map, which was prepared for the Federal Highway Administration (FHWA), summarizes the areas of the United States, except Alaska and Hawaii, where swelling soil problems are likely to occur (fig. 2-1). The basis for classification depends primarily on the estimated volume change of argillaceous or clayey materials within the geologic unit, the presence of montmorillonite, the geologic age, and reported problems due to expansive materials. The stratigraphy and mineralogy are key elements in the classification.

(1) *Classification. The* soils are classified into categories of High, Medium, Low, and Nonexpansive as shown in figure 2-1. The distribution of expansive materials is categorized by the geologic unit on the basis of the degree of expansiveness that relates to the expected presence of montmorillonite and the frequency of occurrence that relates to the amount of clay or shale. The amount refers most significantly to the vertical thickness of the geologic unit, but the areal extent was also considered in the classification. The premises in table 2-1 guide the categorization of soils.

Table 2-1. Premises for Categorization of Soils by the WES Hazard Map

1. Any area underlain by argillaceous rocks, sediments, or soils will exhibit some degree of expansiveness.

2. The degree of expansiveness is a function of the amount of expandable clay minerals present.

3. Generally, the Mesozoic and Cenozoic rocks and sediments contain significantly more montmorillonite than the Paleozoic (or older) rocks. (Damage to structures founded on Permian (Upper Paleozoic) has also been observed.)

4. Areas underlain by rocks or sediments of mixed textural compositions (e.g., sandy shales or sandy clays) or shales or clays interbedded with other rock types or sediments are considered on the basis of geologic age and the amount of agrillaceous material present.

5. Generally, those areas lying north of the glacial boundary are nonexpansive due to glacial drift cover.

6. Soils derived from weathering of igneous and metamorphic rocks are generally nonexpansive.

7. Climate or other environmental aspects are not considered.

8. Argillaceous rocks or sediments originally composed of expandable clay minerals do not exhibit significant volume change when subjected to tectonic folding, deep burial, or metamorphism.

9. Volcanic areas consisting mainly of extruded basalts and kindred rocks may also contain tuffs and volcanic ash deposits that have devitrified and altered to montmorillonite.

10. Areas along the glaciated boundary may have such a thin cover of drift that the expansive character of the materials under the drift may predominate.
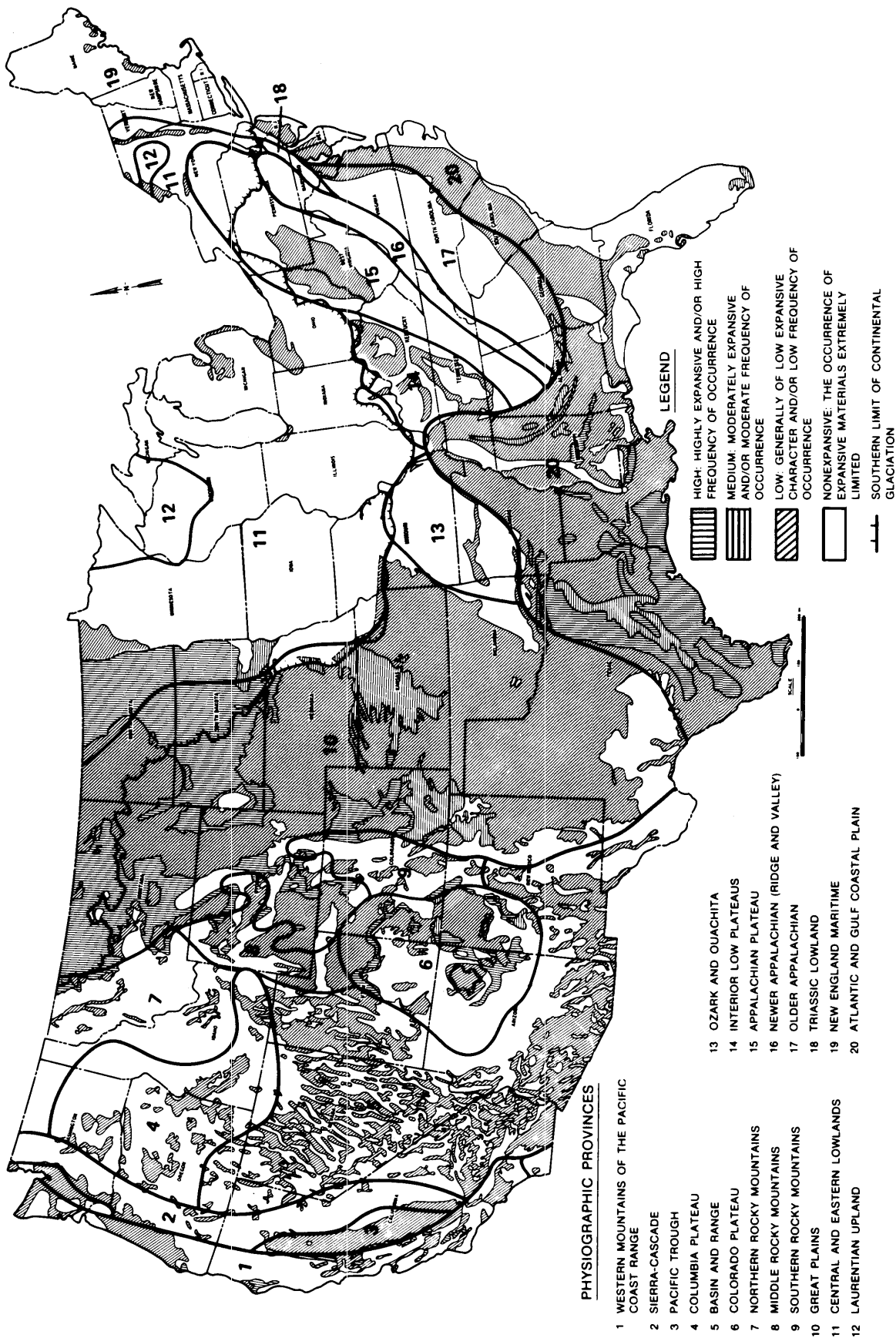
(2) *Physiographic provinces.* Table 2-2 summarizes the potentially expansive geologic units on the basis of the 20 first-order physiographic provinces. Figure 2-1 shows the locations of the physiographic provinces.

b. *Other maps.*

(1) *Area map of susceptible soil expansion problems.* A hazard map was developed by M, W. Witczak (Transportation Research Board, Report 132) on the basis of the occurrence and distribution of expansive soils and expansive geologic units, the pedologic analysis, and climatic data to delineate areas susceptible to expansion problems. Some geologic units for which engineering experiences were not available may have been omitted, and the significance of pedological soil on expansion was not shown on the map.

(2) *Assessment map of expansive soils within the United States. The* major categories for classification of the severity of the swelling soil problem presented by J. P. Krohn and J. E. Slosson (American Society of Civil Engineers, *Proceedings of the Fourth International Conference on Expansive Soils,* Volume 1 (see app. A) correspond to the following modified shrink-swell categories of the Soil Conservation Service (SCS) of the U. S. Department of Agriculture:

High: Soils containing large amounts of montmorillonite and clay (COLE >6 percent)

Moderate: Soils containing moderate amounts of clay with some montmorillonitic minerals (3 percent $\leqslant$ COLE $\leqslant$ 6 percent)

Low: Soils containing some clay with the clay consisting mostly of kaolinite and/or other low swelling clay minerals (COLE <3 percent).

PHYSIOGRAPHIC PROVINCES

1 WESTERN MOUNTAINS OF THE PACIFIC
   COAST RANGE
2 SIERRA-CASCADE
3 PACIFIC TROUGH
4 COLUMBIA PLATEAU
5 BASIN AND RANGE
6 COLORADO PLATEAU
7 NORTHERN ROCKY MOUNTAINS
8 MIDDLE ROCKY MOUNTAINS
9 SOUTHERN ROCKY MOUNTAINS
10 GREAT PLAINS
11 CENTRAL AND EASTERN LOWLANDS
12 LAURENTIAN UPLAND

13 OZARK AND OUACHITA
14 INTERIOR LOW PLATEAUS
15 APPALACHIAN PLATEAU
16 NEWER APPALACHIAN (RIDGE AND VALLEY)
17 OLDER APPALACHIAN
18 TRIASSIC LOWLAND
19 NEW ENGLAND MARITIME
20 ATLANTIC AND GULF COASTAL PLAIN

LEGEND

HIGH: HIGHLY EXPANSIVE AND/OR HIGH
FREQUENCY OF OCCURRENCE

MEDIUM: MODERATELY EXPANSIVE
AND/OR MODERATE FREQUENCY OF
OCCURRENCE

LOW: GENERALLY OF LOW EXPANSIVE
CHARACTER AND/OR LOW FREQUENCY OF
OCCURRENCE

NONEXPANSIVE: THE OCCURRENCE OF
EXPANSIVE MATERIALS EXTREMELY
LIMITED

SOUTHERN LIMIT OF CONTINENTAL
GLACIATION

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Figure 2-1. Occurrence and distribution of potentially expansive materials in the United States, 1977, with boundaries of physiographic provinces.

Table 2-2. Tabulation of Potentially Expansive Materials in the United States

Physiographic Province No.[a]	Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map Category [b]	Remarks
1	Western Mountains of the Pacific Coast Range	Reefridge	Miocene	CA	1	The Tertiary section generally consists of interbedded sandstone, shale, chert, and volcanics
		Monterey	Miocene	CA	1	
		Rincon	Miocene	CA	1	
		Tembler	Miocene	CA	1	
		Umpqua	Paleocene-Eocene	OR	3	
		Puget Gp	Miocene	WA	3	Interbedded sandstones and shales with some coal seams
		Chico Fm	Cretaceous	CA	1	
2	Sierra Cascade	Cascade Gp	Pliocene	OR	4	Predominate material is volcanic
		Columbia Gp	Miocene	WA	4	Interbedded sandstones and shales may occur throughout, particularly in western foot hills
		Volcanics	Paleozoic to Cenozoic	NV	4	
		Volcanics	Paleozoic to Cenozoic	CA	4	
3	Pacific Trough	Troutdale	Pliocene	WA	3	Great Valley materials characterized by local areas of low-swell potential derived from bordering mountains. Some scattered deposits of bentonite
		Santa Clara	Pleistocene	CA	3	
		Riverbank	Pleistocene	CA	3	
4	Columbia Plateau	Volcanics	Cenozoic	WA, OR, ID, NV	4	Some scattered bentonites and tuffs
5	Basin and Range	Valley fill materials	Pleistocene	OR, CA, NV, UT, AZ, NM, TX	3	Playa deposits may exhibit limited swell potential. Some scattered bentonites and tuffs
		Volcanics	Tertiary	OR, CA, NV, UT, AZ, NM, TX	3	
6	Colorado Plateau	Greenriver	Eocene	CO, UT, NM	3	
		Wasatch	Eocene	CO, UT, NM	3	
		Kirkland shale	Upper Cretaceous	CO, UT, NM, AZ	2	Interbedded sandstones and shales
		Lewis shale	Upper Cretaceous	CO, UT, NM, AZ	2	
		Mancos	Upper Cretaceous	CO, UT, NM, AZ	1	
		Mowry	Upper Cretaceous	CO, UT, NM, AZ	1	
		Dakota	Jurassic-Cretaceous	CO, UT, NM, AZ	3	Interbedded sandstones and shales
		Chinle	Triassic	NM, AZ	1	

(Continued)

a Refer to map of physiographic provinces, Figure 2-1.
b Numerical map categories correspond as follows: 1 - high expansion, 2 - medium expansion, 3 - low expansion, and 4 - nonexpansive.

(Sheet 1 of 4)

Table 2-2. (Continued)

Physiographic Province No.	Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map Category	Remarks
7	Northern Rocky Mountains	Montana Gp	Cretaceous	MT	1	Locally some sandstone and siltstone
		Colorado Gp	Cretaceous	MT	2	Locally some siltstone
		Morrison	Jurassic	MT	3	Shales, sandstones, and limestones
		Sawtooth	Jurassic	MT	3	
8	Middle Rocky Mountains	Windriver	Eocene	WY, MT	3	
		Fort Union	Eocene	WY, MT	3	
		Lance	Cretaceous	WY, MT	1	
		Montana Gp	Cretaceous	WY, MT	1	
		Colorado Gp	Cretaceous	WY, MT	2	
		Morrison	Jurassic-Cretaceous	WY, MT	3	
9	Southern Rocky Mountains	Metamorphic and granitic rocks	Precambrian	WY	4	Montana and Colorado Gps may be present locally with some Tertiary volcanic and minor amounts of Pennsylvania limestone (sandy or shaly).
		Metamorphic and granitic rocks	Precambrian	CO	4	
		Metamorphic and granitic rocks	Precambrian to Cenozoic	NM	4	
10	Great Plains	Fort Union	Paleocene	WY, MT	3	
		Thermopolis	Cretaceous	WY, MT	1	
		Montana Gp	Cretaceous	WY, MT, CO, NM	1	
		Colorado Gp	Cretaceous	WY, MT, CO, NM	2	
		Mowry	Cretaceous	WY, MT, CO, NM	1	
		Morrison	Jurassic-Cretaceous	WY, MT, CO, NM	3	
		Ogallala	Pliocene	WY, MT, CO, NM, SD, NE, KS, OK, TX	3	Generally nonexpansive but bentonite layers are locally present
		Wasatch	Eocene	MT, SD	3	
		Dockum	Triassic	CO, NM, TX	3	
		Permian Red Beds	Permian	KS, OK, TX	3	
		Virgillian Series	Pennsylvanian	NE, KS, OK, TX, MO	3	
		Missourian Series	Pennsylvanian	KS, OK, TX, MO	3	
		Desmonian Series	Pennsylvanian	KS, OK, TX, MO	3	
11	Central and Eastern Lowlands	Glacial lake deposits	Pleistocene	ND, SD, NM, IL, IN, OH, MI, NY, VT, MA, NE, IA, KS, MO, WI	3	Some Paleozoic shales locally present which may exhibit low swell

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Table 2-2. (Continued)

Physiographic Province No.	Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map Category	Remarks
12	Laurentian Uplands	Keweenawan	Precambrian	NY, WI, MI	4	Abundance of glacial material of varying thickness
		Huronian	Precambrian	NY, WI, MI	4	
		Laurentian	Precambrian	NY, WI, MI	4	
13	Ozark and Ouachita	Fayetteville	Mississippian	AR, OK, MO	3	May contain some montmorillonite in mixed layer form
		Chickasaw Creek	Mississippian	AR, OK, MO	3	
14	Interior Low Plains	Meramec Series	Mississippian	KY	3	Interbedded shale, sandstone, and limestone
		Osage	Mississippian	KY, TN	3	
		Kinderhook	Mississippian	KY, TN	3	
		Chester Series	Mississippian	KY, IN	3	
		Richmond	Upper Ordovician	KY, IN	3	
		Maysville	Upper Ordovician	KY, IN	3	
		Eden	Upper Ordovician	KY, IN	3	
15	Appalachian Plateau	Dunkard Gp	Pennsylvanian-Permian	WV, PA, OH	3	Interbedded shale, sandstone, limestone, and coal
16	Newer Appalachian	See Remarks	See Remarks	AL, GA, TN, NC, VA, WV, MD, PA	4	A complex of nonexpansive Precambrian and Lower Paleozoic metasedimentary and sedimentary rocks
17	Older Appalachian	See Remarks	Paleozoic	AL, GA, NC, SC, VA, MD	4	A complex of nonexpansive metamorphic and intrusive igneous rocks
18	Triassic Lowland	Newark Gp	Triassic	PA, MD, VA	4	
19	New England Maritime	Glacio-marine deposits	Pleistocene	ME	3	Pleistocene marine deposits underlain by nonexpansive rocks. Local areas of clay could cause some swell potential
20	Atlantic and Gulf Coastal Plain	Talbot and Wicomico Gps	Pleistocene	NC, SC, GA, VA, MD, DE, NJ	4	Interbedded gravels, sands, silts, and clays
		Lumbee Gp	Upper Cretaceous	NC, SC	3	Sand with intermixed sandy shale
		Potomac Gp	Lower Cretaceous	DC	3	Sand with definite shale zones
		Arundel Fm	Lower Cretaceous	DC	1	
		Continental and marine coastal deposits	Pleistocene to Eocene	FL	4	Sands underlain by limestone, local deposits may show low swell potential

(Sheet 3 of 4)

Table 2-2. (Continued)

Physiographic Province No.	Name	Predominant Geologic Unit	Geologic Age	Location of Unit	Map Category	Remarks
20	Atlantic and Gulf Coastal Plain (Cont'd)	Yazoo Clay	Eocene	MS, LA	1	A complex interfacing of gravel, sand, silt, and clay. Clays show varying swell potential
		Porters Creek Clay	Paleocene	MS, AL, GA	1-3	
		Selma Gp	Cretaceous	MS, AL, GA	2-3	
		Loess	Pleistocene	LA, MS, TN, KY	4	A mantle of uniform silt with essentially no swell potential
		Mississippi alluvium	Recent	LA, MS, AR, MO	3	
		Beaumont-Prairie Terraces	Pleistocene	LA, MS, TX	1	Interbedded stringers and lenses of sands, silts, clays, marl, and chalk
		Jackson, Claiborne, Midway	Paleocene-Oligocene	LA, MS	1-3	
		Navarre, Taylor, Austin	Upper Cretaceous	TX	1-2	
		Eagle Ford, Woodbine	Upper Cretaceous	TX	1-3	
		Washita	Lower Cretaceous	TX, OK	1-3	
		Fredricksburg	Lower Cretaceous	TX	3	
		Trinity	Lower Cretaceous	TX	4	

(Sheet 4 of 4)

These categories of classification use the coefficient of linear extensibility (COLE), which is a measure of the change in linear dimension from the dry to a moist state, and it is related to the cube root of the volume change. Premises guiding the categorization of the Krohn and Slosson map include: degree of expansion as a function of the amount of expandable clay; cover of nonexpansive glacial deposits; and low-rated areas with nonexpansive and small quantities of expansive soils. Environmental factors, such as climatic effects, vegetation, drainage, and effects of man, were not considered.

(3) *Soil Conservation Service county soil surveys.* *Survey* maps by SCS provide the most detailed surficial soil maps available, but not all of the United States is mapped. Soil surveys completed during the 1970's contain engineering test data, estimates of soil engineering properties, and interpretations of properties for each of the major soil series within the given county. The maps usually treat only the upper 30 to 60 inches of soil and, therefore, may not fully define the foundation soil problem.

(4) *U.S. and State Geological Survey maps. The* U.S. Geological Survey is currently preparing hazard maps that will include expansive soils.

c. Application of hazard maps. Hazard maps provide basic information indicative of the probable degree of expansiveness and/or frequency of occurrence of swelling soils. These data lead to initial estimates for the location and relative magnitude of the swelling problem to be expected from the foundation soils. The SCS count y survey maps prepared after 1970, if available, provide more detail on surface soils than do the other maps discussed in *b* above. The other maps used in conjunction with the SCS maps provide a better basis for election of the construction site.

(1) Recognition of the problem area at the construction site provides an aid for the planning of field exploration that will lead to the determination of the areal extent of the swelling soil formations and samples for the positive identification and evaluation of potential swell of the foundation soils and probable soil movements beneath the structure.

(2) Problem areas that rate highly or moderately expansive on any of the hazard maps should be explored to investigate the extent and nature of the swelling soils. Structures in even low-rated areas of potential swell may also be susceptible to damages from heaving soil depending on the ability of the structure to tolerate differential foundation movement. These low-rated areas can exhibit significant differential soil heave if construction leads to sufficiently large changes in soil moisture and uneven distribution of loads. Also, low-rated areas on hazard maps may include some highly swelling soil that had been neglected.

(3) Figure 2-1 indicates that most problems with swelling soils can be expected in the northern central, central, and southern states of the continental United States. The Aliamanu crater region of Fort Shafter, Hawaii, is another example of a problem area.

CHAPTER 3

FIELD EXPLORATION

3-1. Scope

The field study is used to determine the presence, extent, and nature of expansive soil and groundwater conditions. The two major phases of field exploration are surface examination and subsurface exploration. The surface examination is conducted first since the results help to determine the extent of the subsurface exploration. In situ tests may also be helpful, particularly if a deep foundation, such as drilled shafts, is to be used.

3-2. Surface examination

a. Site history. A study of the site history may reveal considerable qualitative data on the probable future behavior of the foundation soils. Maps of the proposed construction site should be examined to obtain information on wooded areas, ponds and depressions, water-courses, and existence of earlier buildings. Surface features, such as wooded areas, bushes, and other deep-rooted vegetation in expansive soil areas, indicate potential heave from accumulation of moisture following elimination of these sources of evapotranspiration. The growth of mesquite trees, such as found in Texas, and other small trees may indicate subsurface soil with a high affinity for moisture, a characteristic of expansive soil. Ponds and depressions are often filled with clayey, expansive sediments accumulated from runoff. The existence of earlier structures on or near the construction site has probably modified the soil moisture profile and will influence the potential for future heave beneath new structures.

b. Field reconnaissance. A thorough visual examination of the site by the geotechnical engineer is necessary (table 3-1). More extensive subsurface exploration is indicated if a potential for swelling soil is evident from damages observed in nearby structures. The extent of desiccation cracks, plasticity, slickensides, and textures of the surface soil can provide a relative indication of the potential for damaging swell.

(1) *Cracking in nearby structures.* The appearance of cracking in nearby structures should be especially noted. The condition of on-site stucco facing, joints of brick and stone structures, and interior plaster walls can be a fair indication of the possible degree of swelling that has occurred. The differential heave that may occur in the foundation soil beneath the proposed structure. however, is not necessarily equal to the dif-

ferential heave of earlier or nearby structures. Differential heave depends on conditions such as variation of soils beneath the structure, load distribution on the foundation, foundation depth, and changes in groundwater since construction of the earlier structures.

(2) *Soil gilgai.* The surface soil at the site should also be examined for gilgai. Soil gilgai are surface mounds that form at locations where the subsurface soil has a greater percentage of plastic fines and is thus more expansive than the surface soil. Gilgai begin to form at locations where vertical cracks penetrate into the subsurface soil. Surface water enters and swelling takes place around the cracks leaving fractured zones where plastic flow occurs. These mounds usually have a higher pH than the adjacent low areas or depressions and may indicate subsurface soil that had extruded up the fractures.

(3) *Site access and mobility.* Indicators of site access and mobility (table 3-1) may also influence behavior of the completed structure. For example, nearby water and sewer lines may alter the natural moisture environment. Flat land with poor surface drainage, as indicated by ponded water, may aggravate differential heave of the completed structure if drainage is not corrected during construction. Construction on land with slopes greater than 5 degrees may lead to structural damage from creep of expansive clay surface soils. Trees located within a distance of the proposed structure of 1 to 1.5 times the height of mature trees may lead to shrinkage beneath the structure, particularly during droughts.

c. Local design and construction experience. Local experience is very helpful in indicating possible design and construction problems and soil and groundwater conditions at the site. Past successful methods of design and construction and recent innovations should be examined to evaluate their usefulness for the proposed structure.

3-3. Subsurface exploration

Subsurface exploration provides representative samples for visual classification and laboratory tests. Classification tests are used to determine the lateral and vertical distribution and types of foundation soils. Soil swell, consolidation, and strength tests are needed to evaluate the load/displacement behavior and bearing capacity of the foundation in swelling soil. The struc-

Table 3-1. Field Reconnaissance

Indicators of swelling soil	1.	Desiccation cracks	Cracks appear in the ground surface during dry periods. Larger and more frequent polygon arrangements of cracks indicate greater potential swell. Dry strength of exposed surfaces is high.
	2.	Plasticity	Relative ease to roll into a small thread indicates greater potential swell.
	3.	Slickensides	Slickensides and fissures are abundant in freshly exposed surfaces of many swelling soils.
	4.	Texture	Slick, cohesive soil tending to adhere to shoes or tires of vehicles when wet indicates swelling soil.
	5.	Structure distortion	Relative size and frequency of cracks and distortion in nearby structures indicates the relative potential swell. Potential swell is approximately the sum of the crack widths. Appearance of power lines, fences, or trees often gives an indication of creep behavior.
	6.	Gilgai	Surface mounds of rounded or long, narrow shape.
Indicators of site access and mobility	1.	Restrictions on access.	
	2.	Locations of utilities and restrictions concerning removal of relocation.	
	3.	Locations of existing structures on site and adjacent to the site. Description of foundation types. Obtain photographs if it can be reasonably expected that existing structures may be affected by construction operations.	
	4.	Locations of trees and other major surface vegetation and restrictions concerning removal or disposition.	
	5.	Surface drainage including presence of ponded water.	
	6.	Examination of contour maps of the site: fill areas, slopes, rock outcrops, or other topographic features.	
	7.	Possible condition of ground at time of construction in relation to trafficability of equipment.	

ture interaction effects in swelling soil are complicated by the foundation differential movement caused by soil heave. Sufficient samples should be available to allow determination of the representative mean of the swell and strength parameters of each distinctive soil stratum. The lower limit of the scatter in strength parameters should also be noted.

a. Sampling requirements. The design of lightly loaded structures and residences can often be made with minimal additional subsurface investigations and soil testing if the site is developed, if subsurface features are generally known, and if the local practice has consistently provided successful and economical designs of comparable structures. Additional subsurface investigation is required for new undeveloped sites, multistory or heavy buildings, structures with previously untested or new types of foundations, and special structures that require unusually limited differential movements of the foundation such as deflection/length ratios less than 1/1000. Where the local practice has not consistently provided satisfactory designs, a careful review of the local practice is neces-

sary. Corrections to improve performance compared with earlier structures may prove difficult to devise and implement and may require evaluation of the behavior of the subsurface foundation soils and groundwater conditions.

b. Distribution and depth of borings. The distribution and depth of borings are chosen to determine the soil profile and to obtain undisturbed samples required to evaluate the potential total and differential heave of the foundation soils from laboratory swell tests, as well as to determine the bearing capacity and settlement. Consequently, greater quantities of undisturbed samples may be required in swelling soils than normally needed for strength tests.

(1) Borings should be spaced to define the geology and soil nonconformities. Spacings of 50 or 25 feet and occasionally to even less distance may be required when erratic subsurface conditions (e.g., soils of different swelling potential, bearing capacity, or settlement) are encountered. Initial borings should be located close to the corners of the foundation, and the number should not be less than three unless subsurface condi-

tions are known to be uniform. Additional borings should be made as required by the extent of the area, the location of deep foundations such as drilled shafts, and the encountered soil conditions.

(2) The depth of sampling should be at least as deep as the probable depth to which moisture changes and heave may occur. This depth is called the depth of the active zone X_a. The active depth usually extends down about 10 to 20 feet below the base of the foundation or to the depth of shallow water tables, but it may be deeper (para 5-4c). A shallow water table is defined as less than 20 feet below the ground surface or below the base of the proposed foundation. The entire thickness of intensely jointed or fissured clays and shales should be sampled until the groundwater level is encountered because the entire zone could swell, provided swelling pressures are sufficiently high, when given access to moisture. Continuous sampling is required for the depth range within the active zone for heave.

(3) Sampling should extend well below the anticipated base of the foundation and into strata of adequate bearing capacity. In general, sampling should continue down to depths of 1.5 times the minimum width of slab foundations to a maximum of 100 feet and a minimum of three base diameters beneath the base of shaft foundations. The presence of a weak, compressible, or expansive stratum within the stress field exerted by the entire foundation should be detected and analyzed to avoid unexpected differential movement caused by long-term volume changes in this stratum. Sampling should continue at least 20 feet beneath the base of the proposed foundation. Determination of the shear strength and stress/strain behavior of each soil stratum down to depths of approximately 100 feet below the foundation is useful if numerical analysis by the finite element method is considered.

c. *Time of sampling.* Sampling may be done when soil moisture is expected to be similar to that during construction. However, a design that must be adequate for severe changes in climate, such as exposure to periods of drought and heavy rainfall, should be based on maximum levels of potential soil heave. Maximum potential heaves are determined from swell tests using soils sampled near the end of the dry season, which often occurs toward the end of summer or early fall. Heave of the foundation soil tends to be less if samples are taken or the foundation is placed following the wet season, which often occurs during spring.

d. *Sampling techniques. The* disturbed samples and the relatively undisturbed samples that provide minimal disturbance suitable for certain laboratory soil tests may be obtained by the methods described in table 3-2. Drilling equipment should be well maintained during sampling to avoid equipment failures, which cause delays and can contribute to sample disturbance.

Personnel should be well trained to expedite proper sampling, sealing, and storage in sample containers.

(1) *Disturbed sampling.* Disturbed auger, pit, or split spoon samplers may be useful to roughly identify the soil for qualitative estimates of the potential for soil volume change (para 4-1). The water content of these samples should not be altered artificially during boring, for example, by pouring water down the hole during augering.

(2) *Undisturbed sampling.* Minimization of sample disturbance during and after drilling is important to the usefulness of undisturbed samples. This fact is particularly true for expansive soils since small changes in water content or soil structure will significantly affect the measured swelling properties.

(a) The sample should be taken as soon as possible, after advancing the hole to the proper depth and cleaning out the hole, to minimize swelling or plastic deformation of the soil to be sampled.

(b) The samples should be obtained using a push tube sampler without drilling fluid, if possible, to minimize changes in the sample water content. Drilling fluids tend to increase the natural water content near the perimeter of the soil sample, particularly for fissured soil.

(c) A piston Denisen or other sampler with a cutting edge that precedes the rotating outer tube into the formation is preferred, if drilling fluid is necessary, to minimize contamination of the soil sample by the fluid.

e. *Storage of samples.* Samples should be immediately processed and sealed following removal from the boring hole to minimize changes in water content. Each container should be clearly labeled and stored under conditions that minimize large temperature and humidity variations. A humid room with relative humidity greater than 95 percent is recommended for storage since the relative humidity of most natural soils exceeds 95 percent.

(1) *Disturbed samples.* Auger, pit, or other disturbed samples should be thoroughly sealed in waterproof containers so that the natural water content can be accurately measured.

(2) *Undisturbed samples.* Undisturbed samples may be stored in the sampling tubes or extruded and preserved, then stored. Storage in the sampling tube is not recommended for swelling soils even though stress relief may be minimal, The influence of rust and penetration of drilling fluid or free water into the sample during sampling may adversely influence the laboratory test results and reduce the indicated potential heave. Iron diffusing from steel tubes into the soil sample will combine with oxygen and water to form rust. Slight changes in Atterberg limits, erosion resistance, water content, and other physical properties may occur. In addition, the outer perimeter of a soil sample

Table 3-2. Soil Sampling Methods

Type of Sample	Purpose	Sampler	Description	Application
Disturbed	Profile classification: Specific gravity Grain-size distribution	Auger	Bucket	All soils where wall can be maintained without caving. Continuous flight augers not recommended as the location in the profile cannot be approximated.
	Atterberg limits Water content Physicochemical[a] Lime treatment[b]	Split spoon	Tube sampler split lengthwise	
		Pit	Shallow trench or large borehole	Capable of providing large quantities of soil for special tests such as compaction or chemical stabilization.
Undisturbed	In situ classification:	Pit	Shallow trench or large borehole	Capable of providing large quantites of soil for special tests such as compaction or chemical stabilization.
	Swell behavior Shear strength	Push tube	Pistonless: driving head fixed to sampling tube with ball pressure release valve to bleed off compressed air and form vacuum during sampler withdrawal	Medium to stiff clays free of gravel or small rocks that could damage the leading edge of the tube sampler.
			Free piston: piston locked at lower end of sampler during insertion into hole and resting on top of sample during push. Vacuum assisted sampler withdrawal	Medium to stiff clays free of gravel or small rocks that could damage the leading edge of the tube sampler.
			Fixed piston: piston fixed to drill rig during the push causing vacuum to assist during the push and sampler withdrawal	Medium to stiff clays free of gravel or small rocks that could damage the leading edge of the tube sampler.
		Rotary core barrel	Double-barrel or Denison Sampler: outer barrel with cutter shoe to advance the sampler and inner barrel with cutter edge to fine trim and contain the sample	Hard soils and soils containing gravel.
			Single barrel: with cutter shoe, usually diamond head, to advance and contain the sample	Rock.

[a] Discussed in paragraph 4-1d.
[b] Discussed in paragraph 7-3d.

stored in the sampling tube cannot be scraped to remove soil contaminated by water that may have penetrated into the perimeter of the sample during sampling. The sample may also later adhere to the tube wall because of rust. If samples are stored in tubes, the tubes should be brass or lacquered inside to inhibit corrosion. An expanding packer with a rubber O-ring in both ends of the tube should be used to minimize moisture loss. The following procedures should be followed in the care and storage of extruded samples.

(a) Expansive soil samples that are to be extrubed and stored should be removed from the sampling tubes immediately after sampling and thoroughly sealed to minimize further stress relief and moisture loss. The sample should be extruded from the sampling tube in the same direction when sampled to minimize further sample disturbance.

(b) Samples extruded from tubes that were obtained with slurry drilling techniques should be wiped clean to remove drilling fluid adhering to the surface of the sample prior to sealing in the storage containers. An outer layer of 1/8 to 1/4 inch should be trimmed from the cylindrical surface of the samples so that moisture from the slurry will not penetrate into the sample and alter the soil swelling potential and strength. Trimming will also remove some disturbance at the perimeter due to sidewall friction. The outer perimeter of the soil sample should also be trimmed away during preparation of specimens for laboratory tests.

(c) Containers for storage of extruded samples may be either cardboard or metal and should be approximately 1 inch greater in diameter and 1.5 to 2 inches greater in length than the sample to be encased. Three-ply, wax-coated cardboard tubes with metal bottoms are available in various diameters and lengths and may be cut to desired lengths.

(d) Soil samples preserved in cardboard tubes should be completely sealed in wax. The wax and cardboard containers provide an excellent seal against moisture loss and give sufficient confinement to minimize stress relief and particle reorientation. A good wax for sealing expansive soils consists of a 1 to 1 mixture of paraffin and microcrystalline wax or 100 percent beeswax. These mixtures adequately seal the sample and do not become brittle when cold. The temperature of the wax should be approximately 20 degrees Fahrenheit above the melting point when applied to the soil sample, since wax that is too hot will penetrate pores and cracks in the sample and render it useless, as well as dry the sample. Aluminum foil or plastic wrap may be placed around the sample to prevent penetration of molten wax into open fissures. A small amount of wax (about 0.5-inch thickness) should be placed in the bottom of the tube and allowed to partly congeal. The sample should subsequently be placed in the tube,

completely immersed and covered with the molten wax, and then allowed to cool before moving.

(e) When the samples are being transported, they should be protected from rough rides and bumps to minimize further sample disturbance.

f. Inspection. A competent inspector or engineer should accurately and visually classify materials as they are recovered from the boring. Adequate classification ensures the proper selection of samples for laboratory tests. A qualified engineering geologist or foundation engineer should closely monitor the drill crew so that timely adjustments can be made during drilling to obtain the best and most representative samples. The inspector should also see that all open boreholes are filled and sealed with a proper grout, such as a mixture of 12 percent bentonite and 88 percent cement, to minimize penetration of surface water or water from a perched water table into deeper strata that might include moisture deficient expansive clays.

3-4. Groundwater

Meaningful groundwater conditions and engineering properties of subsurface materials can often best be determined from in situ tests. In situ tests, however, are not always amenable to simple interpretation. The pore water conditions at the time of the test may differ appreciably from those existing at the time of construction. A knowledge of groundwater and the negative pore water pressure are important in evaluating the behavior of a foundation, particularly in expansive soil. Every effort should be made to determine the position of the groundwater level, its seasonal variation, and the effect of tides, adjacent rivers, or canals on it.

a. Measurement of groundwater level. The most reliable and frequently the only satisfactory method for determining groundwater levels and positive pore water pressures is by piezometers with tips installed at different depths. Ceramic porous tube piezometers with small diameters (3/8-inch) risers are usually adequate, and they are relatively simple, inexpensive, and sufficient for soils of low permeability.

b. Measurement of in situ negative pore water pressure, Successful in situ measurements of negative pore water pressure and soil suction have been performed by such devices as tensiometers, negative pore pressure piezometers, gypsum blocks, and thermocouple psychrometer. However, each of these devices has certain limitations, The range of tensiometers and negative pore pressure piezometers has been limited to the cavitation stress of water under normal conditions, which is near one atmosphere of negative pressure. The fluid-filled tensiometer is restricted to shallow soils less than 6 feet in depth. The useable range of the tensiometer is reduced in proportion to the pressure exerted by the column of fluid in the tensiometer. Gyp-

sum blocks require tedious calibration of electrical resistivity for each soil and dissolved salts greatly influence the results. Thermocouple psychrometer cannot measure soil suctions reliably at negative pressures that are less than one atmosphere and require a constant temperature environment. Psychrometer also measure the total suction that includes an osmotic component caused by soluble salts in the pore water, as well as the matrix suction that is comparable with the negative pore water pressure. Tensiometers require constant maintenance, while gypsum blocks and psychrometer tend to deteriorate with time and may become inoperable within one year. A routine field measurement of soil suction is not presently recommended because of the limitations associated with these devices. Alternatively, laboratory measurements of soil suction can be easily performed (para 4-2a).

CHAPTER 4

LABORATORY INVESTIGATIONS

4-1. Identification of swelling soils

Soils susceptible to swelling can be identified by classi-
fication tests. These identification procedures were de-
veloped by correlations of classification test results
with results of one-dimensional swell tests performed
in consolidometers on undisturbed and compacted soil
specimens. Classification data most useful for identi-
fying the relative swell potential include the liquid
limit (LL), the plasticity index (PI), the COLE (para
2-2b(2)), the natural total soil suction τ_{nat}^o, and physico-
chemical tests. Several of the more simple and success-
ful methods recommended for identifying swelling soil
from classification tests described below were devel-
oped from selected soils and locations combined with
the results of limited field observations of heave.
These procedures assume certain environmental condi-
tions for surcharge pressure (e.g., 1 pound per square
inch) and changes in moisture from the initial water
content (e.g., to saturation or zero final pore water
pressure),

a. WES classification. Consolidometer swell tests

were performed on 20 undisturbed clays and clay
shales from the states of Mississippi, Louisiana, Texas,
Oklahoma, Arizona, Utah, Kansas, Colorado, Wyo-
ming, Montana, and South Dakota. Results of these
tests for a change in moisture from natural water con-
tent to saturation at the estimated in situ overburden
pressure (pressures corresponding to depths from 1 to
8 feet) indicated the degrees of expansion and poten-
tial percent swell S_p shown in table 4-1. The S_p repre-
sents the percent increase in the vertical dimension or
the percent potential vertical heave. The classification
may be used without knowing the natural soil suction,
but the accuracy and conservatism of the system are
reduced. Soils that rate low may not require further
swell tests, particularly if the LL is less than 40 per-
cent and the PI is less than 15 percent. Soils with these
Atterberg limits or less are essentially nonexpansive.
However, swell tests may be required for soils of low
swelling potential if the foundation of the structure is
required to maintain small differential movements
less than 1 inch (para 4-2c).

Table 4-1. WES Classification of Potential Swell

Classification of potential swell	Potential swell S_p percent	Liquid limit LL percent	Plasticity index PI percent	Natural soil suction τ_{nat}^o tsf
Low	<0.5	<50	<25	<1.5
Marginal	0.5-1.5	50-60	25-35	1.5-4.0
High	>1.5	>60	>35	>4.0

*b. Texas Department of Highways and Public
Transportation (TDHPT) method.* This procedure,
which is known as Tex-124-E of the TDHPT Manual
of Testing Procedures, is based on the swell test results
of compacted soils from Texas. Field heaves of each
soil stratum in the profile are estimated from a family
of curves using the LL, PI, surcharge pressure on the
soil stratum, and initial water content. The initial wa-
ter content is compared with maximum (0.47 LL + 2)
and minimum (0.2 LL + 9) water contents to evaluate
the percent volumetric change. The potential vertical
rise (PVR) of each stratum is found from a chart using
the percent volumetric change and the unit load bear-
ing on the stratum. These PVRs for depths of as much
as 30 feet or more are summed to evaluate the total
PVR. This method may overestimate the heave of low
plasticity soils and underestimate the heave of high
plasticity soils.

c. Van Der Merwe method. This method evolved
from empirical relationships between the degree of ex-
pansion, the PI, the percent clay fraction, and the sur-
charge pressure, The total heave at the ground surface
is found from

$$\Delta H = \sum_{D=1}^{D=n} F \cdot PE \qquad (4-1)$$

where
ΔH = total heave, inches
D = depth of soil layer in increments of 1 foot
 = increment at the deepest level
F = reduction factor for surcharge pressure, $F = 10^{-D/20}$
PE = potential expansiveness in inch/foot of depth (fig. 4-1)

The PE is found by assumed values of PE = 0, ¼, ½,
and 1 inch/foot for low, medium, high, and very high

levels, respectively, of potential expansiveness, defined in figure 4-1 as functions of the PI and the minus 2μ fraction. The PE values are based on consolidometer swell test results and field observations. This method does not consider variations in initial moisture conditions.

d. Physiochemical tests. These tests include identification of the clay minerals, such as montmorillonite, illite, attapulgite, and kaolinite, with kaolinite being relatively nonexpansive, cation exchange capacity (CEC), and dissolved salts in the pore water. The CEC is a measure of the property of a clay mineral to exchange ions for other anions or cations by treatment in an aqueous solution. The relatively expansive montmorillonite minerals tend to have large CEC exceeding 80 milliequivalents per 100 grams of clay, whereas the CEC of nonexpansive kaolinite is usually less than 15 milliequivalents. The presence of dissolved salts in the pore water produces an osmotic component of soil suction that can influence soil heave if the concentration of dissolved salts is altered. In most cases, the osmotic suction will remain constant and not normally influence heave unless, for example, significant leaching of the soil occurs.

e. Other methods. Other methods that have been successful are presented in table 4-2. These methods lead to estimates of the percent swell S_p or vertical heave assuming that all swell is confined to the vertical direction, and they require an estimate of the depth of the active zone X_a (para 5-4c). Both the TDHPT and Van Der Merwe methods do not require estimates of X_a since computations extend down to depths where the computed heaves become negligible. The Van Der Merwe, McKeen-Lytton, and Johnson methods tend to give maximum values or may overestimate heave, whereas the remaining methods tend to give minimum values or may underestimate heave when compared with the results of field observations at three WES test sections.

f. Application. These identification tests along with the surface examination of paragraph 3-2 can indicate problem soils that should be tested further and can provide a helpful first estimate of the expected in situ heave.

(1) More than one identification test should be used to provide rough estimates of the potential heave because limits of applicability of these tests are not known. In general, estimates of potential heave at the ground surface of more than 1/2 inch may require further laboratory tests, particularly if local experience suggests swelling soil problems. Soil strata in which the degree of expansion is medium or high should also be considered for further swell tests (para 2-2c).

(2) The McKeen-Lytton method of table 4-2 has been applied to the prediction of potential differential heave for average changes in moisture conditions by the Post-Tensioning Institute (PTI) for design and con-
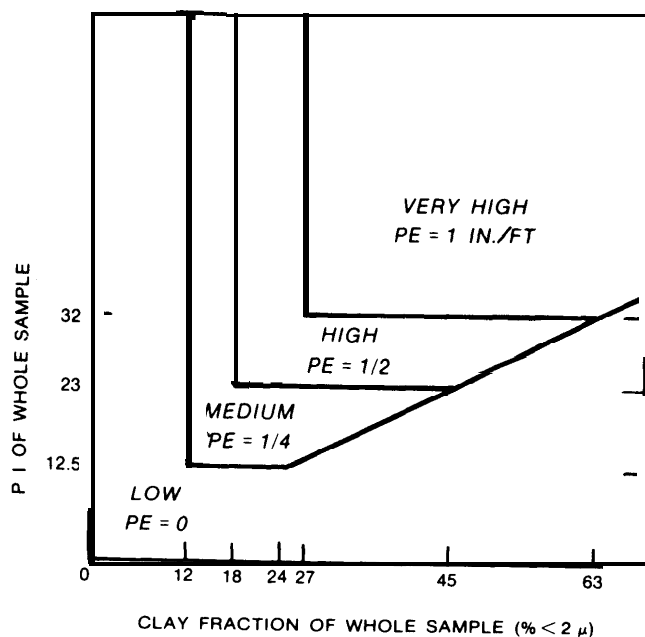
(Based on data from Van Der Merwe, 1964, published in The Civil Engineer with permission granted by the S. A. Institute of Civil Engineers, Johannesburg, South Africa)

Figure 4-1. Relationship used to determine the potential expansiveness for Van Der Merwe's empirical method.

Table 4-2. Other Empirical Methods for Prediction of Potential Heave.

Method	Description[a]
Vijayvergiya and Ghazzaly	$\text{Log } S_p = 1/12(0.44\text{LL} - w_o + 5.5)$ from initial water content to saturation for 0.1-tsf surcharge pressure.
Schneider and Poor	$\text{Log } S_p = 0.9(\text{PI}/w_o) - 1.19$ for no fill or weight on the swelling soil to saturation.

McKeen-Lytton by McKeen	$S_p = -100\gamma_h \log_{10} \dfrac{\bar{\tau}_f}{\bar{\tau}_o}$
	
	 where
	     γ_h = suction compression index
	     τ_f = final weighted in situ suction
	     τ_o = initial in situ weighted suction
	 The γ_h is found from a chart using the CEC, PI, and percent clay. The weighted suction is given by $\bar{\tau} = 0.5\tau_1 + 0.3\tau_2 + 0.2\tau_3$ where τ_1, τ_2, and τ_3 are in situ suctions measured in the top, middle, and bottom third of the active zone.

Johnson	$\text{PI} \geq 40 \quad S_p = 23.82 + 0.7346\text{PI} - 0.1458\text{H} - 1.7w_o + 0.0025\text{PI}w_o - 0.00884\text{PIH}$ $\text{PI} \leq 40 \quad S_p = -9.18 + 1.5546\text{PI} + 0.08424\text{H} + 0.1w_o = 0.0432\text{PI}w_o - 0.01215\text{PIH}$ for 1 psi surcharge pressure to saturation.

[a] S_p = percent swell; LL = liquid limit in percent; PI = plasticity index in percent; w_o = initial water content in percent; H = depth of soil in feet.

struction of stiffened slabs-on-grade in expansive soils. The PTI structural design procedure is described in paragraph 6-3b.

4-2. Testing procedures

Quantitative characterization of the expansive soil from swell tests is necessary to predict the anticipated potential soil heave devaluation of swell behavior and predictions of total and differential heave are determined from the results of tests on undisturbed specimens. Strength tests may be performed to estimate the bearing capacity of the foundation soil at the final or equilibrium water content. A measure of shear strength with depth is also needed to evaluate soil sup-

port from adhesion along the perimeter of shaft foundations or the uplift that develops on the shaft when swelling occurs.
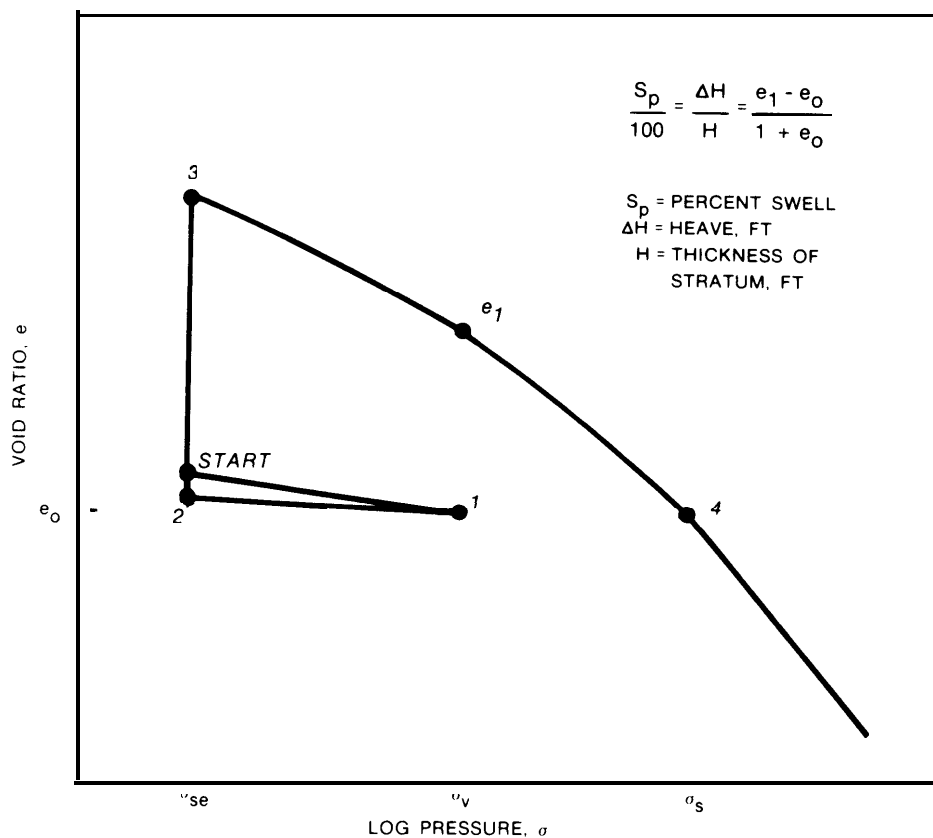
a. Swell tests. Laboratory methods recommended for prediction of the anticipated volume change or potential in situ heave of foundation soils are consolidometer swell and soil suction tests, The WES expansive soil studies show that consolidometer swell tests may underestimate heave, whereas soil suction tests may overestimate heave compared with heaves measured in the field if a saturated final moisture profile is assumed (chap 5). The economy and simplicity of soil suction tests permit these tests to be performed at frequent intervals of depth from 1 to 2 feet.

(1) *Consolidometer.* Recommended consolidometer swell tests include swell and swell pressure tests described in Appendix VIII of EM 1110-2-1906. The swell test may be performed to predict vertical heave AH of soil thickness H when the vertical overburden and structural pressures on thickness H are known prior to the test. The total vertical heave at the ground surface is the sum of the AH for each thickness H in the soil profile. Figure 5-4 illustrates the application of swell test data. The swell pressure test is performed to evaluate the swell pressure d_s and swell index C_s required for prediction of vertical heave by equation

(5-8) discussed in paragraph 5-4e. The confining pressure required to restrain heave is defined as d_s. When little is known about swell behavior or groundwater conditions, an appropriate swell test is given in *(a)* and *(b)* below.

(a) An initial loading pressure, simulating field initial (preconstruction) vertical pressure &, should be applied to determine the initial void ratio $e_{.,}$ point 1 of figure 4-2, then removed to the seating pressure d_{se} (i.e., the lowest possible load) prior to adding distilled water, point 2. The specimen is allowed to expand at the seating pressure until primary swell is complete, point 3, before applying the consolidation pressures.

(b) The swell test of figure 4-2 can eliminate the need for additional tests when behavior is different than that anticipated (e.g., the specimen consolidates rather than swells following addition of water at loading pressures greater than the seating pressure). The void ratio-log pressure curve for final effective pressures, varying from the seating to the maximum applied pressure, can be used to determine heave or settlement with respect to the initial void ratio e_o. Net settlements will occur for final effective pressures exceeding the swell pressure d_s. Figure 4-2 illustrates how the percent swell S_p or heave ΔH may be found with respect to the initial vertical pressure&.

$$\frac{S_p}{100} = \frac{\Delta H}{H} = \frac{e_1 - e_0}{1 + e_0}$$

S_p = PERCENT SWELL
ΔH = HEAVE, FT
H = THICKNESS OF STRATUM, FT

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Figure 4-2. Simple swell test.

(*c*) The d_s in figure 4-2 is defined as confining pressure that must be applied to the soil to reduce the volume expansion down to the (approximated) in situ e_o in the presence of free water. Consolidometer tests in appendix VIII of EM 1110-2-1906 tend to provide lower limits of the in situ swell pressure, while the simple swell test, figure 4-2, tends to provide upper limits. The maximum past pressure is often a useful estimate of the in situ swell pressure at e_o.

(2) *Soil suction.* Soil suction is a quantity that also can be used to characterize the effect of moisture on volume changes and, therefore, to determine the anticipated foundation soil heave. The suction is a tensile stress exerted on the soil water by the soil mass that pulls the mass together and thus contributes to the apparent cohesion and undrained shear strength of the soil. The thermocouple psychrometer and filter paper methods, two of the simplest approaches for evaluation of soil suction and characterization of swelling behavior, are described in appendix B. The suction procedure, which is analogous to the procedure for characterization of swell from consolidometer swell tests, is relatively fast, and the results can increase confidence in characterization of swell behavior.

b. Strength tests. The results of strength tests are used to estimate the soil bearing capacity and load/deflection behavior of shaft or other foundations. The critical time for bearing capacity in many cases is immediately after completion of construction (first loading) and prior to any significant soil consolidation under the loads carried by the foundation. The long-term bearing capacity may also be critical in expansive foundation soils because of reductions in strength from wetting of the soil.

c. Application. Sufficient numbers of swell and strength tests should be performed to characterize the soil profiles. Swell tests may not be necessary on specimens taken at depths below permanent deep groundwater levels.

(1) The representative mean of the swell and strength parameters (and lower limit of the scatter in strength parameters) of each distinctive soil stratum should be determined down to depths of 1.5 times the minimum width of mat slabs to a maximum of 100 feet and to at least three base diameters beneath the base of shaft foundations.

(2) One consolidometer swell and one strength test should be performed on specimens from at least five undisturbed samples at different depths within the depth of the anticipated active zone (e.g., within 10 to 20 feet beneath the base of the foundation). Suction tests may also be performed at relatively frequent depth intervals (e.g., l-foot increments) to better characterize swell behavior and thereby increase confidence in prediction of potential heave discussed in chapter 5.

(3) One consolidometer swell and one strength test should be performed on specimens from each undisturbed sample (or at intervals of 2.5 feet. for continuous sampling) at depths above the base of deep shaft foundations to permit evaluation of the adjacent soil heave and uplift forces exerted on the shaft/soil interface, Suction tests may also be performed to further characterize swell behavior and increase confidence in prediction of potential heave.

(4) Suction test results can characterize the pore pressure profile by indicating depths of desiccation and wetting, which are useful for minimizing potential foundation problems from soil movement and for evaluating remedial measures to correct problems.

APPENDIX A

REFERENCES

Government Publications

Department of the Army

Technical Manuals

TM 5-818-1	Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures)
TM5-818-4	Backfill for Subsurface Structures
TM5-822-4	Soil Stabilization for Roads and Streets

Department of the Army, Corps of Engineers

* U.S. Army Construction Engineering Research Laboratory (CERL), P.O. Box 4005, Champaign, IL 61820

*   TR M-81   Structures on Expansive Soils

    USACE Publications Depot, 2803 52nd Avenue, Hyattsville, MD 20781

      EM 1110-2-1906   Laboratory Soils Testing

    Waterways Experiment Station, P.O. Box 631, Vicksburg, MS 39180

      Miscellaneous Paper   User's Guide for Computer Program, HEAVE

      GL-82-7

Department of the Navy

    Naval Publications and Forms Center, 5801 Tabor Ave., Philadelphia, PA 19120

*   NAVFAC DM-7.2   Foundations and Earth Structures Design Manual

Non-Government Publications

*   American Concrete Institute (ACI), P.O. Box 19150, Detroit, MI 48219

*   304-73   Measuring, Mixing, Transporting, and Placing Concrete

*   (R 1978)

    American Society of Civil Engineers (ASCE), 345 East 47th St., New York, N.Y. 10017

      Assessment of Expensive Soils in the United States, Proceedings of the Fourth International Conference on Expansive Soil, (Volume 1, pp. 596-608, 1980)

    American Society of Testing and Materials (ASTM), 1916 Race Street, Philadelphia, PA 19103

*   A 615-82   Deformed and Plain Billet-Steel Bars for Concrete Reinforcement

*   D 2521-76   Asphalt Used in Canal, Ditch, and Pond Lining

*   (R 1981)

    Post-Tensioning Institute, 301 Osborn, Suite 3500, Phoenix, AZ 85013

*   Design and Construction of Post-Tensioned Slabs-on-Ground, (First Edition, 1980)

    Texas Department of Highways and Public Transportation, Austin, Texas 78703

      Manual of Testing Procedures, 100-E Series, Department D-9, 1978

    Transportation Research Board, 2101 Constitution Ave., N.W., Washington, D.C. 20418

      "Relationship Between Physiographic Units and Highway Design Factors," National Cooperative Highway Research Program, Report 132, 1977

APPENDIX B

CHARACTERIZATION OF SWELL BEHAVIOR FROM SOIL SUCTION

B-1. Introduction

Soil suction is a quantity that can be used to characterize the effect of moisture on volume, and it is a measure of the energy or stress that holds the soil water in the pores or a measure of the pulling stress exerted on the pore water by the soil mass. The total soil suction is expressed as a positive quantity and is defined as the sum of matrix τ_m^o and osmotic τ_s suctions.

a. Matrix suction. The matrix suction τ_m^o is related to the geometrical configuration of the soil and structure, capillary tension in the pore water, and water sorption forces of the clay particles. This suction is also pressure-dependent and assumed to be related to the in situ pore water pressure u_w by

$$\tau_m^o = -u_w + \alpha \delta_m \qquad (B-1)$$

$$\delta_m = \frac{1 + 2K_T}{3} \delta_v \qquad (B-2)$$

where

τ_m^o = matrix soil suction, tons per square foot
α = compressibility factor, dimensionless
δ_m = total mean normal confining pressure, tons per square foot
K_T = ratio of total horizontal to vertical stress in situ
δ_v = total vertical pressure, tons per square foot

The exponent "^{o}" means that the τ_m^o is measured without confining pressure except atmospheric pressure. Experimental results show that the in situ matrix suction τ_m is equivalent to $-u_w$ for soils. The compressibility factor is determined by the procedure in paragraph B-3d.

b. Osmotic suction. The osmotic suction τ_s is caused by the concentration of soluble salts in the pore water, and it is pressure-independent. The effect of the osmotic suction on swell is not well known, but an osmotic effect may be observed if the concentration of soluble salts in the pore water differs from that of the externally available water. For example, swell may occur in the specimen if the external water contains less soluble salts than the pore water. The effect of the osmotic suction on swell behavior is assumed small compared with the effect of the matrix suction. The osmotic suction should not significantly affect heave if the salt concentration is not altered.

B-2. Methods of measurement

Two methods are recommended for determining the total soil suction: thermocouple psychrometer and filter paper. The suction range of thermocouple psychrometers usually is from 1 to 80 tons per square foot while the range of filter paper is from 0.1 to more than 1,000 tons per square foot. Two to seven days are required to reach moisture equilibrium for thermocouple psychrometer, while 7 days are required for filter paper. The thermocouple psychrometer method is simple and can be more accurate than filter paper after the equipment has been calibrated and the operating procedure established. The principal disadvantage is that the suction range is much more limited than the filter paper method, The filter paper method is technically less complicated than the thermocouple psychrometer method; however, the weighing procedure required for filter paper is critical and vulnerable to large error.

a. Calibration. The total soil suction is given on the basis of thermodynamics by the equation

$$\tau^o = -\frac{RT}{v_w} \ln \frac{p}{p_o} \qquad (B-3)$$

where

τ^o = total suction free of external pressure except atmospheric pressure, tons per square foot
R = universal gas constant, 86.81 cubic centimetres-tons per square foot/mole-Kelvin
T = absolute temperature, Kelvin
v_w = volume of a mole of liquid water, 18.02 cubic centimetres/mole
p/p_o = relative humidity
p = pressure of water vapor, tons per square foot
p_o = pressure of saturated water vapor, tons per square foot

Equation (B-3) shows that the soil suction is related to the relative humidity in the soil. Both thermocouple psychrometer and filter paper techniques require calibration curves to evaluate the soil relative humidity from which the soil suction may be calculated using equation (B-3). Calibration is usually performed with salt solutions of various known molality (moles of salt per 1,000 grams of water) that produce a given relative humidity. Table B-1 shows the modalities re-

Table B-1. Calibration Salt Solutions

Measured temperature t, °C	Suction, tsf for cited molality of sodium chloride solution						
	0.053	0.100	0.157	0.273	0.411	0.550	1.000
15	3.05	4.67	7.27	12.56	18.88	25.29	46,55
20	3.10	4.74	7.39	12.75	19.22	25.76	47.50
25	3.15	4.82	7.52	13.01	19.55	26.23	48.44
30	3.22	4.91	7.64	13.22	19.90	26.71	49.37

quired for sodium chloride salt solutions to provide the soil suctions given as a function of temperature.

b. Thermocouple psychrometer technique. The thermocouple psychrometer measures relative humidity in soil by a technique called Peltier cooling. By causing a current to flow through a single thermocouple junction in the proper direction, that particular junction will cool, then water will condense on it when the dewpoint temperature is reached. Condensation of this water inhibits further cooling of the junction. Evaporation of condensed water from the junction after the cooling current is removed tends to maintain a difference in temperature between the thermocouple and the reference junctions. The microvoltage developed between the thermocouple and the reference junctions is measured by the proper readout equipment and related to the soil suction by a calibration curve.

(1) *Apparatus.* Laboratory measurements to evaluate total soil suction may be made with the apparatus illustrated in figure B-1. The monitoring system includes a cooling circuit with the capability of immediate switching to the voltage readout circuit on termination of the current (fig. B-2). The microvoltmeter (item 1, fig. B-2) should have a maximum range of at least 30 microvolt and allow readings to within 0.01 microvolt. The 12-position rotary selector switch (item 2) allows up to 12 simultaneous psychrometer connections. The 0-25 millimeter (item 3), two 1.5-volt dry cell batteries (item 4), and the variable potentiometer (item 5) form the cooling circuit, Equipment is available commercially to perform these measurements of soil suction.

(2) *Procedure.*

(a) Thermocouple psychrometer are inserted into 1-pint-capacity metal containers with the soil specimens, and the assembly is sealed with No. 13-1/2 rubber stoppers. The assembly is inserted into a 1- by 1- by 1.25-foot chest capable of holding six l-pint containers and insulated with 1.5 inches of foamed polystyrene, Cables from the psychrometer are passed through a 0.5-inch-diameter hole centered in the chest cover, The insides of the metal containers are coated with melted wax to inhibit corrosion of the containers.

(b) The apparatus is left alone until equilibrium is attained. Temperature equilibrium is attained within a few hours after placing the chest cover. Time to reach equilibrium of the relative humidity in the air

measured by the psychrometer and the relative humidity in the soil specimen depends on the volume and initial relative humidity in the container. Equilibrium time may require up to 7 days, but may be reduced to 2 or 3 days by repeated testing of soils with similar suctions.

(c) After equilibrium is attained, the microvoltmeter is set on the 10- or 30-microvolt range and zeroed by using a zeroing suppression or offset control. The cooling current of approximately 8 millimeters is applied for 15 seconds and then switched to the microvoltmeter circuit using the switch of item 6 in figure B-2, The maximum reading on the microvoltmeter is recorded. The cooling currents and times should be identical to those used to determine the calibration curves.

(d) The readings can be taken at room temperature, preferably from 20 to 25 degrees Centigrade, and corrected to a temperature of 25 degrees Centigrade by the equation

$$E_{25} = \frac{E_t}{0.325 + 0.027_t} \qquad \text{(B-4)}$$

where

$E_{25}=$ microvolt at 25 degrees Centigrade
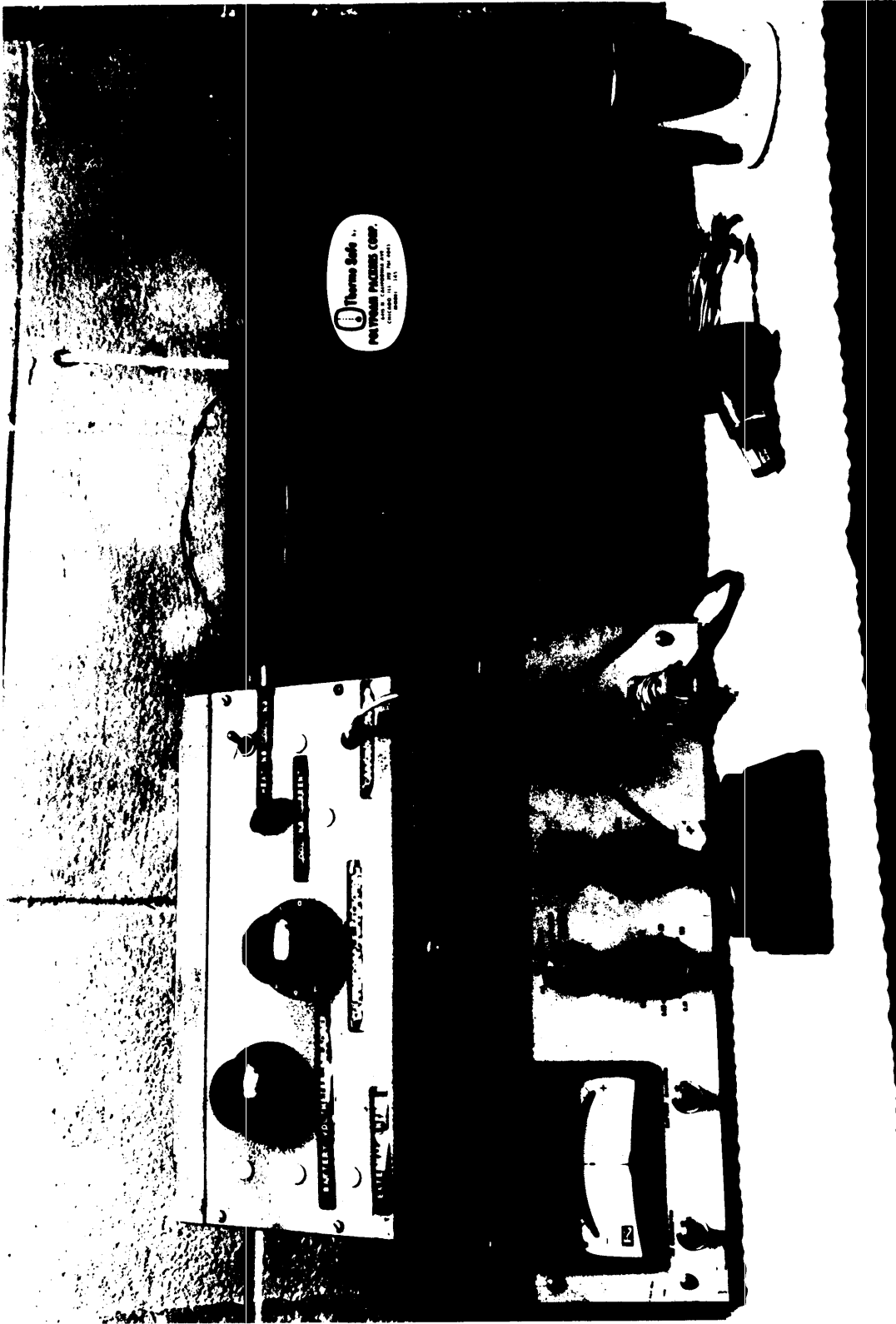
$E_t=$ microvolt at t degrees Centigrade

Placement of the apparatus in a constant temperature room will increase the accuracy of the readings.

(3) *Calibration,* The psychrometer are calibrated by placing approximately 50 millilitres of the salt solutions of known molality (table B-1) in the metal containers and following the procedure in b(2) above to determine the microvolt output. Equilibration time may be reduced to 2 or 3 days by surrounding the psychrometer with filter paper soaked with solution. The suctions given for the known modalities are plotted versus the microvolt output for a temperature of 25 degrees Centigrade. The calibration curves of 12 commercial psychrometer using the equipment of figure B-1 were within 5 percent and could be expressed by the equation

$$\text{To} = 2.65E_{25} - 1.6 \qquad \text{(B-5)}$$

where $\tau°$ is the total soil suction in tons per square foot. The calibration curves using other equipment may be somewhat different.

c. Filter paper technique. This method involves en- — closing filter paper with a soil specimen in an airtight container until complete moisture equilibrium is

Figure B-1. Thermocouple psychrometer monitoring apparatus.

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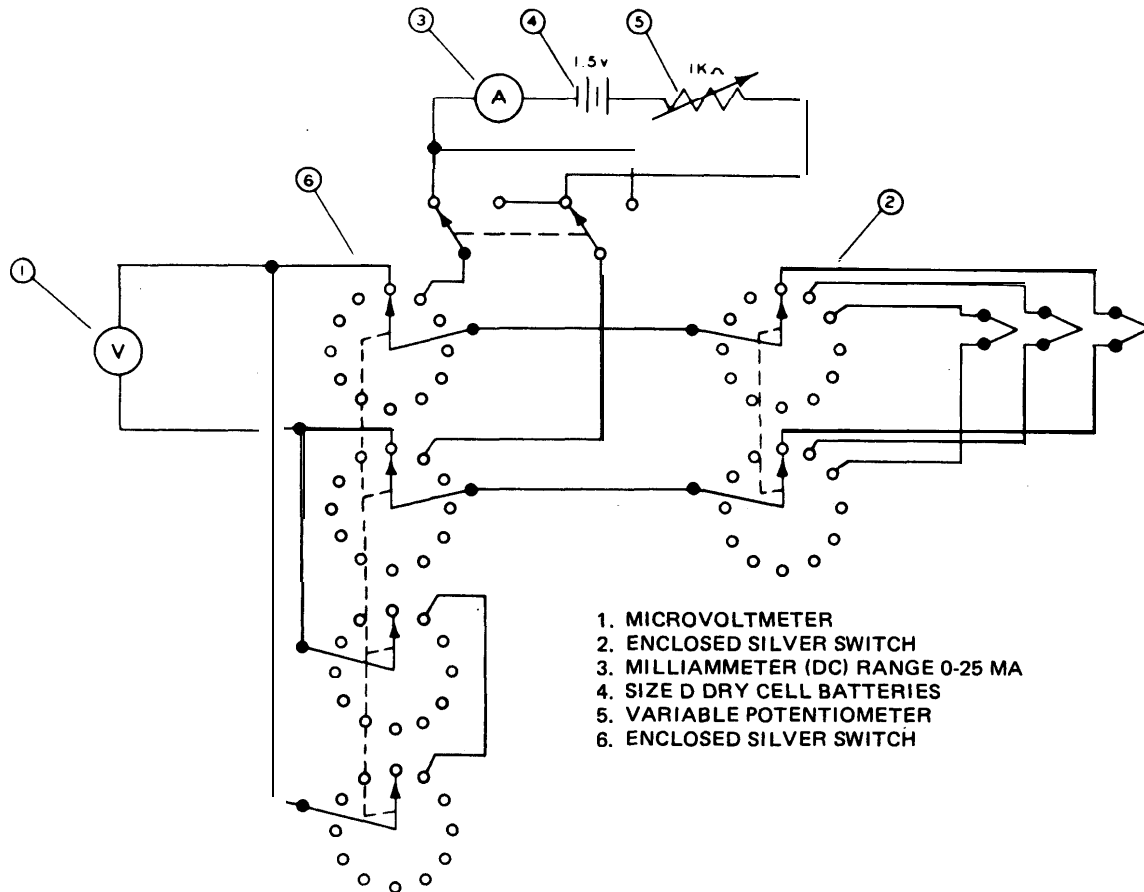
1. MICROVOLTMETER
2. ENCLOSED SILVER SWITCH
3. MILLIAMMETER (DC) RANGE 0-25 MA
4. SIZE D DRY CELL BATTERIES
5. VARIABLE POTENTIOMETER
6. ENCLOSED SILVER SWITCH

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Figure B-2. Electrical circuit for the thermocouple psychrometer.

reached. The water content in percent of the dry weight is subsequently determined, and the soil suction is found from a calibration curve.

(1) *Apparatus.* Materials consist of 2-inch-diameter filter paper, 2-inch-diameter tares, and a gravimetric scale accurate to 0.001 g. A filter paper is enclosed in an airtight container with the soil specimen.

(2) *Procedure.*

(a) The filter paper disc is pretreated with 3 percent reagent grade pentachlorophenol in ethanol (to inhibit bacteria and deterioration) and allowed to air dry. Reagent grade pentachlorophenol is required because impurities in the treatment solution influence the calibration curve. Care is required to keep the filter paper from becoming contaminated with soil from the specimen, free water, or other contaminant (e.g., the filter paper should not touch the soil specimen, particularly wetted specimens).

(b) Seven days are required to reach moisture equilibrium in the airtight container. At the end of 7 days, the filter paper is transferred to a 2-inch-cliameter covered tare and weighed immediately on a gravimetric scale accurate to 0.001 g. The number of

filter papers and tares weighed at one time should be kept small (nine or less) to minimize error caused by water evaporating from the filter paper.

(c) The tare is opened and placed in an oven for **at least 4 hours or overnight at a temperature of 110 ±** 5 degrees Centigrade. The ovendry weight of the filter paper is then determined, and the water content as a percent of the dry weight is compared with a calibration curve to determine the soil suction.

(3) *Calibration.* The ovendry water content of the filter paper is dependent on the time lapse following removal from the drying oven before weighing.

(a) The calibration curves shown in figure B-3 were determined for various elapsed times following removal from the oven. The calibrations are given for Fisherbrand filter paper, Catalog Number 9-790A, enclosed with salt solutions of various molality for 7 days. Calibration curve No. 1 resulted from weighing the filter paper 5 seconds following removal from the oven. Time lapses of 15 minutes and 4 hours lead to a similar calibration curve (No. 3) of significantly smaller water contents than the 5-second curve for identical suctions. Calibration curve No. 2 was determined

by removing 12 specimens from the oven, waiting 30 seconds to cool, then weighing as soon as possible and within 15 minutes.

(b) Calibration curves based on the method used to determine curve No, 3 with a waiting time between 15 and 30 minutes are recommended if the suctions of
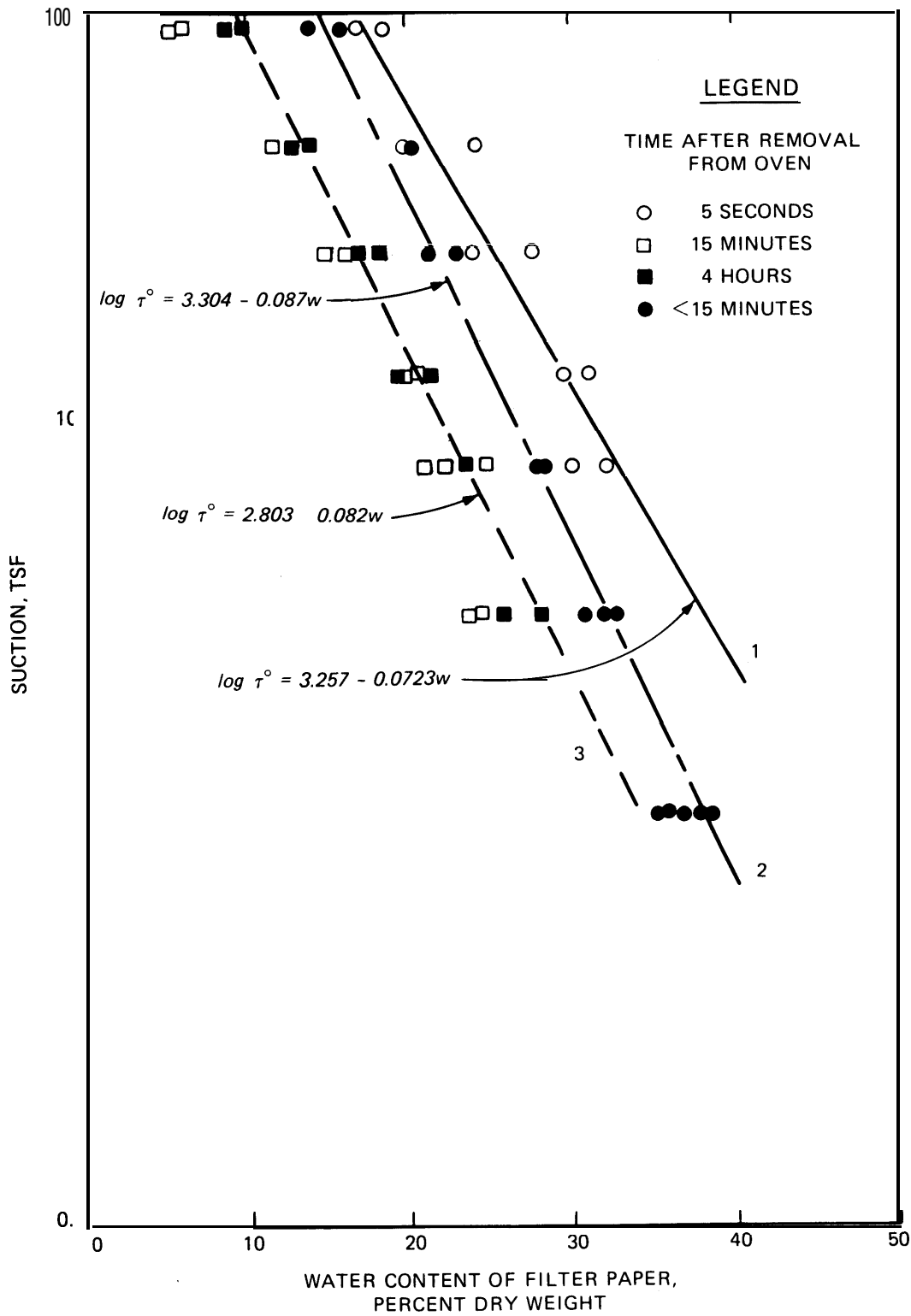

Figure B-3. Calibration of filter paper.

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large numbers of specimens are to be evaluated. However, the accuracy will be less than if curve No. 1 and its procedure are used because curve No. 3 can be influenced by changes in the relative humidity of the testing room. The gravimetric scale should be located near the drying oven for the 5-second calibration curve (No. 1) to be practical. Changes in filter paper weights are normally small (e.g., less than 0.1 g) and require accurate calibration of the gravimetric scale and adherence to a single standardized procedure.

B-3. Characterization of swell behavior

The swell behavior of a particular soil may be characterized from the matrix suction-water content relationship and the compressibility factor α to calculate heave by the equation

$$\frac{\Delta H}{H} = \frac{e_1 - e_0}{1 + e_0} = \frac{C_\tau}{1 + e_0} \log \frac{\tau_{mo}^0}{\tau_{mf}^0} \quad (B-6)$$

where

ΔH = potential vertical heave at the bottom of the foundation, feet

H = thickness of the swelling soil

e_1 = final void ratio following swell

e_0 = initial void ratio

C_τ = $\alpha G_s/100B$, suction index

α = compressibility factor

G_s = specific gravity

B = slope soil suction parameter

τ_{mo}^0 = initial matrix suction without surcharge pressure, tons per square foot

τ_{mf}^0 = final matrix suction without surcharge pressure, tons per square foot

The suction index C_τ is similar to $\gamma_h(1 + e_0)$ where γ_h is the suction compression index of the McKeen-Lytton method in table 4-2. Equation (B-6) is similar to equation (5-2) of paragraph 5-4a and equation (5-8) of paragraph 5-4a from which the total potential heave is calculated. Equation (B-6) will also lead to the same or similar predictions of heave for identical changes in suction. The suction index, a measure of the swelling capability, is analogous to the swell index c_s of consolidometer swell tests, except that the suction index is evaluated with respect to the change in matrix suction without surcharge pressure rather than the change in effective pressure.

a. Matrix suction and water content relationship. This relationship is evaluated from the total soil suction and water content relationship. The total soil suction as a function of water content is found from multiple 1-inch pieces of the undisturbed sample. The pore water may be evaporated at room temperature, for various periods of time up to about 48 hours, from several undisturbed specimens; various amounts of distilled water may also be added to several other undisturbed specimens of each sample to obtain a multipoint water content distribution. Each specimen may

be inserted into a 1-pint metal container with a thermacouple psychrometer or with filter paper to evaluate the total soil suction as previously described. The dry density and void ratio of each undisturbed specimen from which the compressibility factor α is determined may be evaluated by the water displacement method. Using thermocouple psychrometers, collect soil suction data on DA Form 5182-R (Soil Suction, Water Content and Specific Volume). DA Form 5182-R will be reproduced locally on 11- by 8½-inch paper. A copy of DA Form 5182-R for local reproduction purposes can be found at the back of this manual.

(1) The multipoint total soil suction and water content relationship may be plotted as shown in figure B-4 for each undisturbed sample. The open circles in the figure represent natural water content w_0, and the closed circles symbolize water being added to or evaporated from the undisturbed specimens at room temperature. An osmotic suction τ_s is sometimes indicated by a horizontally inclined slope at high water contents, and the magnitude may be estimated by noting the total soil suction at high water contents. Large osmotic suctions appreciably flatten the slope as shown in figure B-4. The matrix suction and water content relationship can be approximated by subtracting the osmotic suction from the total soil suctions and expressing the result as

$$\log \tau_m^0 = A - Bw \quad (B-7)$$

where

τ_m^0 = matrix suction without surcharge pressure, tons per square foot

A = ordinate intercept soil suction parameter, tons per square foot

B = slope soil suction parameter

w = water content, percent dry weight

Information on piezometric pore water pressures is used in approximating the matrix suction and water content relationship in the presence of appreciable osmotic suctions.

(2) The matrix suction and water content relationship of figure B-4 was approximated by noting that the groundwater elevation, at which $u_w = 0$, was 1.5 feet. Hence, the matrix suction at the natural water content of 27 percent was the total mean confining pressure δ_m of approximately 0.1 ton per square foot from equation (B-1). The value δ_m may be estimated from equation (B-2) if K_T can be approximated. The remainder of the curve was approximated by subtracting 26 tons per square foot, which was the total average suction at the natural water content of 27 percent less 0.1 ton per square foot, from the total soil suction observed at smaller water contents. The osmotic suction should be subtracted from the total suction; otherwise heave predictions will be overestimated since the osmotic suction does not appear to cause much heave and
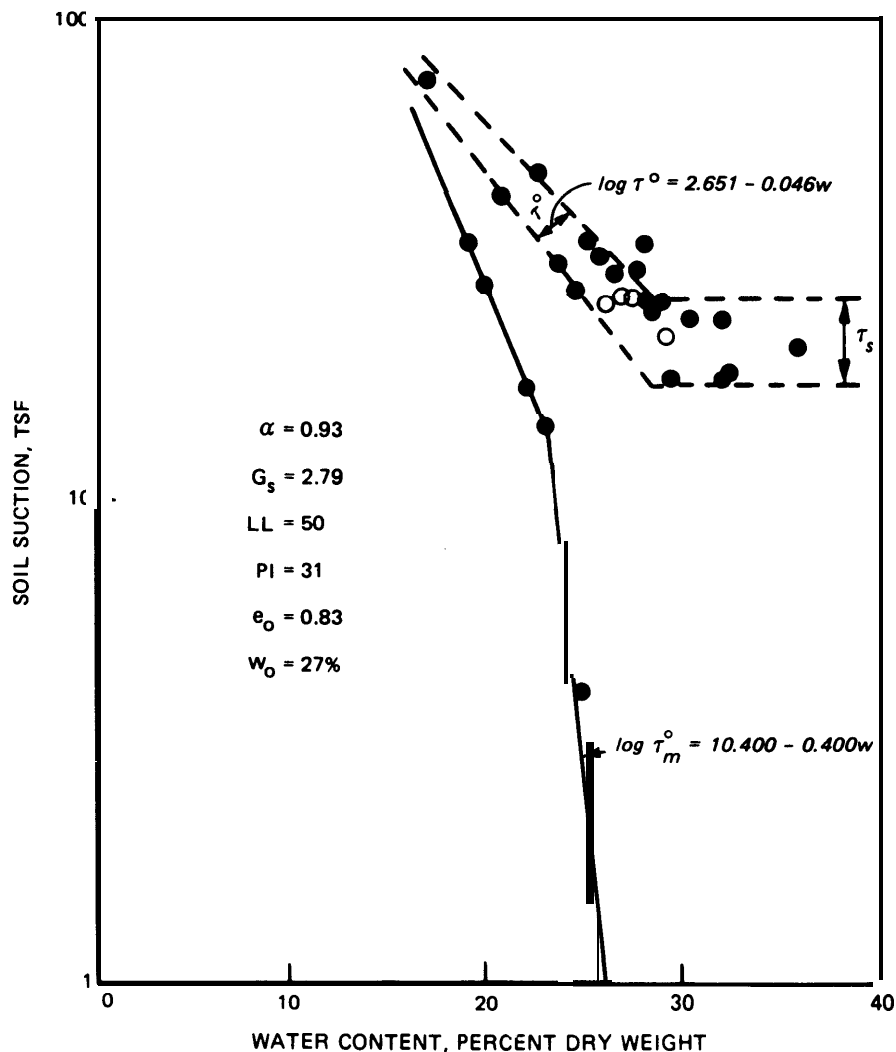
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Figure B-4. Soil suction and water content relationship for Fort Carson overburden at 1 to 3 feet of depth.

if the equilibrium moisture profiles of figure 5-1 (para 5-4b) are used.

b. Initial matrix suction. The initial matrix suction τ&without surcharge pressure may be evaluated using the soil suction test procedure on undisturbed specimens or may be calculated from equation (B-7) and the natural (initial) water content.

c. Final matrix suction. The final matrix suction τ_{mf}^{o} without surcharge pressure may be calculated from the assumption

$$\tau_{mf}^{o} = \left(\frac{1 + 2K}{3} \right) \delta_{v}' \qquad (B-8)$$

K = coefficient of effective lateral earth pressure

δ_{v}' = final vertical effective pressure, tons per square foot or from equation (B-1) setting a = 1 and if K_T can be approximated.

The final vertical effective pressure may be found from

$$\delta_{v}' = \delta_{v} - u_{w} \qquad (B-9)$$

where δ_{v} is the final total vertical pressure. The pore water pressure u_{w} (fig. 5-1) is found from equations (5-3), (5-4), or (5-5).

d. Compressibility factor. The compressibility factor *a* is the ratio of the change in volume for a corresponding change in water content, i.e., the slope of the curve γ_{w}/γ_{d} plotted as a function of the water content where γ_{w} is the unit weight of water and γ_{d} is the dry density. The value of *a* for highly plastic soils is close to 1, and much less than 1 for sandy and low plasticity soils. High compressibility y factors can indicate highly swelling soils; however, soils with all voids filled with water also have a equal to 1.

(1) Figure B-5 illustrates the compressibility factor calculated from laboratory data for a silty clay taken from a field test section near Clinton, Mississippi. Extrapolating the line to zero water content, as shown in the figure, provides an estimate of l/R with

$$R = \frac{W_s}{V_o} \qquad \text{(B-10)}$$

where

R = shrinkage ratio

W_s = mass of a specimen of ovendried soil, grams

V_o = volume of a specimen of ovendried soil, cubic centimetres

(2) The shrinkage limit SL of the clay shown in figure B-5 may be taken as the abrupt change in slope of the curve, which is 23.3 percent. The SL is calculated by the following equation:

$$SL = w - \frac{V - V_o}{W_s} \times 100 \qquad \text{(B-11)}$$

where w is the water content and V is the volume of the wet soil specimen in cubic centimetres. Equation (B-11) assumes that $\alpha = 1$. For soils with α less than 1, the SL varies depending on the initial water content of the specimen. For example, if the initial water content is at the natural water content of 25.7 percent, then equation (B-11) will give

$$SL = 25.7 - (0.658 - 0.588)\,100 = 18.7 \qquad \text{(B-12)}$$

as shown in figure B-5. Other shrinkage limits may be evaluated by drawing straight lines with slope $\alpha = 1$ through other water content points. Soils with the PI less than 40 are more likely to indicate compressibility factors less than 1 than higher plasticity soils. Equation (B-11) is not applicable to soils with α much less than 1.

e. Examples.

(1) The potential heave of the soil characterized by figure B-4 may be calculated from equation (B-6). The final in situ pore water pressure u_w is equal to 0 at the groundwater level of 1.5 feet. If the depth H is 1.5 feet, then $\sigma_v = 0.09$ ton per square foot. From these variables and the parameters in DA Form 5182-R.

$$C_\tau = \frac{\alpha G_s}{100B} = \frac{(0.93)(2.79)}{(100)(0.400)} = 0.065$$

$$\tau_{mo}^o = 10^{10.400-0.400w_o} = 0.398 \text{ ton per square foot}$$

$$\tau_{mf}^o = u_w + \alpha\sigma_v = 0 + 0.93\,(0.09) = 0.084 \text{ ton per square foot}$$

Therefore,

$$\frac{\Delta H}{H} = \frac{c_\tau}{1 + e_o} \log \frac{\tau_{mo}^o}{\tau_{mf}^o}$$

$$= \frac{0.065}{1 + 0.83} \log \frac{0.398}{0.084} = 0.024$$

The potential heave ΔH will be 0.036 foot or 0.4 inch for the 1.5-foot layer of soil overburden. Practically, the computation indicates that ½ inch of heave is expected.

(2) If the osmotic component of suction is not known, then the potential heave may still be roughly approximated by noting that the mean minimum total suction at high water content is 22 tons per square foot in the example of figure B-4. This value may be taken as the final total soil suction τ_f^o. The initial value of total soil suction τ_o^o is found by noting that the mean total soil suction at natural water content is 26 tons per square foot in figure B-4. The slope B of the total soil suction and water content curve is subsequently used to evaluate the suction index C_τ. The potential heave for this case will be

$$CT = \frac{(0.93)\,(2.79)}{(100)\,(0,046)} = 0.564$$

$$\frac{\Delta H}{H} = \frac{C_\tau}{1 + e_o} \log \frac{\tau_o^o}{\tau_f^o}$$

$$= \frac{0.564}{1 + 0.83} \log \frac{26}{22} = 0.022$$

The potential heave ΔH will be 0.033 foot or 0.4 inch for the 1.5-foot layer of soil overburden. The disadvantage of this latter approach is that the equilibrium matrix suction or pore water pressure profile is not known, except that the final matrix suction will be small and probably close to the saturated profile (equation (5-3)). The program HEAVE will compute the potential heave for this case as well as those shown in figure 5-1.
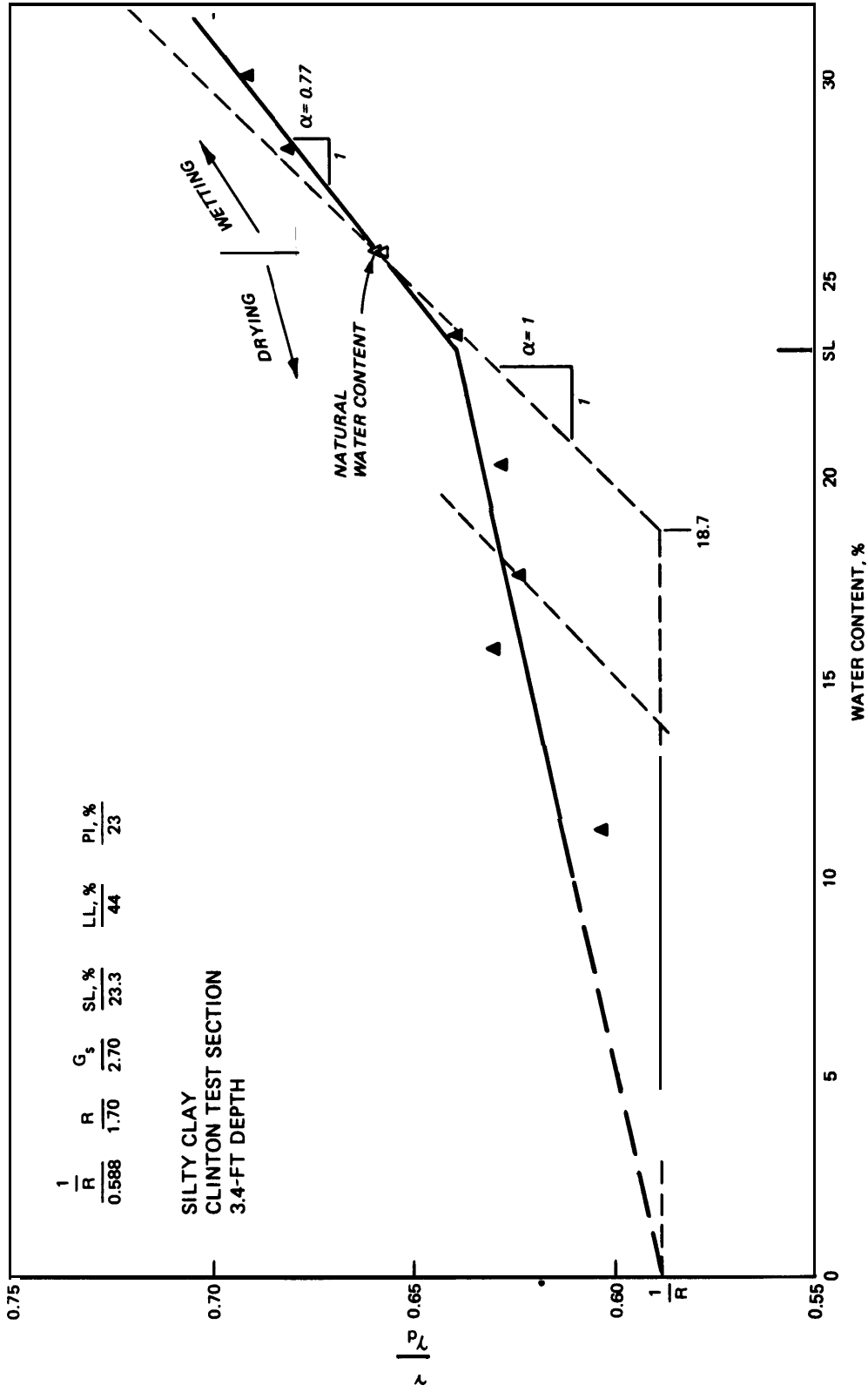
*Figure B-5. Illustration of the compressibility factor.*

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APPENDIX C

FRAME AND WALL CONSTRUCTION DETAILS

Figures C-1 through C-10 illustrate types of construction for expansive foundation soils. These figures were taken from U.S. Army Corps of Engineers Construction Engineering and Research Laboratory Technical Report M-81. The figures show practical wall ties to concrete and steel beams, wall connections with control joints, details of interior partitions, bar joist first floor framing with grade beams, and stiffened mat foundations.
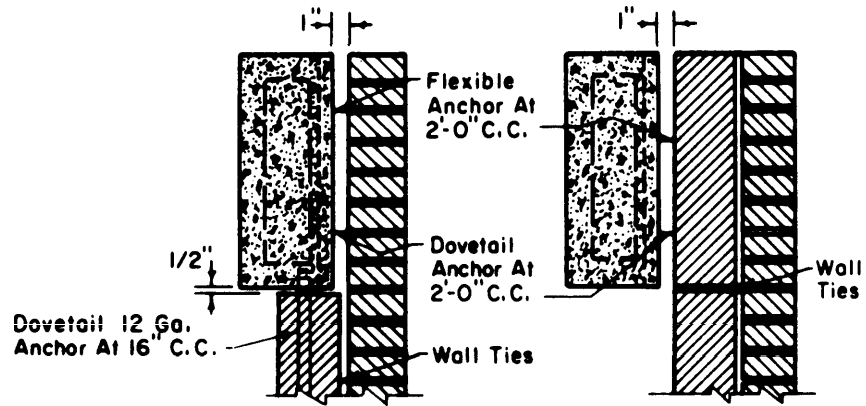
WALL ANCHORAGE TO CONCRETE BEAM
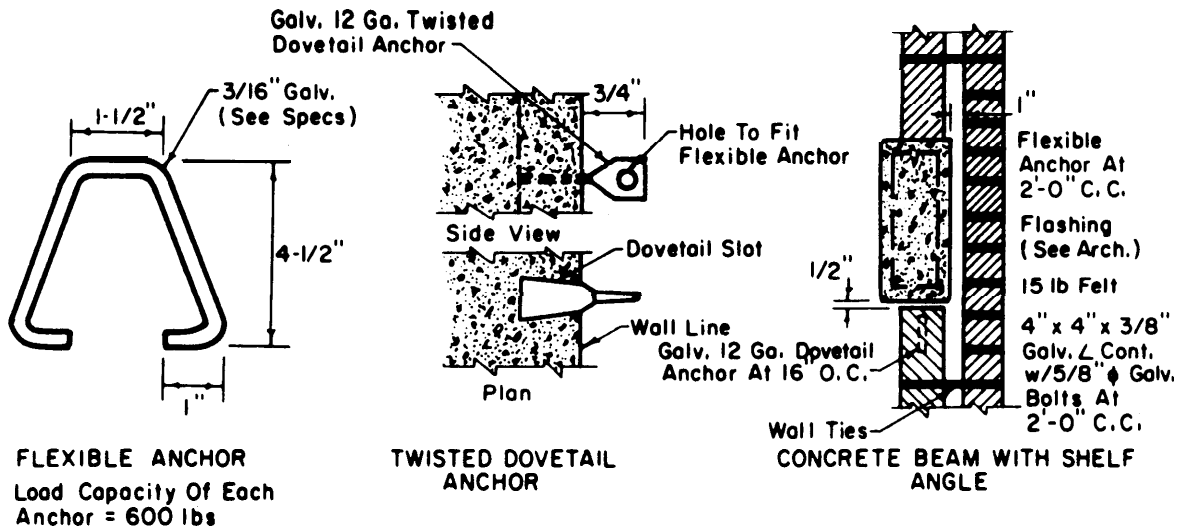
Note:
Ties to beam are required when column ties are omitted.

Figure C-1. Wall ties to concrete beams.

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EXPOSED COLUMN

CONCEALED COLUMN

BYPASSED COLUMN
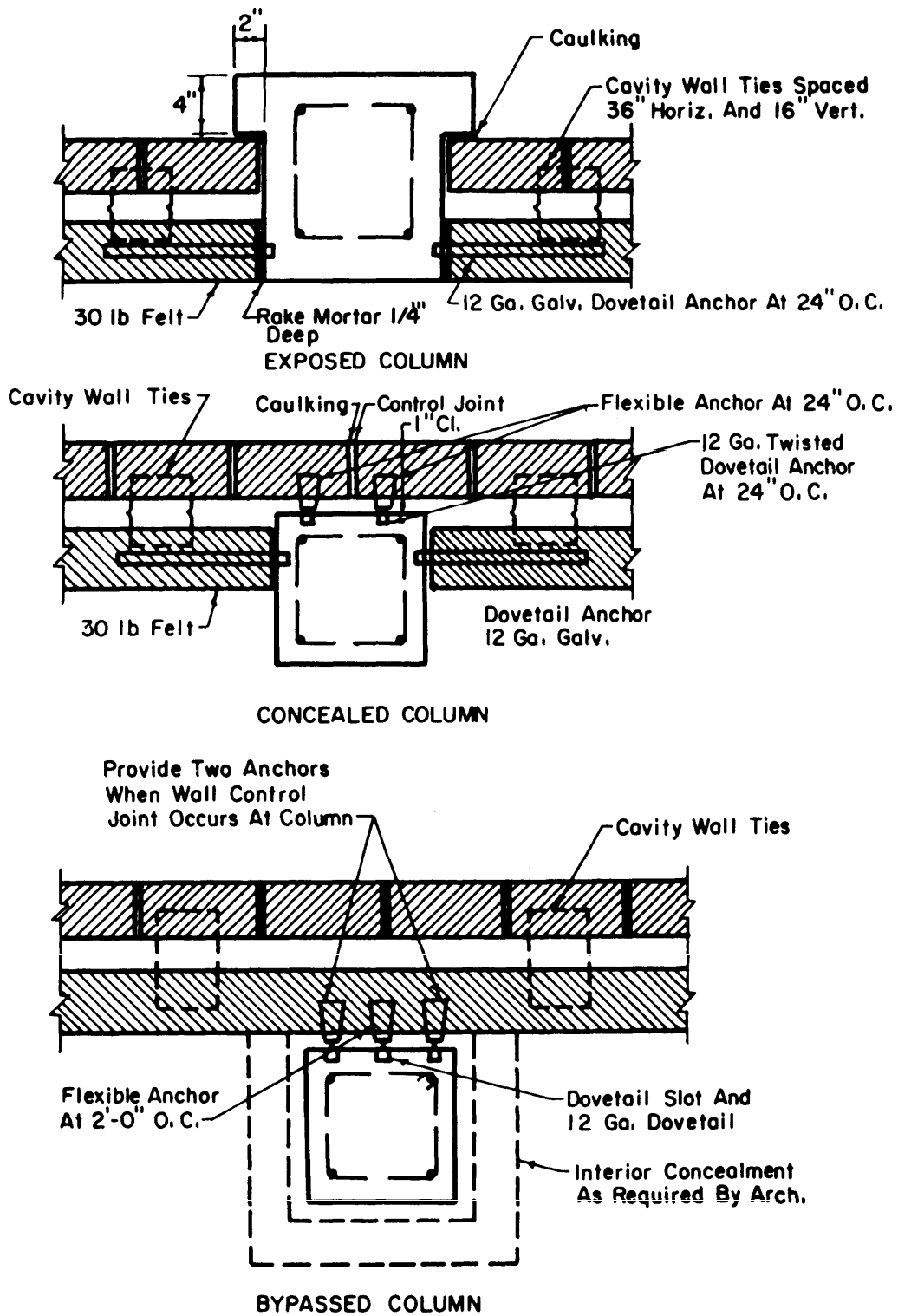
Note: Ties to column are required only when ties to beam are omitted.

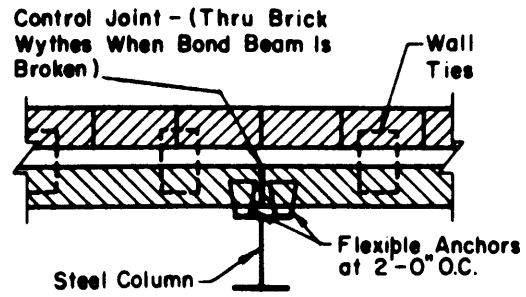*Figure C-2. Wall ties to concrete column.*
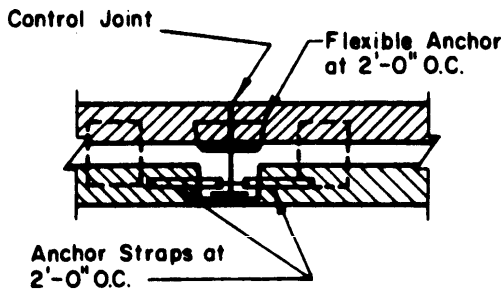
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Steel Column

3/8" Spacer
Plate at 8"O.C.

Masonry
Wall

Cont. 1/4" φ Rod
Welded To Each
Spacer Plate

Flexible Anchor
at 8"O.C.

3/4"

FLEXIBLE ANCHOR

Steel Column
Fill w/Metal Shims
at Anchor Courses
3/8"φ Anchors at 24"O.C.

1/4" Metal Anchor
Clips at 24"O.C.
See Note No.1, Below
Shims as Reqd.

Exterior Face
Concrete or
Mortar Fill
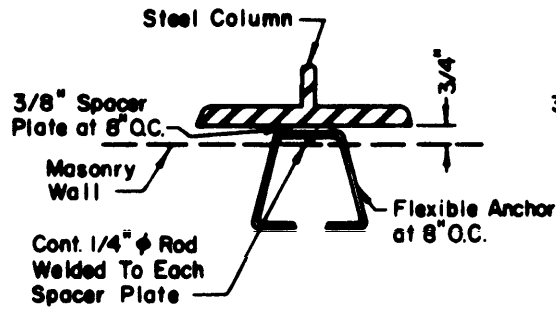
Control Joint

Mortar
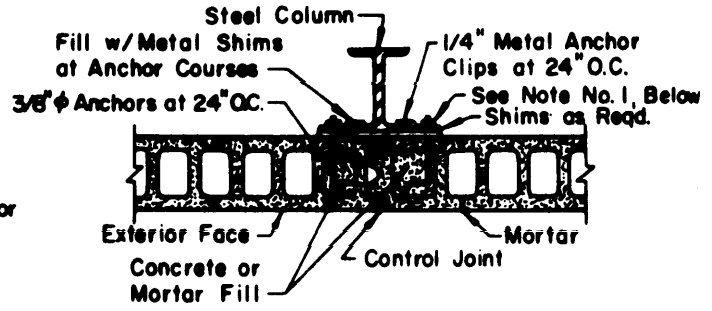
OPTIONAL WALL AND COLUMN CONNECTION

Wall Ties

Flexible Anchor
at 2'-0" O.C.
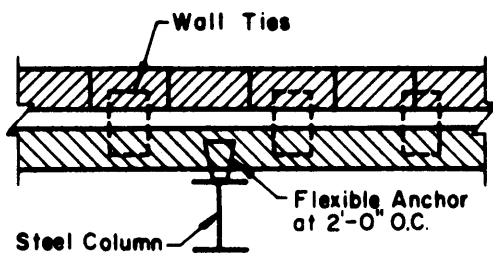
Steel Column

STEEL COLUMN-NO CONTROL JOINT

Control Joint-(Thru Brick
Wythes When Bond Beam Is
Broken)

Wall
Ties

Flexible Anchors
at 2'-0" O.C.

Steel Column

STEEL COLUMN-WITH CONTROL JOINT

Control Joint

Flexible Anchor
at 2'-0" O.C.

Anchor Straps at
2'-0" O.C.

Masonry Shall Be 3/4"
Clear All Around Steel
Column

STEEL COLUMN IN EXTERIOR WALL

Control Joint

Anchor Straps at
2'-0"O.C.
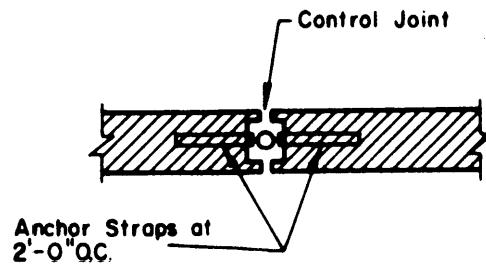
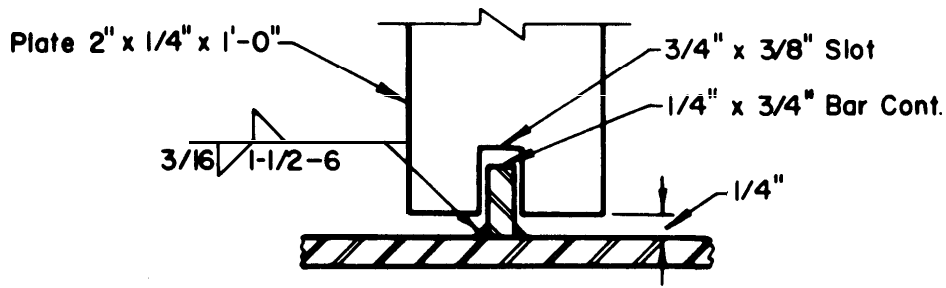Masonry Shall Be 3/4"
Clear All Around Steel
Column

STEEL COLUMN IN INTERIOR WALL

Ties To Columns Are Required Only When Ties To Beam Above Are Omitted.
Do Not Connect Column To Wall At Corners of Buildings

Note:
    1. Nuts Should Not Be Tightened Excessively, Horizontal
    Movement of Wall Is Necessary.
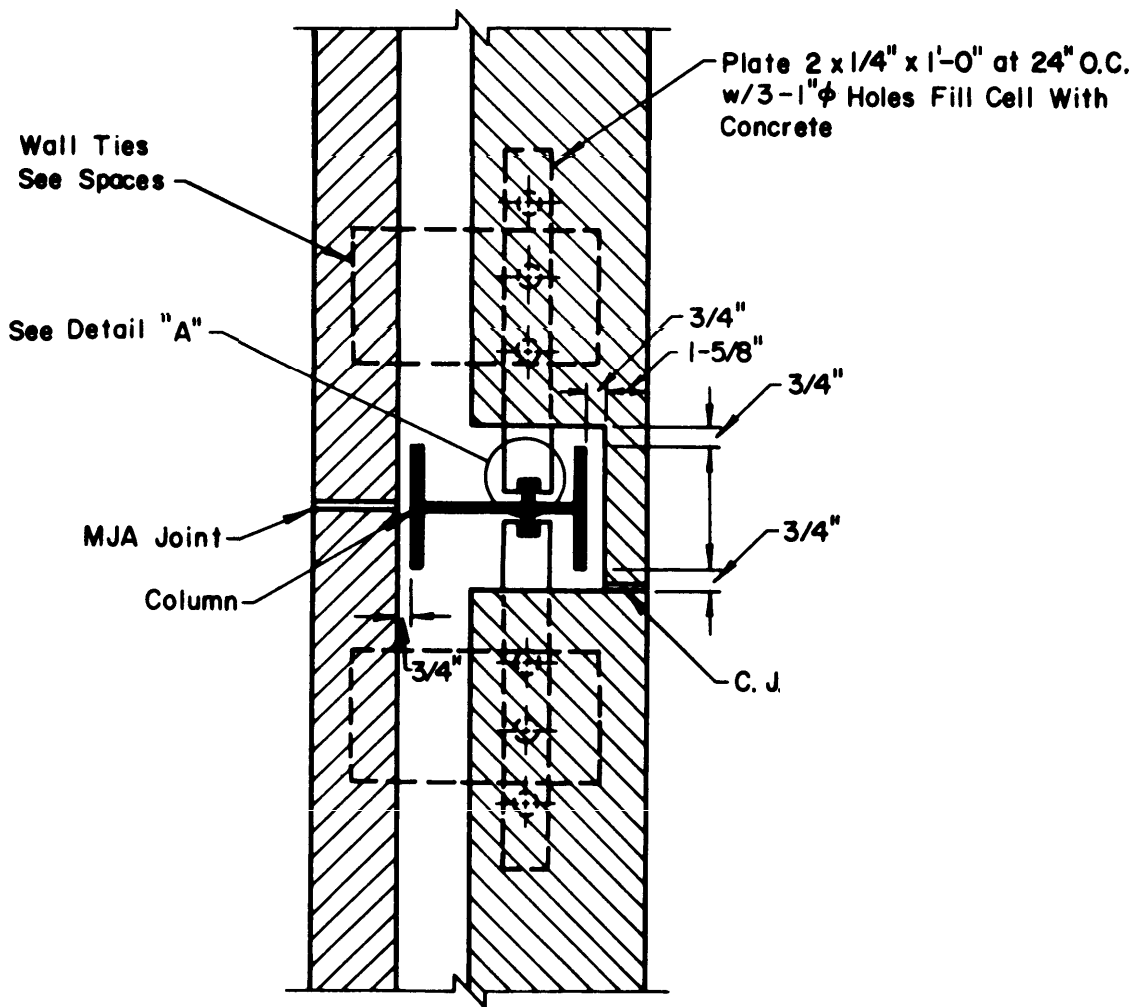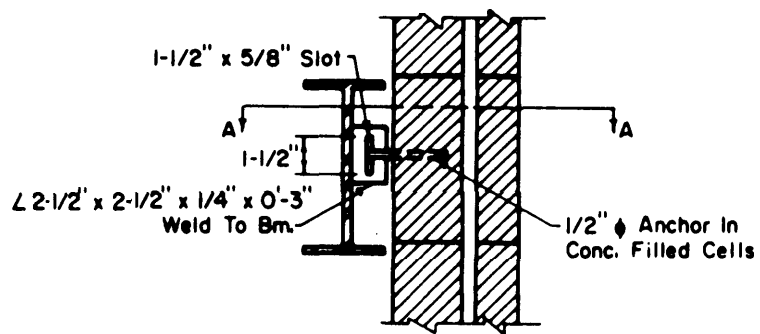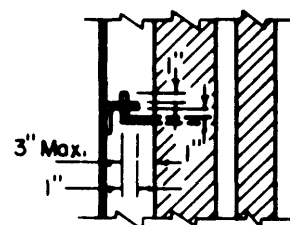
Figure C-3. Wall ties to steel column (Sheet 1 of 2).

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Plate 2" x 1/4" x 1'-0"

3/4" x 3/8" Slot

1/4" x 3/4" Bar Cont.

3/16 $\diagdown$ 1-1/2 - 6

1/4"

DETAIL "A"

Plate 2 x 1/4" x 1'-0" at 24" O.C.
w/3-1"ϕ Holes Fill Cell With
Concrete

Wall Ties
See Spaces

See Detail "A"

3/4"

1-5/8"

3/4"

MJA Joint

Column

3/4"

3/4"

3/4"

C. J.

Figure C-3. (Sheet 2 of 2).

U. S. Army Corps of Engineers

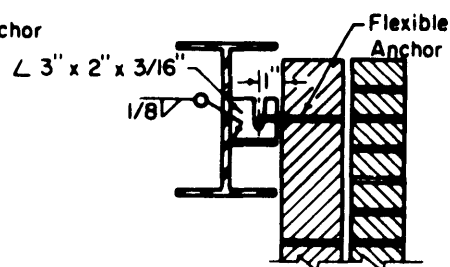I-1/2" x 5/8" Slot

A

I-1/2"

∠ 2-1/2" x 2-1/2" x 1/4" x 0'-3"
Weld To Bm.

1/2" ⌀ Anchor In
Conc. Filled Cells

A

ALTERNATE WALL ANCHOR TO STEEL BEAM ✳

SECTION A-A

3" Max.

1"

∠ 3" x 2" x 3/16"

1/8∇

3/4" Cl.

1/4∇

Bar 2" x 1/4" x 0'-3"

1"

Flexible Anchor

Wall Ties

WALL ANCHORAGE TO STEEL BEAM ✳

∠ 3" x 2" x 3/16"

1/8∇

1"

Flexible
Anchor
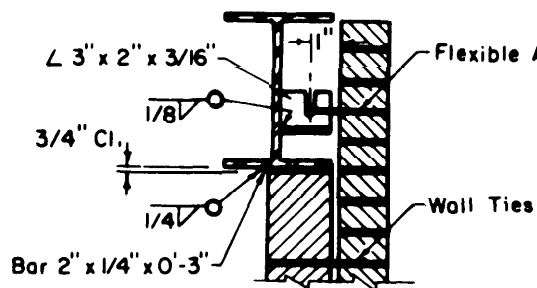
WALL ANCHORAGE TO STEEL BEAM ✳

Note:
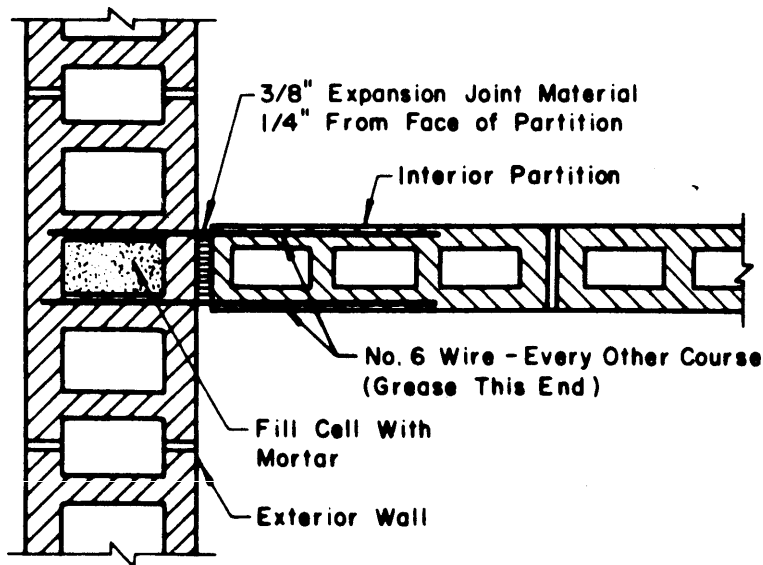   Ties to beam required only when ties to column are omitted.

        ✳ 2'-0" Spacing for Exterior Walls
            4'-0" Spacing for Interior Walls
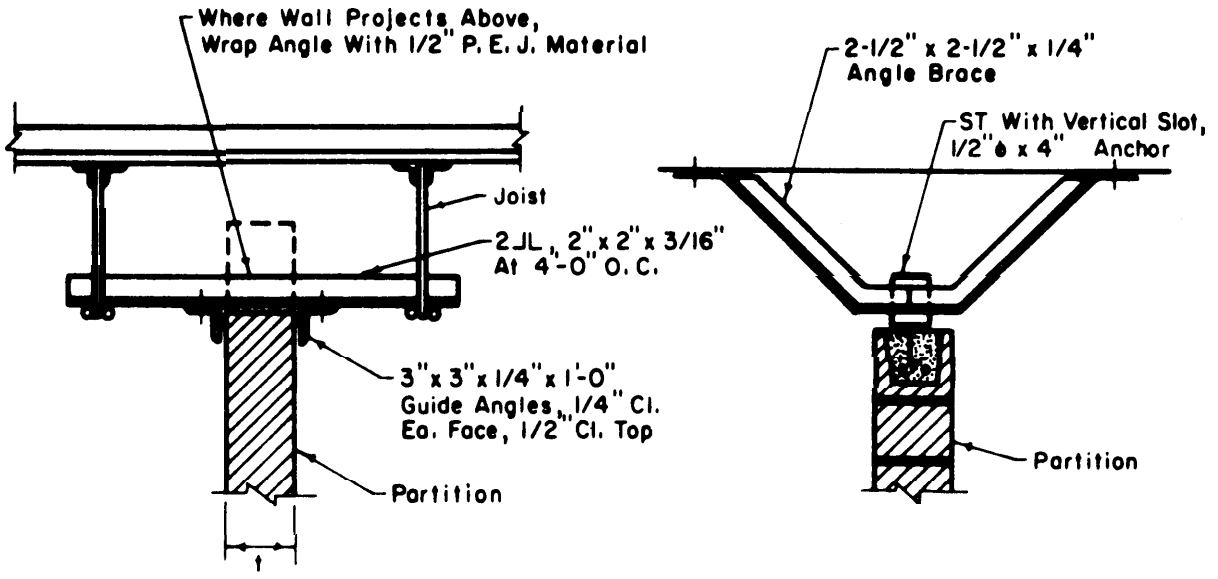
Figure C-4. Wall ties to steel beam.

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3/8" Expansion Joint Material
1/4" From Face of Partition

Grease Horiz. End

Interior Partition

1" x 1/4" x 2'-0" Steel
Anchor - Every 3rd Course

Fill Cell
With Mortar

Exterior Wall

1/4"

3"

2'-0"

1"
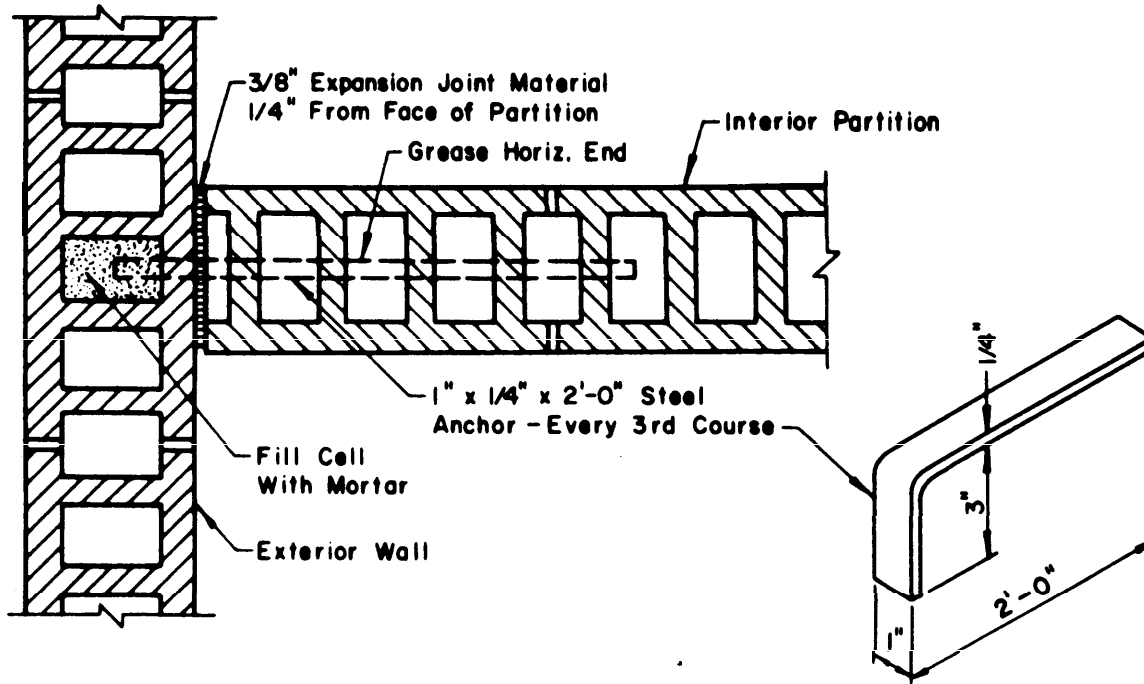
FOR PARTITIONS 6" WIDE OR WIDER

3/8" Expansion Joint Material
1/4" From Face of Partition

Interior Partition

No. 6 Wire - Every Other Course
(Grease This End)

Fill Cell With
Mortar

Exterior Wall

FOR 4" WIDE PARTITIONS
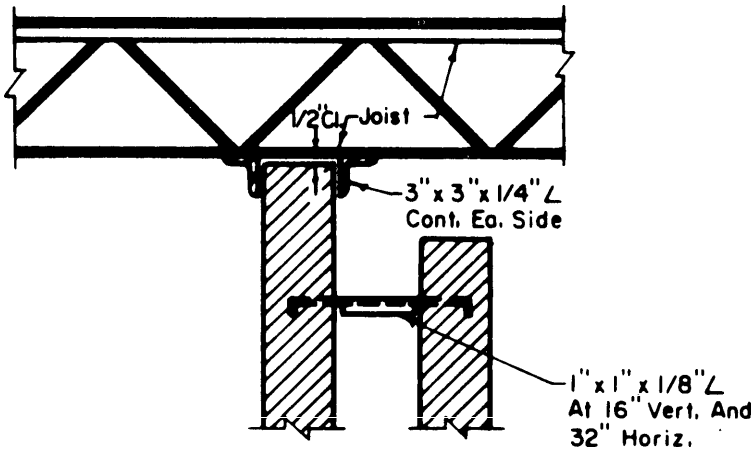
Figure C-5. Wall connections with control joints.

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WITH STRUCTURAL STEEL JOIST

ALTERNATE SUPPORT

Where Wall Projects Above,
Wrap Angle With 1/2" P. E. J. Material

Joist

2 JL, 2"x 2"x 3/16"
At 4'-0" O. C.

3"x 3"x 1/4"x 1'-0"
Guide Angles, 1/4" Cl.
Ea. Face, 1/2" Cl. Top

Partition

2-1/2" x 2-1/2" x 1/4"
Angle Brace

ST With Vertical Slot,
1/2" ⌀ x 4" Anchor

Partition

CHASE PARTITION

1/2" Cl. Joist

3"x 3"x 1/4" ∠
Cont. Ea. Side

1"x 1"x 1/8" ∠
At 16" Vert. And
32" Horiz.

Figure C-6. Typical details of interior partitions.

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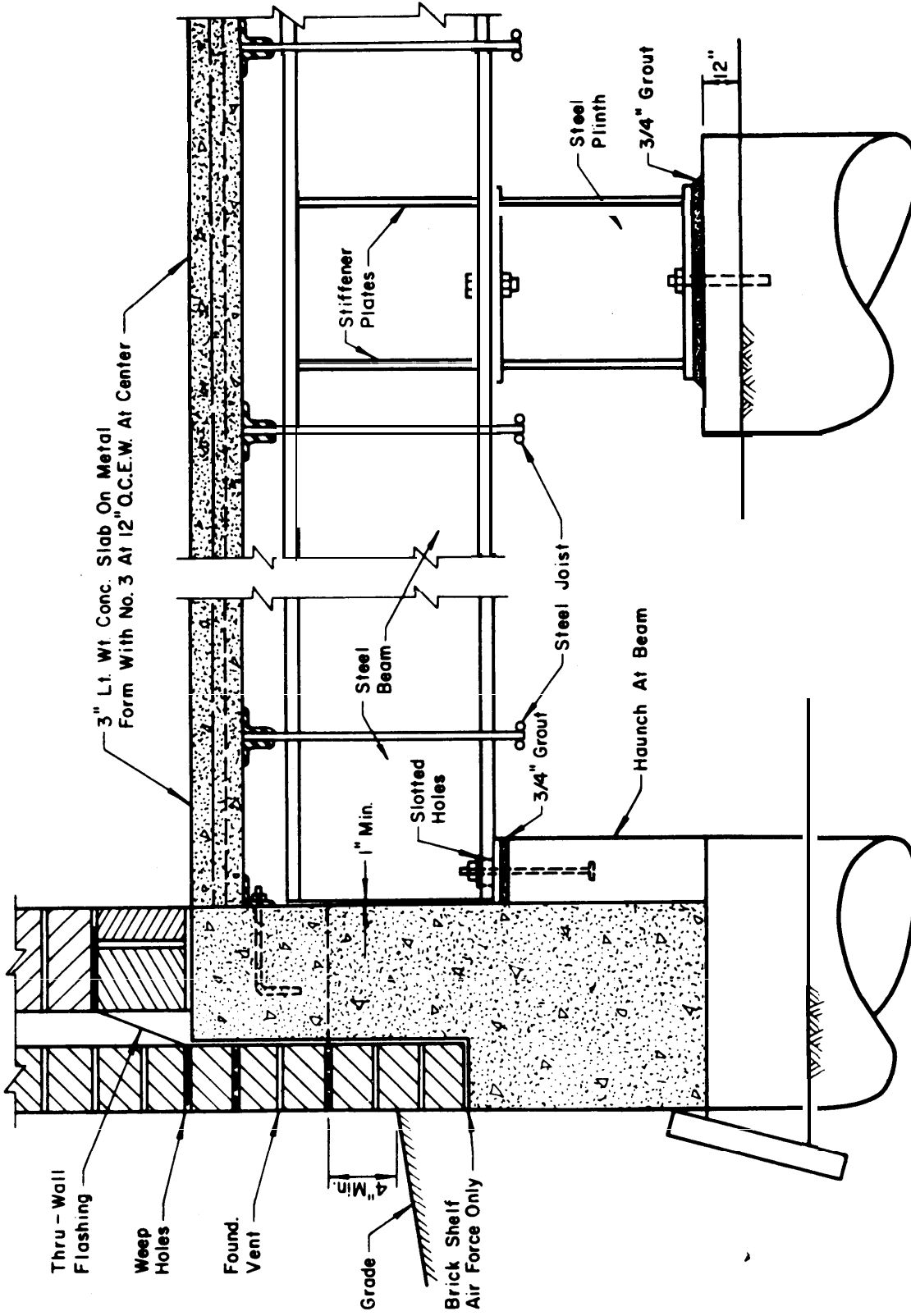
Figure C-7. Typical bar joist first floor framing.
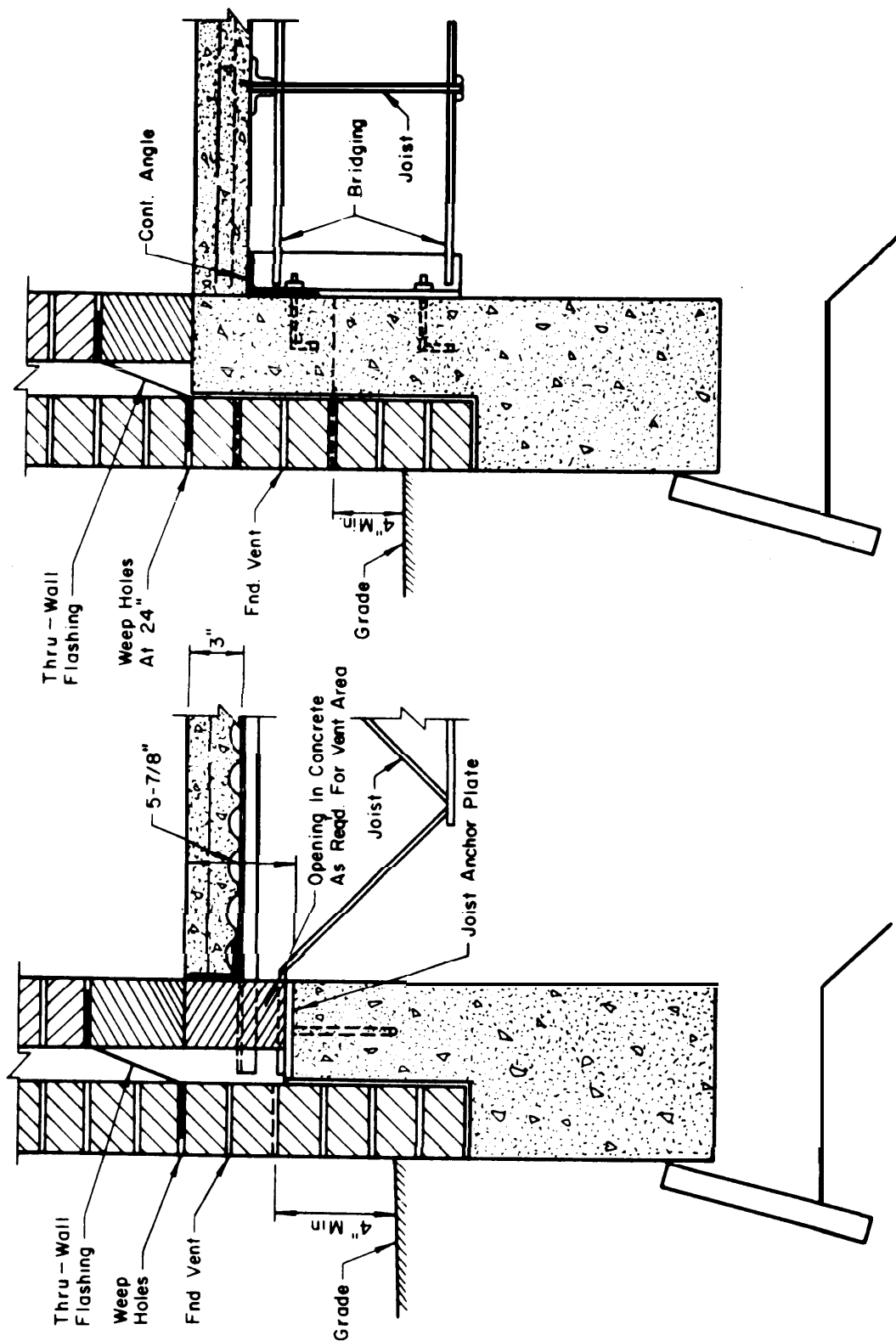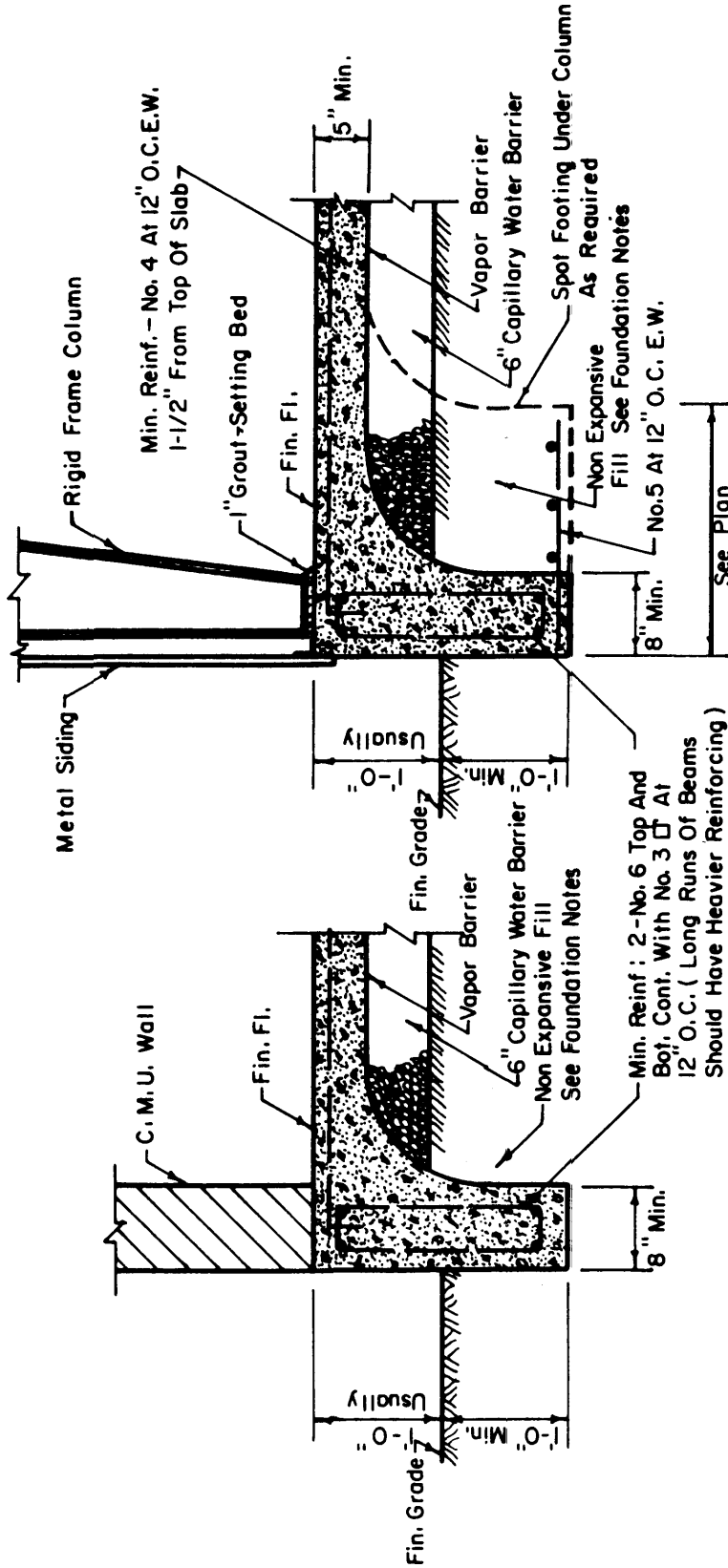
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Cont. Angle

Bridging

Joist

Thru-Wall Flashing

Weep Holes At 24"

Fnd. Vent

Grade

4" Min

3"

5-7/8"

Opening In Concrete As Req'd For Vent Area

Joist

Joist Anchor Plate

Thru-Wall Flashing

Weep Holes

Fnd Vent

Grade

4" Min

Fig C-8. Typical cast-in-place or precast concrete grade beam with steel bar joist floor framing.

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RIGID FRAME CONSTRUCTION

LOAD BEARING MASONRY WALL CONSTRUCTION

Figure C-9. Typical ribbed mat foundations.
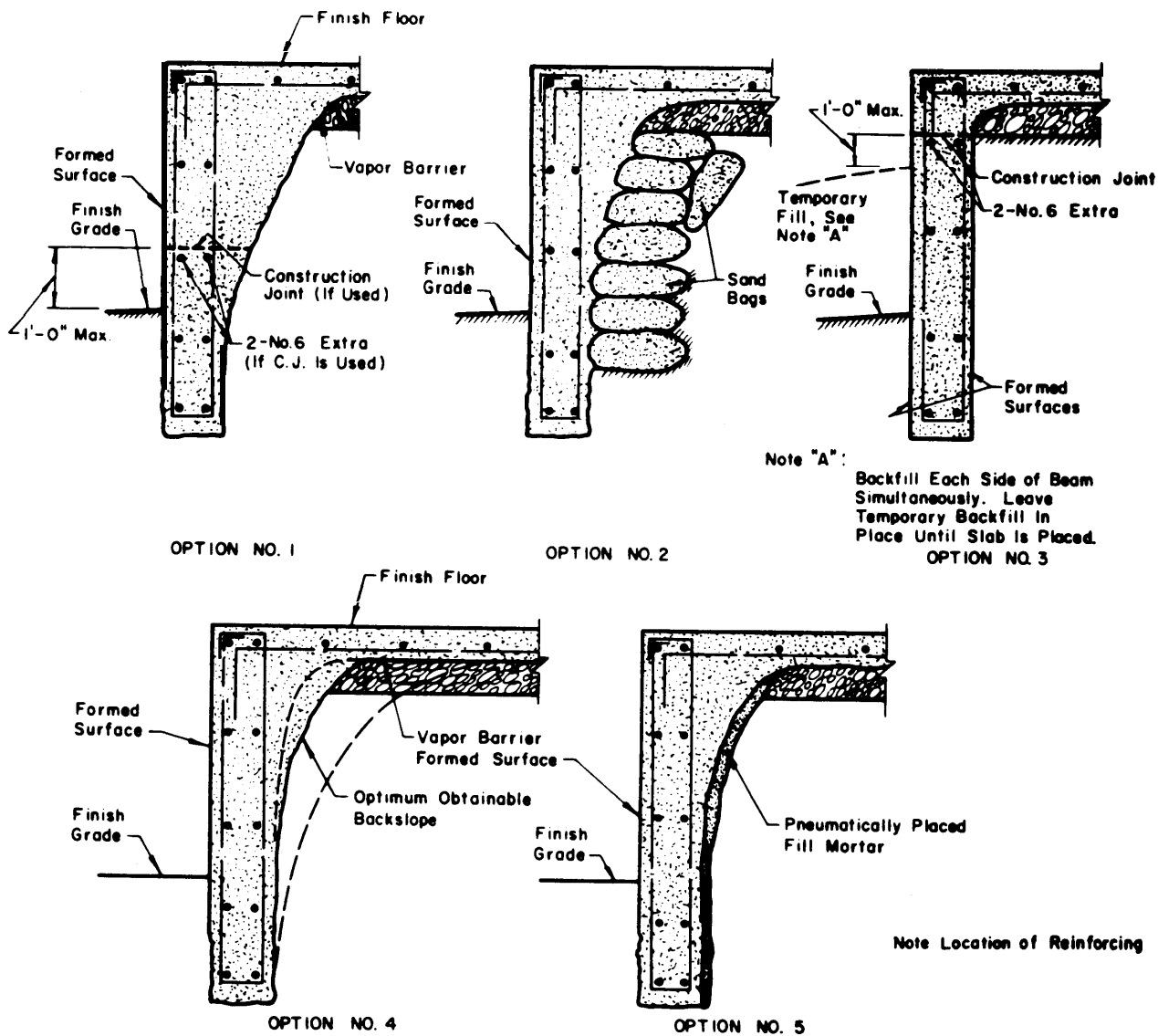
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Figure C-10. Optional construction details of exterior beams (interior beam similar) for ribbed mat construction.

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APPENDIX D

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By Order of the Secretary of the Army:

JOHN A. WICKHAM, JR.
General, United States Army
Chief of Staff

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Major General, United States Army
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SOIL SUCTION, WATER CONTENT, AND SPECIFIC VOLUME

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PROJECT _____ BORING/SAMPLE/DEPTH _____ DATE _____

SOIL SUCTION

- PSYCHROMETER NO.
- SAMPLE CONTAINER NO.
- WATER CONTENT INCREMENT (0, +, -)
- THERMOCOUPLE OUTPUT — t, MILLIVOLTS ; T, °C
- PSYCHROMETER OUTPUT — E_T, MICROVOLTS ; E_{25}, MICROVOLTS
- SOIL SUCTION†, TONS/FT² — τ

WATER CONTENT

- TARE NO.
- WEIGHT IN GRAMS:
  - TARE PLUS WET SOIL
  - TARE PLUS DRY SOIL
  - WATER — W_w
  - TARE
  - DRY SOIL — W_s
- WATER CONTENT, PERCENT — w
- TEST TEMPERATURE OF WATER, °C

WEIGHT-VOLUME RELATIONS

- WEIGHT IN GRAMS:
  - WET SOIL AND WAX IN AIR — w
  - WET SOIL
  - WAX
  - WET SOIL AND WAX IN WATER
  - DRY SOIL†† — W_s
- SPECIFIC GRAVITY OF SOIL — G_s
- VOLUME IN CC:
  - WET SOIL AND WAX‡
  - WAX
  - WET SOIL — V
  - DRY SOIL = W_s/G_s — V_s
- DENSITY PCF:
  - WET DENSITY = $(W/V)\,62.4$ — γ_m
  - DRY DENSITY = $(W_s/V)\,62.4$ — γ_d
- VOID RATIO = $(V - V_s)/V_s$ — e
- POROSITY, % = $[(V - V_s)/V] \times 100$ — n
- DEGREE OF SATURATION, % = $[V_w/(V - V_s)] \times 100$ — S
- SPECIFIC VOLUME = $1/\gamma_d$ — V_T

* T°C = $t/0.0395$

** E_{25} = $E_T/(0.325 + 0.027\,T)$

† SEE INDIVIDUAL PSYCHROMETER CALIBRATION LINE

†† IF NOT MEASURED DIRECTLY, MAY BE COMPUTED AS FOLLOWS: $W_s = \dfrac{W}{1 + 0.01\,w}$

‡ VOLUME OF WET SOIL AND WAX = $\dfrac{\left(\begin{array}{c}\text{WEIGHT OF WET SOIL}\\ \text{AND WAX IN AIR}\end{array}\right) - \left(\begin{array}{c}\text{WEIGHT OF WET SOIL}\\ \text{AND WAX IN WATER}\end{array}\right)}{\text{DENSITY OF WATER AT TEST TEMPERATURE}}$

VOLUME OF WAX = $\dfrac{\text{WEIGHT OF WAX}}{\text{SPECIFIC GRAVITY AT WAX}}$

DA FORM 5182-R, MAR 83

1. The leading cause of foundation heave or settlement in susceptible soils is change in soil moisture, which is attributed to _____.

    a) changes in the field environment from natural conditions
    b) changes related to construction
    c) usage effects on the moisture under the structure
    d) all of the above

2. Damaging edge or dish-shaped heaving or portions of the perimeter may be observed relatively soon after construction. This is called _____.

    a) doming heave
    b) edge heave
    c) cyclic heave
    d) none of the above

3. A building's foundation should _____.

    a) be taken to a depth to protect against heaving/swelling
    b) transmit loads to the ground without excessive settlement
    c) provide foundation to freeze/thaw cycles
    d) all of the above

4. Structures constructed on sites in which the topography relief is greater than _____ gradient may sustain damage from downhill creep of expansive clay surface soil.

    a) 2%
    b) 5%
    c) 9%
    d) 15%

5. Soils are classified into all of the following categories except for _____.

    a) high
    b) medium
    c) swell-prone
    d) non-expansive

6. Hazard maps provide basic information indicative of the probable degree of expansiveness and/or frequency or occurrence of swelling soils.

    a) True
    b) False

7. A study of the site history may reveal considerable qualitative date on the probable future behavior of the foundation soils.

   a) True
   b) False

8. The distribution and depth of borings are chosen to determine the soil profile and to obtain undisturbed samples required to evaluate _____.

   a) the potential total and differential heave of the foundation soils
   b) swelling
   c) bearing capacity
   d) all of the above

9. Meaningful groundwater conditions and engineering properties of subsurface materials can often best be determined from _____.

   a) insitu tests
   b) geological data
   c) field experience
   d) laboratory test

10. A competent inspector or engineer should accurately and visually classify materials as they are recovered from the boring.

   a) True
   b) False

11. _____ samples should be thoroughly sealed in water-proof containers so that the natural water content can be accurately measured.

   a) Auger
   b) Pit
   c) Other disturbed
   d) none of the above

12. Minimization of sample disturbance during and after drilling is important to the usefulness of undisturbed samples. This fact is particularly true for _____ soils since small changes in water content or soil structure will significantly affect the measured swelling properties.

   a) cohesive
   b) expansive
   c) non-swelling
   d) sandy

13. _____ is/are a common method used for identification of swelling soils.

   a) Wes classification
   b) Van der merwe
   c) Physiochemical tests
   d) all of the above

14. Laboratory methods recommended for prediction of the anticipated volume change or potential insitu heave of foundation soils are consolidometer swell and _____.

    a) soil suction tests
    b) van der merwe method
    c) attemberg method
    d) physiochemical tests

15. The results of _____ tests are used to estimate the soil bearing capacity and load/deflection behavior of shaft or other foundations.

    a) soil suction
    b) shear
    c) strength
    d) all of the above