CHAPTER 6

DESIGN OF FOUNDATIONS

6-1. Basic considerations

a. Planning. Swelling of expansive foundation soils should be considered during the preliminary design phase and the level of structural cracking that will be acceptable to the user should be determined at this time.

(1) The foundation of the structure should be designed to eliminate unacceptable foundation and structural distress. The selected foundation should also be compatible with available building materials, construction skills, and construction equipment.

(2) The foundation should be designed and constructed to maintain or promote constant moisture in the foundation soils. For example, the foundation should be constructed following the wet season if possible. Drainage should be provided to eliminate ponded water. Excavations should be protected from drying. Chapter 7 describes the methods of minimizing soil movement.

b. Bearing capacity. Foundation loading pressures should exceed the soil swell pressures, if practical, but should be sufficiently less than the bearing capacity to maintain foundation displacements within tolerable amounts, Present theoretical concepts and empirical correlations permit reasonably reliable predictions of ultimate capacity, but not differential movement of the foundation. Factors of safety (FS) are therefore applied to the ultimate bearing capacity to determine safe or allowable working loads consistent with tolerable settlements. Further details on bearing capacity are presented in TM 5-818-1.

c. Foundation systems. An appropriate foundation should economically contribute to satisfying the functional requirements of the structure and minimize differential movement of the various parts of the structure that could cause damages. The foundation should be designed to transmit no more than the maximum tolerable distortion to the superstructure. The amount of distortion that can be tolerated depends on the design and purpose of the structure. Table 6-1 illustrates foundation systems for different ranges of differential movement or effective plasticity index ($\overline{PI}$) for proper selection of the foundation. Figure 6-1 explains the term $\overline{PI}$. The use of ΔH is preferred to $\overline{PI}$ because ΔH is a more reliable indicator of in situ heave. Also, $\overline{PI}$ is not a satisfactory basis of design in situations such as a 5-foot layer of highly swelling soil overlying nonswelling soil, rock, or sand. Pervious sand strata may provide a path for moisture flow into nearby swelling soil.

(1) *Shallow individual or continuous footings.* Shallow individual or long continuous footings are often used in low swelling soil areas where the predicted footing angular deflection/span length ratios are on the order of 1/600 to 1/1000 or 0.5 inch or less of movement.

(2) *Stiffened mats (slabs).* Stiffened mat foundations are applicable in swelling soil areas where predicted differential movement AH may reach 4 inches. The stiffening beams of these mats significantly reduce differential distortion. The range provided in table 6-1 for beam dimensions and spacings of stiffened slabs for light structures normally provides an adequate design.

(3) *Deep foundations.* A pile or beam on a drilled shaft foundation is applicable to a large range of foundation soil conditions and tends to eliminate effects of heaving soil if properly designed and constructed (para 6-4). The type of superstructure and the differential soil movement are usually not limited with properly designed deep foundations. These foundations should lead to shaft deflection/spacing ratios of less than 1/600.

d. Superstructure systems. The superstructure should flex or deform compatibly with the foundation such that the structure continues to perform its functions, contributes aesthetically to the environment, and requires only minor maintenance. Frame construction, open floor plans, and truss roofs tend to minimize damage from differential movement. Load bearing walls tend to be more susceptible to damage from shear than the relatively flexible frame construction. Wood overhead beams of truss roof systems provide structural tension members and minimize lateral thrust on walls. Table 6-2 illustrates the relative flexibility provided by various superstructure systems.

(1) *Tolerable angular deflection/length ratios. The* ability of a structure to tolerate deformation depends on the brittleness of the building materials, length to height ratio, relative stiffness of the structure in shear and bending, and mode of deformation whether heave (dome-shaped, fig. 1-2) or settlement (dish-shaped, fig. 1-3). The vertical angular deflection/span length (Δ/ℓ) that can be tolerated, therefore, varies considerably between structures. The Δ/ℓ is the differential displacement Δ over the length ℓ between columns as

Table 6-1. Foundation Systems

Predicted Differential Movement, inches	Effective Plasticity Index, $\overline{PI}$	Foundation System	Remarks
1/2	<15	Shallow individual Continuous wall Strip	Lightly loaded buildings and residences.
		Reinforced and stiffened thin mat	Residences and lightly loaded structures; on-grade 4- to 5-in. reinforced concrete slab with stiffening beams; maximum free area between beams 400 ft^2; 1/2 percent reinforcing steel; 10- to 12-in.-thick beams; external beams thickened or deepened, and extra steel stirrups added to tolerate high edge forces as needed; dimensions adjusted to resist loading. Beams positioned beneath corners to reduce slab distortion. Type of Mat — Beam Depth, in. — Beam Spacing, ft
1/2 to 1	15 to 25		Light — 16 to 20 — 20 to 15
1 to 2	26 to 40		Medium — 20 to 25 — 15 to 12
2 to 4	>41		Heavy — 25 to 30 — 15 to 12
No limit		Thick, reinforced mat	Large, heavy structures; mats usually 2 ft or more in thickness.
No limit		Deep foundations, pile or drilled shaft	Foundations for any light or heavy structure; grade beams span between piles or shafts 6 to 12 in. above ground level; suspended floors or on-grade slabs isolated from grade beams and walls. Concrete drilled shafts may be underreamed or straight, reinforced, and cast in place with 3000-psi concrete of 6-in. slump.

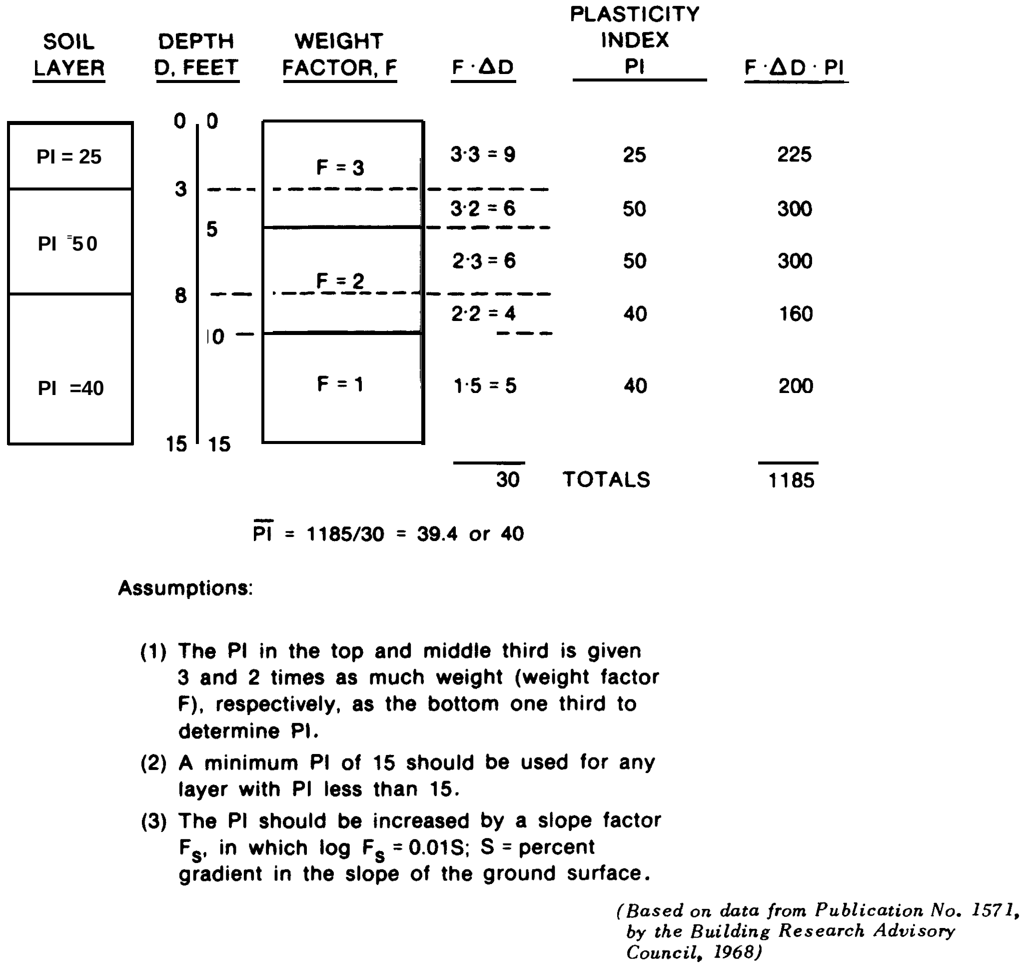

Figure 6-1. Effective plasticity index ($\overline{PI}$) or average $\overline{PI}$ in the top 15 feet of soil beneath the slab.

footings or about twice the A/L ratio of the slab (fig. 5-3). Only rough guidance of the range of tolerable Δ/ℓ ratios can be offered, such as in table 6-2, for different framing systems.

(a) Propagation of cracks depends on the degree of tensile restraint built into the structure and its foundation. Thus, frame buildings with panel walls are able to sustain larger relative deflections without severe damage than unreinforced load-bearing walls. Structural damage is generally less where the dish-shaped pattern develops than in the case of center heaving or edge downwarping because the foundation is usually better able to resist or respond to tension forces than the walls.

(b) A Δ/ℓ ratio of 1/500 is a common limit to avoid cracking in single and multistory structures. Plaster, masonry or precast concrete blocks, and brick walls will often show cracks for Δ/ℓ ratios between 1/600 to 1/1000. However, cracks may not appear in these walls if the rate of distortion is sufficiently slow to allow the foundation and frame to adjust to the new distortions. The use of soft bricks and lean mortar also tend to reduce cracking. Reinforced masonry, reinforced concrete walls and beams, and steel frames can tolerate Δ/ℓ ratios of 1/250 to 1/600 before cracks appear in the structure. Deflection ratios exceeding 1/250 are likely to be noticed in the structure and should usually be avoided. The Δ/ℓ ratios exceeding 1/150 usually lead to structural damage.

(2) *Provisions for flexibility.* The flexibility required to avoid undesirable distress may be provided by joints and flexible connections. Joints should be provided in walls as necessary, and walls should not be tied into the ceiling. Slabs-on-grade should not be tied into foundation walls and columns but isolated using expansion joints or gaps filled with a flexible, impervious compound. Construction items, such as reinforced concrete walls, stud frames, paneling, and gypsum board, are better able to resist distortions and should be used instead of brick, masonry blocks, or plaster walls. The foundation may be further reinforced by making the walls structural members capa-

Table 6-2. Superstructure Systems.

Superstructure system	Tolerable vertical angular deflection/ span length ratios, Δ/ℓ	Description
Rigid	1/600 to 1/1000	Precast concrete block, unreinforced brick, masonry or plaster walls, slab-on-grade.
Semirigid	1/360 to 1/600	Reinforced masonry or brick reinforced with horizontal and vertical tie bars or bands made of steel bars or reinforced concrete beams vertical reinforcement located on sides of doors and windows; slab-on-grade isolated from walls.
Flexible*	1/150 to 1/360	Steel, wood framing; brick veneer with articulated joints; metal, vinyl, or wood panels; gypsum board on metal or wood studs; vertically oriented construction joints; strip windows or metal panels separating rigid wall sections with 25-ft spacing or less to allow differential movement; all water pipes and drains into structure with flexible joints; suspended floor or slab-on-grade isolated from walls (heaving and cracking of slab-on-grade probable and accounted for in design).
Split construction*	1/150 to 1/360	Walls or rectangular sections heave as a unit (modular construction); joints at 25-ft spacing or less between units and in walls; suspended floor or slab-on-grade isolated from walls (probable cracking of slab-on-grade); all water pipes and drains equipped with flexible joints; construction joints in reinforced and stiffened slabs at 150-ft spacing or less and cold joints at 65-ft spacing or less.

* A Δ/ℓ value exceeding 1/250 is not recommended for normal practice, and a Δ/ℓ exceeding 1/150 often leads to structural damage.

ble of resisting bending such as reinforced concrete shear walls. Several examples of frame and wall construction are provided in appendix C.

6-2. Shallow individual or continuous footings

a. Susceptibility y to damage. Structures supported by shallow individual or continuous wall footings are susceptible to damages from lateral and vertical movement of foundation soil if provisions are not made to accommodate possible movement. Dishing or substantial settlement may occur in clays, especially in initially wet soil where a well-ventilated crawl space is constructed under the floor. The crawl space prevents rainfall from entering the soil, but the evaporation of moisture from the soil continues. Center heave or edge downwarping (fig. 1-2) can occur if the top layer of soil is permeable and site drainage is poor. Fractures may appear in walls not designed for differential movement after Δ/ℓ ratios exceed 1/600 or differential movement exceeds about 0.5 inch.

b. Applications. Shallow footings may be used where expansive strata are sufficiently thin to allow location of the footing in a nonexpansive or low-swelling stratum (fig. 6-2).

(1) A structural floor slab should be suspended on top of the footing (fig. 6-2a) or the slab-on-grade should be isolated from the walls (fig. 6-2b). The slab-on-grade should be expected to crack.

(2) Figure 6-3 illustrates examples of interior construction for a slab-on-grade. Interior walls may be suspended from the ceiling or supported on the floor. A flexible joint should be provided in the plenum between the furnace and the ceiling. Sewer lines and other utilities through the floor slab should be permitted to slip freely.

(3) Swelling of deep expansive soil beneath a nonexpansive stratum may cause differential movement of shallow footings if the moisture regime is altered in the deep soil following construction (e.g., change in groundwater level, or penetration of surface water into deep desiccated soil). Excavations for crawl spaces

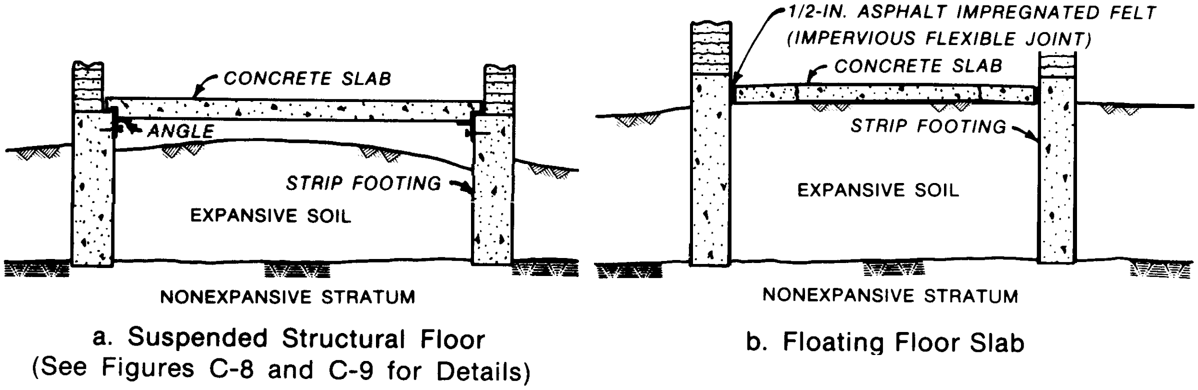

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Figure 6-2. Footings on nonexpansive stratum.

or basements decrease the vertical confining pressure and pore water pressure, which can cause the underlying expansive foundation soil to heave from adjustment of the moisture regime back to the natural pore water pressures.

c. Basements. Basements and long continuous footings constructed in excavations are subject to swell pressures from underlying and adjacent expansive soil.

(1) *Walls.* Basement walls of reinforced concrete can be constructed directly on the foundation soil without footings provided foundation pressures are less than the allowable bearing capacity (fig. 6-4a). However, placing heavy loads on shallow footings may not be effective in countering high swell pressures because of the relative small width of the footings. The stress imposed on the soil is very low below a depth of about twice the width of the footing and contributes little to counter the swell pressure unless the expansive soil layer is thin.

(2) *Voids.* Voids can also be spaced at intervals beneath the walls to increase loading pressures on the foundation soil and to minimize flexing or bowing of the walls (fig. 6-4b). The voids may be made with removable wood forms, commercially available cardboard, or other retaining forms that deteriorate and collapse (para 6-4d).

(3) *Joints.* Joints should be provided in interior walls and other interior construction if slab-on-ground is used (fig. 6-3). The slab should be isolated from the walls with a flexible impervious compound.

(4) *Lateral earth pressure on wall. The* coefficient of lateral earth pressure can exceed one if the backfill is heavily compacted and expansive, or the natural soil adjacent to the wall is expansive. Controlled backfills are recommended to minimize lateral pressures and increase the economy of the foundation (para 7-3a). Steel reinforcement can provide the necessary restraint to horizontal earth pressures, Unreinforced masonry brick and concrete blocks should not be used to construct basement walls.

d. Design. Standard design procedures for foundations of buildings and other structures are presented in TM5-818-1.

6-3. Reinforced slab-on-grade foundations

a. Application. The reinforced mat is often suitable for small and lightly loaded structures, particularly if

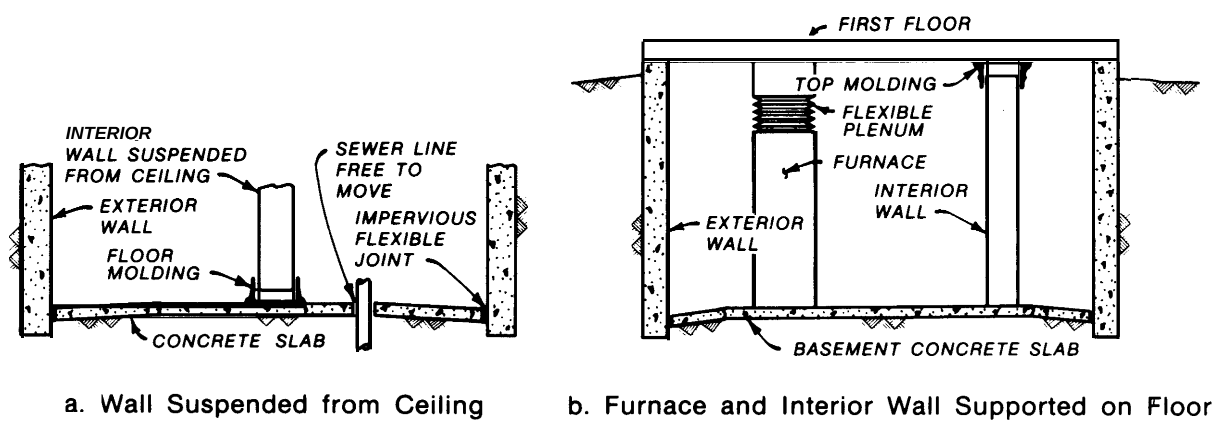

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Figure 6-3. Interior joint details for slab-on-grade.

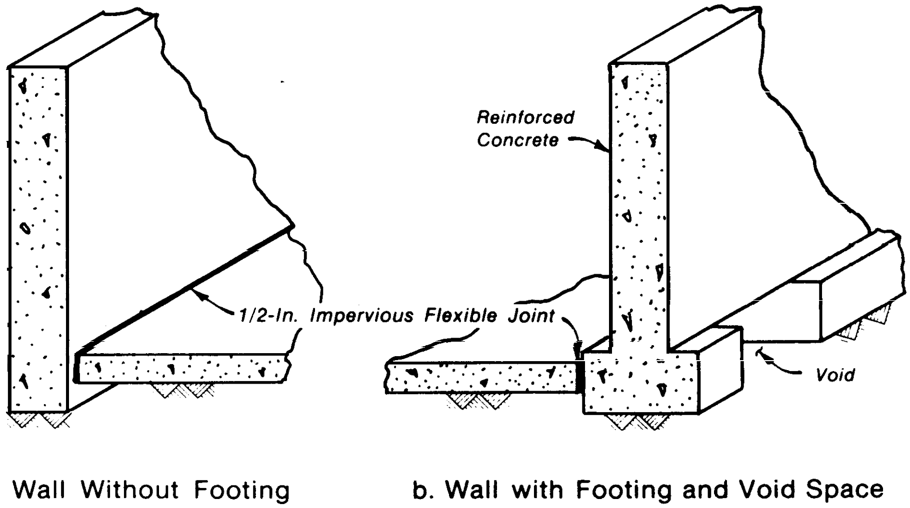

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Figure 6-4. Basement walls with slab-on-grade.

the expansive or unstable soil extends nearly continuously from the ground surface to depths that exclude economical drilled shaft foundations. This mat is suitable for resisting subsoil heave from the wetting of deep desiccated soil, a changing water table, laterally discontinuous soil profiles, and downhill creep, which results from the combination of swelling soils and the presence of slopes greater than 5 degrees. A thick, reinforced mat is suitable for large, heavy structures. The rigidity of thick mats minimizes distortion of the superstructure from both horizontal and vertical movements of the foundation soil.

(1) *Effects of stiffening beams.* Concrete slabs without internal stiffening beams are much more susceptible to distortion or doming from heaving soil. Stiffening beams and the action of the attached superstructure with the mat as an indeterminate structure increase foundation stiffness and reduce differential movement. Edge stiffening beams beneath reinforced concrete slabs can also lessen soil moisture loss and reduce differential movement beneath the slab. However, the actual vertical soil pressures acting on stiffened slabs can become very nonuniform and cause localized consolidation of the foundation soil.

(2) *Placement of nonswelling layer.* Placement of a nonswelling, 6-inch-(or more) thick layer of (preferably) impervious soil on top of the original ground surface before construction of lightly loaded slabs is recommended to increase the surcharge load on the foundation soil, slightly reduce differential heave, and permit the grading of a slope around the structure leading down and away from it. This grading improves drainage and minimizes the possibility that the layer (if pervious) could be a conduit for moisture flow into desiccated foundation expansive soils. The layer should have some apparent cohesion to facilitate trench construction for the stiffening beams.

b. Design of thin slabs for light structures. Stiffened slabs may be either conventionally reinforced or posttensioned. The mat may be inverted (stiffening beams on top of the slab) in cases where bearing capacity of the surface soil is inadequate or a supported first floor is required. The Department of Housing and Urban Development, Region IV, San Antonio Area Office, has documented a series of successful conventionally reinforced and posttensioned slabs for the southern central states. Successful local practice should be consulted to help determine suitable designs.

(1) *Conventionally reinforced. The* conventional reinforced concrete waffle type mat (table 6-1), which is used for light structures, consists of 4- to 5-inch-thick concrete slab. This slab contains temperature steel and is stiffened with doubly reinforced concrete crossbeams. Figure 6-5 illustrates an engineered rebar slab built in Little Rock, Arkansas. Appendix C provides details of drawings of reinforced and stiffened thin mats. The 4-inch slab transmits the self-weight and first floor loading forces to the beams, which resist the moments and shears caused by differential heave of the expansive soil. Exterior walls, roof, and internal concentrated loads bear directly on the stiffening beams. Clearance between beams should be limited to 400 square feet or less. Beam spacings may be varied between the limits shown in table 6-1 to allow for concentrated and wall loads. Beam widths vary from 8 to 12 or 13 inches but are often limited to a minimum of 10 inches.

(a) *Concrete and reinforcement.* Concrete compressive strength f ‘c should be at least 2500 psi and preferably 3000 psi. Construction joints should be placed at intervals of less than 150 ft and cold joints less than 65 ft. About 0.5 percent reinforcing steel should be used in the mat to resist shrinkage and temperature effects.

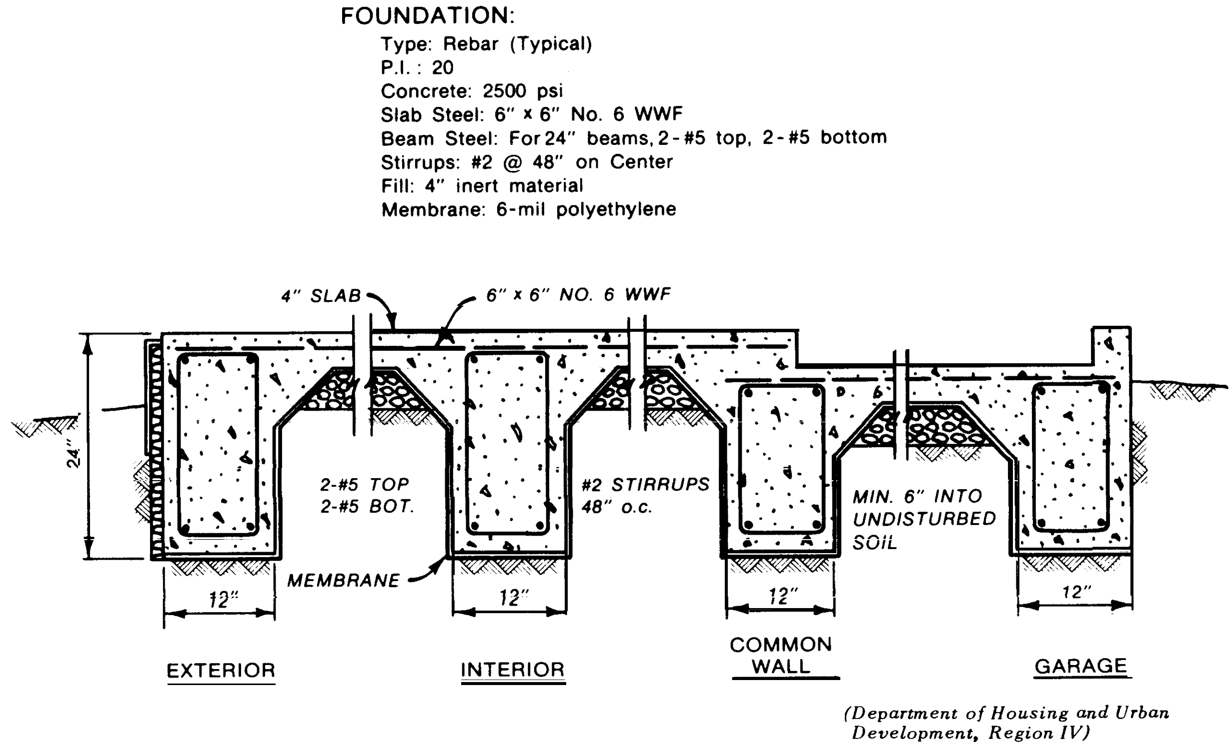


Figure 6-5. Typical conventional rebar slab in Little Rock, Arkansas, for single-family, single-story, minimally loaded frame residence with 11- to 12-foot wall spacing.

(b) Preliminary design, The three designs for reinforced and stiffened thin mats presented in table 6-1 differ in the beam depth and spacing depending on the predicted ΔH or $\overline{PI}$. The beam depths and spacings for each of the light, medium, and heavy slabs are designed for Δ/ℓ ratios of 1/500 and tend to be conservative in view of still undetermined fully acceptable design criteria and relatively high repair cost of reinforced and stiffened slabs. Stirrups may be added, particularly in the perimeter beams, to account for concentrated and exterior wall loads.

(2) *Post-tensioned.* Figure 6-6 illustrates an example of a posttensioned slab. Properly designed and constructed posttensioned mats are more resistant to fracture than an equivalent section of a conventional rebar slab and use less steel. However, post-tensioned slabs should still be designed with adequate stiffening beams to resist flexure or distortion from differential heave of the foundation soil, Experienced personnel are necessary to properly implement the posttensioning.

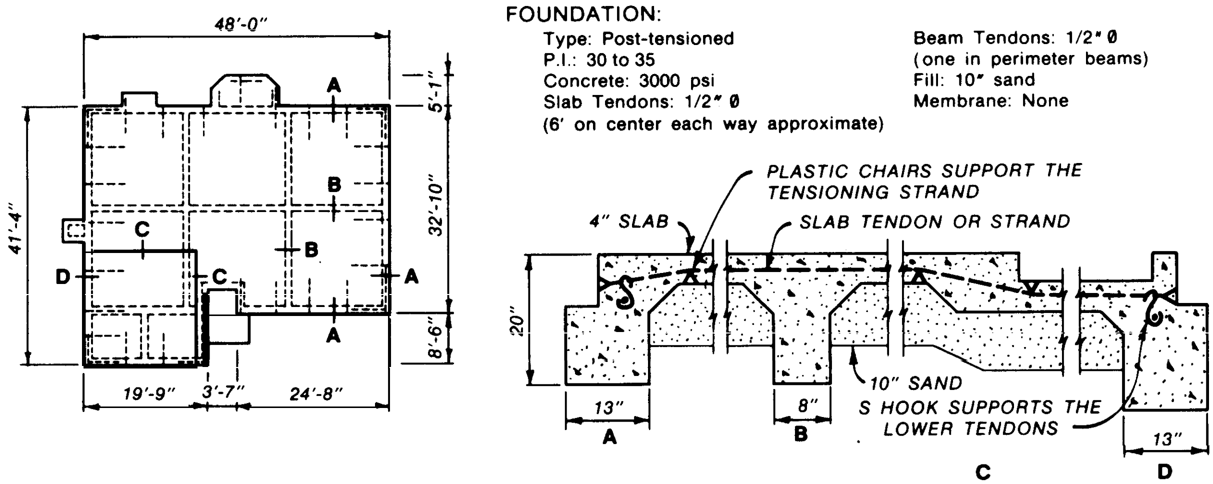
(3) *Assumptions of design parameters.* Design parameters include effects of climate, center and edge modes of differential swelling, perimeter and uniform loads, and structural dimensions.

(a) The effects of climate and differential swelling are accounted for by predictions of the maximum differential heave AH and the maximum edge lift-off distance e_m. Procedures for prediction of ΔH are provided in chapter 5. Reasonable values of the e_m are correlated with the Thornthwaite Moisture Index (TMI) in figure 6-7. The TMI, a climate related parameter roughly estimated from figure 6-8, represents the overall availability of water in the soil. The TMI can vary 10 to 20 or more (dimensionless) units from year to year. The e_m should be picked toward the top of the range in figure 6-7 for fissured soils. Since the e_m may exceed the range given in figure 6-7, depending on the activity of the soil or extreme changes in climatic conditions (e.g., long droughts and heavy rainfall), the value of e_m in feet may also be approximated by $2.5\Delta H$ with ΔH in inches for $\Delta H \leqslant 4$ inches.

(b) The loading distribution depends on the architectural arrangement of the building and often cannot be significantly altered. Perimeter and concentrated loads should be supported directly on the stiffening beams.

(c) The length and width of the slab are usually fixed by the functional requirement. Beam spacing depends on the slab geometry and varies between 10 and 20 feet. The depth of stiffening beams is controlled by the moment and shear capacity. The beam depth is adjusted as needed to remain within the allowable limits. The width of the stiffening beam is usually controlled by the excavation equipment and soil bearing capacity.

(4) *Structural design procedure,* The design proce-



(Department of Housing and Urban Development, Region IV)

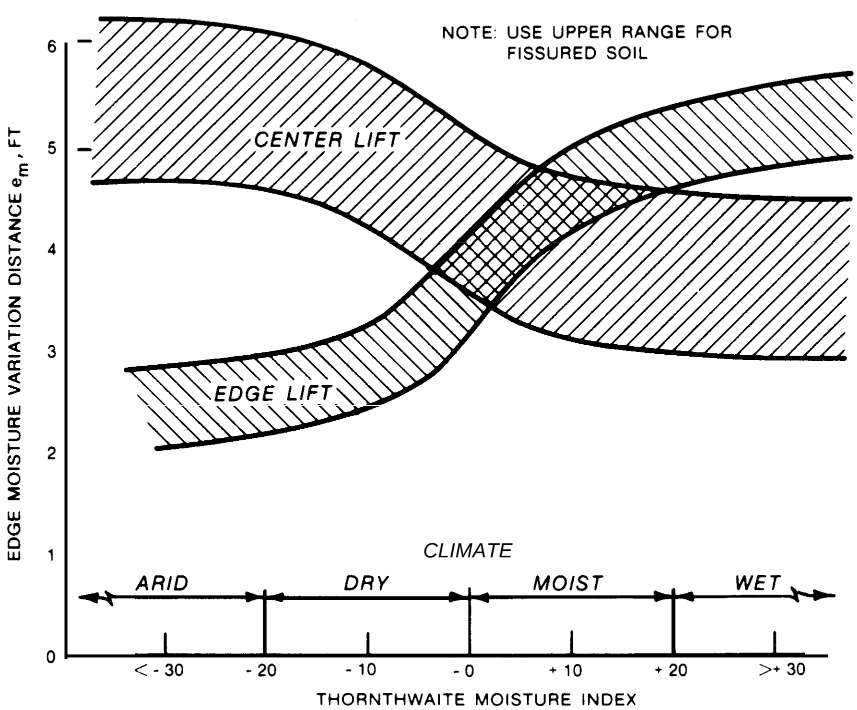
Figure 6-6. Post-tensioned slab in Lubbock, Texas, for single-family, single-story, minimally loaded frame residence.

dure should provide adequate resistance to shear, moment, and deflections from the structural loading forces, while overdesign is minimized. An economically competitive procedure that builds on the early work of the Building Research Advisory Board of the National Academy of Sciences is that developed for the Post-Tensioning Institute (PTI).

(a) The PTI procedure is applicable to both conventionally reinforced and posttensioned slabs up to 18 inches thick. It considers the previously discussed assumptions of the design parameters.

(*b*) The e_m and maximum differential heave y_m of the unloaded soil determined by the PTI procedure reflect average moisture conditions and may be exceeded if extreme changes in climate occur.

(c) Material parameters required by the PTI pro-



(Based on data from W. K. Wray, 1980, published in Proceedings, Fourth International Conference on Expansive Soils, Vol I, with permission of the American Society of Civil Engineers)

Figure 6-7. Approximate relationship between the Thornthwaite Moisture Index and the edge lift-off distance.

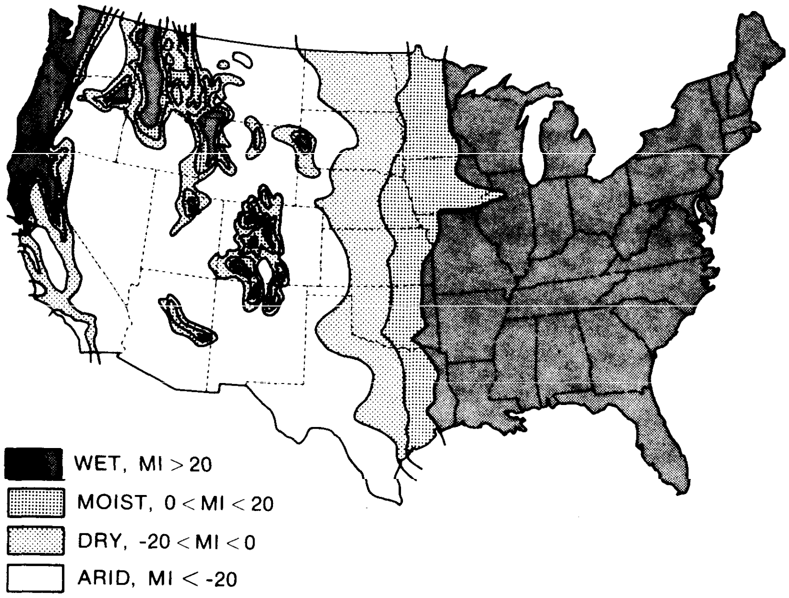


Figure 6-8. Approximate distribution of the Thornthwaite Moisture Index (MI) in the United States.

cedure are the compressive strength of concrete; allowable tensile and compressive stresses in concrete; type, grade, and strength of the prestressing steel; grade and strength of the mild steel reinforcement; and slab subgrade friction coefficient, The amount of reinforcing steel recommended by this procedure should be considered a minimum. The slab-subgrade coefficient of friction should be 0.75 for concrete cast on polyethylene membranes and 1.00 if cast on-grade.

(*d*) The allowable Δ/ℓ ratio must be estimated. This ratio may be as large as 1/360 for center heave and 1/800 for edge heave. The smaller edge Δ/ℓ ratio criterion is recommended by the PTI because edge lift is usually much less than center lift deflections and the stems of the beams resisting the positive bending moment may be unreinforced.

c. Design of thick mats. The state of the art for estimating spatial variations in soil pressures on thick mats is often not adequate. These mats tend to be heavily overdesigned because of the uncertainty in the loading and the relatively small extra investment of some overdesign.

(1) *Description.* Concrete mats for heavy structures tend to be 3 feet or more in thickness with a continuous two-way reinforcement top and bottom. An 8-foot-thick mat supporting a 52-story structure in Houston, Texas, contains about 0.5 percent steel, while the 3-foot-thick mat of the Wilford Hall Hospital complex at Lackland Air Force Base in Texas also contains about 0.5 percent steel. The area of steel is 0.5 percent of the total area of the concrete distributed equally each way both top and bottom. The steel is overlapped near the concentrated loads, and a 3-inch cover is provided over the steel. The depth of the excavation that the mats are placed in to achieve bearing capacity and tolerable settlements eliminates seasonal edge effects such that the edge lift-off distance is not applicable.

(2) *Procedure.* The thick mat is designed to determine the shear, moment, and deflection behavior using conventional practice, then modified to accommodate swell pressures and differential heave caused by swelling soils. The analyses are usually performed by the structural engineer with input on allowable soil bearing pressures, uplift pressures (hydrostatic and swell pressures from expansive soils) and estimates of potential edge heave/shrinkage and center heave from the foundation engineer. Computer programs are commonly used to determine the shear, moments, and deflections of the thick mat.

(a) Structural solutions. The structural solution may be initiated with an estimate of the thickness of a spread footing that resists punching shear and bending moments for a given column load, concrete compressive strength, and soil bearing capacity. Following an estimation of the initial thickness, hand solutions of mat foundations for limited application based on theory of beams on elastic foundations are available from NAVFAC DM-7. More versatile solutions are available from computer programs based on theory of beams on elastic foundations such as BMCOL 2, which is available at the U.S. Army Corps of Engineer Waterways Experiment Station, and finite element analysis.

(b) Foundation soil/structure solutions. The BMCOL 2 soil-structure interaction program permits nonlinear soil behavior. Finite element programs are

also available, but they are often burdened with hard to explain local discontinuities in results, time-consuming programming of input data, and need of experienced personnel to operate the program. The finite element program originally developed for analysis of Port Allen and Old River Locks was applied to the analysis of the Wilford Hall Hospital mat foundation at Lackland Air Force Base in Texas. Figure 6-9 compares predicted with observed movement of the 3.5-foot-thick mat at Wilford Hall. Foundation soils include the fissured, expansive Navarro and upper Midway clay shales. These computer programs help refine the design of the mat and can lead to further cost reductions in the foundation.

6-4. Deep foundations

The deep foundation provides an economical method for transfer of structural loads beyond (or below) unstable (weak, compressible, and expansive) to deeper stable (firm, incompressible, and nonswelling) strata. Usually, the deep foundation is a form of a pile foundation. Numerous types of pile foundations exist of which the most common forms are given in table 6-3. Occasionally when the firm-bearing stratum is too deep for the pile to bear directly on a stable stratum, the foundation is designed as friction or floating piles and supported entirely from adhesion with the surrounding soil and/or end bearing on underreamed footings.

a. General applications. Each of the types of piling is appropriate depending on the location and type of structure, ground conditions (see table 3-1 for examples), and durability. The displacement pile is usually appropriate for marine structures. Any of the piles in table 6-3 may be considered for land applications. Of these types the bored and cast in situ concrete drilled shaft is generally more economical to construct than driven piles.

b. Application of drilled shafts. Table 6-4 describes detailed applications of drilled shaft foundations including advantages and disadvantages. Detailed discussion of drilled shaft foundations is presented below because these have been most applicable to the solution of foundation design and construction on expansive clay soils.

(1) A drilled shaft foundation maybe preferred to a mat foundation if excavating toward an adequate bearing stratum is difficult or the excavation causes settlement or loss of ground of adjacent property.

(2) A drilled shaft foundation 20 to 25 feet deep tends to be economically competitive with a ribbed mat foundation,

(3) Drilled shafts may be preferred to mat foundations if differential heave ΔH exceeds 4 inches or Δ/ℓ ratios exceed 1/250, Mat foundations under such conditions may tilt excessively leading to intolerable distortion or cracking.

(4) The shaft foundation may be economical com-

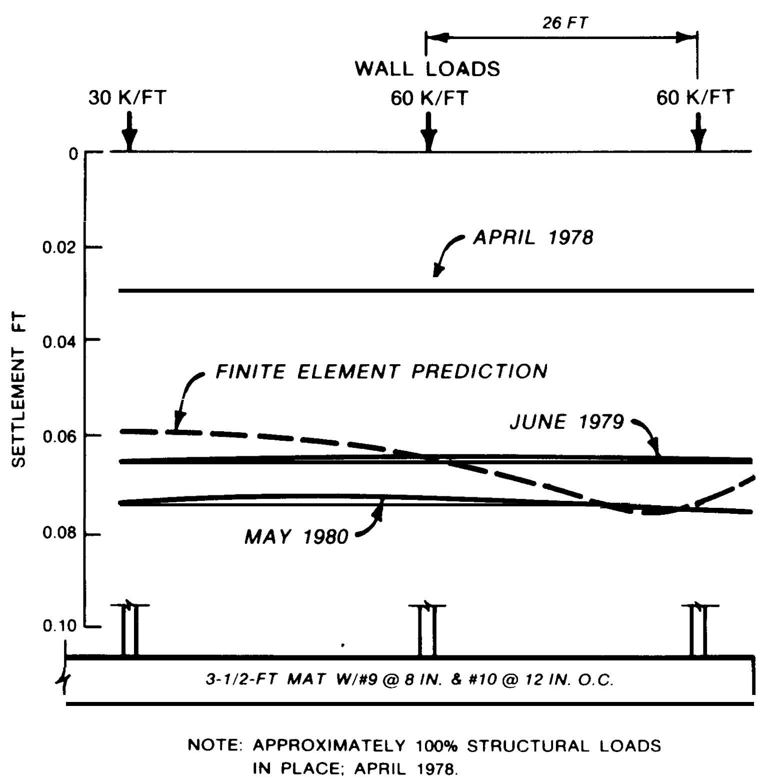


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Figure 6-9. Settlement and deflection of a mat foundation.

Table 6-3. Classification of Piles

Classification	Type	Description
Displacement	Timber Precast concrete Steel circular or rectangular Tapered timber or steel	Driven piles with solid circular or rectangular cross section or hollow section with closed bottom end. Piles hammered or jacked down into place.
Small displacement	Precast concrete Prestressed concrete Steel H section Steel circular or rectangular Screw	Small cross-section pile consisting of open-end cylinder, rectangular, H section, or screw configuration.
Nondisplacement	Drilled shaft Tubes filled with concrete Precast concrete Injected cement mortar Steel section	Piles placed in open boreholes. Usually concrete placed in holes drilled by rotary auger, baling, grabbing, airlift, or reverse circulation methods.
Combination	Steel-driven tube replaced by concrete Precast shell filled with concrete Jointed pile of different materials	Combination of different forms of piles.

pared with traditional strip footings, particularly in open construction areas and with shaft lengths less than 10 to 13 feet, or if the active zone is deep, such as within areas influenced by tree roots.

c. *General considerations.*

(1) *Causes of distress.* The design and construction of drilled shaft foundations must be closely controlled to avoid distress and damage. Most problems have been caused by defects in construction and by inadequate design considerations for effects of swelling soil (table 6-5). The defects attributed to construction techniques are discontinuities in the shaft, which may occur from the segregation of concrete, failure to complete concreting before the concrete sets, and early set of concrete, caving of soils, and distortion of the steel reinforcement. The distress resulting from inadequate design considerations are usually caused by wetting of subsoil beneath the base, uplift forces, lack of an air gap beneath grade beams, and lateral movement from downhill creep of expansive clay.

(2) *Location of base.* The base of shafts should be located below the depth of the active zone, such as below the groundwater level and within nonexpansive soil. The base should not normally be located within three base diameters of an underlying unstable stratum.

(a) Slabs-on-grade isolated from grade beams and walls are often used in light structures, such as residences and warehouses, rather than the more costly structural slabs supported by grade beams and shafts. These slabs-on-grade will move with the expansive soil and should be expected to crack.

(b) To avoid "fall-in" of material from the granular stratum during underreaming of a bell, the base may be placed beneath swelling soil near the top of a granular stratum.

(3) *Underreams.* Underreams are often used to increase anchorage to resist uplift forces (fig. 6-10). The belled diameter is usually 2 to 2.5 times the shaft diameter D_s and should not exceed 3 times D_s. Either 45- or 60-degree bells may be used, but the 45-degree bell is often preferred because concrete and construction time requirements are less. Although the 45-degree bell may be slightly weaker than the 60-degree bell, no difference has been observed in practice. The following considerations are necessary in comparing underreamed shafts with straight shafts.

(a) Straight shafts may be more economical than underreams if the bearing stratum is hard or if subsoils are fissured and friable. Soil above the underream may be loose and increase the upward movement needed to develop the bell resistance.

(b) The shaft can often be lengthened to eliminate the need for an underream, particularly in soils where underreams are very difficult to construct. Friction resistance increases rapidly in comparison with end bearing resistance as a function of the relative shaft-soil vertical movement.

(c) Underreams reduce the contact bearing pressure on potentially expansive soil and restrict the minimum diameter that may be used.

(4) *Uplift forces.* If bells or underreams are not feasible, uplift forces (table 6-5) may be controlled by the following methods:

(a) The shaft diameter should be the minimum required for downloads and construction procedures and control.

Table 6-4. Applications of Drilled Shafts

Applications	Advantages	Disadvantages
Absence of a shallow, stable founding stratum; support of structures with shafts drilled through swelling soils into zones unaffected by moisture changes.	Personnel, equipment, and materials for construction usually readily available; rapid construction due to mobile equipment; careful inspection of excavated hole usually possible; noise level of equipment less than some other construction methods; low headroom needed.	Accurate predictions of load and settlement behavior not always possible.
Support of moderate-to-high column loads; high column loads with shafts drilled into hard bedrock; moderate column loads with underreamed shafts bottomed on sand and gravel.	Excavated soil examined to check the projected soil conditions and profile; excavation possible for a wide variety of soil conditions.	Careful design and construction required to avoid defective foundations; careful inspection necessary during construction; inspection of concrete after placement difficult.
Support of light structures on friction shafts.	Heave and settlement at the ground surface normally small for properly designed shafts.	Inadequate knowledge of design methods and construction problems leading to improper design; strict requirements for investigations.
Rigid limitations on allowed structure deformations at site where differential heave or settlement is predicted to exceed 3 to 4 in.; large lateral variations in soil conditions.	Disturbance of soil minimized by drilling, thus reducing consolidation and dragdown due to remolding compared to other methods of placing deep foundations such as driving.	Construction techniques sometimes very sensitive to subsurface conditions: susceptible to "necking" in squeezing ground; difficult to concrete requiring tremie if hole filled with slurry or water; cement washing out if water is under artesian pressure; pulling casing disrupting continuity of concrete in the shaft or displacing/distorting the reinforcing cage.
Structural configurations and functional requirements or economics precluding a mat or other foundation; resisting uplift forces from swelling soils; and providing anchorage to pulling, lateral, or overturning forces.	A single shaft carrying very large loads.	Heave beneath base of shaft aggravating movement beneath slab-on-grade.
	Pile caps eliminated.	Failures difficult and expensive to correct.
	Changes in geometry (diameter, penetration, underream) made during construction if required by subsurface conditions.	

Table 6-5. Defects Associated with Drilled Shafts

Defects from Construction Techniques	
Defect	**Remarks**
Discontinuities in the shaft	Do not leave cuttings in the borehole prior to concreting. Too rapid pulling of casing can cause voids in the concrete. Avoid ground-water pressure greater than concrete pressure, inadequate spacing in steel reinforcement, and inadequate concrete slump and workability.
Reduced diameter from caving soil	Caving or squeezing occurs along the shaft in cohesionless silt, rock flour, sand or gravel, and soft soils, especially below the water table. Coarse sands and gravels cave extensively during drilling and tend to freeze casing in place. Soft soils tend to close open boreholes. Raising the auger in soft soils may "suck" the borehole to almost complete closure.
Distortion of reinforcement	Distortion of steel reinforcement cages can occur while the casing is pulled. Horizontal bands or ties should be placed around reinforcing steel.
Defects Attributed to Swelling Soil	
Mode of Defect	**Remarks**
Subsoil wetting below base of shaft	Moisture may migrate down the concrete of the shaft from the surface or from perched water tables into deeper desiccated zones, causing the entire shaft to rise. Shafts may also heave from a rising deep water table. Rise is sometimes avoided by increasing the shaft length or placing the base in nonswelling soil or within a water table.
Uplift	Heave of surrounding desiccated swelling clays can cause friction forces, which in time cause the shaft to rise and even fracture from excessive tensile stress. Rise can be reduced by placing an underreamed base in nonswelling soil, increasing steel reinforcement along the entire shaft length and underreamed base to resist the tensile stress, and providing sleeving to reduce adhesion between the shaft and soil.
Grade beams on swelling soil	Lack of an air gap between the surface of swelling soil and the grade beam can cause the grade beam to rise.
Lateral swell	Shaft foundations have low resistance to damage from lateral swell. Downhill creep of expansive clays contributes to damaged foundations.

(b) The shaft length may be extended further into stable, nonswelling soil to depths of twice the depth of the active zone X_a.

(c) Widely spaced shafts may be constructed with small diameters and a total loading force Q_w that exceeds the maximum uplift thrust (fig. 6-11) expressed as

$$Q_u = \pi D_{s_o} \int^{L_n} f_s dL < Q_w \qquad (6-1)$$

where

Q_u = maximum uplift thrust on perimeter of shaft, tons

D_s = diameter of shaft, feet

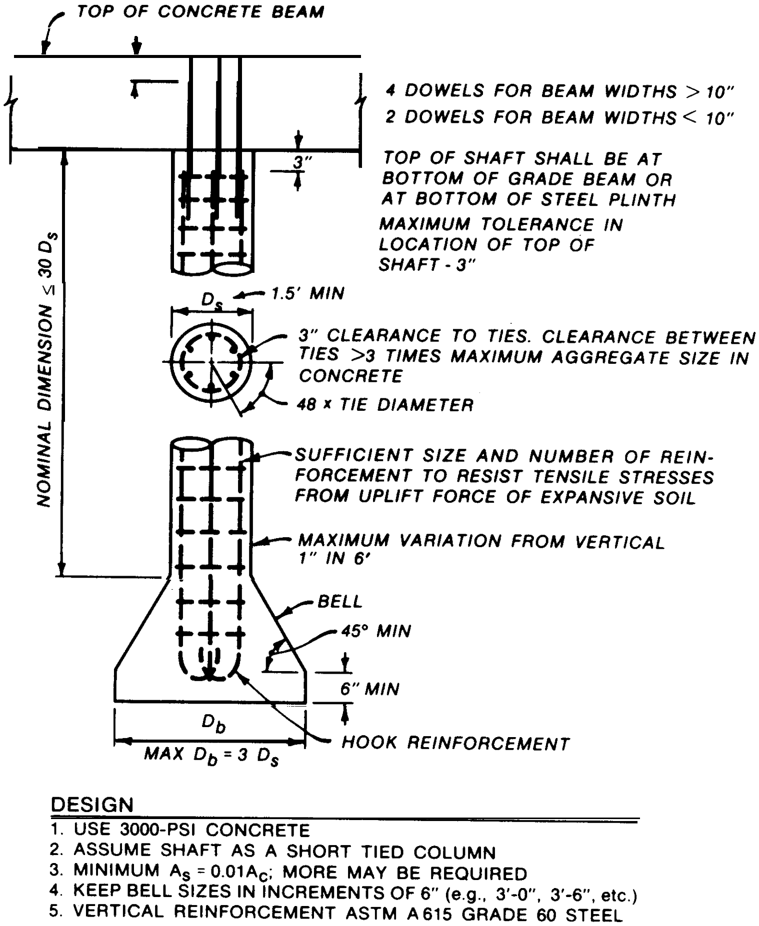
L_n = thickness of the swelling layer moving up relative to the shaft, feet

f_s = skin resistance, tons per square foot

dL = differential increment of shaft length L, feet

The point n in figure 6-11 is the neutral point. The value of L_n should be approximately equal to the depth X_a. The maximum skin resistance f_s is evaluated in *d* below. The loading force Q_w should also be less than or equal to the soil allowable bearing capacity. Wide spans between shafts also reduce angular rotation of the structural members. The minimum spacing of shafts should be 12 feet or 8 times the shaft diameter (whichever is smaller) to minimize effects of adjacent shafts.

(d) The upper portion of the shaft should be kept vertically plumb (maximum variation of 1 inch in 6 feet shown in fig. 6-10) and smooth to reduce adhesion between the swelling soil and the shaft. Friction reducing material, such as roofing felt, bitumen slip layers, polyvinyl chloride (PVC), or polyethylene sleeves, may be placed around the upper shaft to re-



(Based on data from U. S. Army Construction Engineering Research Laboratory TR M-81 by W. P. Jobes and W. R. Stroman)

Figure 6-10. Drilled shaft details.

duce both uplift and downdrag forces. Vermiculite, pea gravel, or other pervious materials that will allow access of water to the expansive material should be avoided.

d. Design. The heave or settlement of the foundation usually controls the design and should not exceed specified limits set by usage requirements and tolerances of the structure. The design of drilled shafts, in addition to bearing capacity, should consider the method of construction, skin resistance, and uplift forces. The computer program HEAVE (WES Miscellaneous Paper GL-82-7) may be used to help determine the movement of drilled shafts for different lengths and diameters of the shaft, and the diameter of the underream for different loading forces.

(1) *Skin resistance.* Skin resistance develops from small relative displacements between the shaft and the adjacent soil. Positive (upward directed) skin friction, which helps support structural loads, develops when the shaft moves down relative to the soil. Uplift of adjacent swelling soils also transfers load to the shaft foundation by positive skin friction and can cause large tensile stresses to develop in the shaft. Negative skin friction, which adds to the structural loads and increases the end bearing force, develops when the surrounding soil moves down relative to the shaft, Negative skin friction is associated with the settling of the adjacent fill, loading of surrounding compressible soils, or lowering of the groundwater level.

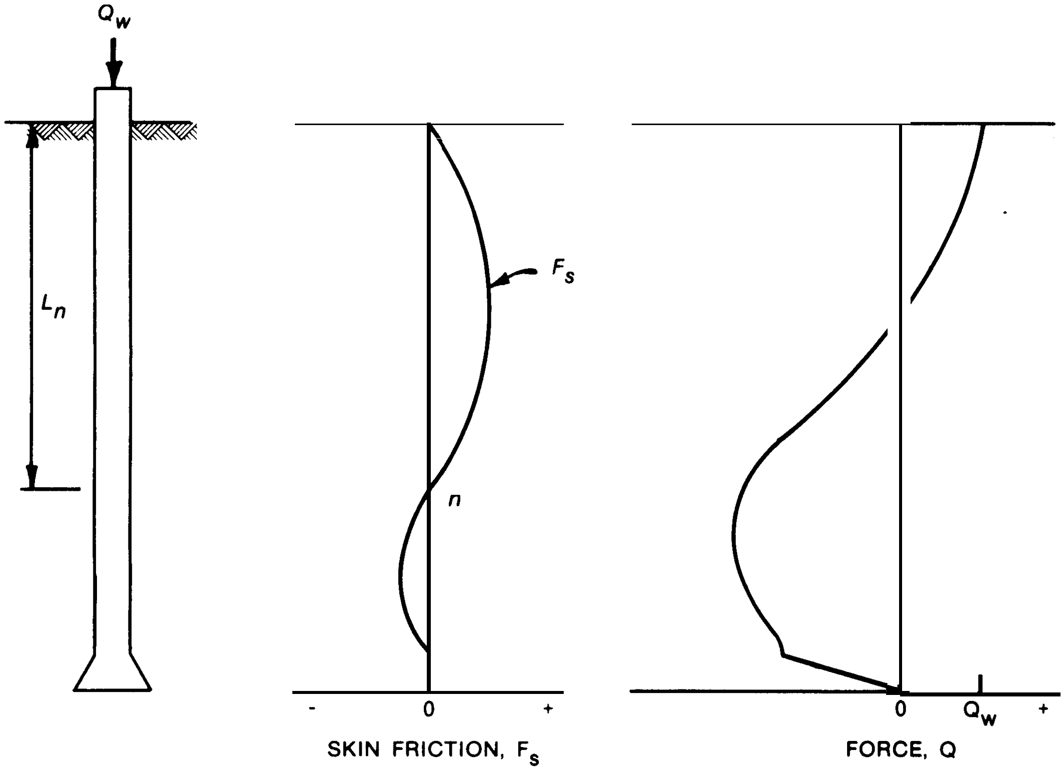
(*a*) The maximum skin friction f_s may be evaluated by the equation

$$f_s = c_a + K\sigma_v' \tan \phi_a \qquad (6\text{-}2)$$

where

c_a = adhesion, tons per square foot
K = ratio of horizontal to vertical effective stress
σ_v' = vertical effective stress, tons per square foot
ϕ_a = angle of friction between the soil and shaft, degrees

The angle ϕ_a is close to, although less than, the effective angle of internal friction ϕ' for remolded cohesive



U. S. Army Corps of Engineers

Figure 6-11. Distribution of load from uplift of swelling soil.

soil against concrete. The skin resistance, which is a function of the type of soil (sand, clay, and silt), is usually fully mobilized with a downward displacement of 1/2 inch or less or about 1 to 3 percent of the shaft diameter. These displacements are much less than those required to fully mobilize end bearing resistance.

(b) The fully mobilized skin resistance has been compared with the undrained, undisturbed shear strength c_u for all clays by

$$f_s = c_a = \alpha_f c_u \tag{6-3}$$

in which α_f is a reduction coefficient that has been found to vary between 0.25 and 1.25 depending on the type of shaft and soil conditions. The reduction factor is the ratio of mobilized shearing resistance to the undrained, undisturbed shear strength. The α_f appears to be independent of soil strength. Also, the in situ reduction factor may appear greater than one depending on the mechanism of load transfer. For example, the shaft load may be transferred over some thickness of soil such that the effective diameter of the shaft is greater than the shaft diameter D,. The reduction factor concept, although commonly used, is not fully satisfactory because α_f is empirical and should be evaluated for each shaft foundation. The average α_f for use in stiff overconsolidated clays is about 0.5 to 0.6. An α_f of 0.25 is recommended if little is known about the soil or if slurry construction is used.

The reduction factor approaches zero near the top and bottom of the shafts in cohesive soils, reaching a maximum near the center. The reduction of α_f at the top is attributed to soil shrinkage during droughts and low lateral pressure, while the reduction at the bottom is attributed to interaction of stress between end bearing and skin resistance.

(c) Skin resistance may also be evaluated in terms of effective stress from results of drained direct shear tests

$$f_s = c' + K\sigma'_v \tan\phi' = \beta\sigma'_v \tag{6-4}$$

where c' is the effective cohesion and ϕ' is the effective angle of internal friction. The effective cohesion is assumed zero in practical applications and eliminated from equation (6-4). Most of the available field data show that $K \tan\phi'$ or β varies from 0.25 to 0.4 for normally consolidated soils, while it is about 0.8 for overconsolidated soils. Reasonable estimates of β can also be calculated for normally consolidated soils by

$$\beta = (1 - \sin\phi')\tan\phi' \tag{6-5a}$$

and in overconsolidated soils by

$$\beta = (1 + 2K_o)\frac{\cos\phi' \sin\phi'}{3 - \sin\phi'} \tag{6-5b}$$

where K_o is the lateral coefficient of earth pressure at rest. If K_o is not known, a reasonable minimum estimate of β is given by assuming $K_o = 1$. The effective cohesion is often assumed to be zero.

(2) *Uplift forces.* Uplift forces, which area direct function of swell pressures, will develop against sur-

faces of shaft foundations when wetting of surrounding expansive soil occurs. Side friction resulting in uplift forces should be assumed to act along the entire depth of the active zone since wetting of swelling soil causes volumetric expansion and increased pressure against the shaft. As the shaft tends to be pulled upward, tensile stresses and possible fracture of concrete in the shaft are induced, as well as possible upward displacement of the entire shaft.

(a) The tension force T (a negative quantity) may be estimated by

$$T = Q_w - Q_u \qquad (6\text{-}6)$$

where Q_w is the loading force from the structure and includes the weight of the shaft. Limited observations show that the value of K required to evaluate Q_u (equation (6-1)) from the skin resistance f_s (equations (6-3) and (6-4)) varies between 1 and 2 in cohesive soils for shafts subject to uplift forces. The same swelling responsible for uplift also increases the lateral earth pressure on the shaft. Larger K values increase the computed tension force.

(b) The shaft should be of proper diameter, length, and underreaming, adequately loaded, and contain sufficient reinforcing steel to avoid both tensile fractures and upward displacement of the shaft. ASTM A 615 Grade 60 reinforcing steel with a minimum yield strength f_s of 60,000 psi should be used. The minimum percent steel required if ASTM A 615 Grade 60 steel is used is given approximately by

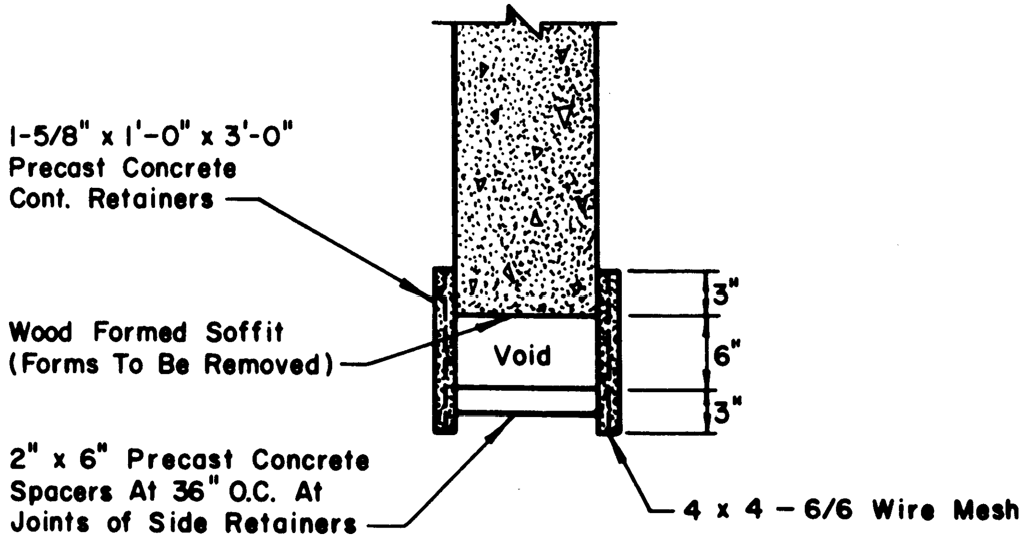
$$\text{Percent } A_s \cong -0.03 \frac{T}{D_s^2} \qquad (6\text{-}7)$$

where T is the tension force in tons and the shaft diameter D_s is in feet. The minimum percent steel A_s should be 1 percent of the concrete area A_c (fig. 6-10), but more may be required. The reinforcing steel should be hooked into any existing bell as shown in figure 6-10, and it may also be hooked into a concrete grade beam. Maximum concrete aggregate size should be one third of the openings in the reinforcement cage.

d. Grade beams. Grade beams spanning between shafts are designed to support wall loads imposed vertically downward. These grade beams should be isolated from the underlying swelling soil with a void space beneath the beams of 6 to 12 inches or 2 times the predicted total heave of soil located above the base of the shaft foundation (whichever is larger). Steel is recommended in only the bottom of the grade beam if grade beams are supported by drilled shafts above the void space. Grade beams resting on the soil without void spaces are subject to distortion from uplift pressure of swelling foundation soil and are not recommended.

(1) *Preparation of void space.* Construction of grade beams with void spaces beneath the beams may require overexcavation of soil in the bottom of the grade beam trench between shafts. The void space may be constructed by use of sand that must later be blown away at least 7 days after concrete placement, or by use of commercially available cardboard or other retainer forms that will support the concrete. The cardboard forms should deteriorate and collapse before swell pressures in underlying soil can deflect or damage the grade beams. The resulting voids should be protected by soil retainer planks and spacer blocks. Figure 6-12 illustrates some void details.

(2) *Loading.* Interior and exterior walls and concentrated loads should be mounted on grade beams. Floors may be suspended from grade beams at least 6 inches above the ground surface, or they maybe placed directly on the ground if the floor slab is isolated from the walls. Support of grade beams, walls, and suspended floors from supports other than the shaft foundation should be avoided. Figure 6-13 illustrates typical exterior and interior grade beams.



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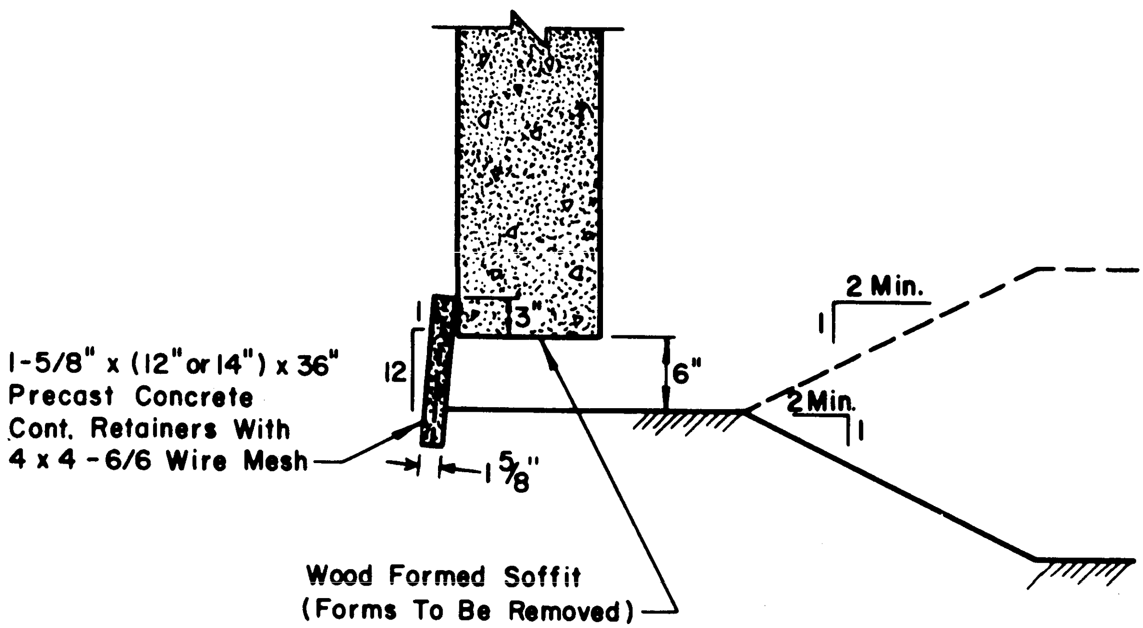

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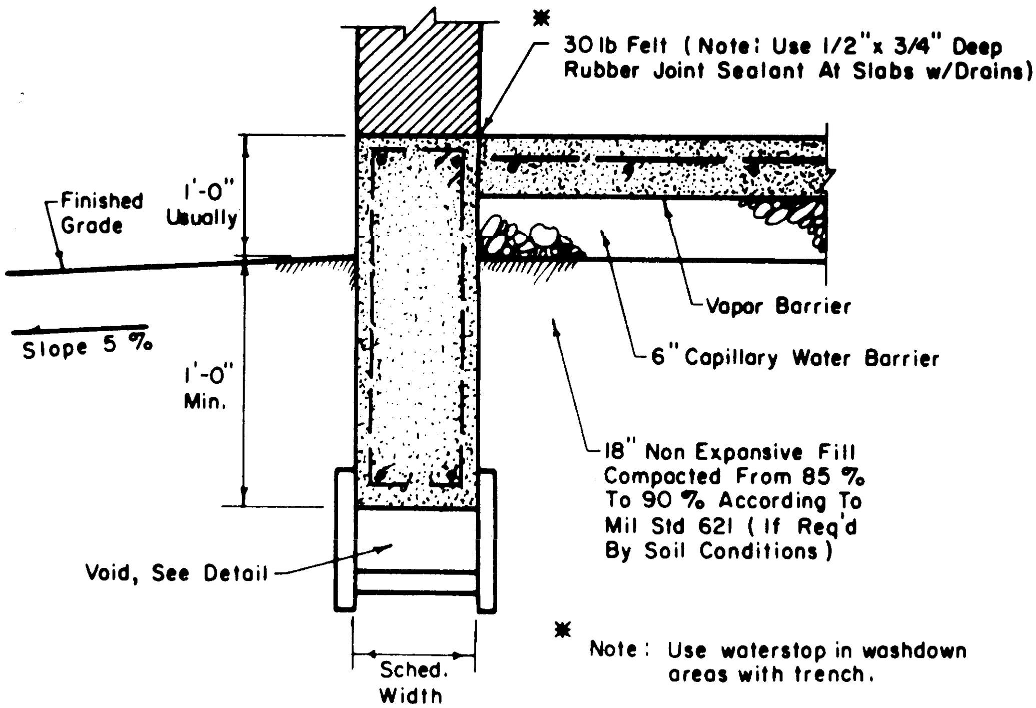
(Based on data from U. S. Army Construction Engineering Research Laboratory TR M-81 by W. P. Jobes and W. R. Stroman)

Figure 6-12. Typical grade beam void details.



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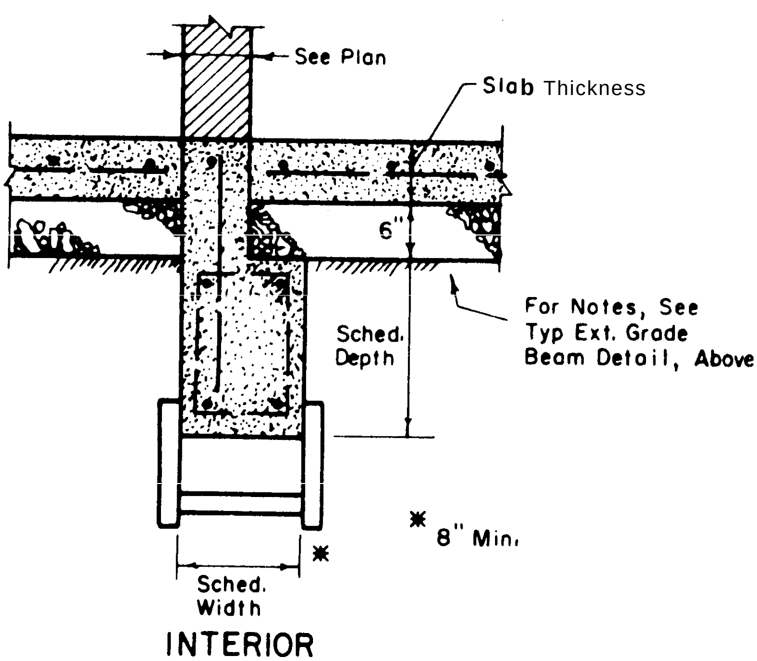

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(Based on data from U. S. Army Construction Engineering Research Laboratory TR M-81 by W. P. Jobes and W. R. Stroman)

Figure 6-13. Typical exterior and interior grade beams.