CHAPTER 1
INTRODUCTION

1-1. Purpose and Scope. This manual presents guidelines for calculation of vertical displacements and settlement of soil under shallow foundations (mats and footings) supporting various types of structures and under embankments.

a. Causes of Soil Displacements. Soil is a nonhomogeneous porous material consisting of three phases: solids, fluid (normally water), and air. Soil deformation may occur by change in stress, water content, soil mass, or temperature. Vertical displacements and settlement caused by change in stress and water content are described in this manual. Limitations of these movements required for different structures are described in Chapter 2.

(1) Elastic deformation. Elastic or immediate deformation caused by static loads is usually small, and it occurs essentially at the same time these loads are applied to the soil. Guidance for tests and analyses to estimate immediate settlements of foundations, embankments, pavements, and other structures on cohesionless and cohesive soils for static loading conditions is given in Sections I and II of Chapter 3.

(2) Consolidation. Time delayed consolidation is the reduction in volume associated with a reduction in water content, and it occurs in all soils. Consolidation occurs quickly in coarse-grained soils such as sands and gravels, and it is usually not distinguishable from elastic deformation. Consolidation in fine-grained soils such as clays and organic materials can be significant and take considerable time to complete. Guidance for tests and analyses to estimate consolidation settlement of foundations, embankments, pavements, and other structures on cohesive soil for static loading conditions is given in Section III of Chapter 3.

(3) Secondary compression and creep. Secondary compression and creep are associated with the compression and distortion at constant water content of compressible soils such as clays, silts, organic materials, and peat. Guidance for tests and analyses to estimate secondary compression settlement is given in Section IV of Chapter 3.

(4) Dynamic forces. Dynamic loads cause settlement from rearrangement of particles, particularly in cohesionless soil, into more compact positions. Guidance to estimate settlement for some dynamic loads is given in Chapter 4.

(5) Expansive soil. Expansive soil contains colloidal clay minerals such as montmorillonite that experience heave and shrinkage with changes in the soil water content. Guidance for calculation of soil movements in expansive soil is given in Section I of Chapter 5.

(6) Collapsible soil. Collapsible soil usually consists of cohesive silty sands with a loose structure or large void ratio. The cohesion is usually caused by the chemical bonding of particles with soluble compounds such as calcareous or ferrous salts. Collapse occurs when the bonds between particles are dissolved. Guidance for calculation of settlement in collapsible soil is given in Section II of Chapter 5.

b. Coping with Soil Movements. Soil movements may be minimized by treating the soil prior to construction by numerous methods such as removal of poor soil and replace with suitable soil, precompression of soft soil, dynamic consolidation of cohesionless soil, and chemical stabilization or wetting of expansive or collapsible soil. Foundations may be designed to tolerate some differential movements. Remedial techniques such as underpinning with piles, grouting, and slabjacking are available to stabilize and repair damaged foundations. Methodology for minimizing and coping with settlement is given in Chapter 6.

c. Limitations of the Manual. This manual excludes settlement caused by subsidence and undermining by tunnels, subsidence due to buried karst features or cavities, thermal effects of structures on permafrost, effects of frost heave, loss in mass from erosion, loss of ground from rebound and lateral movement in adjacent excavations, and loss of support caused by lateral soil movement from landslides, downhill creep, and shifting retaining walls.

(1) Horizontal deformation. Horizontal deformation of structures associated with vertical deformations may also occur, but such analysis is complex and beyond the scope of this manual.

(2) Deep foundations. Deep foundations are driven piles and drilled shafts used to transmit foundation loads to deeper strata capable of supporting the applied loads. Guidelines on settlement analysis of deep foundations is given in TM 5-809-7, "Design of Deep Foundations (Except Hydraulic Structures)".

(3) Landfills. Settlement of domestic and hazardous landfills are unpredictable and cannot be readily estimated using techniques presented in this manual.

1-2. Applicability. This manual applies to all Corps of Engineers field operating activities. Applications include, but are not limited to, design analysis of alternatives for new construction, analyses for rationalizing in-service performance, forensic investigations, and damage assessments and repair/rehabilitation design.

1-3. References. Standard references pertaining to this manual are listed in Appendix A, References. Each reference is identified in the text by the designated Government publication number or performing agency. Additional reading materials are listed in the Bibliography and are indicated throughout the manual by numbers (item 1, 2, etc.) that correspond to similarly numbered items in Appendix B.

1-4. Rescission. This manual supersedes EM 1110-2-1904, "Settlement Analysis", Chapter 4, dated January 1953.

1-5. General Considerations and Definitions. Placement of an embankment load or structure on the surface of a soil mass introduces stress in the soil that causes the soil to deform and leads to settlement of the structure. It is frequently necessary to estimate the differential and total vertical soil deformation caused by the applied loads. Differential movement affects the structural integrity and performance of the structure. Total deformation is significant relative to connections of utility lines to buildings, grade and drainage from structures, minimum height specifications of dams (i.e., freeboard), and railroad and highway embankments. Soils and conditions described in Table 1-1 require special considerations to achieve satisfactory design and performance. Early recognition of these problems is essential to allow sufficient time for an adequate field investigation and preparation of an appropriate design.

a. Preconsolidation Stress. The preconsolidation stress or maximum effective past pressure σ'_p experienced by a foundation soil is a principle factor in determining the magnitude of settlement of a structure supported by the soil. σ'_p is the maximum effective stress to which the in situ soil has been consolidated by a previous loading; it is the boundary between recompression and virgin consolidation, which are described in Section III, Chapter 3. Pressures applied to the foundation soil that exceed the maximum past pressure experienced by the soil may cause substantial settlement. Structures should be designed, if practical, with loads that maintain soil pressures less than the maximum past pressure.

(1) Geological evidence of a preconsolidation stress. Stresses are induced in the soil mass by past history such as surcharge loads from soil later eroded away by natural causes, lowering of the groundwater table and desiccation by drying from the surface.

(a) Temporary groundwater levels and lakes may have existed causing loads and overconsolidation compared with existing effective stresses.

(b) Desiccation of surface soil, particularly cyclic desiccation due to repeated wetting and drying, creates significant microscale stresses which in turn cause significant preconsolidation effects. Such effects include low void ratios as well as fissures and fractures, high density, high strength and high maximum past pressures measured in consolidation tests.

(c) A high preconsolidation stress may be anticipated if $\frac{N}{15} \bullet \frac{1}{\sigma_{oz}} > 0.25$ where N is the blowcount from standard penetration test (SPT) results and σ_{oz} (tons/square foot or tsf) is the total overburden pressure at depth z (Table 3-2, TM 5-818-1).

(2) Evaluation from maximum past thickness. Local geologic records and publications when available should be reviewed to estimate the maximum past thickness of geologic formations from erosion events, when and amount of material removed, glacial loads, and crustal tilt.

(a) The minimum local depth can sometimes be determined from transvalley geologic profiles if carried sufficiently into abutment areas to be beyond the influence of valley erosion effects.

(b) The maximum past pressure at a point in an in situ soil is estimated by multiplying the unit wet soil weight (approximately 0.06 tsf) by the total estimated past thickness of the overlying soil at that point.

(c) Results of the cone penetration test (CPT) may be used to evaluate the thickness of overburden soil removed by erosion if the cone tip resistance q_c increases linearly with depth (refer to Figure 7 in item 56). The line of q_c versus depth is extrapolated back above the existing surface of the soil

Table 1-1

Problem Soils and Conditions[a]

a. Problem Soils

Soil	Description
Organic	Colloids or fibrous materials such as peats, organic silts, and clays of many estuarine, lacustrine, or fluvial environments are generally weak and will deform excessively under load. These soils are usually not satisfactory for supporting even very light structures because of excessive settlements.
Normally consolidated clays	Additional loads imposed on soil consolidated only under the weight of the existing environment will cause significant long-term settlements, particularly in soft and organic clays. These clays can be penetrated several centimeters by the thumb. The magnitude and approximate rate of settlement should be determined by methods described in Section III, Chapter 3, in order to determine acceptability of settlements for the function and characteristics of the structure. Bottoms of excavations may heave and adjoining areas settle unless precautions are taken to prevent such movement.
Sensitive clays	The ratio of undisturbed to remolded strength is the sensitivity of a clay. Clays having remolded strengths 25 percent or less of the undisturbed strength are considered sensitive and subject to excessive settlement and possible catastrophic failure. Such clays preconsolidated by partial desiccation or erosion of overlying soil may support shear stresses caused by foundation loads if these loads are well within the shear strength of the clay. Refer to paragraph 3-12 on apparent preconsolidation for analysis of settlement.
Swelling and shrinking clays and shales	Clays, especially those containing montmorillonite or smectite, expand or contract from changes in water content and are widely distributed throughout the United States and the world. Clay shales may swell significantly following stress relief as in a cut or excavation and following exposure to air. Foundations in these soils may have excessive movements unless the foundation soil is treated or provisions are made in the design to account for these movements or swell pressures developed in the soil on contact with moisture. Refer to Section I, Chapter 5, for details on analysis of heave and shrinkage.
Collapsible soils	The open, porous structure of loosely deposited soil such as silty clays and sands with particles bonded with soluble salts may collapse following saturation. These soils are often strong and stable when dry. Undisturbed samples should be

[a] Based on information from the Canadian Foundation Engineering Manual, 2nd edition.

Table 1-1. Continued

Soil	Description
	taken to accurately determine the in situ density. Refer to Section II, Chapter 5, for details on settlement analysis.
Loose granular soils	All granular soils are subject to some densification from vibration, which may cause significant settlement and liquefaction of soil below the water table; however, minor vibration, pile driving, blasting, and earthquake motion in loose to very loose sands may induce significant settlement. Limits to potential settlement and applicable densification techniques should be determined. Refer to Chapter 4 for analysis of dynamic settlements in these soils.
Glacial tills	Till is usually a good foundation soil except boulders and soft layers may cause problems if undetected during the field investigation.
Fills	Unspecified fills placed randomly with poor compaction control can settle significantly and provide unsuitable foundation soil. Fills should usually be engineered granular, cohesive materials of low plasticity index < 12 and liquid limit < 35. Suitable materials of the Unified Soil Classification System include GW, GM, GC, GP, SW, SP, SM, SC, and CL soils. Compaction beneath structures to ≥ 92 percent of optimum density for cohesive fill or 95 percent for cohesionless fill using ASTM Standard Test Methods D 1557 has provided highly successful constructability and in-service performance. Refer to EM 1110-2-1911 for construction control of earth and rockfill dams.

b. Problem Conditions

Condition	Description
Meander loops and cutoffs	Soils that fill abandoned waterways are usually weak and highly compressible. The depth of these soils should be determined and estimates made of potential settlement early in design to allow time for development of suitable measures for treating the soil or accommodating settlement.
Landslides	Potential landslides are not easily detected, but evidence of displacement such as bowed trees and tilted or warped strata should be noted. Sensitive clays and cutting action of eroding rivers significantly increase the risk of landslides. Slopes and excavations should be minimized, seasonal variations in the local water table considered in the design, and suitable arrangements for drainage provided at the top and toe of slopes.

Table 1-1. Concluded

Soil	Description
Kettle holes	The retreating continental ice sheet left large blocks of ice that melted and left depressions, which eventually filled with peat or with soft organic soils. Lateral dimensions can vary from a few to several hundred feet. Depths of kettle holes usually do not exceed 40 percent of lateral dimensions and can sometimes be identified as shallow surface depressions.
Mined areas and sinkholes	Voids beneath the surface soil may lead to severe ground movements and differential settlement from subsidence or caving. Sink holes are deep depressions formed by the collapse of the roofs of underground caverns such as in limestone. Maps of previous mined areas are helpful when available. Published geological data, nondestructive in situ tests and past experience help indicate the existence of subsurface cavities. Investigations should be thorough to accurately determine the existence and location of any subsurface voids.
Lateral soil distortions	Lateral distortions are usually not significant, but can occur in highly plastic soils near the edge of surface loads. These distortions can adversely affect the performance of foundations of structures and embankments. Driven piles can cause large lateral displacements and excessive pressures on retaining walls.
Downdrag	Compression of fills or consolidation of soft soil adjacent to wall footings or piles cause downdrag on the footing or pile. This leads to substantial loads at the base of the foundation that can exceed the bearing capacity of the underlying soil supporting the footing or pile. Failure of the foundation can occur with gross distortion.
Vibrations	Cohesionless soil, especially loose sands and gravels, can densify and settle when subject to machine vibration, blasts, and earthquakes. Distortion with negligible volume change can occur in loose, saturated sands due to liquefaction. Low level sustained vibration can densify saturated sands.

to the elevation where q_c is zero assuming the original cohesion is zero. The difference in elevation where q_c is zero and the existing elevation is the depth of overburden removed by erosion. This depth times the unit wet weight γ is the total maximum past pressure σ_p . The cohesion for many clays is not zero, but contributes to a q_c approaching one tsf. Extrapolating the line above the existing ground surface to $q_c = 1$ tsf produces a more conservative depth of overburden clay soil. This latter estimate of overburden depth is recommended.

(3) Evaluation from overconsolidation ratio. The preconsolidation stress σ'_p may be evaluated from the overconsolidation ratio (OCR), σ'_p/σ'_{oz} , where σ'_{oz} is the effective vertical overburden pressure at depth z .

(a) The initial vertical effective pressure in a saturated soil mass before placement of an applied load from a structure is given by

$$\sigma'_{oz} = \gamma z - u_w \tag{1-1}$$

where

σ'_{oz} = initial vertical effective stress at depth z , tsf
γ = saturated unit weight of soil mass at depth z , tsf
z = depth, ft
u_w = pore water pressure, tsf

u_w usually is the hydrostatic pressure $\gamma_w \cdot z_w$ where γ_w is the unit weight of water, 0.031 tsf, and z_w is the height of a column of water above depth z . γz is the total overburden pressure σ_{oz} .

(b) The overconsolidation ratio has been related empirically with the coefficient of earth pressure at rest K_o , $\sigma'_{hz}/\sigma'_{oz}$, and the plasticity index PI in Figure 3-21, TM 5-818-1. σ'_{hz} is the effective horizontal pressure at rest at depth z . Normally consolidated soil is defined as soil with OCR = 1. Overconsolidated soil is defined as soil with OCR > 1.

(c) The results of pressuremeter tests (PMT) may be used to evaluate the effective horizontal earth pressure σ'_{hz} . K_o may be evaluated if the effective vertical overburden pressure σ'_{oz} at depth z is known and the OCR estimated as above.

(4) Laboratory tests. The preconsolidation stress may be calculated from results of consolidation tests on undisturbed soil specimens, paragraph 3-12.

(a) A high preconsolidation stress may be anticipated if the natural water content is near the plastic limit PL or below or if $C_u/\sigma_{oz} > 0.3$ where C_u is the undrained shear strength (Table 3-2, TM 5-818-1).

(b) An empirical relationship between the preconsolidation stress and liquidity index as a function of clay sensitivity, ratio of undisturbed to re-molded undrained shear strength, is given in Figure 1-1. The preconsolidation stress may also be estimated from (NAVFAC DM-7.1)

$$\sigma'_p = \frac{C_u}{0.11 + 0.0037PI} \tag{1-2}$$

where

σ'_p = preconsolidation stress, tsf
C_u = undrained shear strength, tsf
PI = plasticity index, percent

b. <u>Pressure Bulb of Stressed Soil</u>. The pressure bulb is a common term that represents the volume of soil or zone below a foundation within which the

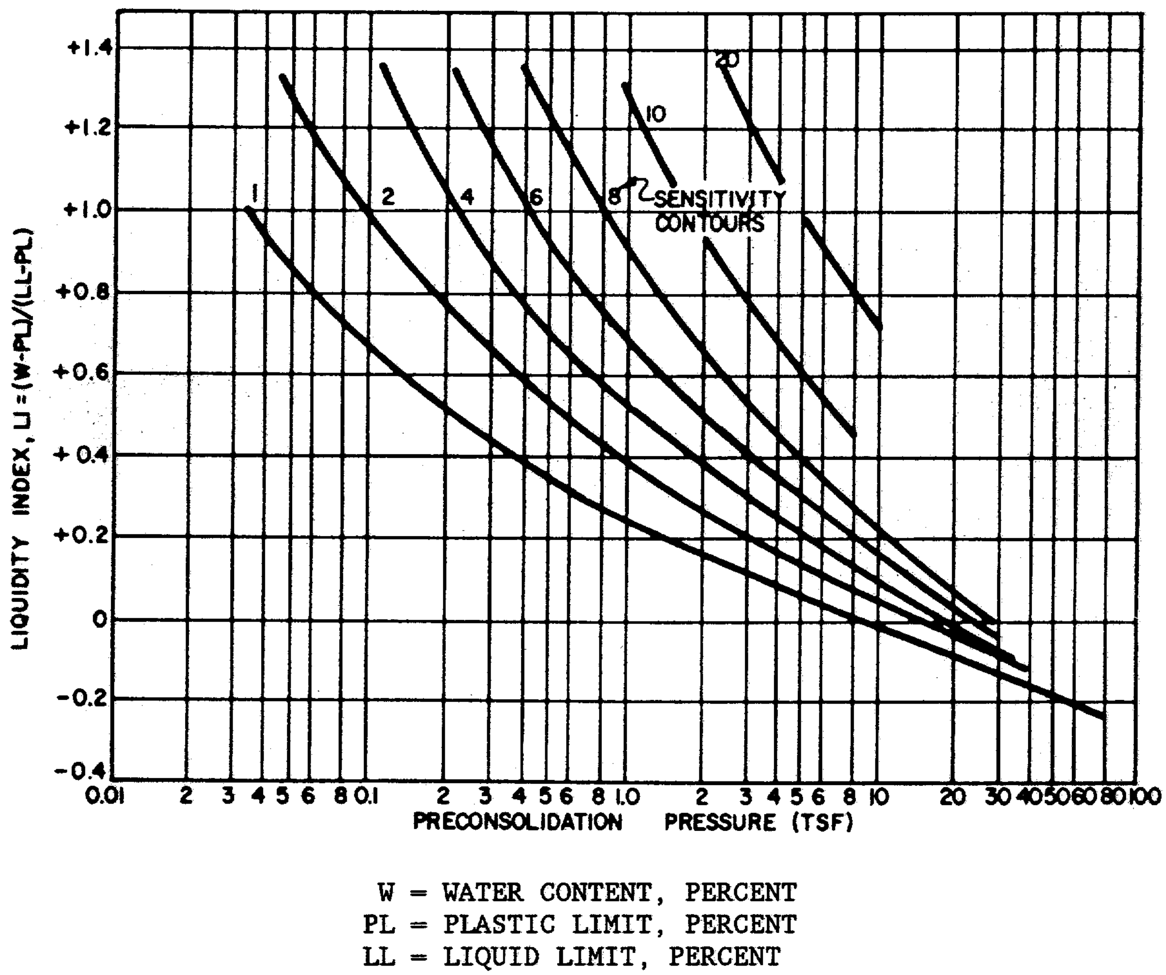

Figure 1-1. Preconsolidation Stress as a function of Liquidity Index LI and clay sensitivity (ratio of undisturbed to remolded shear strength) (After NAVFAC DM 7.1)

foundation load induces appreciable stress. The stress level at a particular point of soil beneath a foundation may be estimated by the theory of elasticity.

(1) Applicability of the theory of elasticity. Earth masses and foundation boundary conditions correspond approximately with the theory of plasticity (item 52).

(2) Stress distribution. Various laboratory, prototype, and full scale field tests of pressure cell measurements in response to applied surface loads on homogeneous soil show that the measured soil vertical stress distribution corresponds reasonably well to analytical models predicted by linear elastic analysis for similar boundary conditions.

(a) The Boussinesq method is commonly used to estimate the stress distribution in soil. This distribution indicates that the stressed zone decreases toward the edge of the foundation and becomes negligible (less than 10 percent of the stress intensity) at depths of about 6 times the width of an infinite strip or 2 times the width of a square foundation, Figure 1-2.

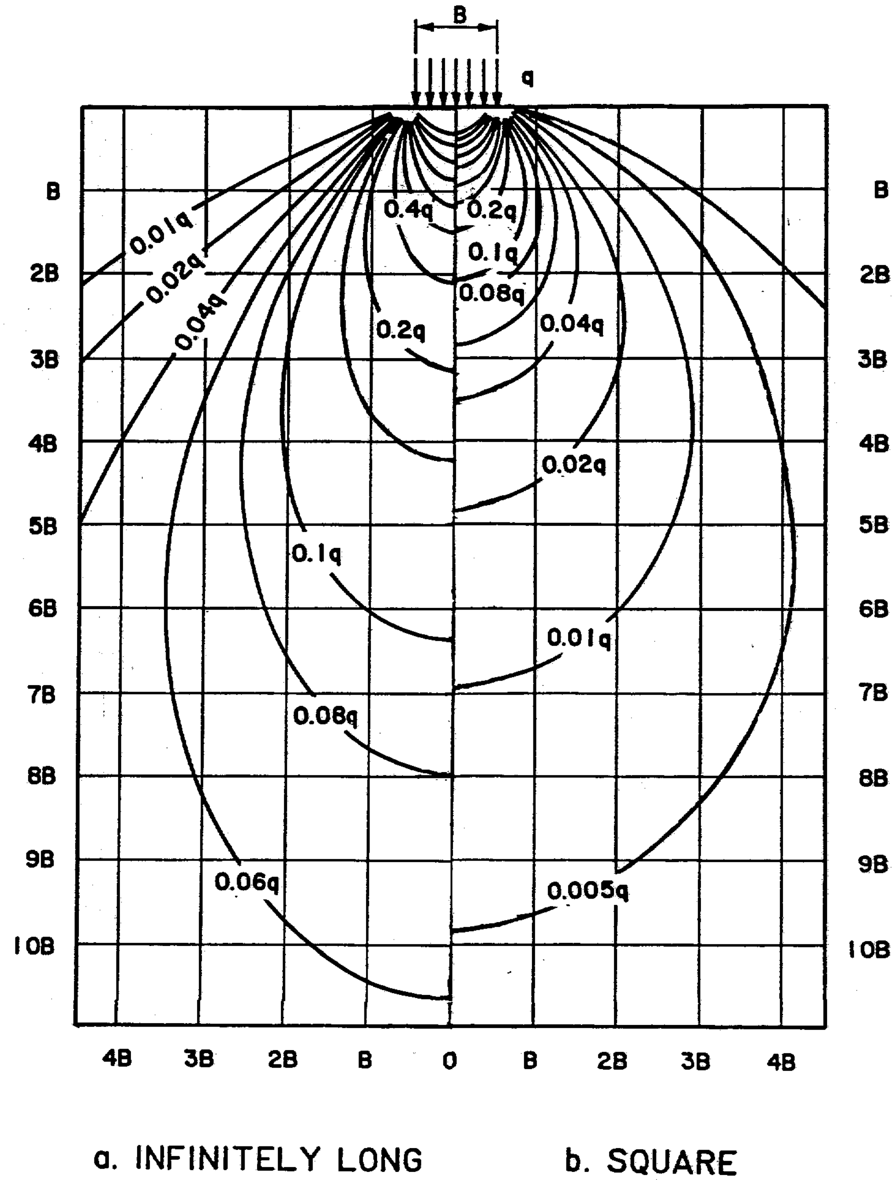

Figure 1-2. Contours of equal vertical stress beneath a foundation in a semi-infinite elastic solid by the Boussinesq solution

(b) The recommended depth of analysis is at least twice the least width of the footing or mat foundation, 4 times the width of infinite strips or embankments, or the depth of incompressible soil, whichever comes first.

(c) The distribution of vertical stress in material overlain by a much stiffer layer is more nearly determined by considering the entire mass as homogeneous rather than a layered elastic system.

(d) Methods and equations for estimating stresses in foundation soils required for analysis of settlement are provided in Appendix C, Stress Distribution in Soil.

(3) Applicability to settlement calculations. The ability to predict settlements using elastic theory depends much more strongly on the in situ nonlinearity and material inhomogeneity than errors in the distribution of stresses. These settlements directly depend on the assumed constitutive material law and on the magnitude of the required soil parameters. Refer to Appendix D for further information on elasticity theory.

c. Contact Pressure and Deformation Pattern. The shape of the deformation pattern varies depending on flexibility of the foundation and type of soil. Figure 1-3 illustrates the relative distribution of soil contact pressures and displacements on cohesionless and cohesive soil. Linear contact pressure distributions from uniformly applied pressure q are often assumed for settlement analysis, Figure 1-3c and 1-3d. An applied load Q may cause an unequal linear soil contact pressure distribution, Figure 1-3e.

(1) Cohesionless soil. Cohesionless soil is often composed of granular or coarse-grained materials with visually detectable particle sizes and with little cohesion or adhesion between particles. These soils have little or no strength when unconfined and little or no cohesion when submerged. Apparent adhesion between particles in cohesionless soil may occur from capillary tension in pore water. Settlement usually occurs rapidly with little long-term consolidation and secondary compression or creep. Time rate effects may become significant in proportion to the silt content such that the silt content may dominate consolidation characteristics.

(a) Uniformly loaded rigid foundations (footings of limited size or footings on cohesionless soil) may cause less soil contact pressure near the edge than near the center, Figure 1-3a, because this soil is pushed aside at the edges due to the reduced confining pressure. This leads to lower strength and lower modulus of elasticity in soil near the edge compared with soil near the center. The parabolic soil contact pressure distribution may be replaced with a saddle-shaped distribution, Figure 1-3b, for rigid footings or mats if the soil pressure does not approach the allowable bearing capacity.

(b) The distortion of a uniformly loaded flexible footing, mat, or embankment on cohesionless soil will be concave downward, Figure 1-3c, because the soil near the center is stressed under higher confining pressure such that the modulus of elasticity of the soil is higher than near the edge.

(c) The theory of elasticity is not applicable to cohesionless soil when the stress or loading increment varies significantly throughout the soil such that an equivalent elastic modulus cannot be assigned. Semi-empirical and numerical techniques have been useful to determine equivalent elastic parameters at points in the soil mass based on stress levels that occur in the soil.

(2) Cohesive soil. Cohesive soil often contains fine-grained materials consisting of silts, clays, and organic material. These soils have significant strength when unconfined and air-dried. Most cohesive soil is relatively impermeable and when loaded deforms similar to gelatin or rubber; i.e., the undrained state. Cohesive soils may include granular materials with bonding agents between particles such as soluble salts or clay aggregates. Wetting of soluble agents bonding granular particles may cause settlement in loose or high void ratio soil. Refer to Section II, Chapter 5, for evaluation of settlement in collapsible soil.

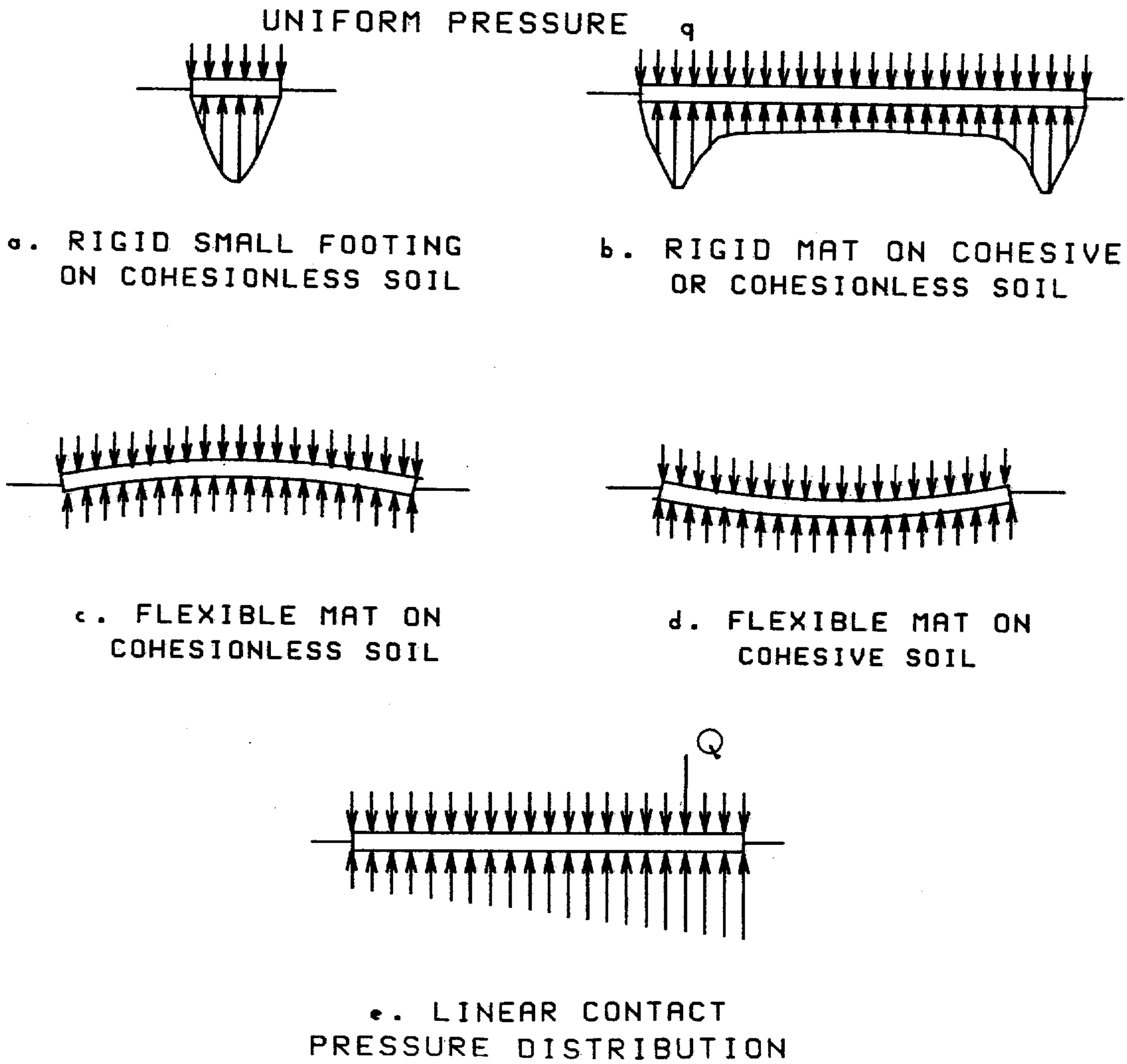

Figure 1-3. Relative distribution of soil contact pressures and displacements of rigid and flexible mats or footings on cohesionless and cohesive soils

(a) A uniform pressure applied to a rigid foundation on cohesive soil, Figure 1-3b, can cause the soil contact pressure to be maximum at the edge and decrease toward the center because additional contact pressure is generated to provide stress that shears the soil around the perimeter.

(b) A uniform pressure applied to a flexible foundation on cohesive soil, Figure 1-3d, causes greater settlement near the center than near the edge because the cumulative stresses are greater near the center as a result of the pressure bulb stress distribution indicated in Figure 1-2. Earth pressure measurements from load cells beneath a stiffening beam supporting a large, but flexible, ribbed mat also indicated large perimeter earth pressures resembling a saddle-shaped pressure distribution similar to Figure 1-3b (item 29).

(c) Elastic theory has been found useful for evaluation of immediate settlement when cohesive soil is subjected to moderate stress increments. The modulus of elasticity is a function of the soil shear strength and often in creases with increasing depth in proportion with the increase in soil shear strength.

(d) Cohesive soil subject to stresses exceeding the maximum past pressure of the soil may settle substantially from primary consolidation and secondary compression and creep.

d. Sources of Stress. Sources of stress in soil occur from soil weight, surface loads, and environmental factors such as desiccation from drought, wetting from rainfall, and changes in depth to groundwater.

(1) Soil weight. Soil strata with different unit weights alter the stress distribution. Any change in total stress results in changes in effective stress and pore pressure. In a saturated soil, any sudden increase in applied total stress results in a corresponding pore pressure increase, Equation 1-1. This increase may cause a flow of water out of the soil deposit, a decrease in pore pressure, and an increase in effective stress. Changes in pore water pressure such as the raising or lowering of water tables also lead to a reduction or increase in effective stress.

(2) Surface loads. Loads applied to the surface of the soil mass increase the stress within the mass. The pressure bulb concept, Figure 1-2, illustrates the change in vertical stress within the soil mass. Placement of a uniform pressure over a foundation with a minimum width much greater than the depth of the soil layer will cause an increase of vertical stress in the soil approximately equal to the applied pressure.

(3) Rules of thumb for static loads. Preliminary settlement analyses are sometimes performed before the structural engineer and architect are able to furnish the design load conditions.

(a) Some rules of thumb for line and column loads for buildings described in Table 1-2 are based on a survey of some engineering firms. Tall multistory structures may have column loads exceeding 1000 tons. Column spacings are often 20 to 25 ft or more. The average pressure applied per story of a building often varies from 0.1 to 0.2 tsf. Refer to TM 5-809-1/AFM 88-3, Chapter 1, "Load Assumptions for Buildings", for estimating unfactored structural loads.

(b) Vertical pressures from embankments may be estimated from the unit wet weight times height of the fill.

(c) Vertical pressures from locks, dams, and retaining walls may be estimated by dividing the structure into vertical sections of constant height and evaluating the unit weight times the height of each section.

Table 1-2

Some Typical Loads on Building Foundations

Structure	Line Load, tons/ft	Column Load, tons
Apartments	0.5 to 1	30
Individual housing	0.5 to 1	< 5
Warehouses	1 to 2	50
Retail Spaces	1 to 2	40
Two-story buildings	1 to 2	40
Multistory buildings	2 to 5	100
Schools	1 to 3	50
Administration buildings	1 to 3	50
Industrial facilities		50