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IEEE Guide for Transmission Structure Foundation Design and Testing

IEEE Power Engineering Society

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Transmission and Distribution Committee

and the

American Society of Civil Engineers

Sponsored by the
Transmission Structure Foundation Design Standard Committee

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IEEE-SA Standards Board

Abstract: The design of foundations for conventional transmission line structures, which include lattice towers, single or multiple shaft poles, H-frame structures, and anchors for guyed structures is presented in this guide.
Keywords: anchor, foundation, guyed structure, H-frame structure, lattice tower, multiple shaft pole, single shaft pole, transmission line structure

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Introduction

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This design guide is intended for the use of the practicing professional engineer engaged in the design of foundations for electrical transmission line structures. This guide is not to be used as a substitute for professional engineering competency, nor is it to be considered as a rigid set of rules. Of all building materials, soil is the least uniform and most unpredictable; therefore, the methods described in this guide may not be the only methods of design and analysis, nor may they be appropriate in all situations. Design and analysis must be based upon sound engineering principles and relevant experience.

This design guide is the result of a major effort to consolidate the results of published reports and data, ongoing research, and experience into a single document. It is also an outgrowth of the previously published efforts of a joint committee of the American Society of Civil Engineers and the Institute of Electrical and Electronic Engineers, which combined the knowledge, expertise, and experience of both organizations in the field of transmission line structure foundation design. Electrical transmission line structures are unique when compared with other structures, primarily in that no human occupancy is involved and the loading requirements are different from other structure types. The primary loading of most conventional structures or buildings is a dead load or sustained live load and lateral wind forces or seismic loads. The primary loading of a transmission line structure is caused by meteorological loads, such as wind and ice, or combinations thereof [B68].[1] Under normal weather or operating conditions, the loads may be only a fraction of the ultimate capacity of tangent structures, but the application of the design load is short term and sometimes violent as nature unleashes its fury. In addition, a finite probability exists that the design load could be exceeded.

Foundations for transmission line structures are called on to resist loading conditions consisting of various combinations. Lattice tower foundations typically experience uplift or compression and horizontal shear loads. H-frame structures experience combinations of uplift or compression and horizontal shear and moment loads. Single pole structures experience horizontal shear loads and large overturning moments. Foundations for transmission structures must satisfy the same fundamental design criteria as those for any other type of structure—adequate strength and stability, tolerable deformation, and cost-effectiveness. In addition, transmission line structures may be constructed hundreds or thousands of times in a multitude of subsurface conditions encountered along the same route. Therefore, optimization and standardization for cost-effectiveness is highly desirable.

This design guide addresses fundamental performance criteria and the design methods associated with transmission line structure modes of loading, much of which is not found in geotechnical engineering textbooks.

Many alternative approaches can be used for the geotechnical design of foundations for transmission line structures. It is the intent of this design guide to provide several approaches to the design of various foundation types that are consistent with the present state of geotechnical engineering practice. Where several methods are presented for the design of a particular type of foundation, the design engineer should exercise sound engineering judgment in determining which method is most representative of the situation.

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Contents

1. Overview 1

1.1 Scope 1
1.2 System design considerations 1
1.3 Other considerations 2

2. Loading and performance criteria 3

2.1 Loading 3
2.2 Foundation performance criteria and structure types 5

3. Subsurface investigation and selection of geotechnical design parameters 10

3.1 General 10
3.2 Phases of investigation 10
3.3 Types of boring samples 13
3.4 Soil and rock classification 15
3.5 Engineering properties 18

4. Design of spread foundations 23

4.1 Structural applications 23
4.2 Analysis 31
4.3 Traditional design methods 66
4.4 Construction considerations 73
4.5 General foundation considerations 74

5. Design of drilled shaft and direct embedment foundations 77

5.1 Types of foundations 77
5.2 Structural applications 79
5.3 Drilled concrete shaft foundations 80
5.4 Direct embedment foundations 110
5.5 Precast-prestressed, hollow concrete shafts and steel casings 113
5.6 Design and construction considerations 113

6. Design of pile foundations 115

6.1 Pile types and orientation 116
6.2 Pile stresses 121
6.3 Pile capacity 122
6.4 Pile deterioration 137
6.5 Construction considerations 139

7. Design of anchors 139

7.1 Anchor types 139
7.2 Anchor application 142
7.3 Design analysis 144
7.4 Group effect 163
7.5 Grouts 163

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7.6 Construction considerations 164

8. Load tests 167

8.1 Introduction 167
8.2 Instrumentation 169
8.3 Scope of test program 170

Annex A (informative) Bibliography 177

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1. Overview

1.1 Scope

The material presented in this design guide pertains to the design of foundations for conventional transmission line structures, which include lattice towers, single or multiple shaft poles, H-frame structures, and anchors for guyed structures. It discusses the mode of loads that those structures impose on their foundations and applicable foundation performance criteria. The design guide addresses subsurface investigations and the design of foundations, such as spread foundations (footings), drilled shafts, direct embedded poles, driven piles, and anchors. The full-scale load testing of the above-listed foundation types is also presented.

This design guide does not include the structural design of the foundations nor the design of the structure. Citations [B5][1] and [B50] provide guidance for the design of lattice towers and tubular steel poles, respectively. The foundation engineer should have an understanding of the magnitudes and time-history of various loading conditions imposed on the foundations in order to provide a suitable foundation to support the transmission line structures under the actual loading conditions that may be reasonably expected in actual service.

1.2 System design considerations

A transmission line is a system of interconnected elements, each individually designed. The overall line must integrate all of these individual design elements into a coordinated structural system.

Every decision made for the system should consider total installed cost, of which foundations are a major consideration. For example, wire tensions are sometimes increased to minimize the number and/or height of the supporting structures. However, if a significant number of angles is in the line, total installed costs may be higher because of increased angle structure and foundation costs. Similarly, when developing structure configurations, a wider base structure could be considered to reduce foundation loads and thereby decrease the foundation cost. This must be evaluated against the added cost of widening the structure.

[1]The numbers in brackets correspond to those of the bibliography in Annex A.

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When designing a transmission line, the engineer has the option to design each foundation for site-specific loadings and subsurface conditions or to develop standard designs that can be used at predetermined similar sites. The preferred approach is one that will minimize the total installed cost of the line, and it may also involve a combination of site-specific and standard foundation designs.

A custom design at each site has the advantage of avoiding costly overdesign. However, this approach will require a more extensive subsurface investigation in advance of the design and involve added engineering investment to prepare the many individual designs required. A custom foundation design may be justified at angle structures, or at lightly loaded structures that will not develop the full capacity of a standard structure.

Foundations may be standardized by limiting the number to only one or two designs for each standard structure type, considering each to cover a preselected range of subsurface conditions and foundation loads. The extent of subsurface investigations can be reduced to a level necessary to identify the general subsurface conditions along the line. This approach enables the engineer to select an appropriate standard foundation. Verification of subsurface conditions at each structure site should be made during construction excavation. This approach allows for greater efficiencies in fabrication and assembly of foundation types, such as steel grillages. Using standard foundation designs will result in utilizing foundations having greater load-carrying capacity at some structure locations. Construction excavation may reveal locations that require site-specific foundations because the actual subsurface conditions are outside the limits of the preselected range. The benefits of standardization should be weighted against the cost of site-specific foundation designs and against the additional cost of redesigning the foundation when unusual subsurface conditions are encountered during construction.

The amount and extent of standardization will vary with each foundation type. Steel grillages that are entirely shop fabricated are almost always designed to cover the maximum loads for a given tower type and the majority of subsurface conditions expected along the line. An advantage of the grillage-type foundation is that concrete is not required at the site with the attendant transporting and curing requirements. In addition, grillages may be shipped to the site with the rest of the tower steel. A drilled shaft foundation can be varied to suit the actual soil conditions by providing different depths and/or diameters. Usually, the only change to prefabricated materials, required to modify drilled shaft foundations, is the length or quantity of steel reinforcing bars, and this can usually be readily accomplished at a small additional cost. Likewise, many types of pile foundations can be adapted to actual site conditions by providing standard foundations with various numbers of driven piles of varying lengths, as required.

1.3 Other considerations

Whereas this design guide is primarily aimed at the design of new foundations, the principles are applicable to the investigation of the geotechnical capacity of existing foundations for purposes of determining line capacity or for upgrading or refurbishing the line. If the foundations are upgraded to meet new loading requirements, care must be taken to assure the structural adequacy of the foundation. The investigation and design for restoration of a line after natural or man-made disasters must adhere to the same careful principles of investigation and design as a new line.

Documentation of the design and “as-built” construction data of foundations is vital, particularly if a line is to be refurbished or upgraded at a later date.

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2. Loading and performance criteria

2.1 Loading

Each utility normally has a unique agenda of loading cases for the design of transmission line systems. Based on this information, the engineer should analyze the structural system and calculate appropriate combinations of axial, shear, and moment loads acting on every foundation for each loading case. For a given structure type, different load cases may control foundation design depending on line angles and other design factors.

Foundation design methods must be compatible with the foundation type and loading conditions. Similarly, the subsurface exploration program must be compatible with these factors to provide the required geotechnical design data.

The foundation designer should consider the following sources for the determination of foundation loads:

a) Legislated Loads
b) ASCE Guidelines for Electrical Transmission Line Structural Loading [B68]
c) State-Specific Loading Criteria (e.g., California General Order 95)
d) Utility-Specific Loading Criteria

Legislated loads provide minimum structural loading criteria for the design of transmission lines. An example of legislated loads is the National Electric Safety Code (NESC) [B117], which is a legislated code in many U.S. states.

The American Society of Civil Engineers' Committee on Electrical Transmission Structures has published a guide [B68] that provides transmission line designers with procedures for the selection of design loads and load factors. A load resistance factor design (LRFD) format is presented for the development of attachment point loads for the design of any transmission structure. The same design loads and load factors apply to structures made of steel, reinforced concrete, wood, or other materials, as well as to their foundations, with only the resistance factors differing.

Based on specific service area requirements and experience, many utilities have developed their own structural loading agenda. The structural loading agenda may include legislated loads, ASCE, and utility-specific loading criteria.

The foundation design engineer should establish the strength of the foundation relative to the strength of the structure it supports. A foundation could be designed to be stronger than the structure; thus, in the event the structure fails, its replacement can be erected on the same foundation. A foundation could be designed to have the same strength as the structure it supports, thus, developing the full capacity of the structure while minimizing foundation first-cost expenditures. In some cases, the foundation engineer may find that the foundation could be designed to carry loads that are less than the capacity of the structure (where a standard structure is used at less than its design load capacity). In this case, the designer should recognize the probability of a foundation failure if the structure is ever subjected to a load greater than the load required by the structure application. An analysis weighing all values and probabilities should be made to determine the foundation that meets requirements and provides economy.

It is generally recommended that loading cases be separated into steady-state, transient, construction, and maintenance loads. These loading cases are considered separately in the following discussion.

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2.1.1 Steady-state loads

Steady-state loads are those loads imposed on a structure for a long or continuous time period. Examples of these types of loads are

- Vertical loads due to the dead weight of the structure, bare weight of conductors and shield wires, insulators, and any hardware, such as suspension clamps and dampers
- Loads due to horizontal or vertical angles in the line
- Differential line tension
- Termination of the line (dead ends)

2.1.2 Transient loads

Transient loads are those loads imposed on a structure for a short time duration. Examples of these types of loads are

- Wind loads on bare or ice-covered conductors, shield wires, structure, insulators, and hardware
- Extreme event loads (including broken wire, hardware failure, loss of structure, etc.)
- Stringing loads due to conductor hanging-up in the stringing block during wire installation, where no work crews are endangered
- Ice loads (including ice shedding and galloping)

2.1.3 Construction loads

Construction loads are those loads imposed during the erection of the structure and wire installation. Examples of these types of loads are

- Horizontal shear loads on a foundation used in tilt-up construction of the structure
- Temporary terminal loads that occur during wire installation
- Wire installation load where work crews are endangered

It is anticipated that construction loads will have a higher load factor than transient loads. Thus, wire installation loads, which endanger work crews, are grouped under construction loads, whereas wire installation loads, which do not endanger work crews, are grouped under transient loads.

2.1.4 Maintenance loads

Maintenance loads are those loads that are a result of line maintenance activities (insulator replacement, etc.).

2.1.5 Design loads

The design loads are the combination of loading conditions used to design the foundations. The time duration that a load is applied to a foundation may often be taken advantage of to reduce foundation costs. An example of this is a foundation in a cohesive soil that can resist design loads for a short duration of time without experiencing significant movements; but when the design loads are applied over the service life of the structure, they will result in excessive displacements. In this situation, the foundation should be designed to resist the maximum combined loading condition; however, displacement could be based on steady-state loads only.

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In summation, a foundation should be designed to resist the maximum combined design loads acting on it. On the other hand, displacements could be estimated using steady-state loads in the case of foundations constructed in cohesive soils or using the maximum combined design loads in the case of granular soils. Design loads may be steady-state, transient, construction, and maintenance loads. Variations in subsurface conditions from one structure location to another, subsurface variations between foundations of the same structure, uncertainties of the foundation analysis, and foundation construction procedures are additional factors that must be considered in each individual foundation design.

2.2 Foundation performance criteria and structure types

The establishment of performance criteria for the design of safe and economical foundations is essential. In establishing performance criteria, the definition of foundation failure and damage limits should be thoroughly understood by the foundation designer and the structure designer. Foundation failure limit performance criteria are the failure capacity of the foundation and/or the magnitude of displacement (differential and total) at which failure of the structure is imminent. The damage limit performance criteria are the load capacity of the foundation or the displacement (differential and total) that would damage but not fail the structure. Differential settlements may result in foundation elevation differences that cause warping of the structure, inducing unanticipated loads in the structural members and creating difficulties in tower erection. Unfortunately, little work has been done to quantify the levels of failure and damage limit displacements for lattice and H-frame type structures. However, it is known that the amount of allowable total and differential displacement is dependent on the type of structure.

2.2.1 Lattice towers

Lattice tower foundation loads consist of vertical forces (uplift or compression) combined with horizontal shear forces. For tangent and small line angle towers, the vertical loads on a foundation may be either uplift or compression. For terminal and large line angle towers, the foundations on one side may always be loaded in uplift while the foundations on the other side may always be loaded in compression. The distribution of horizontal forces between the foundations of a lattice tower vary with the bracing and geometry of the structure. Care should be taken to include the transverse and the longitudinal load components of all tower members connected to the foundations. A free-body diagram for lattice tower foundation loads is shown in Figure 1.

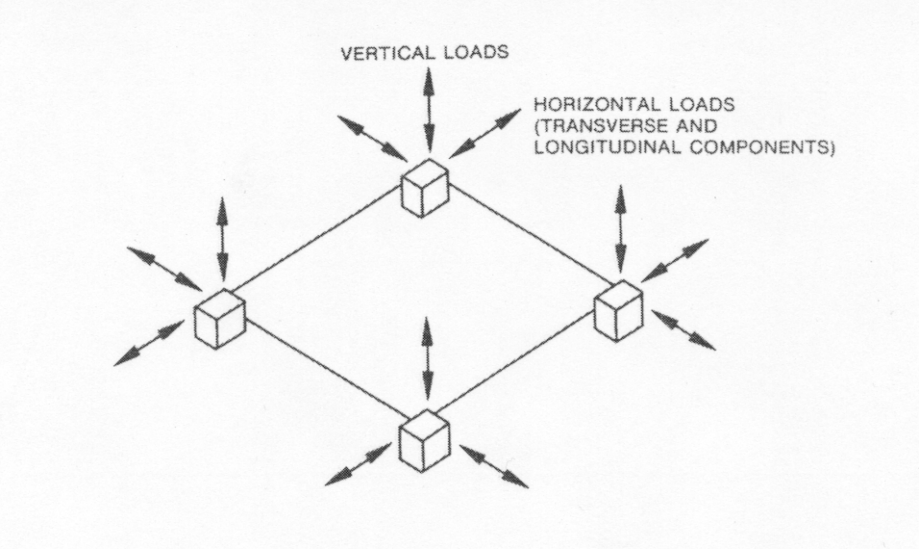


Figure 1—Typical loads acting on lattice tower foundations

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When the foundations of a tower displace and the geometric relationship of the four tower foundations remains the same, any increase in load due to this displacement will have a minimal effect on the tower and its foundations. However, foundation movements that change the geometric relationship between the tower's four foundations will redistribute the loads in the tower members and foundations. This will usually cause greater reactions on the foundation that moves less relative to the other tower foundations, which in turn will tend to equalize this differential displacement.

At the present time, the effects of differential foundation movements are normally not included in tower design. Several options are available should the engineer decide to consider differential foundation displacements in the tower design. These options include designing the foundations to satisfy performance criteria that will not cause significant secondary loads on the tower, or designing the tower to withstand specified differential foundation movements.

2.2.2 Single pole structures

Single pole structures can be made of tubular steel, wood, or concrete. These structures have one foundation so that differential foundation movement is precluded. The foundation reactions consist of a large overturning moment and usually relatively small horizontal, vertical, and torsional loads. A free-body diagram for a free-standing single shaft structure is shown in Figure 2.

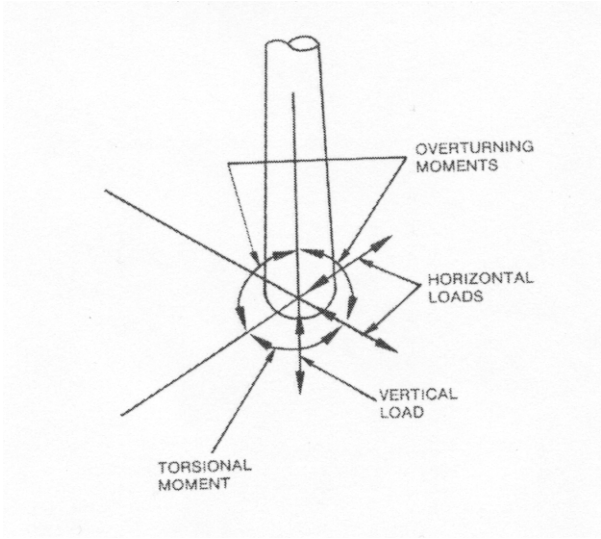


Figure 2—Typical loads acting on foundations for single shaft structures

For single shaft structures, the foundation movement of concern is the angular rotation and horizontal displacement of the top of the foundation. When these displacements and rotations have been determined and combined with the deflections of the structure, the resultant displacement of the conductor support can be computed. Under high wind loading, a corresponding deflection of the conductors perpendicular to the transmission line can be permitted if electrical clearances are not violated. Accordingly, under infrequent temporary loads, larger ground line displacements and rotations of the foundation could also be permitted.

In establishing performance criteria for single-shaft structure foundations, consideration should be given to how much total, as well as permanent, displacement and rotation can be permitted. In some cases, large permanent displacements and rotations might be aesthetically unacceptable and replumbing of the structures and/or their foundations may be required. In establishing performance criteria, the cost of replumbing should be compared with the cost of a foundation that is more resistant to displacement and rotation.

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For terminal and large line angle structures, large foundation deflections parallel to the conductor probably are not tolerable. For these structures, the deflection may excessively reduce the conductor-to-ground clearance or increase the loads on adjacent structures. There are also problems in the stringing and sagging of conductors if the deflections are excessive. These problems are usually resolved by construction methods or use of permanent guys.

2.2.3 H-frame structures

The foundation loads for H-frame structures are dependent on the structural configuration and the relative stiffness of the members. Although foundation reactions for moment-resisting H-frames are statically indeterminate, they can be approximated by making assumptions that result in a statically determinate structure. Also, the statically indeterminate structures can be analyzed using any of the classic long-hand analysis methods or by using computer programs. Figure 3 shows a free-body diagram of the foundation loads for an H-frame structure.

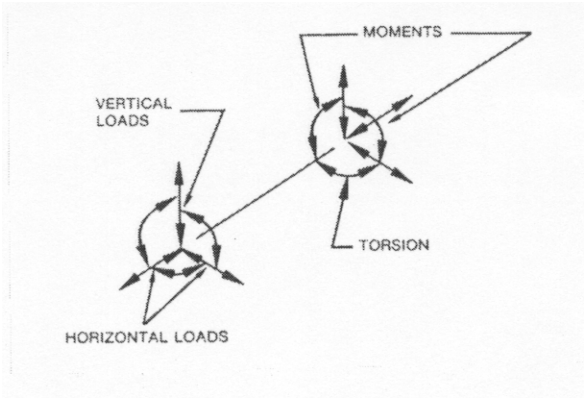


Figure 3—Typical loads acting on foundations for H-frame structures

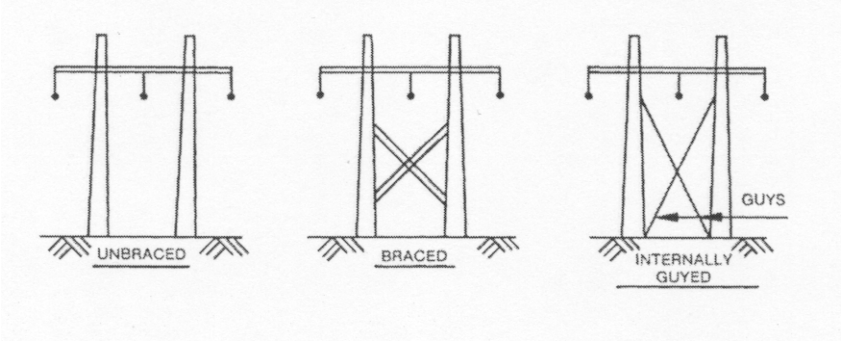


Figure 4—Typical H-frame structures

Many different types of two-legged H-framed structures are in use in transmission lines. This has been particularly true in recent years because visual impact has become of greater concern.

The H-frame structure is particularly applicable for wood, tubular steel, or concrete poles. The cross arm may be pin-connected to the poles, in which case an unbraced structure behaves essentially as two single poles connected by the cross arm. These structures may be unbraced, braced, or internally guyed, as shown in Figure 4.

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As with lattice towers, past practice has not normally included the influence of foundation displacement and rotation in H-frame structure design. Significant foundation movements will redistribute the frame and foundation loads. The foundations can be designed to experience movements that will not produce significant secondary stresses in the structure, or the structure can be designed for a predetermined maximum allowable total and differential displacement and rotation.

2.2.4 Externally guyed structures

Three general types of externally guyed structures exist [B49]. For all types, the guys produce uplift loads on the guy foundations and compression loads on the structure foundation. The guys are generally adjustable in length to permit plumbing of the structure during construction and to account for creep in the guy and movement of the uplift anchor.

Several types of externally guyed structures are shown in Figure 5. The guys are located out-of-plane, both ahead and in back of the structure. In this case, the shaft or shafts of the structures usually have a ball-and-socket base connection to the foundation to permit free rotation without transmitting moment to the foundation. This will produce compression loading with a small shear load.

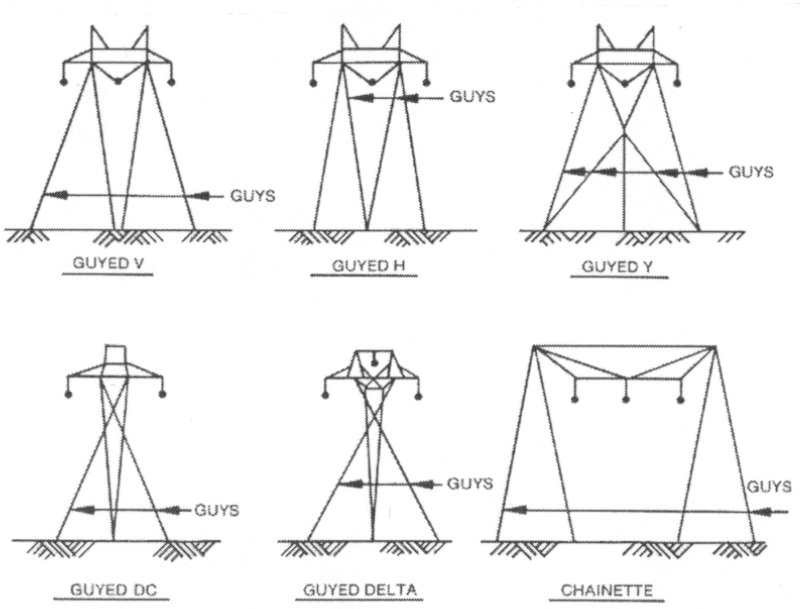


Figure 5—Typical externally guyed structures

This type of guyed structure can generally tolerate large foundation movements if guy stability is maintained. Consideration in establishing performance criteria are similar to those discussed in 2.2.2 for single pole structures.

A single-pole type externally guyed structure is shown in Figure 6. This type of structure is often used as a terminal and large line angle structure and is quite flexible, allowing most of the load to be resisted by tension in the guys and compression in the main shaft.

This type of guyed structure can generally tolerate significant foundation movement as far as its structural integrity is concerned; but like the terminal and large line angle poles discussed in 2.2.2, if excessive guy anchor slippage occurs, conductor-to-ground clearance, security of adjacent structures, and stringing and sagging conductors can become problems.

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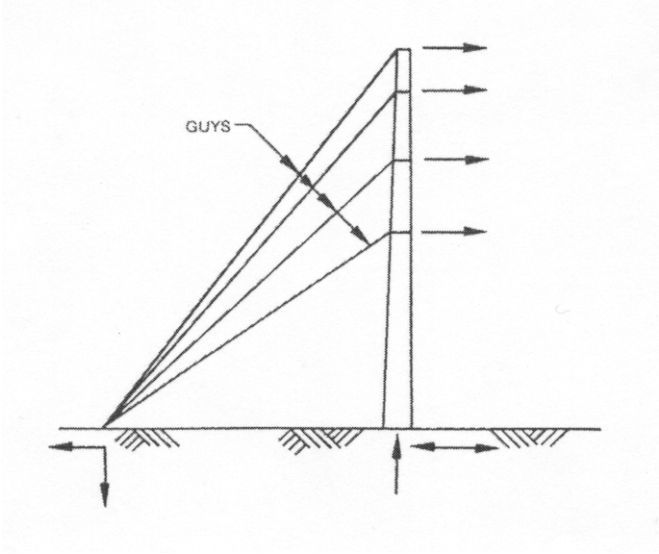


Figure 6—Single-pole, externally guyed structures

Another type of externally guyed structure is a conventional lattice tower guyed to reduce its leg loads and foundation reactions. This approach, which has often been used to upgrade existing towers, can lead to problems, as the relative distribution of the loads between the guys and the tower depend on the guy pretensions and the potential creep of the foundation. The flexibility of the guy, together with the flexibility of the tower, are needed to compute the foundation reactions and anchor loads. The maximum amount of anchor slippage can be selected, and the tower and anchors designed accordingly. The initial and final modulus of elasticity of the guys, together with the creep of the guys, should be considered. The amount of pretension in the guys should be specified and guys prestressed. Load testing of the guy anchors is recommended to ensure against excessive slippage. Figure 7 shows a typical installation.

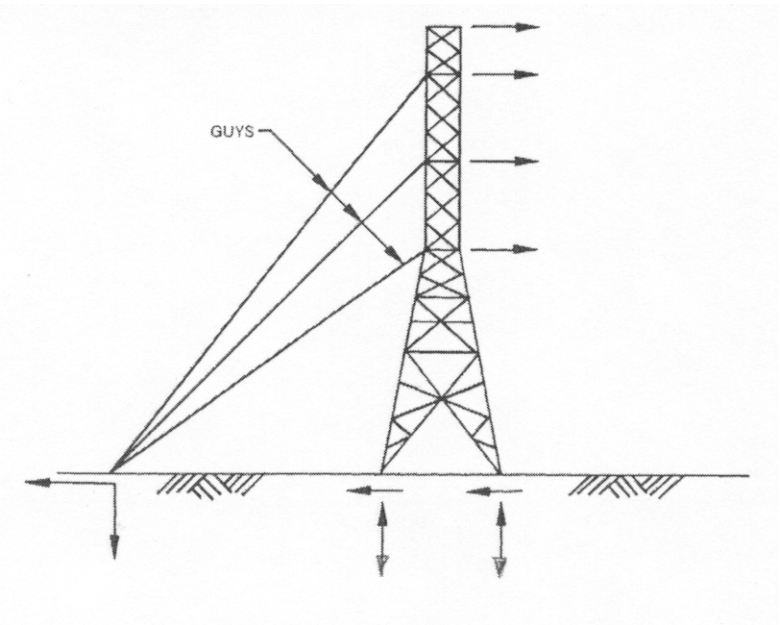


Figure 7—Externally guyed lattice tower

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The guyed-lattice tower leg foundations are required to resist horizontal shear forces and vertical compression or uplift loads. As in the case of the lattice towers, discussed in 2.2.1, the load distribution in the component structural elements is sensitive to the foundation performance. Differential displacements of the legs of the tower will result in load redistribution and may affect the integrity of the tower.

3. Subsurface investigation and selection of geotechnical design parameters

3.1 General

Subsurface investigation for electrical transmission tower foundation should be carried out along the right-of-way (r/w) of the transmission line to obtain geotechnical parameters required to successfully design the transmission structure foundations at a minimum cost.

As a minimum, the investigation should provide geotechnical parameters required to establish the ultimate load-bearing capacity of the subsurface material, and to determine the allowable movement of the foundation.

3.2 Phases of investigation

The investigation consists of the following three phases:

— Preliminary investigation to establish feasibilities
— Detailed investigation to finalize designs and details
— Design verification during construction and documentation

3.2.1 Preliminary investigation

The preliminary investigation should consist of collecting existing data relating to local and subsurface conditions, and of making a geotechnical field reconnaissance of the line route. If considered cost-effective, preliminary boring, penetration, and pressuremeter tests can be added to verify and increase the confidence level in existing data and finalize the reconnaissance mapping.

3.2.1.1 Existing data

A considerable amount of data regarding local geology, including distribution of surface water, depth of groundwater, depth and physical characteristics of bedrock, and type and thickness of soil cover, is available from several sources.

Topographic maps and aerial photographs, available from the various U.S. Geological Survey offices and commercial aerial surveying firms, typically provide data on the distribution of surface and ground waters, soil conditions and rock types, the areas of exposed bedrock, and the geomorphologic landform. They also show the location of man-made features such as radio towers, quarries, highways, other transmission lines, and building constructions. Often, due to proximity, useful information along the proposed r/w may be obtained on the foundation conditions by simple extrapolation of the available data.

Other excellent sources of information on the soil distribution and rock types are state and federal geological surveys and the geology departments of nearby universities.

The other potential sources of information are the Natural Resources Conservation Service (NRCS) of the U.S. Department of Agriculture, the U.S. Bureau of Public Roads, and county or regional planning boards. More information concerning sources of geological data may be found in [B165].

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3.2.1.2 Field reconnaissance

Another useful means of obtaining information during the preliminary investigation is to perform a field reconnaissance survey of the transmission line route. The reconnaissance should be performed by a geotechnical engineer or an engineering geologist. The purpose of the reconnaissance is to develop a map of the surficial soils showing areas that may offer particular foundation problems such as deep peat or soft organic silt, bedrock outcrops, areas of high groundwater table, and areas of potential slope instability. The soil and rock classifications used in the mapping should be based on engineering properties, not on geological or agricultural distinctions. By comparing the information from the field reconnaissance and existing published information, a preliminary line route map showing basic soil or rock types, inferred depth to bedrock, and elevation of the groundwater table can be developed.

3.2.1.3 Preliminary borings

The development of a surficial map with adequate subsurface interpretation usually is the final step in the preliminary investigation. To achieve such objective, it may be cost-effective to obtain a few preliminary borings in those areas where subsurface interpretation is difficult and where it may affect the foundation design significantly.

Preliminary borings are generally used for soil classification purposes only and disturbed samples are thus satisfactory. The most common methods of obtaining disturbed samples are auger boring and using a heavy walled split-barrel sampler which is driven into the soil at selected intervals in the boring. Test pits and probes can also be used.

When the boring has been advanced to the required depth, the sample is taken by driving the split-barrel sampler into the soil. This Standard Penetration Test (SPT) is covered in ASTM D1586 [B14]. Samples usually are taken at intervals of not more than 1.5 m (5 ft) in depth, and at every change in stratification where such change can be detected by the driller. Closer sampling intervals may be necessary if the soil stratification is complex or thinly stratified. When the scope of the investigation requires that borings be made, it is important to have a knowledgeable person with experience in geotechnical engineering present to ensure correct interpretation of the data obtained from the boring program. Dutch cone tests [B16] or pressuremeter tests [B19] may be used in lieu of the standard penetration tests to determine the in-situ stress, deformability and strength.

Since ground water affects many elements of foundation design and construction, its location should be established as accurately as possible. It is generally determined by measuring to the water level in the borehole after a suitable time lapse. A period of 24 hr is a typical time interval. However, in clays and other soils of low permeability, it may require several days to weeks to determine a meaningful water level. Standpipes or other perforated casings may be used to prevent the borehole from caving during this period.

3.2.2 Design investigation

The purpose of the design investigation is to provide the foundation engineer with sufficient subsurface information to

- Select the types of foundation most suitable at each structure location
- Determine the size and depth of the selected foundations to adequately support the power transmission along the line
- Evaluate potential problems during construction

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The information required to achieve these goals includes:

— Type of structure and allowable foundation movements
— Magnitude and duration of structure loadings at the ground line
— Stratigraphy of the subsurface materials
— Elevation of the ground water table
— Engineering properties of the subsurface materials

On any transmission line route these five factors may vary considerably, and the detailed investigation should provide the required information in a cost-effective manner. Ideally, a detailed subsurface investigation would involve boring at each structure site. However, this may not be necessary if the results of the preliminary investigation have shown that subsurface conditions in a specific section of the line route are reasonably uniform.

Indirect methods of subsurface investigation include geophysical exploration techniques such as seismic refraction, electrical resistivity, and gravimetric surveys. These methods generally are used to survey large areas. While not well suited to investigate the small area at each structure location, they may be helpful as supplemental data between boring locations. These indirect methods only assist in defining general stratigraphy.

The designer should be aware of the opportunity to save substantial project cost, since there may be a large number of foundation designs. The saving in cost due to failure to administer adequate subsurface investigation must be weighed, however, against the cost of the risks involved.

Coincident with selecting the locations for the subsurface investigations, decisions should be made concerning the type and depth of exploration. The type of exploration is mainly a function of soil types expected at a given site and the type of foundation being considered for the site. For example, if the structure is located where the expected subsurface material is sand, a boring that obtains disturbed samples and records the standard penetration test results will usually be adequate. On the other hand, the same foundation type located in clay may require a boring that will allow undisturbed samples to be obtained.

Guidance for determining the most satisfactory boring may be obtained from considering the following question: Can the foundation be designed in a cost effective manner from empirical correlations between classification tests and engineering properties of the soil or rock? If so, then boring to obtain disturbed samples with standard or Dutch cone penetration test will be sufficient.

If a cost effective design can be determined only by accurate knowledge of the engineering properties, then undisturbed sample borings must be made, and laboratory or *in situ* tests conducted to determine the required engineering properties. Empirical relationships between engineering properties and classification tests performed on disturbed soil samples can be developed for a specific project. On large projects, this correlation can result in a reduction in boring costs by reducing the number of undisturbed sample borings and engineering property measurements.

The depth of each exploration should extend through any unsuitable or questionable foundation materials, and to a depth sufficient so that imposed stresses below that depth (due to foundation loads) will not result in adverse performance for the types of foundations being considered. As a general guide, unless bedrock is encountered first, explorations should be made to a depth at which the net increase in soil stress from the maximum design load is 10% of the *in situ* vertical effective stress in the soil at the depth. For spread foundations, this translates into depths which are 2.0 to 2.5 times the equivalent diameter of the foundation. The net increase in stress may be computed from the Boussinesq and Mindlin equations [B113]. Poulos [B129] and Westergaard provided various stress distribution charts. Further discussion regarding the depth of the subsurface investigation may be found in [B146] and [B151].

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3.2.3 Construction verification

The owner should have representation in the field during foundation construction to determine if the actual subsurface conditions are similar to those conditions used in the foundation design. If the subsurface conditions used in the foundation design differ *significantly* from the actual conditions, it may be necessary to enlarge the foundation or change the foundation type.

3.3 Types of boring samples

The purpose of making a boring is to obtain samples of the subsurface materials for visual description, classification, and testing to determine design parameters. Each sample should be visually examined preferrably in the field by a geotechnical engineer or an engineering geologist and the appropriate manual tests performed to allow the soil to be classified according to the Unified Soil Classification System [B37]. ASTM D2488 [B15] may be used for routine field classification.

In making borings, the hole is advanced by drilling with a bit to cut away the soil and circulating drilling fluid through the bit to carry away the cuttings, or the hole is advanced by an auger. Augers, either conventional or hollow-stem type, should be used with caution when sampling below the groundwater table. Upward seepage of water in pervious soils (or even in many silts) may disturb and loosen the soil to such an extent that penetration tests will indicate erroneously low blow-counts and increase the moisture contents of the soil. It is essential that at all times the water level in the borehole be kept above the groundwater table. In granular soils, even above the water table, loading of the soil by the blades of a hollow stem auger may cause higher blow counts in the penetration test than would be measured in other types of boring.

Three kinds of samples can be taken by boring operations: disturbed soil, undisturbed soil, and rock core. The foundation designer should be familiar with the detailed means of subsurface exploration and sampling methods described in [B80].

3.3.1 Disturbed soil samples

Thick-walled samplers may be used for obtaining samples suitable for identification and index property tests. The barrels of such samplers may be solid tubes of the split-barrel type that facilitates removal and examination of samples. Samplers of this type range in diameter from 5 cm to 11 cm (2 in to 4.5 in). They may be used to recover samples in many soils, although there may be difficulties with coarse gravel or rock fragments unless the sampler is equipped with a flap valve or basket retainer. The equipment and procedures for making Standard Penetration Tests (SPT), determining the standard penetration resistance (N), and obtaining split-barrel samples are covered in ASTM D1586 [B14].

The SPT resistance should not be used for estimating the strength and compressibility of cohesive soils (clays). The strength and compressibility of cohesive soils are greatly influenced by their soil structure (particle arrangement) which is a function of mode of deposition, mineralogy, and stress history. Since first described by Casagrande [B36], the importance of the structure of clay has been well documented. The vast majority of clays are sensitive, since their strength is reduced and their compressibility increased when their structure is disturbed. The act of driving a thick-walled sampler, used to measure the SPT resistance, disturbs the clay sufficiently so that this technique is unsuitable for estimating the engineering properties of clays.

The strength and compressibility of cohesionless soils (sands and gravel) usually are not greatly influenced by soil structure, and these soils typically are insensitive. Their strength and compressibility are mainly a function of grain size and density (degree of compactness). Therefore, the SPT resistance can be used to estimate the adequacy of cohesionless soils for supporting the loading associated with transmission tower foundations. In addition to their insensitivity, a second important reason for the applicability of the SPT to cohesionless soils is that these soils are relatively incompressible and have high shear strength; except in unusual cases, the loads imposed by transmission structure foundations will not cause large deformations.

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Having stated that the SPT is a useful classification test for cohesionless soils, it is necessary to point out one important exception. The designer must be aware of the special case of cohesionless silts (*they* do not have dry strength). Because of their small particle size, the behavior of silts is influenced by particle arrangement or structure. The strength and compressibility of silts cannot be evaluated from standard penetration tests. Silts should be treated similarly to clays and undisturbed samples should be obtained to permit measurement of strength and compressibility.

A number of the additional factors affecting the results of the SPT have been discussed in the literature. For potential errors inherent in this exploration procedure, see [B48], [B99], [B123]. For example, minor amounts of gravel exceeding 6.35 mm (0.25 in) in size may affect the SPT results. Because of its sensitivity to gravel, the test is not dependable in coarse-grained soils including medium to course gravel.

Customary practice is to take samples at intervals of approximately 1.5 m (5 ft). With the standard sampler, about 45.7 cm (18 in) of soil are usually recovered, which results in about 30% of the soil column being available for examination. This is usually sufficient, although closer spacing of sampling should be used if soils vary markedly with depth. In soil masses where the individual strata are relatively thin, as is frequently the case in estuarine or fluvial deposits, intermittent sampling may give quite misleading results. In such deposits, continuous sampling should be done in a sufficient number of holes to define the stratigraphy more accurately.

At least 15 cm (6 in) of each sample should be sealed in an airtight container and sent to the soils laboratory for further classification and testing. Dependence on a driller for field classification of soils is not good practice, because drillers rarely have the requisite technical training to adequately classify soils.

3.3.2 Undisturbed soil samples

Equipment and procedures for obtaining undisturbed samples of soils of a quality suitable for quantitative testing of strength and deformation characteristics have been given in [B80]. Briefly, taking undisturbed samples requires using a thin-walled sampler with proper clearance at the cutting edge. The sampler must be forced into the soil smoothly and continuously.

To permit taking undisturbed samples in dense soils or soils containing gravel or other hard particles that tend to deform a conventional thin-walled sampler, samplers such as the Denison or Pitcher have been developed in which a thin-walled, nonrotating inner sampler barrel is forced into the soil mass, while the soil surrounding the barrel is removed by a rotating, carbide-toothed outer barrel. Good quality samples in difficult soils can usually be obtained with such equipment.

In most soils of soft to stiff consistency, samples of a quality suitable for quantitative testing can be obtained using thin-walled Shelby tube samplers a minimum of 5 cm (2 in) diameter, providing there is a proper cutting edge [B80]. Normally, the tube is pushed into the soil for a distance of about 15 cm to 20 cm (6 in to 8 in) less than the length of the tube. Preferably the sampler should be pushed downward in one continuous movement. After the sampler has been forced down, the drill rods are rotated to shear the end of the sample and the sample is removed. Friction between the sample and the tube retains the sample as the sampler is withdrawn. A special valve or piston arrangement also may be attached to create a pressure differential (suction) to aid in retaining the sample. To reduce deficiencies with respect to sample length and sample disturbance due to side friction between the sample and the walls of the sampler (while the sampler is being advanced into the soil), various piston and foil samplers have also been developed. These are described in more detail by Hvorslev [B80] and may be used to obtain undisturbed samples in soft soils or soils in which recovery is difficult using a conventional Shelby tube sampler.

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3.3.3 Rock coring

Where investigation of the bedrock is necessary, pertinent data to be obtained include:

- Elevation of the rock surface and variation over the site
- Rock type and hardness
- Permeability
- Extent and character of weathering (including alteration of mineral constituents)
- Extent and distribution of solution channels in soluble rocks such as limestones
- Discontinuities such as bedding planes, faults, and joints
- Foliation or cleavage

Identification and classification of rock types for engineering purposes may be limited to broad, basic classes in accordance with accepted geological standards.

The behavior of rock subjected to foundation loadings is a function of the deformation characteristics of the rock mass which are controlled by rock discontinuities such as weathering, joints, and bedding planes. Locating and evaluating the effects of such discontinuities requires carefully planned and executed investigations made by experienced, well-equipped drillers under the guidance of a competent specialist in the field.

Other significant factors affecting the behavior of rock as a foundation material include weathering and hardness. There are no generally accepted criteria for these, although the Rock Quality Designation (RQD) suggested by Deere [B47] is useful. The RQD is defined as the modified core recovery percentage in which all pieces of sound core over four inches in length are counted as recovered. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. The RQD may be used for core boring as an indication of the effects of weathering aid discontinuities. It should be noted that if RQD is to be determined, double-tube NX size core barrels with nonrotating inner barrels that produce approximate 5 cm (2 in) ϕ diameter core must be used.

The drillers should proceed with maximum care for maximum possible recovery. Drillers should also pull the core whenever they feel a blockage, grinding, or other indication of poor core recovery. The material that is not recovered is frequently the most significant in deciding upon proper design. The time required to drill each foot, total recovery, physical condition, length of pieces of core, joints, weathering, evidence of disturbance, or other effects should be noted on the drilling log.

Any comments by the driller with regard to the character of the drilling and difficulties encountered should be included. Where massive rocks such as unweathered granite are encountered, good recoveries may be obtained with smaller diameter drills, such as BX and AX sizes. Stepping down to these smaller sizes may be necessary when in bouldery areas of deep weathering.

3.4 Soil and rock classification

Classification of soil and rock samples by visual description and simple manual tests is an important aspect of a subsurface investigation program. The written visual description is the first means of conveying to the engineer the types of subsurface materials along the r/w. This information will be used to determine the parameters selected for designing the foundation.

Based on the visual classification of the soils or rocks, a series of index property tests are performed that further aid in classification of the materials into categories and permit the engineer to decide what field or laboratory tests, if any, will best describe the engineering properties of the soil and rock on a given project.

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3.4.1 Soil classification

3.4.1.1 Index properties

Soil classification by index properties (that is, classifying them into broad groups having similar engineering properties) is used primarily to qualitatively describe the soil. Engineering properties (strength, compressibility, and permeability) are usually expensive and time-consuming to determine, especially since they must be measured either *in situ* or from undisturbed samples which are tested in the laboratory. It is impractical and uneconomical to try to measure the engineering properties everywhere throughout a large mass of soil.

Index properties can be measured more economically and quickly than engineering properties. With some exceptions, they can be measured on disturbed samples which can be obtained with less difficulty and expense than undisturbed samples.

Index properties are useful because they can be roughly correlated with the engineering properties. From his knowledge of the empirical correlation between the index properties and engineering properties of soils or rock, the designer can make use of the index properties for the following purposes:

— To select sites that have the most favorable subsoil conditions for a given transmission line
— To make a preliminary estimate of the engineering properties of the soil at a given site
— To select the most critical zones in the subsoils for more extensive investigation of the engineering properties

Useful index properties for cohesionless and cohesive soils are summarized below:

(Cohesionless)	(Cohesive)
Grain size	Water content
Specific gravity	Degree of saturation
Relative density	Atterberg limits
Unit weight	Specific gravity
Degree of saturation	Void ratio
Standard penetration resistance	Undrained strength
Cone penetration test	—

The undrained strength of cohesive soils referred to in this context is the strength measured in the field by means of a pocket penetrometer or vane shear device [B85]. These measurements are made on both undisturbed samples at each end of a tube sample and disturbed samples from a standard penetration test. The measurements, which are quickly and easily performed when combined with the water content and Atterberg limits, provide an excellent means for classifying cohesive soils and selecting specific samples on which engineering property measurements can be made.

The standard penetration resistance is one of the most commonly used index properties for cohesionless soils. A number of empirical relationships between SPT and the compressibility and shear strength of sands have been developed. It should be emphasized that the standard penetration test is an index test and that care must be emphasized when using only the SPT as the basis of a foundation design. The SPT is not listed as an index property test in cohesive soils, since its application to the classification of cohesive soils is subject to serious question, as discussed previously.

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3.4.1.2 Visual classification

Soil classification, like the index properties, is used to convey qualitative information about the engineering properties. Of the many soil classification systems in use by engineers, geologists, and pedologists, the Unified Soil Classification System [B37] is best suited for conveying significant information about the engineering properties of soils.

Soils are divided into three broad categories in the Unified Soil Classification System: Coarse-grained, fine-grained, and highly organic. A whole spectrum of soil types, overlapping two or all three of these broad categories, can be found in nature. Subdivisions within the broad categories make it possible to classify these more complex soil types.

3.4.2 Rock classification

Generally, the engineering properties of a rock mass cannot be predicted with the precision expected in a soil investigation. Although there are many field and laboratory tests available, there are no widely accepted index properties that correlate with the engineering properties of the rock mass.

As mentioned in 3.3.3, the engineering properties of a rock mass are largely a function of the number, type, spacing, and orientation of rock defects such as

— Joints
— Weathering
— Faults
— Bedding Planes
— Shear Zones
— Foliation
— Solution Channels

The geotechnical engineer or geologist should provide a lithologic description of the rock core, including the geologic name given to the rock type on the basis of its mineralogical composition, texture, and in some cases, its origin. Such names as granite, basalt, sandstone, shale, etc., evolve from such schemes and are generally understood by the foundation design engineer.

In addition to textural description, a generalized description of rock hardness should be included in the rock description. As mentioned previously, even a soft rock generally will have adequate engineering properties to support transmission structure foundations. However, as an aid in describing the rock core, the relative terms soft, medium, or hard should be used to describe rock hardness.

In addition to the lithologic and textural description, additional rock drilling information should be obtained during the coring operation. This information includes

— Rate of drilling with emphasis on the unusual
— Water losses
— Groundwater level
— Core recovery

An index used to evaluate the rock mass in terms of its discontinuities is the RQD; see 3.3.3. An RQD approaching 100% denotes an excellent quality rock mass with properties similar to that of an intact specimen. RQD values ranging from 0 to 50% are indicative of a poor quality rock mass having a small fraction of the strength and stiffness measured for an intact specimen.

Problems arise in the use of core fracture frequencies and RQD for determining the in situ rock mass quality. The RQD and fracture frequency evaluate fractures in the core caused by the drilling process, as well as natural fractures previously existing in the rock mass. For example, when the core hole penetrates a fault zone

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or a joint, additional breaks may form that, although not natural fractures, are caused by the natural planes of weakness existing in the rock mass. These breaks should be included in the estimated rock quality. However, some fresh breaks occur during drilling and handling of the core that are not related to the quality of the rock mass. In certain instances, it may be advisable to include all fractures when estimating RQD and fracture frequency. Considerable judgement is involved in the logging of rock core samples.

3.5 Engineering properties

To design foundations for transmission structures or evaluate the foundation performance under the loads applied to the structure, it is necessary that certain geotechnical engineering properties be determined or estimated. The performance of a transmission structure foundation and the dimensions and type of foundation required is governed primarily by the shear strength and compressibility of the supporting soil. Estimated values for the engineering properties required to compute ultimate capacity (for example, bearing, lateral, uplift) or settlement of the foundation may often be obtained from correlations with various index properties of the soil in which the foundation is constructed. Laboratory test procedures are available to measure the shear strength and compressibility of various soil samples [B97].

3.5.1 Index property correlations

Various engineering properties pertaining to the shear strength or compressibility characteristics of both cohesionless and cohesive soils may be estimated from appropriate index properties. While other correlations exist, several useful relationships between engineering properties and index properties are discussed below.

The shear strength of soils is normally expressed by the Mohr-Coulomb equation as:

$$s = c + \sigma_n \tan\phi \quad (1)$$

where

s is shear strength,
c is cohesion,
σ_n is normal stress,
ϕ is angle of internal friction.

In general, the shear strength of a soil determines the ultimate load carrying capacity of a foundation and, consequently, must be estimated to design or analyze potential foundations for transmission structures. The use of the engineering properties, c and ϕ, in determining the capacities of various foundation types will be shown in later sections of this guide.

In cohesionless soils (c = 0), the value of ø and, therefore, the shear strength may be related to the gradation, grain shape, and relative density of the soil mass, among other properties. The influence of grain shape and gradation on the magnitude of ø may be discussed qualitatively. As the angularity of the soil grains increases, the amount of particle interlocking increases. Well-graded soils (those containing roughly equal amounts of a wide range of grain sizes) usually have a lower void ratio since the voids between larger particles are partially filled with the smaller soil particles. Both of these factors result in increases in the value of the angle of internal friction, ϕ.

An approximate quantitative relationship exists between ϕ and the relative density of cohesionless soils, which may be determined from laboratory test procedures or estimated from standard penetration tests conducted during sampling operations in the field.

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The Atterberg limits are laboratory tests to determine the influence of moisture content on the consistency of cohesive soils. The liquid limit is defined as the water content at which transition from a plastic state to a liquid state occurs and the plastic limit is the moisture content at which the soil behavior changes from nonplastic to plastic state (test procedures to determine the Atterberg limits have been standardized and are discussed in any basic text on soil mechanics). The plasticity index (the numerical difference between the liquid limit and the plastic limit) provides a measure of the range of water contents over which the soil remains plastic.

Empirical correlations have been obtained which relate index properties to the compressibility of clay soils. For normally consolidated clays (clay soils that have not previously experienced consolidation pressures greater than the existing effective overburden pressure), the compression index, C_c, contained in the consolidation settlement equations presented in Clause 4 may be related to the liquid limit as:

$$C_c = 0.009(W_l - 10) \tag{2}$$

where

W_l is liquid limit in percent.

This discussion illustrates the usefulness of several index properties in estimating various engineering properties. Basic texts on soil mechanics and foundation analysis and design will provide other useful empirical relationships that have been developed to provide estimates of engineering properties required for the analysis and design of the various foundation types used to support transmission structures. The use of index properties to estimate engineering properties should be done with caution, and the engineer should be aware of how the relationships were developed and for what material. Whenever possible, correlation should be verified with appropriate laboratory testing. The empirical relationships should not be accepted as a substitute for laboratory tests to determine the engineering properties of soils along the route of the transmission line. They may, however, often be used to supplement or reduce the amount of laboratory tests conducted and may aid the engineer in selecting the areas along the route where more extensive investigation of engineering properties is required.

3.5.2 Laboratory testing

As mentioned previously, the performance and load carrying capacity of various types of foundations depend upon the shear strength and compressibility of the soil on which the foundation is constructed. Various laboratory tests have been developed to investigate these properties of soil. Brief descriptions outlining several useful laboratory tests are presented in this section to aid in the selection of appropriate tests to determine the engineering properties required in the analytical techniques presented in subsequent sections of this guide.

The shear strength of soils is dependent not only on soil type, but also on test method and loading or drainage conditions imposed during testing of a sample. The two test methods most commonly used to determine the shear strength of soils are the direct shear test and triaxial test.

The direct shear test is one of the earlier methods developed to determine the shear strength of various soils. The test consists of shearing a soil sample across a predetermined failure plane. The soil specimen is enclosed in a box consisting of an upper and lower half. The upper half is usually free to move vertically and can slide horizontally with respect to the lower half of the box. A horizontal force is applied to the upper half of the box either by controlling the loading rate or the rate at which the upper half of the box is displaced horizontally, and both the displacement and load applied to the box are monitored. A stress-displacement curve is obtained by plotting the shear stress versus shear displacement. Failure may be defined either at the peak stress (for dense sand or stiff clays) or at an arbitrary displacement value (for loose cohesionless soil or soft clays). At least three tests using different normal stresses (applied vertically to the top half of the box)

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are required to determine the Mohr-Coulomb failure envelope defined by Equation (1); see [B24], [B96] for detailed descriptions of laboratory test procedures.

The direct shear test is relatively simple and inexpensive to perform, but often has been criticized because the failure plane is predetermined. In addition, it is difficult to control sample volume and drainage conditions or to obtain pore pressure measurements during testing. Consequently, some uncertainty may exist with respect to the actual effective stresses existing in the sample during testing and at failure.

The triaxial test eliminates most of these difficulties. This test is conducted inside a cylindrical cell on cylindrical samples encased in rubber membranes. Hydrostatic confining pressure is applied to the sample by application of pressure to the fluid inside the cell. Shear stresses in the sample are usually controlled by applying an additional vertical stress (the deviator stress). Drainage from the sample may be controlled during application of both the confining pressure and deviator stress, and pore pressures generated in the sample during the test may be monitored. To obtain the Mohr-Coulomb failure envelope (and consequently, ϕ and c), several tests are performed using various confining pressures.

The shear strength parameters obtained from triaxial tests are dependent on the consolidation and drainage conditions imposed prior to and during application of the deviator stress. Three conditions under which these tests are conducted are described below:

a) *Unconsolidated-Undrained Test (UU Test)*. No drainage is allowed during application of the confining pressure or the deviator stress. The unconfined compression test is a special case of the unconsolidated-undrained test with confining pressure equal to zero. The deviator stress at failure is the unconfined compressive strength, q_u, which is equal to two times the undrained shear strength, S_u.

b) *Consolidated-Undrained Test (CU Test)*. Drainage is allowed during application of the confining stress. The sample is allowed to consolidate with respect to the applied pressure as observed via drainage measurements. No drainage is allowed during the application of the deviator stress.

c) *Consolidated-Drained Test (CD Test)*. Drainage takes place during the entire test. The deviator stress is applied slowly enough so that pore pressures do not build up during shearing of the specimen.

Detailed descriptions of equipment and test procedures are contained in [B24] and [B96].

For soils of low permeability (such as clays), the CD test may require long periods of time to conduct so that pore pressures will not be generated during shear; consequently, the test would be more expensive to conduct for this type of soil. The drained strength can be evaluated during the quicker CU test if pore water pressures are measured.

With cohesionless soils, which drain relatively freely both during testing and in situ, the CD test is appropriate and does not have the time restraints that are imposed when cohesive soils are tested. Table 1 provides representative values for the angle of internal friction, ø, for various soil types and triaxial test conditions.

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Table 1 Representative values for angle of internal friction ϕ

Soil	Type of test[a]		
	Unconsolidated-undrained UU	Consolidated-undrained CU	Consolidated-drained CD
Gravel			
Medium size	40°–55°		40°–55°
Sandy	35°–50°		35°–50°
Sand			
Loose dry	28°–34°		
Loose saturated	28°–34°		
Dense dry	35°–46°		43°–50°
Dense saturated	1°–2° less than dense dry		43°–50°
Silt or silty sand			
Loose	20°–22°		27°–30°
Dense	25°–30°		30°–35°
Clay	0° if saturated	3°–20°	20°–42°

NOTES:
1—Use larger values as unit weight, γ, increases.
2—Use larger values for more angular particles.
3—Use larger values for well-graded sand and gravel mixtures (GW, SW).
4—Average values for
Gravels: 35°–38°
Sands: 32°–34°

[a]See a laboratory manual on soil testing for a complete description of these tests, e. g., Bowles (1986b) .

For cohesive soils, the value of the cohesion term, c, in Equation (1) is dependent upon mineral content, triaxial test conditions, and previous (geological) stress history.

The engineering properties governing the compressibility of soils may also be determined from laboratory tests. In general, the settlement of a foundation in cohesionless soils is governed primarily by elastic/plastic compression and is normally computed using expressions derived from the theory of elasticity (see Clause 4). Settlement of foundations in cohesive soils may have both an immediate (elastic) component and a time-dependent consolidation component.

The analysis to estimate the elastic or immediate settlement component of settlement for both cohesionless and cohesive soils requires the determination or estimation of a stress-strain modulus (or modulus of elasticity) and frequently a value for Poisson's ratio. Various methods have been proposed for determining stress-strain moduli from both conventional and cyclic triaxial tests [B27].

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Engineering properties governing the consolidation settlement of cohesive soils (for example, clays) are normally determined from laboratory consolidation or oedometer tests. Consolidation of a soil may be defined as the time-dependent reduction in void ratio due to the application of an applied compressive stress, such as might be generated below the foundation of a transmission structure. The compressibility of a cohesive soil is dependent upon the stress history of the soil. If the effective vertical stress below a foundation is less than the maximum effective stress previously experienced by the soil, the settlement will be governed by the recompression index, C_r, determined from laboratory consolidation tests. The void-ratio effective-stress relationship for stress levels exceeding the past maximum effective stress is governed by the so-called virgin compression curve and the compression index, C_c. Detailed discussions of these parameters are presented in various texts on soil mechanics and foundation engineering [B97], [B27], [B123] and a description of test procedures and equipment [B96]. The use of the compression and recompression indexes in estimating consolidation settlement is demonstrated in 4.2.2.2.

The consolidation test and shear strength tests described above are normally conducted on undisturbed samples obtained during the subsurface investigation. It should be emphasized that the results of such laboratory tests are very dependent upon the quality of the samples tested. Consequently, care should be exercised in sampling, handling, and trimming the samples in preparation for testing. Undisturbed samples are difficult to obtain for many cohesionless soils. However, recompacted samples will generally provide useful results provided that care is taken to ensure that the recompacted soil is tested in the same condition (for example, density) as existed in the field.

In addition to the laboratory tests discussed in this section, other specialized tests have been developed to determine the engineering properties of soils. They are treated in laboratory soil testing manuals [B96].

3.5.3 In situ testing

In situ tests that measure the engineering properties of the subsurface materials in place are valuable for designing transmission structure foundations. The most common types of in situ tests that may be useful are

— Vane shear
— Pressuremeter
— Plate loading

The vane shear test is used to measure the undrained shear strength of soft to medium clays. A small, four-bladed vane attached to the end of a rod is pushed into the undisturbed clay at the bottom of a boring. The rod is rotated at the ground surface, and torque and angle of rotation are measured. The measured torque can be related to the shearing resistance developed on the periphery of the cylinder formed by the vanes rotating in the clay. Apparatus and procedures for conducting vane shear tests are described in [B85]. The vane shear test is not suitable in clays containing sand or silt layers, gravel, shells, or organic material.

Comparative studies between the undrained shear strength measured by the vane shear test and laboratory tests on undisturbed samples indicate that the vane shear test can give results either above or below laboratory strength measurements [B152]. Proper interpretation of vane shear test data requires careful sampling and identification of the soil; therefore, the vane shear test should be performed under the direction of a geotechnical engineer.

The pressuremeter is an instrument designed to measure the *in situ* modulus of deformation and may be used to determine the *in situ* state of stress and strength. The pressuremeter consists of an expandable probe that is lowered into a borehole and expanded to contact the sides of the boring. The expandable probe, activated by water pressure, is connected to a volumeter-manometer on the ground surface. After lowering the probe to the desired depth, it is expanded by applying pressure that can be determined by the volume; hence, a curve of pressure versus volume is obtained. This data may be used to determine a horizontal modulus of deformation. It is recommended that pressuremeter testing be performed under the direction of a geotechnical engineer.

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Menard [B109] has proposed a means of using the pressuremeter to determine the horizontal subgrade modulus. The horizontal subgrade modulus is used to design drilled pier foundations (see Clause 5).

The plate-loading test is a means of estimating the bearing capacity and determining the modulus of vertical subgrade reaction by obtaining a load versus deformation curve from which a modulus of deformation is computed. The general procedure for performing a plate-loading test is described in ASTM D1194-94 [B13]. Particular attention is drawn to Note 3 in [B13], which points out that the deflection of a foundation to a given load is a function of the foundation size and shape and the groundwater table location with respect to the bottom of the foundation.

When plate-loading tests are being considered, an alternative method would be to construct a concrete foundation of one-half or one-third scale at the depth of the final foundation. Data from a field test of this scale will be more readily interpreted and applied to the final foundation design.

It is important that field tests be located at those sites that are representative of the majority of soil conditions on the line route. Generally, if only one test is performed, it will be at a location that is judged to represent the poorest subsurface conditions. If the purpose of the field test is to refine the foundation design for a large number of foundations, then the field test should be performed at a location that is representative of a large number of foundation locations. However, considerable experience and judgement is required in the application of *in situ* test results to the design of foundations.

4. Design of spread foundations

4.1 Structural applications

The spread foundation is suitable and commonly used as support for lattice transmission towers. Less common applications are for single shaft and framed structures. The most frequently used types are steel grillages, pressed plates, cast-in-place concrete, and precast concrete. A description of each of these foundation types is presented in the following.

4.1.1 Foundation types

4.1.1.1 Steel grillages

Figure 8 indicates three typical types of steel grillages. Figure 8, part A, is a pyramid arrangement in which the leg stub is connected to four smaller stubs which are connected to the grillage at the base. The advantage of this type of construction is that the pyramid can transfer the horizontal shear load down to the grillage base by truss action. However, the pyramid arrangement does not permit much flexibility for adjusting the assembly, if needed. In addition, it is difficult to compact the backfill inside the pyramid.

Figure 8, part B shows a grillage foundation which has the single leg stub carried directly to the grillage base. The horizontal shear is transferred through shear members that engage the passive lateral resistance of the adjacent compacted soil. It is important that the bottom shear member and diagonal be connected to the leg stub at an adequate depth below the ground surface to mobilize the passive resistance of the compacted backfill.

Figure 8, part C also has the single leg stub carried directly to the grillage base. This type of grillage foundation has a leg reinforcer which increases the area for mobilizing passive soil pressure as well as increasing the leg strength. The shear is transferred to the soil via the leg and reinforcer and resisted by passive soil pressure.

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The base grillage of these three typical foundations consists of steel beams, angles, or channels which transfer the bearing or uplift loads to the soil.

The advantages of steel grillage foundations include: low cost, ease of installation, and immediate tower installation, and they can be purchased with the tower steel, while concrete is not required at the site. The disadvantage is that these foundations may have to be designed before any soil borings are obtained and then may have to be enlarged by pouring a concrete base around the grillage if actual soil conditions are not as good as those assumed in the original design. In addition, large grillages are difficult to set with required accuracy.

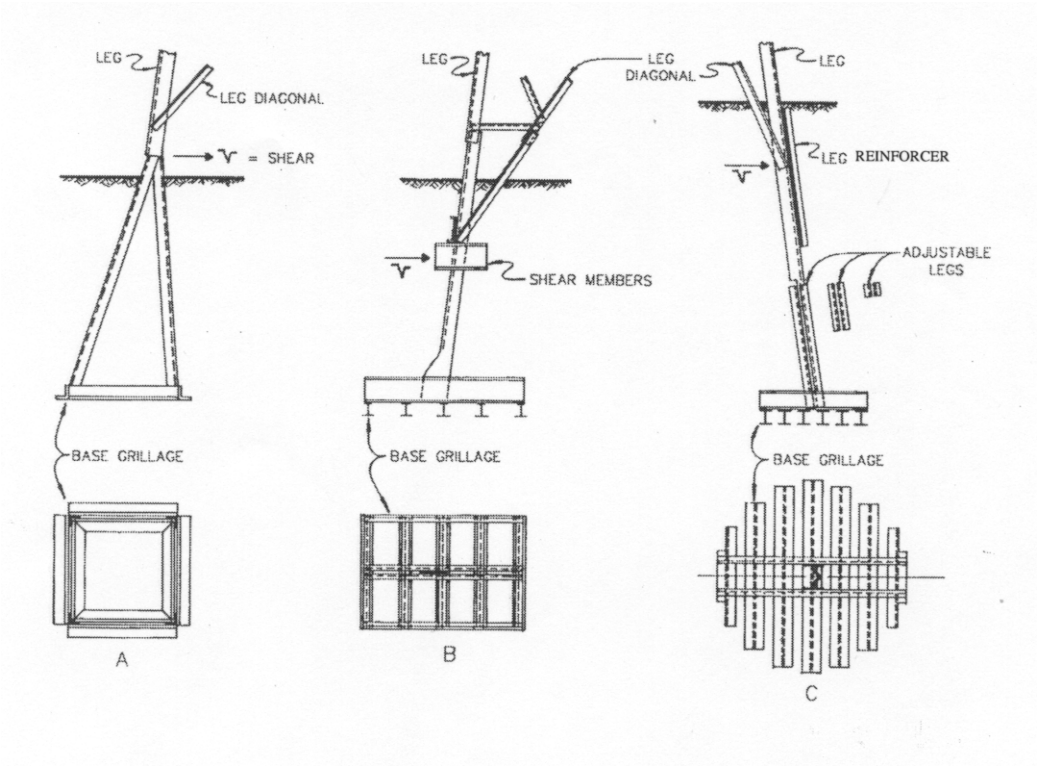


Figure 8—Various steel grillage foundations

4.1.1.2 Pressed plates

A typical pressed plate foundation is shown in Figure 9. This arrangement is similar to the grillages shown in Figure 8, part B except that the base grillage is replaced by a pressed plate. Figure 10 indicates a bipod foundation which has a truss in one direction. In both of the designs shown, the net horizontal shear at the level where the diagonal is attached to the stub is resisted by the passive soil pressure. An apparent disadvantage of this type of foundation is the possibility of loose sand under the dish portion of the plate which could increase settlement.

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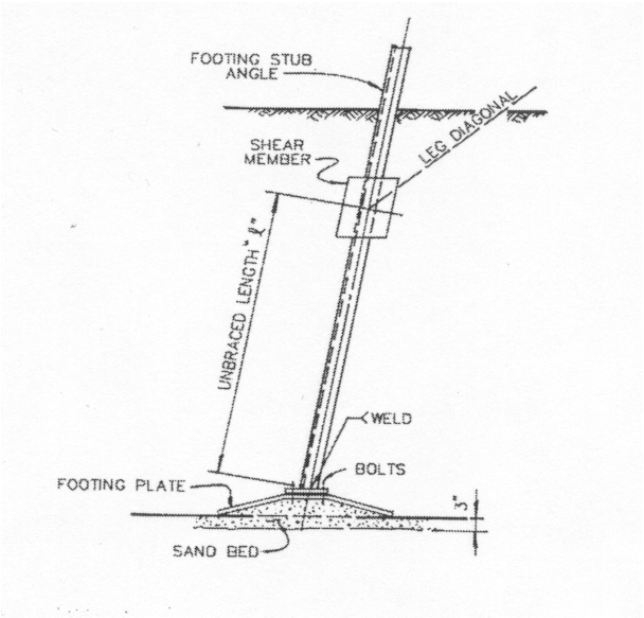


Figure 9—Typical plate foundation

4.1.1.3 Cast-in-place concrete

This type of foundation consists of a base mat and a square or cylindrical pier. It is constructed of reinforced or plain concrete, and several variations exist as indicated in Figure 11. The stub angle can be bent and the pier and mat centered. Alternatively, the mat can be located so that the projection from the stub angle intersects the centroid of the mat, or the pier itself can be battered to the tower leg slope.

Since the mat is required to resist both compression and uplift loads, top and bottom reinforcing steel may be provided to resist the bending moments developed. As required, a construction joint should be provided between the mat and the pier.

Stub angles are embedded in the top of the pier so that the upper exposed section can be spliced directly to the main tower leg and diagonals. The embedded members should be of adequate size to resist the axial loads transmitted from the main leg and diagonals, plus any secondary bending moment from the horizontal shear, if applicable. The embedded member must be embedded in the concrete to a sufficient depth to transmit the load to the concrete. Bolted clip angles, welded stud shear connectors, or bottom plates may be added on the end of the stub angle to reduce this length, as shown in Figure 12. Anchor bolts can also be used in lieu of the direct embedment stub angle, as shown in Figure 11, part C. ANSI/ASCE 10-97, Section 9 [B5] describes the latest embedment design.

4.1.1.4 Precast concrete

This type of foundation is very similar to the cast-in-place concrete foundation, except that the mat is precast elsewhere and delivered to the construction site. Stub angles or anchor bolts may be embedded in the piers during fabrication to provide a connection with the superstructure. The piers may also be cast in the field after the precast mat has been placed and a suitable connection installed prior to pouring the concrete. Care should be exercised to ensure that a uniform contact surface is provided between the precast mat and the soil, and that the soil immediately below the mat is well-compacted.

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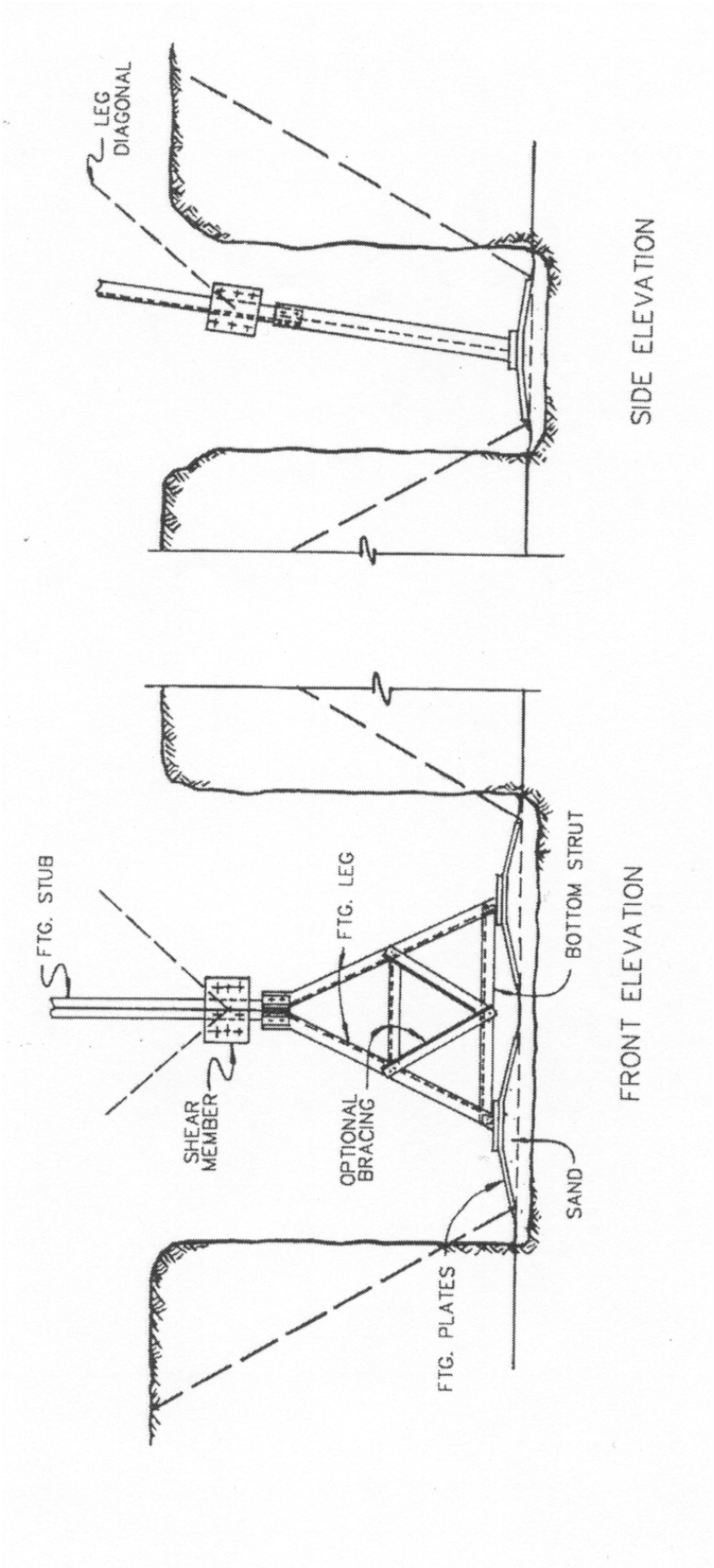


Figure 10—Typical bipod footing

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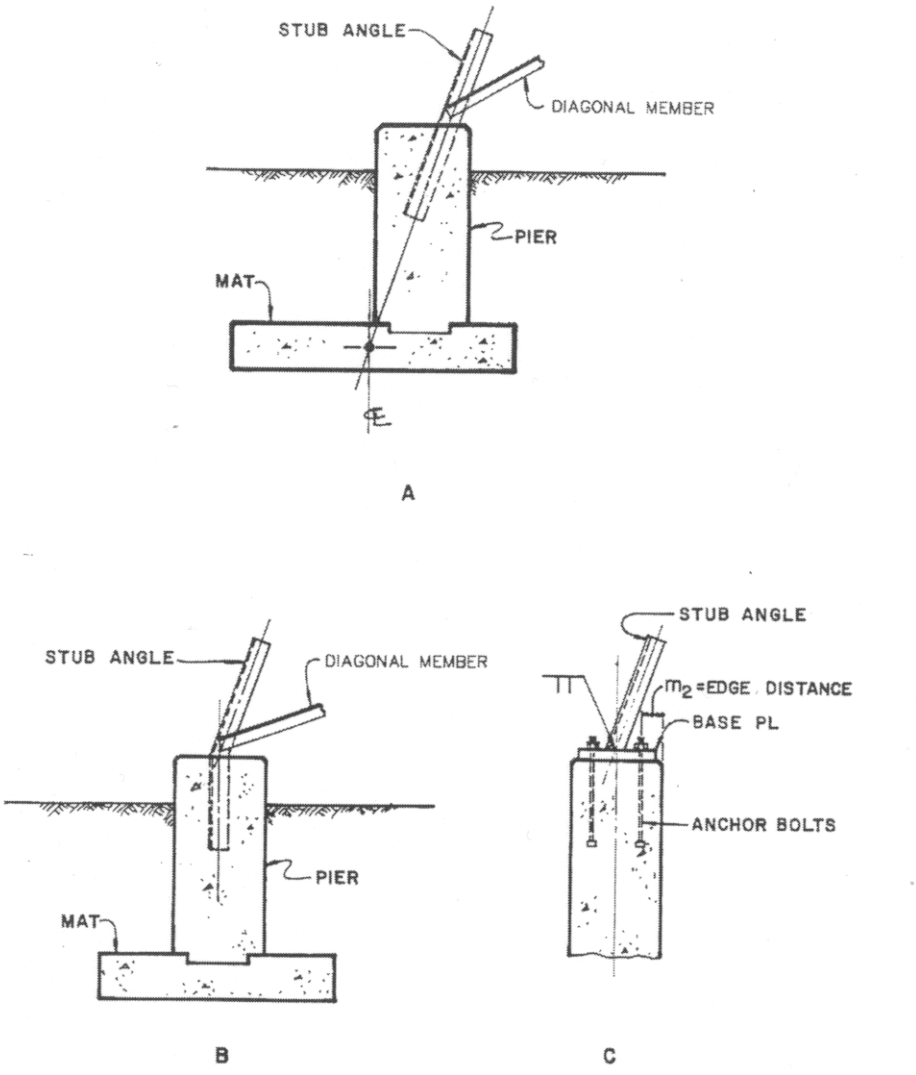


Figure 11—Cast in place concrete foundation

4.1.1.5 Rock foundations

Many areas of the United States have bedrock either exposed at the ground surface or covered with a thin mantle of soil. Relatively simple, economical, and efficient rock foundations may be installed where this type of terrain is encountered. A rock foundation can be designed to resist both uplift and compression loads plus horizontal shear and, in some structure applications, bending moments. Where suitable bedrock is encountered at the surface or close to the surface, a rock foundation, as shown in Figure 13, can be installed.

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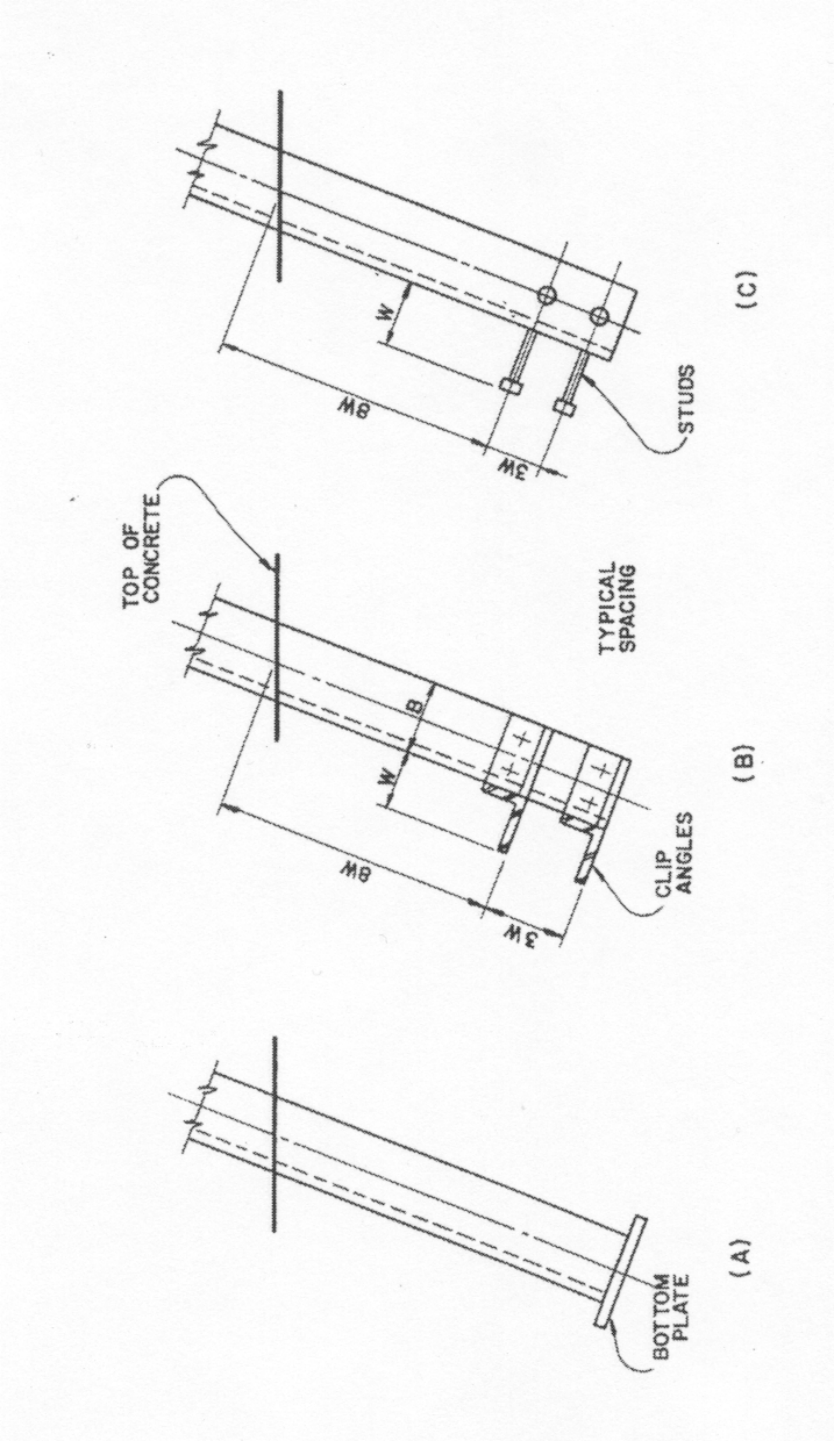


Figure 12—Stub angles

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The determination of whether a rock formation is suitable for installation of rock foundations is an engineering judgment based on a number of factors which were discussed previously in Clause 3. Test holes, field inspection of the excavation, knowledge of the local geology, past experience, and load tests should be considered in this evaluation. The Rock Quality Designation (RQD) is useful in helping to evaluate rock suitability [B46].

Since the bearing capacity of rock is usually much greater than the uplift capacity, care must be exercised in designing for uplift [B88]. The rock sockets can be roughened, grooved, or shaped to increase the uplift capacity [B88]. The design of foundations in rock to resist uplift loads is similar to the design of rock anchors discussed in 7.3.1.

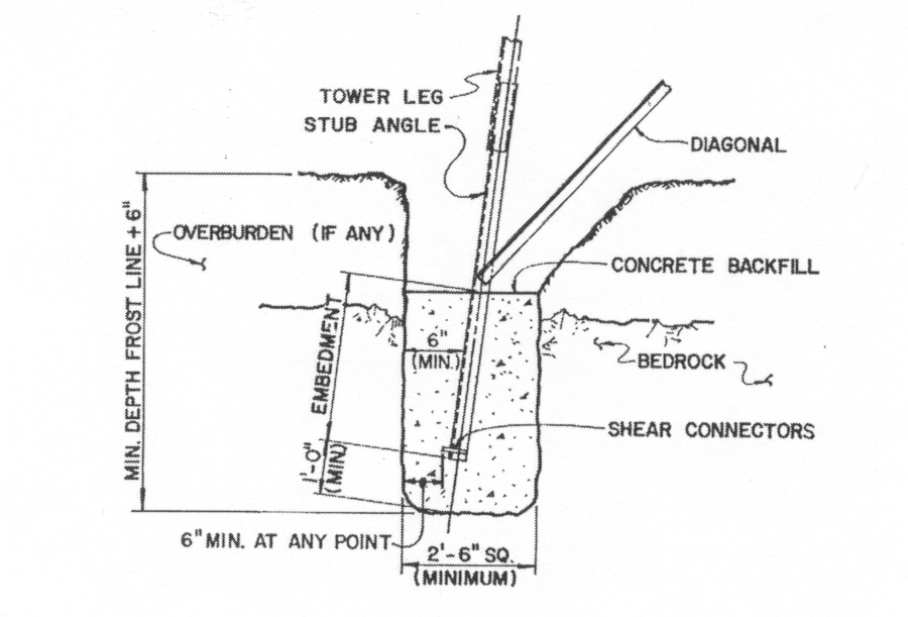


Figure 13—Rock foundation

4.1.2 Foundation orientation

The foundations for lattice towers can be installed with a vertical pier or a pier battered to the same slope as the tower leg, as shown in Figure 14. The pier may be round, square, or rectangular in cross-section and may be of constant section or be tapered to a greater width at the bottom to provide extra strength for the bending moment caused by the horizontal shear at the top of the pier. Generally, the tapered pier will prove to be less economical because of the more complex formwork required.

The pier may also be vertical, as shown in Figure 11, part A, but offset to allow the center of gravity of the stub angle to intersect the centroid of the mat. Alternatively, the pier may be vertical and the stub angle bent, as indicated in Figure 11, part B. The piers and mats can be oriented as shown for Section A-A or for Section B-B in Figure 14. Normally, the orientation of Section A-A gives a better resolution of forces from the two tower faces.

The disadvantage of the vertical pier shown in Figure 14, Section B-B, is the necessity of designing for a large horizontal shear at the top of the pier.

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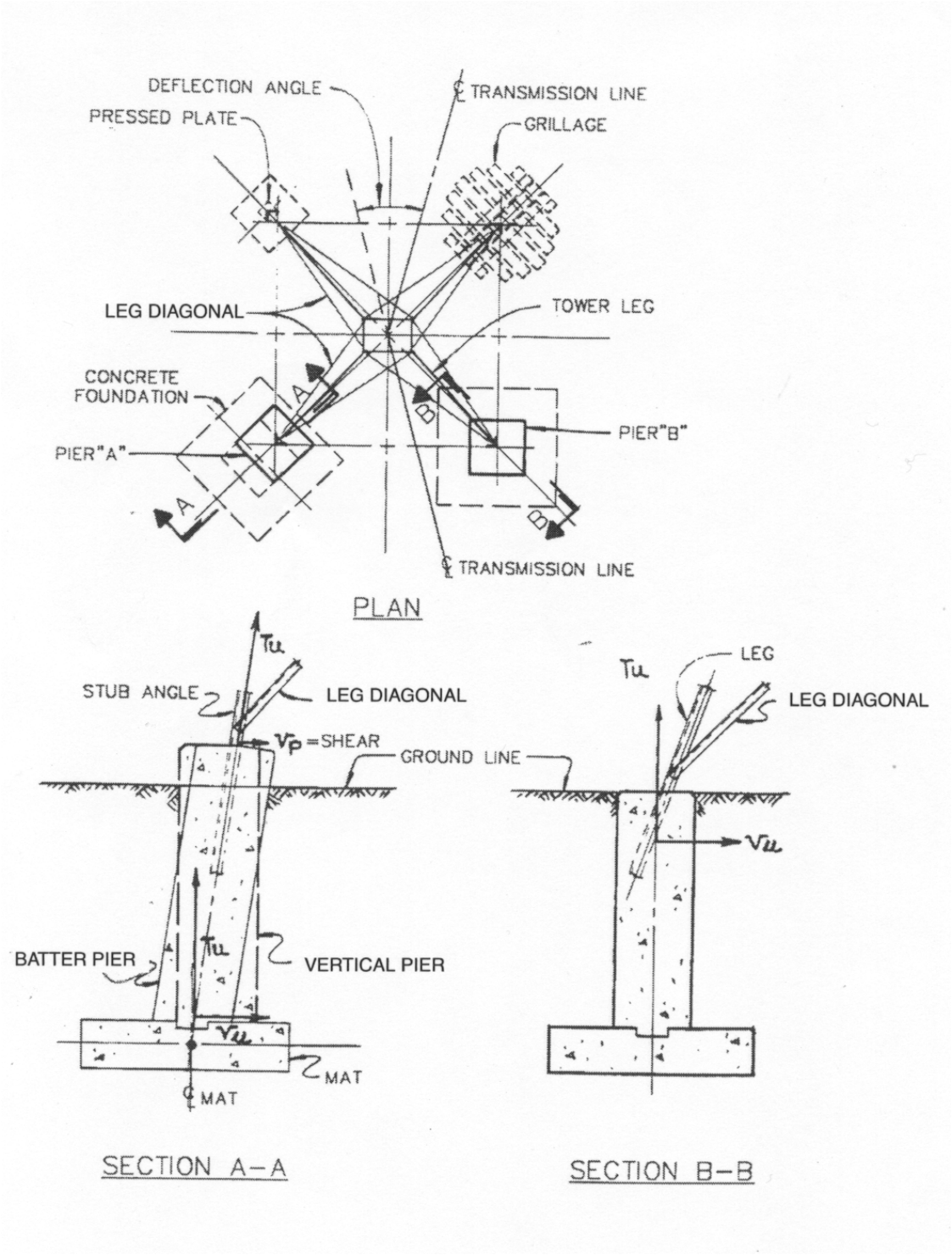


Figure 14—Footing orientation

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When the pier is oriented as shown in Figure 14, Section A-A, the axial forces will continue down through the pier to the center of the mat. Consequently, the horizontal shear load at the top of the pier is greatly reduced for dead-end and large line angle towers. The remaining shear load at the top of the pier can be resisted either by passive soil pressure or by pier bending or a combination of both. Therefore, with dead-end and large line angle tower foundations, the piers and mats can be designed more economically as shown in Figure 14, Section A-A. For tangent tower foundations, the differential shear between straight and battered piers is usually not significant.

As shown in Figure 14, the grillage and plate foundations are relatively easy to orient and adjust as required.

4.2 Analysis

The design of spread foundations for transmission towers must consider the following:

— Load direction
— Load magnitude
— Load duration
— Static vs. cyclic loads
— Foundation movement

This section presents methods of estimating the uplift and compression (bearing) capacities and the settlement of spread foundations. Additional details on uplift and compression analysis of spread foundations for transmission structures are contained in References [B82], [B3], [B168], [B148], and [B158].

Although concrete foundations are used in the discussion, the methods presented here are applicable to other spread foundation types. Minor modifications to the methods are suggested as necessary to consider the type or geometry of the foundation.

4.2.1 Compression capacity

The allowable compression capacity of a spread foundation may be controlled either by the stability of the soil-foundation system (bearing capacity) or by the need to limit the total or differential settlement of the structure. The methods to compute the bearing capacity and settlement are given in the following sections.

4.2.1.1 Bearing capacity

The maximum load per unit area that can be placed on a soil at a given depth is the ultimate bearing capacity, q_{ult}. As shown in Figure 15, q_{ult} is the maximum load, Q, divided by the foundation area, B × L, at depth D. Q includes the structure loads, weight of the foundation, and weight of the backfill within the volume B × L × D. In Figure 15, the soil within the shear surface is assumed to behave as a rigid plastic medium which is idealized by an active Rankine zone (I), a radial Prandtl zone (II), and a passive Rankine zone (III). The soil above the foundation base is treated as an equivalent surcharge.

The general solution is the Buisman-Terzaghi equation given below:

$$q_{ult} = cN_c + \frac{1}{2}B\gamma N_\gamma + qN_q \quad (3)$$

where

c is soil cohesion,
B is foundation width,
q is surcharge (γD),

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D is foundation depth,
γ is soil unit weight,
N_c, N_γ, N_q are dimensionless bearing capacity factors.

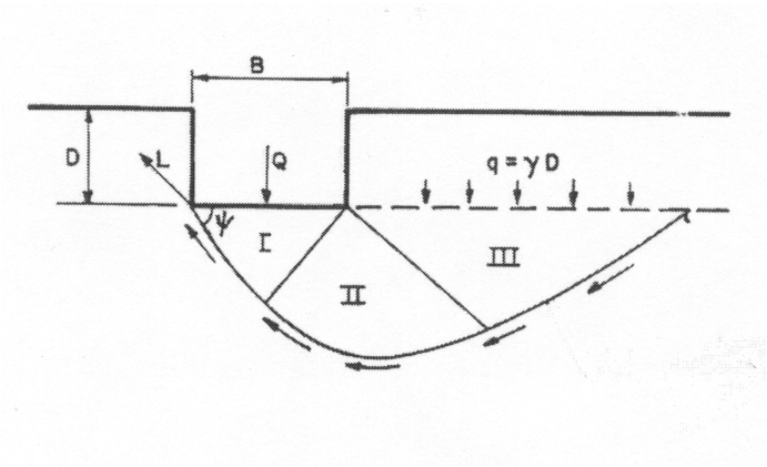


Figure 15—General description of bearing capacity

This equation includes the Prandtl and Reissner solutions for a load on a weightless medium, resulting in:

$$N_q = e^{\pi \tan\phi} \tan^2(45 + \phi/2) \quad (4)$$

$$N_c = (N_q - 1)\cot\phi \quad (5)$$

NOTE—As $\phi \rightarrow 0$, $N_c \rightarrow 5.14$

where

ϕ is soil angle of friction.

Values of N_c and N_q are given in Table 2 and Figure 16. The N_γ term is given as:

$$N_\gamma \approx 2(N_q + 1)\tan\phi \quad (6)$$

which is Vesic's approximation [B162] of the numerical solution by Caquot and Kerisel [B35] that uses $\psi = 45° + \phi/2$ in Figure 15. The solid line (for N_γ) in Figure 16 is Vesic's approximation, which is within 5% for $\phi = 20°$ to 40°.

Equation (3) has been developed for the following idealized conditions:

— General shear failure in the soil
— Horizontal ground surface
— Horizontal, infinitely long, strip foundation at shallow depth
— Vertical loading, concentrically applied

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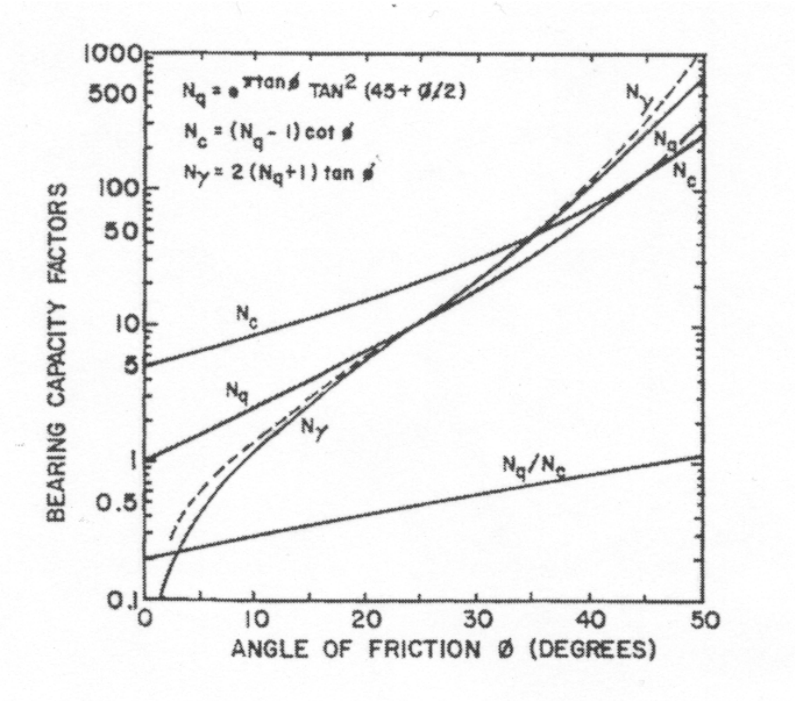


Figure 16—Bearing capacity factors for shall foundations

Table 2—Bearing-capacity factors N_c, N_q and N_γ

ϕ	N_c	N_q	N_γ	N_q/N_c	$\tan\phi$
0	5.14	1.00	0.00	0.20	0.00
1	5.38	1.09	0.07	0.20	0.02
2	5.63	1.20	0.15	0.21	0.03
3	5.90	1.31	0.24	0.22	0.05
4	6.19	1.43	0.34	0.23	0.07
5	6.49	1.57	0.45	0.24	0.09
6	6.81	1.72	0.57	0.25	0.11
7	7.16	1.88	0.71	0.26	0.12
8	7.53	2.06	0.86	0.27	0.14
9	7.92	2.25	1.03	0.28	0.16
10	8.35	2.47	1.22	0.30	0.18
11	8.80	2.71	1.44	0.31	0.19
12	9.28	2.97	1.69	0.32	0.21

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Table 2—Bearing-capacity factors N_c, N_q and N_γ (continued)

ϕ	N_c	N_q	N_γ	N_q/N_c	tanϕ
13	9.81	3.26	1.97	0.33	0.23
14	10.37	3.59	2.29	0.35	0.25
15	10.98	3.94	2.65	0.36	0.27
16	11.63	4.34	3.06	0.37	0.29
17	12.34	4.77	3.53	0.39	0.31
18	13.10	5.26	4.07	0.40	0.32
19	13.93	5.80	4.68	0.42	0.34
20	14.83	6.40	5.39	0.43	0.36
21	15.82	7.07	6.20	0.45	0.38
22	16.88	7.82	7.13	0.46	0.40
23	18.05	8.66	8.20	0.48	0.42
24	19.32	9.60	9.44	0.50	0.45
25	20.72	10.66	10.88	0.51	0.47
26	22.25	11.85	12.54	0.53	0.49
27	23.94	13.20	14.47	0.55	0.51
28	25.80	14.72	16.72	0.57	0.53
29	27.86	16.44	19.34	0.59	0.55
30	30.14	18.40	22.40	0.61	0.58
31	32.67	20.63	25.99	0.63	0.60
32	35.49	23.18	30.22	0.65	0.62
33	38.64	26.09	35.19	0.68	0.65
34	42.16	29.44	41.06	0.70	0.67
35	46.12	33.30	48.03	0.72	0.70
36	50.59	37.75	56.31	0.75	0.73
37	55.63	42.92	66.19	0.77	0.75
38	61.35	48.93	78.03	0.80	0.78
39	67.87	55.96	92.25	0.82	0.81
40	75.31	64.20	109.41	0.85	0.84

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Table 2—Bearing-capacity factors N_c, N_q and N_γ (continued)

ϕ	N_c	N_q	N_γ	N_q/N_c	tanϕ
41	83.86	73.90	130.22	0.88	0.87
42	93.71	85.38	155.55	0.91	0.90
43	105.11	99.02	186.54	0.94	0.93
44	118.37	115.31	224.64	0.97	0.97
45	133.88	134.88	271.76	1.01	1.00
46	152.10	158.51	330.35	1.04	1.04
47	173.64	187.21	403.67	1.08	1.07
48	199.26	222.31	496.01	1.12	1.11
49	229.93	265.51	613.16	1.15	1.15
50	266.89	319.07	762.89	1.20	1.19

To extend this equation to actual field conditions, modifiers have been developed by a number of authors. Those presented below are based primarily upon the consistent interpretations of the available data by Vesic [B162] and Hansen [B70] and as summarized by Kulhawy, et al. [B88].

In its general form the bearing capacity equation is given as:

$$q_{ult} = \frac{Q}{B'L'} = cN_c\zeta_{cs}\zeta_{cd}\zeta_{cr}\zeta_{ci}\zeta_{ct}\zeta_{cg} \quad (7)$$

$$+ \frac{1}{2}B_\gamma N_\gamma \zeta_{\gamma s}\zeta_{\gamma d}\zeta_{\gamma r}\zeta_{\gamma i}\zeta_{\gamma t}\zeta_{\gamma g}$$

$$+ qN_q\zeta_{qs}\zeta_{qd}\zeta_{qr}\zeta_{qi}\zeta_{qt}\zeta_{qg}$$

The ζ modifiers are doubly subscripted to indicate which term it applies to (N_c, N_γ, N_q) and which phenomenon it describes (s for shape of foundation, d for depth of foundation, r for soil rigidity, i for inclination of the load, t for tilt of the foundation base, and g for ground surface inclination). The B' and L' terms take into account load eccentricity. The equations for ζ modifiers are given in Table 3, with definitions of the geometric terms given in Figure 17. It should be noted that these modifiers only include geometric terms, the soil strength parameters, c and ϕ, and the soil rigidity index, I_r which will be defined subsequently.

Equation (7) represents the most general formulation for the bearing capacity of the foundation for a c-ϕ soil. However, caution must be exercised in evaluating the soil strength parameters because very few natural soils have a true cohesion. Those which do fall into special categories, such as naturally cemented soils, very stiff, overconsolidated clays which show an effective stress cohesion that normally decays with time, and partially saturated cohesive fill, in which the cohesion is lost upon saturation. Part of the problem in evaluating the strength parameters correctly is that the strength envelope for many soils in nonlinear and the in-situ or laboratory testing commonly is limited.

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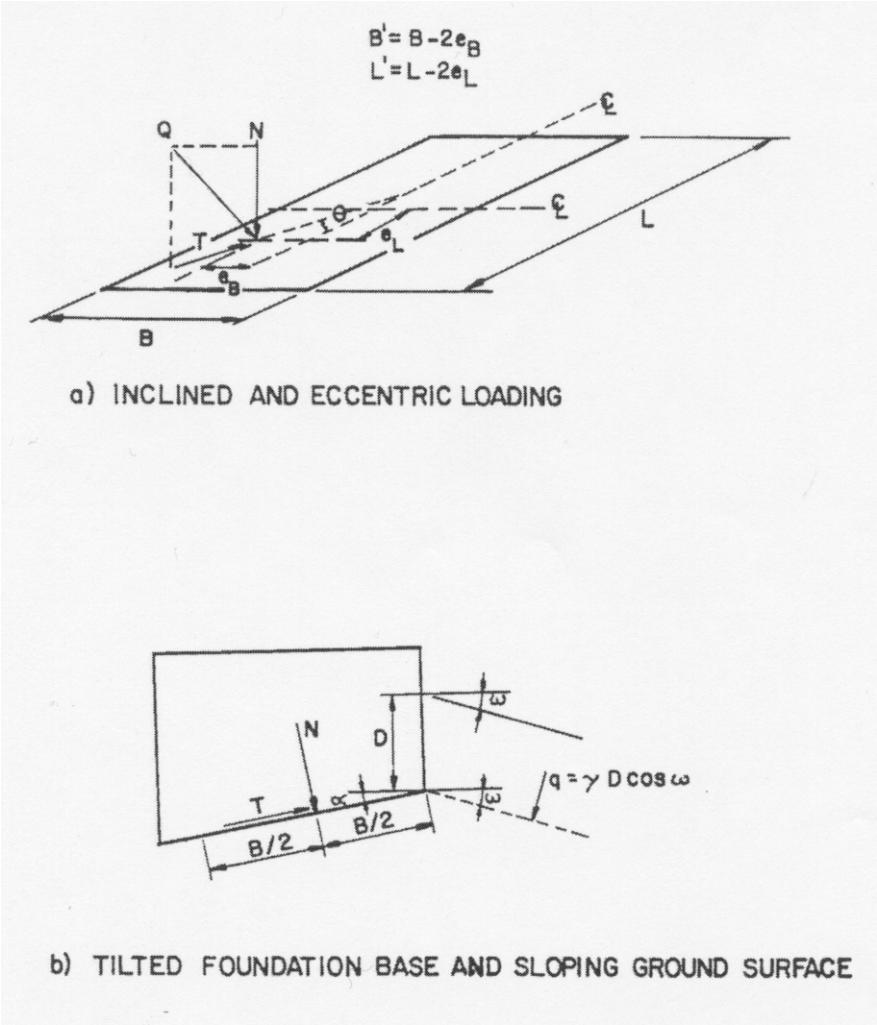


Figure 17—Definitions of geometric terms in bearing capacity equation

Figure 18 shows a common situation which arises. Three tests were conducted on a granular soil at three different normal stresses. The common tendency would be to evaluate this data using the dotted linear approximation. This would be satisfactory if all that one was seeking was the total value of strength within the testing stress range. However, granular soil is cohesionless and the true failure envelope is nonlinear, as shown by the solid line in Figure 18. This nonlinear envelope can be approximated well from the three data points, knowing that the curve must go through the origin. Once this envelope has been established, successive secants from the origin to the envelope are taken to evaluate the variation of ϕ with σ, as shown in Figure 19. For bearing capacity calculations, the value of ϕ to use will be that from Figure 19, consistent with the stress level for the problem at hand.

4.2.1.2 Bearing capacity for drained loading

Equation (7) is used most commonly in either of two derivative forms, which depend primarily on the soil type and rate of loading. The first is for drained loading, which develops under most loading conditions in coarse-grained soils such as sands and for long-term sustained loading of fine-grained soils such as clays. The second is undrained loading, described in the next section.

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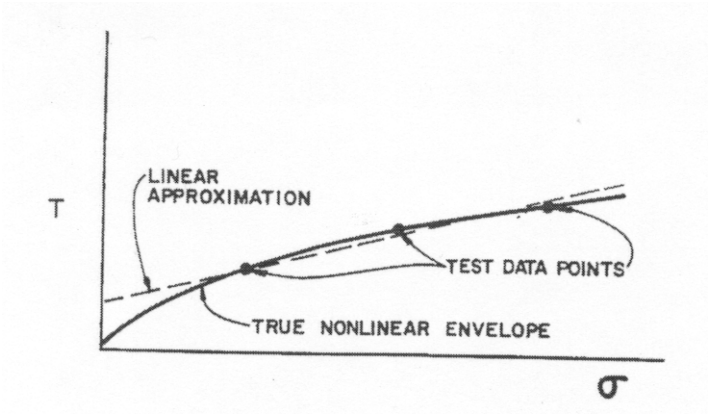


Figure 18—Strength envelope determination

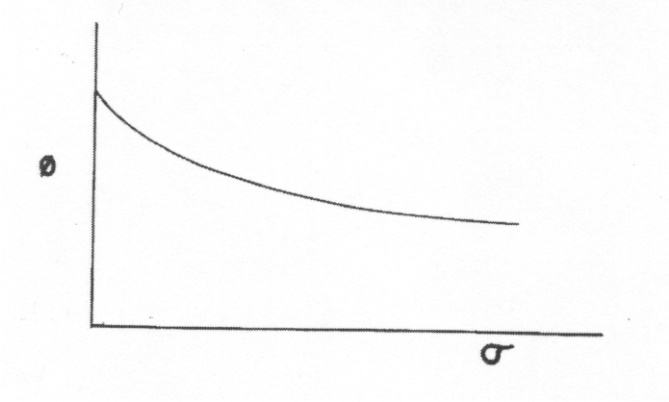


Figure 19—Actual variation of ϕ with σ

For drained loading, c = 0 as described previously, and therefore, Equation (7) becomes:

$$q_{ult} = \frac{Q}{B'L'} = \frac{1}{2}B\gamma N_\gamma \zeta_{\gamma s}\zeta_{\gamma d}\zeta_{\gamma r}\zeta_{\gamma i}\zeta_{\gamma t}\zeta_{\gamma g}$$

$$+ qN_q\zeta_{qs}\zeta_{qd}\zeta_{qr}\zeta_{qi}\zeta_{qt}\zeta_{qg} \quad (8)$$

where

q_{ult}	is ultimate bearing capacity,
Q	is maximum load (including structure load, effective weight of foundation, and effective weight of backfill within the volume $B \times L \times D$),
B	is foundation width or diameter (minimum dimension),
L	is foundation length or diameter,
D	is foundation depth,
B' and L'	are reduced B and L because of load eccentricity,
γ	is average effective soil unit weight from depth D to $D + B$,
q	is effective overburden stress at depth D,
N_γ and N_q	are bearing capacity factors defined in Equation (6) and Equation (4), respectively, and
ζ_{xy}	is bearing capacity modifiers given in Table 3.

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Table 3—Bearing capacity modifiers for general solution

Modification	Shape	Value	Notes
Shape	ζ_{cs}	$1 + (B/L)\ (N_q/N_c)$	—
	$\zeta_{\gamma s}$	$1 - 0.4\ (B/L)$	—
	S_{qs}	$1 + (B/L)\ \tan\phi$	—
Depth	ζ_{cd}	$\zeta_{qd} - [(1 - \zeta_{qd})/(N_c\ \tan\phi)]$	—
	$\zeta_{\gamma d}$	1	—
	ζ_{qd}	$1 + 2\ \tan\phi\ (1 - \sin\phi)^2\ \tan^{-1}\ (D/B)$	a)
Rigidity	ζ_{cr}	$\zeta_{qr} - [(1 - \zeta_{qr})/(N_c\ \tan\phi)]$	—
	$\zeta_{\gamma r}$	ζ_{qr}	—
	ζ_{qr}	$\exp\{[(-4.4 + 0.6\ (B/L))\ \tan\phi] + [(3.07\ \sin\phi)\ (\log_{10} 2I_{rr})/(1 + \sin\phi)]\}$	—
Load inclination	ζ_{cl}	$\zeta_{ql} - [(1 - \zeta_{ql})/(N_c\ \tan\phi)]$	b)
	$\zeta_{\gamma i}$	$\{1 - [T/(N + B'L'\ c\ \cot\phi)]\}^{n+1}$	b), c), d)
	ζ_{qi}	$\{1 - [T/(N + B'L'\ c\ \cot\phi)]\}^{n}$	b), c), d)
Base tilt	ζ_{ct}	$\zeta_{qt} - [(1 - \zeta_{qt})/(N_c\ \tan\phi)]$	b), e)
	$\zeta_{\gamma t}$	$(1 - \alpha\ \tan\phi)^2$	b), c), e)
	ζ_{qt}	$\approx \zeta_{\gamma t}$	b), e)
Sloping ground surface	ζ_{cg}	$\zeta_{qg} - [(1 - \zeta_{qg})/(N_c\ \tan\phi)]$	b), g)
	$\zeta_{\gamma g}$	$\approx \zeta_{qg}$	b), g)
	ζ_{qg}	$(1 - \alpha\ \tan\omega)^2$	b), c), g), h)

a) $\tan^{-1}$ in radians
b) Check for sliding
c) See Figure 17 for notation.
d) $n = \dfrac{[2 + L/B]}{1 + L/B}\cos^2\theta + \dfrac{[2 + B/L]}{1 + B/L}\sin^2\theta$
e) Limited to $\alpha < 45°$
f) α in radians
g) limited to $\omega < 45°$ and $\omega < \phi$; for $\omega > \phi/2$, check slope stability
h) ω in radians

The ζ terms for shape, depth, load inclination, base tilt, and sloping ground surface are a function only of the geometry and the soil friction angle, ϕ, which should be evaluated at the average effective vertical stress within the shear zone or, more specifically, at a depth D + B/2. It should be noted (footnote b) in Table 3) that a check is warranted to ensure that any lateral load component, T, does not exceed the maximum resistance to sliding, given by:

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$$T_{max} = N \tan\delta \tag{9}$$

where

N is axial load component defined in Figure 17,
δ is angle of friction for the soil-foundation interface.

For cast-in-place concrete, $\delta = \phi$; for smooth steel, $\delta = \phi/2$; and for rough steel, $\delta = 3\,\phi/4$ [B94] [B126].

The ζ terms for rigidity include the same geometry and ϕ terms, plus the soil rigidity index, defined as:

$$I_r = \frac{G}{c + \sigma \tan\phi} \tag{10}$$

where

I_r is rigidity index,
G is shear modulus,
c is soil cohesion (equal to 0 for most cases as described previously),
σ is effective vertical stress at depth D + B/2, and
ϕ is soil friction angle as described above.

The shear modulus is commonly expressed in terms of Young's modulus, E, and Poisson's ratio, ν, so that, for drained loading with c = 0, the rigidity index becomes:

$$I_r = \frac{E}{2(1+\nu)} \cdot \frac{1}{\sigma \tan\phi} \tag{11}$$

Young's modulus can be evaluated directly from a number of different field or laboratory tests, corresponding to the stress conditions at depth D + B/2, or can be estimated from correlations in the literature [B137], from empirical techniques [B147], or from case history evaluation [B33]. Of particular interest in this regard is that, for 55 spread foundations in drained uplift, Callanan and Kulhawy found an apparent lower limit for $E/\bar{\sigma}_{vm}$ equal to 200, in which $\bar{\sigma}_{vm}$ equals mean vertical effective stress over depth, D. This apparent limit is a convenient first approximation.

Poisson's ratio approximately ranges from about 0.1 to about 0.4 for granular soils and can be estimated from: [B158]

$$\nu = 0.1 + 0.3\phi_{rel} \tag{12}$$

in which ϕ_{rel} is a relative friction angle given by:

$$\phi_{rel} = \frac{\phi - 25^\circ}{45^\circ - 25^\circ} = \frac{\phi - 25^\circ}{20^\circ} \tag{13}$$

with limits of 0 and 1.

Once the rigidity index is evaluated, it is reduced for volumetric strains [B180] to yield:

$$I_{rr} = I_r/(1 + I_r\Delta) \tag{14}$$

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where

I_{rr} is reduced rigidity index, and
Δ is volumetric strain.

Based on Vesic's [B162] guidelines, Trautmann and Kulhawy [B180] showed that Δ can be estimated conveniently by:

$$\Delta \approx 0.005\sigma(1 - \phi_{rel}) \quad (15)$$

with σ defined in Equation (10), in units of tsf, up to a limit of 10 tsf, and ϕ_{rel} as defined in Equation (13).

After the soil rigidity index has been computed, it is compared with the theoretically based critical rigidity index, I_{rc}, given by [B130]:

$$I_{rc} = \frac{1}{2}\exp[(3.30 - 0.45B/L)\cot(45 - \phi/2)] \quad (16)$$

If $I_{rr} > I_{rc}$, the soil behaves as a rigid-plastic material, general shear failure would result, and therefore $\zeta_{cr} = \zeta_{\gamma r} = \zeta_{qr} = 1$. If $I_{rr} < I_{rc}$, the soil stiffness is low, local, or punching shear failure would result, and therefore, ζ_{cr}, $\zeta_{\gamma r}$, and ζ_{qr} will be less than 1 and must be computed to reduce the ultimate bearing capacity.

4.2.1.3 Bearing capacity for undrained loading

For undrained loading, which occurs when loads are applied relatively rapidly to fine-grained soils such as clays, pore water pressures build up in the soil at constant effective stress and lead to the analysis procedure commonly known as the total stress or $\phi = 0$ method. For this $\phi = 0$ method, $N_c = 5.14$, $N_\gamma = 0$, $N_q = 1$, and $\zeta_{qs} = \zeta_{qd} = \zeta_{qr} = \zeta_{qt} = \zeta_{qg} = 1$, therefore, Equation (7) reduces to:

$$q_{ult} = \frac{Q}{B'L'} = 5.14 s_u \zeta_{cs}\zeta_{cd}\zeta_{cr}\zeta_{ci}\zeta_{ct}\zeta_{cg} + \zeta_{qi} \quad (17)$$

in which

q_{ult} is ultimate bearing capacity,
Q is maximum load (including structure load, total weight of foundation, and total weight of backfill within the volume $B \times L \times D$),
B is foundation width or diameter (minimum dimension),
L is foundation length or diameter,
D is foundation depth,
B' and L' are reduced B and L dimensions because of load eccentricity,
s_u is c = average undrained shear strength from depth D to $D + B$,
q is total overburden stress at depth D,
ζ_{xy} is bearing capacity modifiers given in Table 4.

The ζ terms for shape, depth, base tilt, and sloping ground surface are a function only of the geometry while the ζ term for load inclination includes s_u and the geometry. It should be noted [footnote b) in Table 4] that a check is warranted to ensure that any lateral load component, T, does not exceed the maximum resistance to sliding, given by:

$$T_{max} = c_a B'L' \quad (18)$$

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where

c_a is adhesion for the soil-foundation interface.

For cast-in-place concrete, $c_a \approx s_u$; for smooth steel, $c_a \approx s_u/2$; and for rough steel, $c_a \approx 3\, s_u/4$ [B126] [B137].

The ζ term for rigidity includes the geometry and the soil rigidity index, defined as:

$$I_r = \frac{G}{s_u} = \frac{E}{2(1+\nu)} \cdot \frac{1}{s_u} = \frac{E}{3s_u} = I_{rr} \tag{19}$$

where

I_r is rigidity index,
G is shear modulus,
E is Young's modulus,
ν is Poisson's ratio, which is equal to 0.5 for saturated cohesive soil during undrained loading.

Since $\nu = 0.5$, no volumetric strains occur, and therefore, the reduced rigidity index, I_{rr}, is equal to I_r.

Young's modulus can be evaluated directly from a number of different field or laboratory tests, corresponding to the stress conditions at depth D + B/2, or can be estimated from correlations in the literature [B137], from empirical techniques [B147], or from case history evaluation [B33]. Of particular interest in this regard is that, for 20 spread foundations in undrained uplift, Callanan and Kulhawy [B33] found an apparent lower limit for E/σ_{vm} equal to about 175, in which σ_{vm} is the mean vertical total stress over depth D. This apparent limit is a convenient first approximation.

After the soil rigidity index has been computed, it is compared with the theoretically based critical rigidity index, I_{rc}, given by [B162]:

$$I_{rc} = \frac{1}{2}\exp(3.30 - 0.45B/L) \tag{20}$$

which will vary from 8.64 for a square or circular foundation (B = L) to 13.56 for an infinite strip foundation ($L \rightarrow \infty$). If $I_{rr} > I_{rc}$, the soil behaves as a rigid-plastic material, general shear failure would result, and therefore, $\zeta_{cr} = 1$. If $I_{rr} < I_{rc}$, the soil stiffness is low, local or punching shear failure would result, and therefore, ζ_{cr} will be less than 1 and must be computed to reduce the ultimate bearing capacity.

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Table 4—Bearing capacity modifiers for undrained ($\phi = 0$) loading

Modification	Symbol	Value	Footnotes
Shape	ζ_{cs}	1+ 0.20 (B/L)	—
	ζ_{qs}	1	—
Depth	ζ_{cd}	$1 + 0.33 \tan^{-1}$ (D/B)	a)
	ζ_{qd}	1	a)
Rigidity	ζ_{cr}	$0.32 + 0.12$ (B/L) $+ 0.60 \log_{10} I_{rr}$	—
	ζ_{qr}	1	—
Load inclination	ζ_{ci}	$1 - [(nT)/(5.14\ s_u\ B'L')]$	b),c),d)
	ζ_{qi}	$[1 - (T/N)]^n$	b),c),d)
Base Tilt	ζ_{ct}	$1 - [2\alpha/(\pi + 2)]$	b),c),e),f)
	ζ_{qt}	1	b),e)
Sloping ground surface	ζ_{cg}	$1 - [2\alpha/(\pi + 2)]$	b),c),g),h)
	$\zeta_{\gamma g}$	1	b),g),i)
	ζ_{qg}	1	b),g)

a) $\tan^{-1}$ in radians
b) check for sliding
c) See Figure 17 for notation
d) $$n = \left[\frac{2 + L/B}{1 + L/B}\right]\cos^2\theta + \left[\frac{2 + B/L}{1 + B/L}\right]\sin 2\theta$$
e) limited to $\alpha < 45°$
f) α in radians
g) limited to $\omega < 45°$ and $\omega < \phi$; for $\omega > \phi/2$, check slope stability
h) ω in radians
i) $1/2\ B\gamma N\gamma\ \zeta$ term is necessary for $\phi = 0$ loading when $\omega > 0$; for this case, $N_\gamma = -2\sin\omega$ with ω in degrees, $\zeta_{\gamma s} = 1 - 0.4$ (B/L), $S_{\gamma i} = [1 - (T/N)]^{n+1}$, and $\zeta_{\gamma d} = \zeta_{\gamma r} = \zeta_{\gamma g} = 1$.

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4.2.2 Settlement of spread foundations

4.2.2.1 Immediate settlement

Immediate settlements are those that occur as soon as the load is applied to the soil mass. While these settlements are not truly elastic, most solutions are based on the assumption that the soil may be modeled as a linear elastic half-space. Consequently, immediate settlements are often referred to as elastic settlements.

Elastic settlements (S_i) of saturated or near saturated clays can be determined by the equation [B137]:

$$S_i = I_w q B(1 - \nu^2)/E \quad \text{[for a one-layer system]} \tag{21}$$

where

I_w is geometric factor which reflects the foundation shape, flexibility, and the point on the foundation for which settlement is being calculated,
q is bearing pressure,
ν is the Poisson's Ratio for the soil,
E is modulus of elasticity of the soil,
B is least lateral dimension of the foundation.

The value of Poisson's Ratio (ν) for saturated clay is commonly assumed equal to 0.5. Typical values for I_w for flexible foundations for a square and circular loaded area are 0.95 and 0.85, respectively. These are average values for the entire area. Various points such as the center, corner, and side of the foundation have different values (e.g., see [B133]).

Equation (21) is applicable to granular soils where the elastic parameters depend substantially upon the confining pressure. An alternative in this case is the method proposed by Schmertmann [B139] which has the additional advantage that it is applicable to layered soils. The settlement is given as:

$$S_i = C_1 C_2 = \sum_{i=1}^{n} (I_z/E)_i \Delta Z_i \tag{22}$$

where

C_1 is the correction factor to incorporate strain relief because of embedment and is given by:

$$C_1 = 1 - 0.5\frac{(\sigma'_o)}{q} \geq 0.5 \tag{23}$$

and σ'_o is the effective overburden pressure at the foundation depth. C_2 is a coefficient of time-dependent increase in settlement for cohesionless soils and may be expressed as:

$$C_2 = 1 + 0.2\log_{10}\frac{t}{0.1} \tag{24}$$

where

t is time in years.

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