PDHonline Course E475 (5 PDH)

Substations - Volume VIII - Site and Foundation Design

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Substation Design
Volume VIII
Site & Foundation Design

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This series of courses are based on the "Design Guide for Rural Substations", published by the Rural Utilities Service of the United States Department of Agriculture, RUS Bulletin 1724E-300, June 2001.

Preface

This course is one of a series of thirteen courses on the design of electrical substations. The courses do not necessarily have to be taken in order and, for the most part, are stand-alone courses. The following is a brief description of each course.

Volume I, Design Parameters. Covers the general design considerations, documents and drawings related to designing a substation.

Volume II, Physical Layout. Covers the layout considerations, bus configurations, and electrical clearances.

Volume III, Conductors and Bus Design. Covers bare conductors, rigid and strain bus design.

Volume IV, Power Transformers. Covers the application and relevant specifications related to power transformers and mobile transformers.

Volume V, Circuit Interrupting Devices. Covers the specifications and application of power circuit breakers, metal-clad switchgear and electronic reclosers.

Volume VI, Voltage Regulators and Capacitors. Covers the general operation and specification of voltage regulators and capacitors.

Volume VII, Other Major Equipment. Covers switch, arrestor, and instrument transformer specification and application.

Volume VIII, Site and Foundation Design. Covers general issues related to site design, foundation design and control house design.

Volume IX, Substation Structures. Covers the design of bus support structures and connectors.

Volume X, Grounding. Covers the design of the ground grid for safety and proper operation.

Volume XI, Protective Relaying. Covers relay types, schemes, and instrumentation.

Volume XII, Auxiliary Systems. Covers AC & DC systems, automation, and communications.

Volume XIII, Insulated Cable and Raceways. Covers the specifications and application of electrical cable.

Chapter 1
Site Design

This chapter covers the design factors related to the substation site. The objective of site work design for a substation yard is to provide an easily accessible, dry, maintenance-free area for the installation and operation of electrical substation equipment and structures. The utility should take advantage of the natural drainage and topographical features in the design consistent with the electrical layout since coordination of the two is essential.

Types of Graded Yards

There are generally three basic profiles for substation yards:

1. Flat—most prevalent
2. Sloped—occasionally required
3. Stepped—seldom required

1. Flat Yards

The basic flat yard is more desirable for the layout and operational function of a substation. It permits uniformity in foundation elevations and structure heights. Unless there are property restrictions, severe topographical features, subterranean rock, or other considerations dictate otherwise, the yard should be graded nominally flat. See Figure 1.

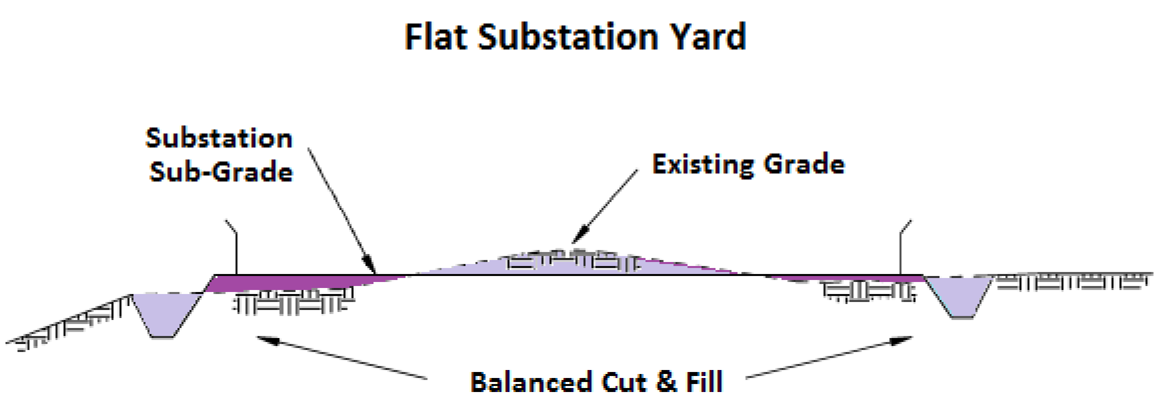


Figure 1

2. Sloped Yards

Occasionally, property restrictions or economic considerations will outweigh the desirability for a flat yard, and a continuously sloping yard may be advantageous. See Figure 2,

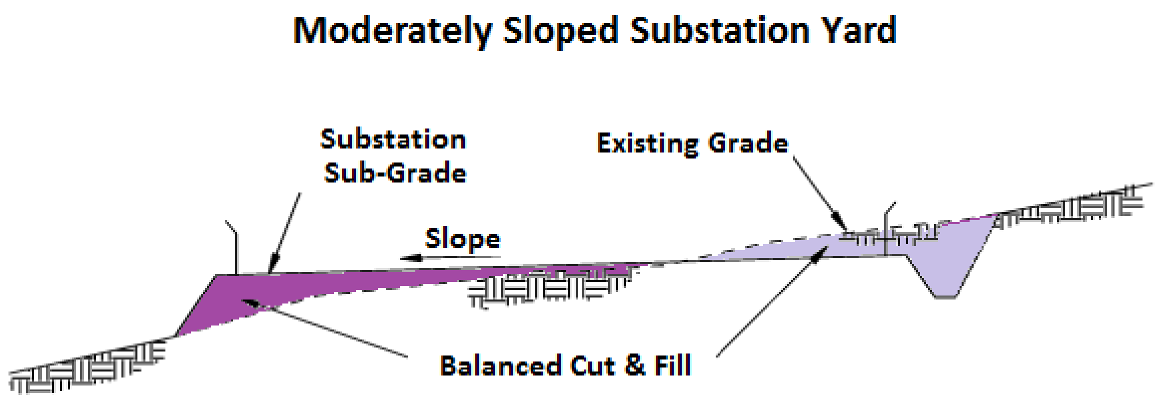


Figure 2

3. Stepped Yards (Two or More Levels)

Stepped yards are usually created by extreme property restrictions, adverse mountainous terrain, or underlying rock formations making excavation uneconomical. See Figure 3.

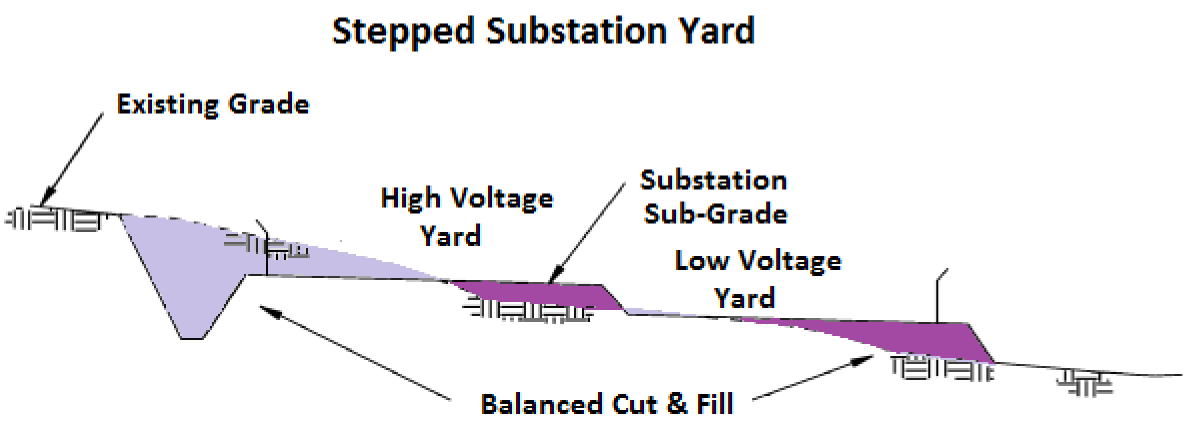


Figure 3

Modification of any of the three types may be necessary to arrive at the optimum yard design. Sloped and stepped sites entail extra design considerations and close coordination with the electrical layout. There may be more structures required and variable foundation elevations.

Preliminary Requirements

The following lists some of the basic information required for the site preparation design for a substation yard:

- Area maps

- Topographic drawing of immediate area showing:
    - Ground elevations on a grid system at 50-foot spacing
    - Location and elevation of existing roads, railroads, ditch inverts, and culverts
    - Location of pertinent overhead or underground utilities, particularly the exact location and depth of any pipelines
    - Property plan
    - Legal description of property
    - Location of the area's drainage exits
    - High water elevation in area, if any
    - Flood zone designation with base flood elevation, if any
- Soil borings in immediate site area

Drainage Considerations

Review state and local government regulations for stormwater management requirements. Many local governments have adopted storm drainage criteria and require that stormwater detention or retention basins be provided, and a few require zero discharge from the site.

Generally all three profiles lend themselves to a surface runoff system. Such a system consists of a gently sloping (0.5 percent to 0.75 percent) ground surface so that the water drains to the edge of the yard or to shallow ditches within the yard. The ditches may discharge into culverts or shallow open channels removing the runoff from the yard.

A *closed drainage system* is a network of catch basins and storm sewer pipe that provides a more positive means of yard drainage. This type of system is quite costly. Circumstances other than economics, however, may require the use of this system.

The yard surface drainage has to be coordinated with the location of cable trenches and roads within the yard. The yard profile (flat, sloped, or stepped) may present varying drainage design considerations. Careful review of the quantity, quality, and particularly the location of the discharge water from the yard is emphasized. Planning the initial drainage system for a future substation addition is sometimes required. Generally a good rule to follow is do not discharge any more water into an existing drainage area outlet than what originally occurred. Small interceptor ditches strategically located will prevent erosion of slopes or embankments.

Whenever it is necessary to calculate the amount of rainfall runoff for the design of culverts, storm sewer pipes, detention or retention ponds, or ditches, a widely used and accepted method is the *Rational Method*. The Rational Method is not recommended for drainage areas much larger than 100 to 200 acres.

The Rational Method formula is expressed by the following equation,

$$Q = CiA$$

Where:
A = Drainage area in acres
i = Average rainfall intensity inches/hr for the period of maximum rainfall of a storm of a given frequency of occurrence, having a duration equal to the time of concentration. The *Time of Concentration*, "tc", is the time required for runoff from remotest part of drainage area to reach the point under design
C = Runoff coefficient
Q = Quantity, cfs

Table 1 and Figure 4 provide information to assist in determining the rainfall intensity for different geographical areas, durations, and recurrence intervals. Additional information concerning rainfall and frequency is available from U.S. Weather Bureau Offices and other technical and hydrological publications.

Table 1
Rainfall Intensity and Conversion Factors

Rainfall Intensity Conversion Factors				Recurrence Interval Factors	
Duration (Minutes)	**Factor**	**Duration (Minutes)**	**Factor**	**Recurrence Interval (Years)**	**Factor**
5	2.22	40	0.8	2	1.0
10	1.71	50	0.7	5	1.3
15	1.44	60	0.6	10	1.6
20	1.25	90	0.5	25	1.9
30	1.00	120	0.4	50	2.2

Frequency of storm occurrence should be either two or five years, and the time of concentration should be between 15 and 20 minutes for a reasonable and economical design. Figure 4 shows two year, 30-Minute Rainfall Intensity (inches/hr).

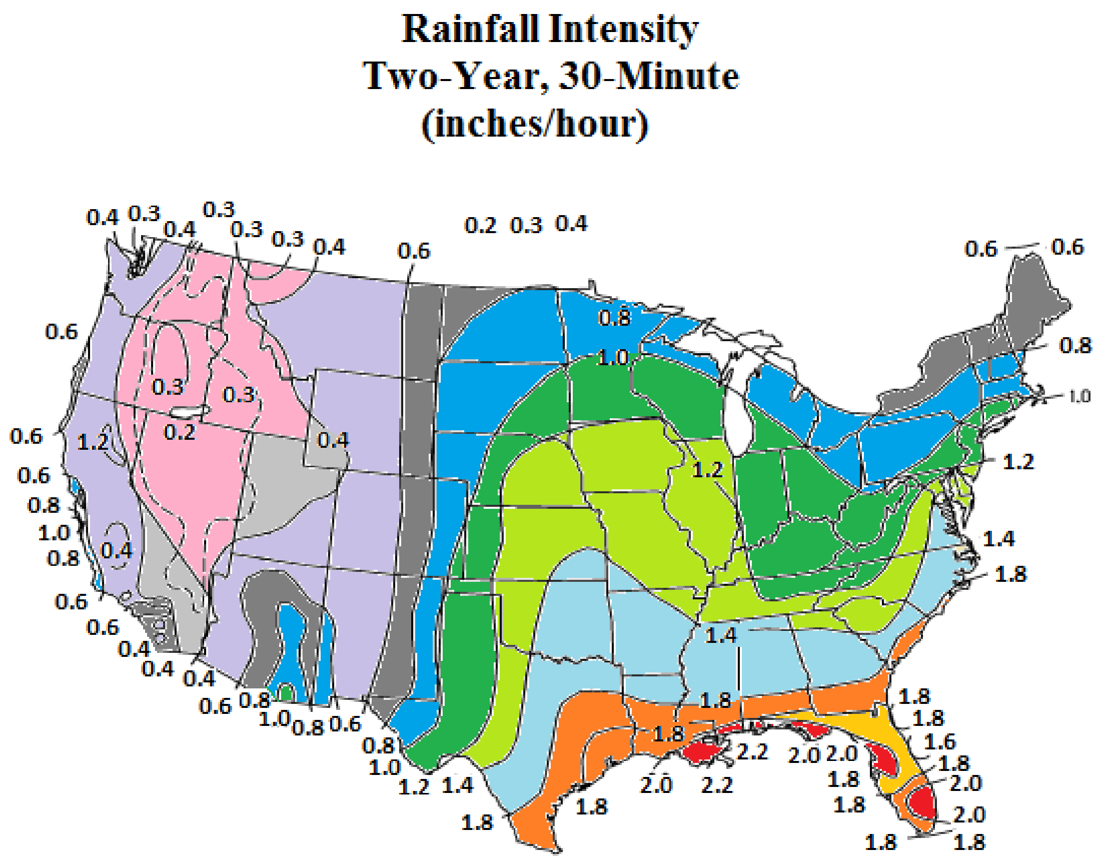


Figure 4

The following design example illustrates a sample problem. Design a culvert in a substation in Lansing, Michigan, given the following information:

- Drainage area = 0.5 acres, or 21,780 ft^2
- Time of concentration = 15 minutes
- Period of recurrence = 5 years
- Coefficient of runoff = 0.50

From Table 1 and Figure 4 determine "I" for a 5-year storm of 15 minutes' duration. From Table 1 the 15 minute rainfall intensity factor is 1.44. The five year reoccurrence interval is 1.3. From Figure 4, for Lansing Michigan, the rainfall intensity is 1.0 inches/hour.

i = (2-yr., 30-min. duration) * (15-min. duration factor) * (5-yr. recurrence factor)

Therefore,

i = 1 in/hr * 1.44 * 1.3

i = 1.87 in/hr or 0.16 ft/hr

The culvert then has to convey:

Q = 0.50 * 0.16 in/hr * 21,780 ft^2 = 1,742 ft^3/hr , or 0.48 ft^3/sec

Once the quantity of water is determined by the Rational Method, the actual size of storm sewer pipe, culverts, or ditches may be determined using,

$$\mathbf{Q = A * V = A * \frac{1.486}{n} * R^{2/3}3 * S^{1/2}}$$

Where:
Q = Volume of pipe or ditch discharge, cfs
A = Cross-sectional area of pipe or ditch flow, ft^2
V = Velocity = ft/s
n = Roughness coefficient for pipe or ditch (assume 0.024)
R = Hydraulic radius of pipe or ditch, ft. For pipe,

$$\mathbf{R = \left(\frac{Area\ of\ Section}{Wetted\ Perimeter}\right) = \left(\frac{D}{4}\right)}$$

S = Hydraulic gradient, ft/ft (slope of pipe or ditch)

Assuming that an 8 inch diameter (area = 0.35ft^2), corrugated, metal pipe will handle the flow, and using the following equation, determine the slope of the pipe as follows,

R = D/4 = (8/12) / 4 = 0.167

$$0.48 = 0.35 * \frac{1.486}{0.024} * 0.167^{2/3} * s^{1/2}$$

Solving for “s”,

S = 0.27 ft/ft

The velocity is Q = A * V

Since Q = 0.48 ft^3/sec and the area is 0.35 ft^2, the velocity is,

$$v = \frac{0.48}{0.35} = 1.37 \text{ ft/sec}$$

For pipes to be *self-cleaning* a minimum velocity of 3 ft/s is required to prevent silting. The velocity at the minimum slope to handle the discharge volume is not quite self-cleaning, but would probably be okay. After the size of pipe or ditch is determined, both minimum and maximum flow velocities should be reviewed. Water in ditches, however, should be allowed to flow approximately 1 or 2 ft/s in unprotected ditches and up to a maximum of about 5 ft/s in sodded channels before erosion occurs. Special erosion protection at open discharge ends of storm sewers and both ends of culverts should be made. Flared end sections and riprap or concrete headwalls should be specified to protect these areas from scour and erosion.

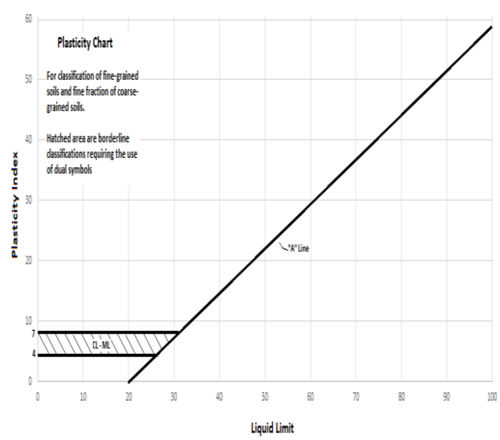
Earthwork Considerations and Design

The computation of earthwork quantities is usually the first step in establishing the nominal rough grade elevation of the yard.

Clearing and grubbing of the site is required, and all vegetation should be removed and properly disposed of. Generally, the topsoil in the substation area is removed and stockpiled for future use in areas requiring seeding.

When the natural grade of the proposed site is essentially flat, it may be necessary to bring in fill material (borrow) to improve the drainage condition of the yard. However, the engineer should avoid the use of borrow in the site design if possible.

The borrow material should consist of a satisfactory soil free from sod, stumps, roots, large rocks, or other perishable or deleterious matter. It should be capable of forming a stable embankment when compacted in accordance with the requirements of this section. Acceptable soils for borrow as identified by the Unified Soil Classification System are GW, GP, GM, GC, SW, SP, SM, and SC (see Table 2).

<table>
<tr><th colspan="9">Table 2
Unified (ASTM) Soil Classification System</th></tr>
<tr><th colspan="3">Major Divisions</th><th>Group Symbols</th><th>Typical Names</th><th colspan="4">Classification Criteria</th></tr>
<tr><td rowspan="8">Coarse-Grained Soils
More than 50% retained on #200 Sieve</td><td rowspan="4">Gravels 50% or more of Coarse Fraction Retained on #4 Sieve</td><td rowspan="2">Clean Gravels</td><td>GW</td><td>Well graded gravels & gravel sand mixtures, little or no fines</td><td rowspan="8">Classification on Basis of Percentage of Fines
Less than 5% pass #200 Sieve GW, GP, SW, SP
More than 12% pass #200 Sieve GM, GC, SM,SC
5% to 12% pass #200 Sieve Borderline classification requiring dual</td><td colspan="3">$C_u = \frac{D_{60}}{D_{10}}$, >4
$C_z = \frac{D_{30}^2}{D_{10}+D_{60}}$, 1<>3</td></tr>
<tr><td>GP</td><td>Poorly graded gravels and gravel-sand mixtures, little or no fines</td><td colspan="3">Not meeting both criteria for GW</td></tr>
<tr><td rowspan="2">Gravels with Fines</td><td>GM</td><td>Silty gravel, gravel-sand-silt mixtures</td><td colspan="2">Atterberg limits plot below “A” line or Plasticity index < 4</td><td rowspan="2">Atterberg Limits plotting in hatched area are borderline classifications requiring dual symbols</td></tr>
<tr><td>GC</td><td>Clayey gravels, gravel-sand-clay mixtures</td><td colspan="2">Atterberg Limits Plot above “A” Line and Plasticity Index >7</td></tr>
<tr><td rowspan="4">Sands more than 50% Coarse Fraction passes #4 Sieve</td><td rowspan="2">Clean Sands</td><td>SW</td><td>Well graded sands and gravelyy sands, little or no fines</td><td colspan="3">$C_u = \frac{D_{60}}{D_{10}}$, >6
$C_z = \frac{D_{30}^2}{D_{10}+D_{60}}$, 1<>3</td></tr>
<tr><td>SP</td><td>Poorly graded sands and gravelly sands, little or no fines</td><td colspan="3">Not meeting both criteria for SW</td></tr>
<tr><td rowspan="2">Sands with Fines</td><td>SM</td><td>Silty sands, sand-silt mixtures</td><td colspan="2">Atterberg limits plot below “A” line or Plasticity index < 4</td><td rowspan="2">Atterberg Limits plotting in hatched area are borderline classifications requiring dual symbols</td></tr>
<tr><td>SC</td><td>Clayey sands, sand-clay mixtures</td><td colspan="2">Atterberg Limits Plot above “A” Line and Plasticity Index >7</td></tr>
<tr><td rowspan="6">Fine-Grained Soils
50% or more passes #200 Sieve</td><td colspan="2" rowspan="3">Silts and Clays Liquid Limit 50% or less</td><td>ML</td><td>Inorganic clays, very fine sands, rock flour, silty or clayey fine sands</td><td colspan="4" rowspan="6">
</td></tr>
<tr><td>CL</td><td>Inorganic clays of low to miedium plasticity gravelly clys, sandy clays, silty clays, lean clays</td></tr>
<tr><td>OL</td><td>Organic silts and organic silty clays of low plasticity</td></tr>
<tr><td colspan="2" rowspan="3">Silts and Clays Liquid Limit Greater than 50%</td><td>MH</td><td>Organic silts, micaceous or diatomaceous fine ssands or silts, elastic silts</td></tr>
<tr><td>CH</td><td>Inorganic clays of hgiht plasticity, fat clays</td></tr>
<tr><td>OH</td><td>Organic clays of medium to high plasticity</td></tr>
<tr><td colspan="3">Highly Organic Soils</td><td>PT</td><td>Peat, muck, and other highly organic soils</td><td colspan="4">Visual Manual Identification, See ASTM D2488</td></tr>
</table>

The borrow pit should be located on the property if possible. If the borrow pit is located a remote distance from the site, the engineer should reevaluate the site design to avoid hauling borrow long distances. Removing topsoil on flat natural sites increases the borrow or fill requirements. Conditions when it would be excessively uneconomical to remove all the topsoil might be:

- Excessive depth of topsoil - 18" and deeper
- When borrow material has to be hauled long distances

The engineer should evaluate alternatives to stripping the topsoil in such circumstances. One alternative when conditions do not seem favorable for removing topsoil is to uniformly mix the topsoil with the underlying soil. The mixture is very often suitable for embankments up to 3 feet.

The mixture may also be compacted in place and serve as a non-bearing base upon which to build the embankment. The engineer should make certain that the soil to be mixed with the topsoil is predominantly granular soil. Silts or clays would not be suitable. The mixture should consist of one or more parts of good soil to one part of topsoil. When alternatives to topsoil removal are considered, the foundation design should take into account the depth at which the soil conditions have been altered.

On other than flat natural grade conditions, the nominal elevation of the yard is usually determined from a balance between the required earth *fill* for the embankment and the available earth that has to be excavated or *cut* from the higher areas of the site. All cut-and-fill slopes should be one vertical to four horizontal if possible.

There are several software programs available with which the engineer can input digitized topography and roadway templates and the programs will output earthwork quantities. The computer software programs contain a *digital terrain model* (DTM), which is a graphical representation of the topography of the site. The DTM is created from using a *triangulated irregular network* (TIN) model of points from the site survey. Changing the surface terrain is simulated by building new surfaces to create the final grade and merging them with the existing terrain into the revised site representation. The software program then compares the two surfaces to compute the earthwork volumes.

Cut and fill quantities can also be computed by the *average end area method*, which is explained in most surveying books. Briefly, the method consists of drawing cross sections taken every 50 feet or 100 feet. The areas of cut and fill are then determined from the computed sections. The sections are usually drawn with a vertical scale exaggeration of ten times the horizontal scale. The sections show both the existing profile and the proposed profile.

To compute the earthwork, the "cut" and "fill" areas of each section are totaled separately and added to the "cut" and "fill" quantities of the adjacent section. The average of the cut summation and the average of the fill summation for each pair of adjacent sections are multiplied by the distance between sections to obtain the volumes of cut and fill. This procedure is followed at each section plotted across the substation yard. Usually several adjustments to the proposed elevation are necessary to balance the earthwork. Only 80 to 85 percent of cut volume, as previously computed, is assumed to be available for fill. The 15 to 20 percent reduction allows for losses due to compaction, spillage, and unsuitable material.

Adequate compaction during placement of the fill is necessary to develop the required soil bearing capacity and lateral resistance for the foundation design. It is necessary also to prevent settlement due to consolidation of the embankment, which may result in ponding, broken ducts, conduits, cable trenches, etc. All fill areas should be compacted in 8-inch layers to 95 percent of the maximum density obtained by AASHTO Std. T180. The base upon which the embankment is constructed should also be loosened and compacted.

Upon completion of the site work, all excavated earth not used in backfilling should be leveled off or shaped to present a neat appearance and not obstruct any drainage. Borrow pits should be graded to a smoothly contoured shape. It may be necessary to provide seeding mulching to such areas.

Roads and Other Access

Access roads into substation yards have to be adequate to sustain heavy equipment under all weather conditions. Long access roads require design considerations similar to most secondary county or state roads. Any culverts or sewer crossings also need to be designed for anticipated heavy equipment loads.

The maximum grade on the access road should generally not exceed 7 percent so that heavy transformers may be transported to and from the yard by normal movers without problems. Ten percent grades may be tolerated for short distances of 200 to 300 feet. The inside radius of the access road at 90 degree intersections with major roads should not be less than 50 feet in order to provide sufficient turning space for long vehicles. Smaller radii may be adequate for substations below 230 kV. Where space allows, access roads should be about 20 feet wide. The road should be crowned at the center for drainage. The sub-grade for the road should be prepared and compacted to the same requirements as the embankment for the yard.

The wearing course for access roads in substations up to 69 kV may consist of a 8-inch deep aggregate base course. State highway department standard specifications usually contain several different types of base course material.

For larger substations, the access road may consist of a 8-inch aggregate base course and a 4-inch aggregate surface course. Highway standard specifications include several types and specify the related material and gradation requirements for the base and surface course material. Application of the wearing courses should be made in accordance with highway standard specifications.

Railroad spurs may be economically feasible at some substation locations. Coordination with the responsible railroad company will usually determine the requirements for making the turnout. Often the railroad company will insist on installing the track for a specified length from the main line. Normally, the railroad company's standards are specified in regard to ballast, ties, rails, and connections. The compaction requirements used for the yard embankment are adequate for the spur track subgrade.

Many substations do not have any specific drives or roads within the fenced yard. The entire yard is considered as driveable by light traffic. If it is desirable to have specific drives within the fenced yard for access to transformer banks or as a perimeter drive, the wearing surface can be the same as for the access road. The width may be reduced to 16 feet or even less. Inside radii for interior drives may be 25 feet or less as space allows. Culverts and cable trenches should also be designed for anticipated heavy equipment loads.

Erosion Protection

All cut and fill slopes, ditches, and all other areas outside the fenced yard from which topsoil or vegetation was removed should be protected from wind and water erosion. In most cases the placement of topsoil, fertilizer, mulch, and seed are sufficient and economical for erosion protection. Topsoil should be placed about 4 inches thick. Consult the local agricultural extension office or the highway department for appropriate types and application rates of fertilizer and seed. Slopes greater than 1 vertical to 2 ½ horizontal may require sodding. The engineer should attempt to keep slopes at 1 vertical to 4 horizontal for erosion and maintenance purposes. Riprap should be used at corners and intersections of ditches where erosion is likely.

Because of the large amount of land under construction each year, erosion and sedimentation control during construction have become a problem. Many states and localities now have laws or ordinances to control soil erosion on construction sites and the sedimentation of adjoining waterways.

The engineer should become aware of such state laws, which are usually enforced by the counties. Many counties have extensive guidelines that have to be strictly adhered to. Consult county agricultural extension offices for these requirements. The soil erosion and sedimentation control statutes may mean considerably more engineering time to develop drawings to show compliance with the requirements of the statute.

Yard Surfacing Material

It is desirable to have 4 to 6 inches of crushed stone or rock cover the entire substation yard and to extend 3 feet beyond the substation fence. In some locations clam and oyster shell may also be used. The yard surface material helps reduce the danger of potentially hazardous mesh and touch potentials; minimizes weed growth; provides a clean, reasonably dry walking surface during wet periods; dissipates erosion effect from rain; and contributes to better access drives for light vehicles.

In cases where a more substantial and unlimited drive area is desirable, a 4-inch layer of well-graded gravel (highway aggregate base course material) is placed and rolled firm. A 3-inch layer of crushed stone or rock may then be placed on top. In areas where clam and oyster shell is available, a very durable drive surface is easily obtained.

The size of stone for yard surfacing material should generally vary between 3/8 to 1 inch with the percentage finer than 3/8 inch limited to 5 to 10 percent. Usually the state highway department has gradations in this range. All faces of yard surface material should be crushed. Use of round rock (river rock) is to be avoided due to rutting problems.

The material selected for yard surfacing may be affected by the electrical grounding design. Because of electrical fault currents, surfacing material may need to be specified differently than would otherwise be the case. It may be desirable to have a 4 to 6-inch layer of crushed rock as coarse as can reasonably be walked on with as few fines as practical or nominally available.

Before the yard surfacing material is installed, the yard surface should be brought to its proposed elevations and rolled to a reasonably firm condition. A soil sterilizer may be applied to prevent the growth of grass and weeds at this time. The yard stone or shell should then be spread as evenly as practical but need not be rolled. This work should not begin until all substation work is essentially completed.

Security Fence

The fence should be installed as soon as practicable after the site work is completed. This work is usually done by a fence contractor and is not necessarily a part of the general contract.

Chapter 2
Foundations

This chapter provides an overview of the design issues for substation foundations. Foundation design primarily depends on the in-place density and strength/strain properties of the soil on or in which foundations are located. The heterogeneous characteristics of soils or their localized variability make foundation design a much less exacting engineering problem than structural design or some facets of electrical design, but the inexactness of soil mechanics need not be a reason for ultraconservatism and costly foundations. Further, with the enhancements of convenient computer programs, tedious repetition of design practices for foundation design has been reduced. While this chapter is intended as a general guide, it cannot serve as a guarantee against foundation problems.

A thorough knowledge of geotechnical subsurface engineering parameters is essential to providing a reliable and cost-effective foundation design. Geotechnical engineering or soil analysis is beyond the scope of this document and generally requires the services of a geotechnical engineer to prepare site-specific recommendations. When designing foundations, it is important to address the following design issues:

- The allowable load-bearing capacity of the subsurface materials
- The allowable deformations permitted upon the structure/foundation under loading

In addition to site-specific borings and laboratory testing, additional data may be gathered pertinent to the site through reference to state or federal geologic mapping, aerial photos, Natural Resource Conservation Service mapping, and other hydrological references. A number of federal and state agencies frequently have available information that can be consulted and will provide either regional or localized data to supplement a site subsurface investigation.

Soil Information

A subsurface investigation should be conducted for each substation. Temporary installations and small distribution substations may only require a minimal amount of information. At these locations augered probe borings, in situ quasi-static cone penetrations (ASTM D3441), vane shear tests (ASTM D2573), or the pressure meter (ASTM D4719) can be used to provide an indication of the soil's engineering characteristics.

Soil probes or borings should be taken primarily at critical foundations. These normally are for line support structures (i.e., deadends) and transformers. The number of borings may vary from three at small substations to six or ten at larger substations. The depth of borings should be about 30 feet below the final grade of the substation yard, but may vary dependent upon:

1. Intensity of structural loads
2. Softness or wetness of subsurface soils
3. Depth of groundwater level

Soil Classification

Because of design considerations, the geotechnical engineer will typically determine whether to design the foundations as if the soil behaves as a *cohesive* (fine-grained clay-like) soil or as a granular or *cohesionless* (coarse-grained sand-like) soil. The description of the material noted on the soil boring log should be described in accordance with the Unified Soil Classification SysteASTM D2487 and D2488).

Material described as "sandy clay" can, for example, be assumed to behave predominately as cohesive material, whereas "clayey sand" will probably behave as granular material. The relative quantities of cohesive and granular materials can appreciably affect the soil properties and cause concern. Therefore, when in doubt, design the foundations both ways and use the most conservative design. To assist with the visual classification of soil types, specific index classification tests can be performed. Typical index classification tests for cohesionless and cohesive soils are as follows:

Cohesionless

Grain Size Sieve Analysis	ASTM D422
Moisture Content	ASTM D2216
Specific Gravity	ASTM D854

Cohesive

Atterberg Limits	ASTM D4318
Moisture Content	ASTM D2216
Unit Weight	ASTM D2216
Specific Gravity	ASTM D854
Hydrometer	ASTM D422

Index tests provide data that can frequently be correlated with other engineering characteristics of the soils such as strength, swell potential, collapse potential, consolidation, and degree of compaction. Although index tests should not be viewed as a total replacement to undisturbed engineering tests, they frequently can be used to supplement an investigation program and confirm either consistency or variability at site.

Specific references are available in research that relate index properties to engineering properties; these should be reviewed by an individual familiar with geotechnical engineering design who understands their limitations and appropriateness for use. Typically, it is not recommended to simply utilize presumptive engineering properties from tables or texts that rely on standard penetration tests, visual classifications, and limited index tests alone, particularly for cohesive soils where greater variability in strength and deformation may occur locally. Presumptive values for allowable bearing pressures on soil or rock should be used only as a means of guidance or for preliminary or temporary design. Further, the degree of disturbance to soil samples, whether standard penetration test samples or relatively undisturbed samples, have to be kept to a minimum prior to laboratory testing through the use of appropriate drilling, sampling, and transportation methods.

Bearing Values

Bearing values or engineering parameters developed from laboratory tests upon cohesive soils should be obtained from relatively undisturbed samples in accordance with "Practice for Thin-Walled Tube Geotechnical Sampling of Soils," ASTM D1587, or "Practice for Ring-Lined Barrel Sampling of Soils," ASTM D3550.

Engineering parameters for granular soils can generally be estimated reliably by correlations with standard penetration blow counts. The parameter obtained is usually the internal friction angle. Unconfined compressive strengths for cohesive or clay-like soils are best determined using "Test Method for Unconfined Compressive Strength of Soil" (ASTM D2166), although rough correlations relative to standard penetration tests have been developed. In situ strengths derived from the quasi-static cone penetrometer (ASTM D3441), vane shear (ASTM D2573), or pressure meter (ASTM D4719) are frequently more reliable, particularly in very soft soils, because of their minimization of sample disturbance and testing of the soils in place.

Rock coring to obtain sample recovery, Rock Quality Designation (RQD), and unconfined strengths may be required for sites with anticipated rock formations. It is also important to be familiar with local engineering characteristics of not only the individual rock formations present, but also their global or rock mass properties. Give special design consideration to swelling hales, steeply dipping rock, highly weathered or decomposed rock units, fractured or jointed rock, or clay or bentonite seamed rock units. These conditions may impact both the strength and deformation performance of structures founded in or upon rock.

Groundwater Level

The elevation of the groundwater level is important in foundation design for several reasons. Open-cut excavations below it within permeable stratas require dewatering and increased costs. The water level also has considerable influence on the bearing capacity and total settlement in granular soils. The bearing capacity of a spread footing in granular soil is derived from the density of the soil below the footing and the density of the soil surrounding the footing (backfill or surcharge). A rise of the water level above a depth greater than the width of the footing up to the top of the surcharge in effect reduces the effective overburden pressures within the sand to roughly half their original values, and further reduces the stiffness or strain carrying capacity of the granular soils. Therefore, the footing pressure that is expected to produce a 1-inch total settlement when the water level is at the surface is only about half that required to produce a 1-inch total settlement when the water level is at or below a depth equal to the footing width below the base of the footing.

The effect of the water level is to reduce the effective overburden pressure or density of the granular soil because of buoyancy. The submerged density of granular soil is about half of the moist or dry density. If the water level is at or exceeds a depth equal to the footing width below the base of the footing, the bearing capacity is not affected. If the water level is at the bottom of the footing or may rise to the ground surface, the portion of bearing capacity obtained by the density of soil below the footing is reduced in half. If the water level is at the top of the backfill, the portion of bearing capacity obtained by the density of the surcharge is also reduced in half.

Differential Settlement

Minor differential settlement between foundations in substations is generally acceptable. However, there are certain soils and conditions that have to be carefully reviewed and avoided if possible. Silts and silty sands are usually problem soils, both from strength relationships and degree of compaction. Carefully examine weak strata of soil under a thin layer of dense or good soil and take them into consideration regarding settlement. Organic or swamp-like soils, or uncompacted fills, pose a risk to differential longterm settlement to structures. Soils that may expand, collapse, or disperse upon wetting require special attention for design by a geotechnical engineer.

Chemical Tests

Soils may impact foundations and other buried systems as a result of chemical attack on concrete, corrosion of steel, and other detrimental weathering. Consult a corrosion specialist or engineer familiar with these chemical tests to determine the appropriate cement type based on prevailing soluable sulfate and chloride ion concentrations within the soil or groundwater. Resistivity of the soil, pH, sulfides, Redox readings, and nitrates may also have detrimental effects on concrete, steel, or grounded structures. These conditions require localized testing and design.

Seismic Evaluation

Seismic evaluation of substation sites is currently beyond the scope of this document, but should be reviewed locally by a geotechnical engineer, and geologist, where required. Delineating the location of faults and surface disruptions, and the potential for sand boils, soil flows, slope slides, and liquefiable soils, is essential to designing foundations that remain stable under extreme seismic events. If liquefaction is determined to be likely at a site under certain seismic events, then design of the foundations to extend beyond such zones or mitigation measures to improve the soils will need to be pursued in final design.

Foundation Types

The various types of foundations for substation structures and equipment include drilled shafts (augered piers), spread footings, piles, slabs on grade, rock anchors, and direct embedment for wood or concrete poles.

Drilled Shafts (Piers)

The drilled shaft is constructed by augering, drilling, or coring a hole in the ground, placing reinforcing steel, and filling with concrete. The anchor bolts may be cast in the shaft at this time or set in a cap constructed at some later time. Drilled shaft construction and design has been developed and researched by the Association of Drilled Shaft Contractors (ADSC) and the American Concrete Institute (ACI), each of which has developed typical standard specifications for their construction.

When there is a sufficient quantity of foundations, the drilled shaft foundations are usually more economical than other types because of the “assembly line” installation procedure. Drilled shafts are most economical when soil conditions are not wet and sandy, although, with care and quality control oversight during construction, casings, bentonite slurries, and polymer muds are currently available to promote their construction.

If wet and sandy soil is below a level where a spread footing would bear, the spread footing should be evaluated and may be selected. However, if wet and sandy conditions also exist above the spread footing level, consider the drilled shaft, allowing extra costs for encasement. This design may be more economical than attempting to install a spread foundation in wet, sandy soil, requiring either dewatering to stabilize the excavation or pumped concrete below water.

Drilled shafts are best suited to resist overturning shears and moments. Uplift and compressive forces are also adequately resisted by drilled shafts. Where soils permit belling of the shaft, additional uplift or compressive capacity is often gained economically.

Common sizes for substation foundations range from 24 inches to 60 inches in diameter, in 6-inch increments. Drilled shafts above 84 inches in diameter are typically installed in 12-inch increments with a maximum diameter of 120 inches available for extreme substation applications.

The engineer should attempt to utilize the same shaft diameter for as many foundations as practical. *Belling* of the shaft is less frequently performed for substation foundations, but has cost advantages when permissible.

For most substation equipment support structures and line support structures, the foundations are required to resist moderate shear forces and overturning moments. For A-frame and lattice-type line support structures, shear, uplift, and compression are typical design loads. Drilled shafts also provide improved performance against differential settlement for critical transformer structures and have been used to provide support against overturning loads resulting from wind or seismic design conditions.

<u>Drilled Shaft Design</u>

Drilled shaft foundations derive their support to carry loads based on the shaft's shape (straight versus belled), soil/rock stratification, groundwater level, and applied loads. Generally, compressive loads are carried by a combination of skin friction along the shaft's perimeter and end bearing at the base. In cases where shafts bear upon or in rock, the overlying soil skin friction is typically discounted for design. Uplift loads are carried as skin friction alone for straight shafts. Shafts with bells require special review as to their capacity and the load transference mechanism based upon soil strata types and depth-to-diameter ratios of shaft and bell embedment. Lateral loads (shear and/or moment) are carried by the lateral resistance of soil/rock against the shaft's cross section.

The spacing of shafts for adjacent structures, commonly referred to as group action, should be evaluated to prevent or minimize stress overlap from skin friction or lateral stresses. A common rule of thumb is to maintain a minimum shaft spacing (centerline to centerline) of not less than three times the diameter, although selected research has indicated that some soil types may require a greater shaft spacing. Typically, compression and uplift loads are designed separate from lateral design analyses. Conventional factors of safety are applied to compression and uplift soil resistance to account for potential variability of subsurface conditions. Depending on the method of lateral load analysis used for design, the lateral design may or may not use factors of safety. Methods employed that evaluate shaft deflection and rotation performance typically do not use factors of safety since they are determining the predicted load/deformation characteristics of the drilled shafts under the design loading.

For purposes of lateral design of drilled shaft foundations, several techniques exist to evaluate shaft performance. With the advent of computers and the ability to evaluate multiple length and diameter shaft combinations readily, lateral analysis of shafts has been performed by methods employing computer stress/strain relationships rather than the older rough calculations by hand using ultimate strength of a passive pressure wedge.

The design of drilled shafts for lateral loads should account for the effects of soil/rock–structure interaction between shaft and subsurface stratas to account for deflection and rotation of the shafts. Final design methods employing this conventional process are described under Method A presented below. Two other methods – conveniently named Method B and Method C – are also mentioned below.

Method A—Lateral Drilled Shaft Design (Computer-Modeled Soil Interaction Curves):
Drilled shaft response is dependent upon the applied load, the soil/rock stratigraphy (including groundwater level), and the dimensional and physical features of the drilled shaft. Under applied load, soil–structure interaction behaves in either a linear-elastic, elastic-plastic, or ultimate-plastic state, depending on the soil subgrade modulus. The resulting load-displacement relationship has been described as nonlinear subgrade modulus P-Y curves.

The use of P-Y curves describing the relationship between lateral pressure "P" and lateral displacement Y at individual stations along the length of the shaft permits the computer program to solve for deflected shapes of the shaft, as developed for the specific stratigraphic soils present. It is possible to assign rock fixity at any point along the shaft and to account for additional rotational restraints, such as grade beams or slab inter-ties.

Output from the computer analysis includes deflections, rotations, internal moments, internal shears, and soil reactions along the entire shaft length. Moments and shears generated by the program are used by structural engineers to design the amount and spacing of longitudinal and tie steel. The program further permits evaluation of shaft performance under the cracked section analysis, resulting in a more accurate representation of shaft and structure movement under load.

Method B—Lateral Shaft Resistance Design: With the increasing use of both concrete and steel poles as replacements for wood poles in selected instances, a method of evaluating pole embedment using an earlier referenced approach prepared by Hansen in 1961 for stratified soils has been documented. The design analogies applied to direct-embedded poles can be further extended to drilled shafts installed in stratified soils. It is used for the preliminary selection of embedment depths for steel and concrete transmission poles sustaining relatively large overturning moments. This method has limitations due to the proposed national scope of use and variations in localized performance.

Method C—Lateral Drilled Shaft Design (Equipment Support Structures): Equipment support structures designed by the working stress method having overturning moments at the column bases may be easily designed by a method developed by E. Czerniak and published in the Journal of the Structural Division of the Proceedings of the American Society of Civil Engineers in March 1957. Method C procedures for evaluating the ultimate capacity and deflection, based on hand calculation procedures, should be used primarily for preliminary design or cost estimating purposes. Convenient nomographs and soil values make the tedious formulas unnecessary. Some formulas are presented here for purposes of illustration and as an aid in design. See Figure 5 for the force diagram.

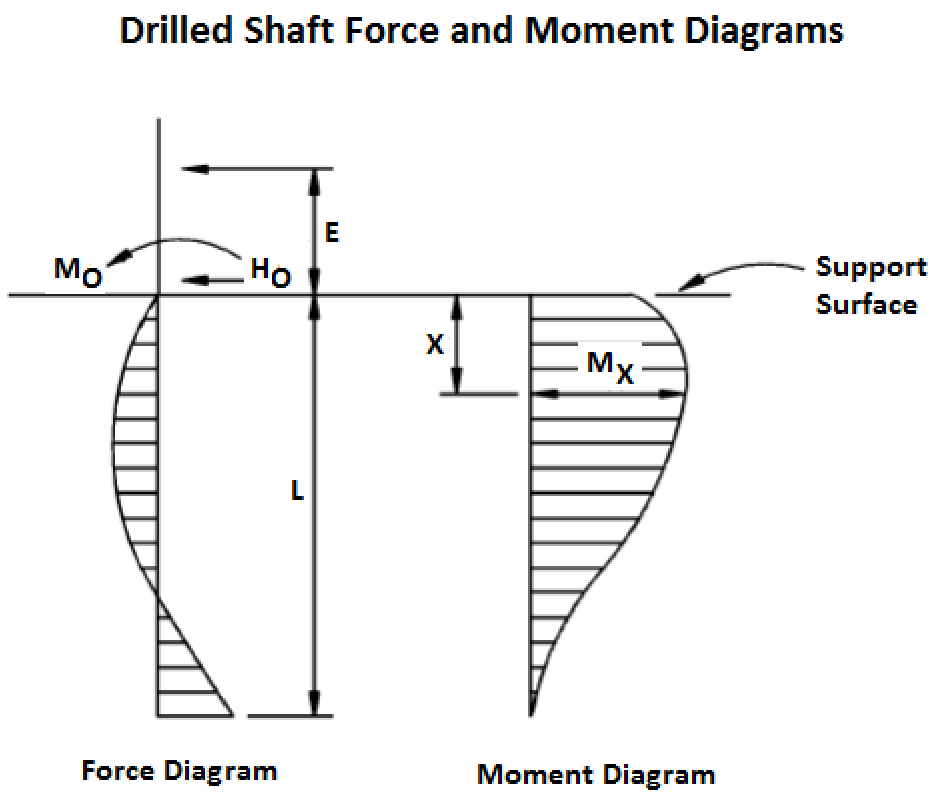


Figure 5

$$\mathbf{L^3 - 14.14 * \frac{H_O * L}{R} - 18.85 * \frac{M_O}{R} = 0,} \qquad \textbf{for Round Piers}$$

Where:
L = Embedment length in feet
H_o = Lateral force per foot of pier diameter, in lbs/ft
M_o = Moment per foot of pier diameter, applied at the resisting surface, in ft-lbs/ft
R = Allowable lateral soil resistance, in pounds per square foot per foot of depth

Lateral soil pressure (R) values for design are given in Table 3. The maximum bending moment in the shaft is obtained from,

$$\mathbf{M_{Max} = C_M * H_O * LD}$$

Where:

D = Shaft diameter, in feet
C_M = Moment coefficient

Table 3 **Recommended Lateral Soil Pressure (R)**	
Material	**Value (PSF/ft)**
Rock in Natural Beds	500
Medium Hard Caliche	400
Fine Caliche with Sand Layers	400
Compact well-graded Gravel	400
Hard Dense Clay	350
Compact Coarse Sand	300
Compact Coarse and Fine Sand	300
Medium Stiff Clay	250
Compact Fine Sand	200
Ordinary Silt	200
Sandy Clay	200
Adobe	200
Compact Inorganic Sand & Silt Mixture	200
Soft Clay	100
Loose Organic Sand & Silt Mixtures and Muck or Bay Mud	0

C_M, the moment coefficient, can be calculated from either the following equation or the maximum value can be obtained from Table 4.

$$\mathbf{C_M = \left(\frac{E}{L} + \frac{X}{L}\right) - \left(\frac{4E}{L} + 3\right)\left(\frac{X}{L}\right)^3 + \left(\frac{3E}{L} + 2\right)\left(\frac{X}{L}\right)^4}$$

Where:

$$\mathbf{E = \frac{M_O}{H_O}}$$

Ho = Lateral force per foot of pier diameter, in lbs/ft

Mo = Moment per foot of pier diameter, applied at the resisting surface, in ft-lbs/ft
L = Depth of the foundation, feet.
X = Distance from supporting surface to point of maximum bending moment (for the force diagram shown X = 0.35 L ±)

Table 4
Moment Coefficients

E/L	0	0.25	0.5	0.75	1.0	1.25	1.5	1.75	2.0
C_M	0.26	0.48	0.70	0.92	1.16	1.40	1.64	1.88	2.12

Consider the following example as shown in Figure 6. For an augered pier with a 24" diameter, what pier design is sufficient for a 12,000 ft-lb moment?

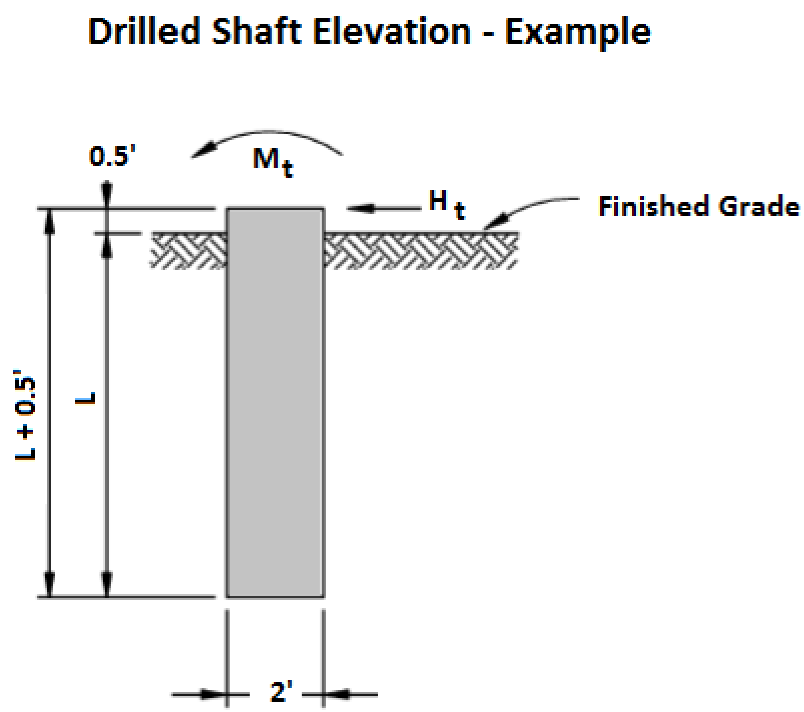


Figure 6

For this foundation use six 5/8" diameter bars at equal spacing for reinforcement; see Tables 4 and Table 5. Use 3/8" diameter bars for ties.

Table 5 **Maximum Moment for Augered Piers with 6 Straight Bars** (Ft-KIPS)				
Diameter of Augered Pier	**6 - #5**	**6 - #6**	**6 - #7**	**6 - #8**
12”	5.75	8.2	11.1	14.7
18”	10.8	15.3	20.8	27.4
24”	15.8	22.4	30.5	40.1
30”	20.7	29.4	40.2	52.9
36”	25.7	36.5	49.8	65.6
42”	30.7	43.6	59.5	78.3
48”	35.7	50.7	69.2	91.1
(Fy = 40 KSI, Fc’=3,000 PSI) (Based on simplified working stress assumptions)				

Table 6 **Maximum Moment for Augered Piers with 8 Straight Bars** (Ft-KIPS)				
Diameter of Augered Pier	**8 - #5**	**8 - #6**	**8 - #7**	**8 - #8**
12”	8.4	12.0	16.3	21.5
18”	15.7	22.3	30.4	40.0
24”	22.9	32.6	44.4	58.5
30”	30.2	42.8	58.4	76.9

36”	37.4	53.1	72.4	95.4
42”	44.7	63.4	86.5	113.9
48”	51.9	73.7	100.5	132.3
(Fy = 40 KSI, Fc’=3,000 PSI (Based on simplified working stress assumptions)				

Drilled Shaft Design - Line Support Structure

For pole-type line support structure foundations, a different method of foundation design is employed. These structures are designed on the basis of yield stress, and appropriate overload factors are applied to the structure loads. To complement these loadings, a foundation design is used utilizing the ultimate strength of the soil. See Figure 7.

Drilled Shaft Section

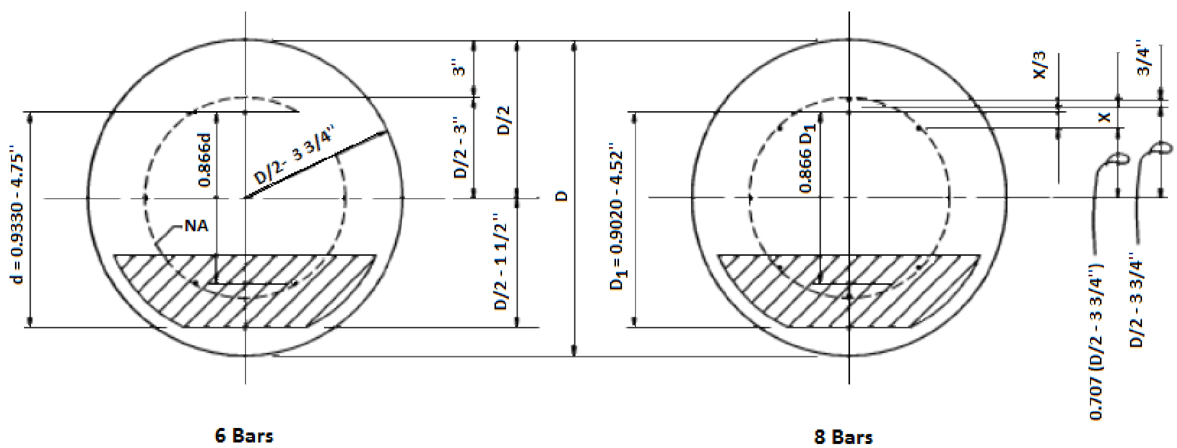


Figure 7

Some of this information is presented here as an aid in design:

kp = Coefficient of passive earth pressure = $\tan^2(45 + \phi/2)$

ϕ = Angle of internal friction of soil @ 28.5° + N/4

γ = Effective unit weight of soil, in pcf

c = Cohesive strength of soil, ½ unconfined compressive strength (q_u), in psf

N = Number of blows per foot from standard penetration test

H = M/Q
q_u = Unconfined compressive strength @ N/4
q = Q/9cD

For a drilled shaft foundation in cohesive soil use these equations,

$$\mathbf{L = 1.5 * D + q[1 + \sqrt{2 + \frac{(4H + 6D}{q}}}$$

$$\mathbf{M_{max} = M + 1.5 * QD + \frac{Q^2}{18 * c * D}}$$

Note: this is below top of foundation. See Figure 8 for a drilled shaft elevation view.

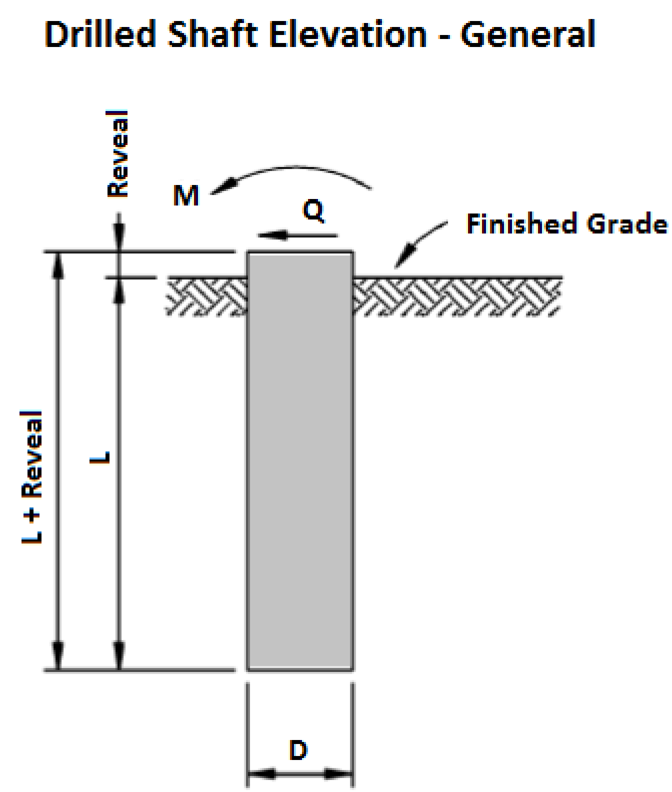


Figure 8

For a drilled shaft foundation in granular soil, use the following equations,

$$\mathbf{L^3 - \frac{2 * Q * L}{k_p * \ \gamma * D} - \frac{2 * M}{k_p * \ \gamma * D} = 0}$$

$$\mathbf{M_{max\ bending} = M + \frac{0.545 * Q * \sqrt{Q}}{\sqrt{k_p * \gamma * D}}}$$

This equation is at,

$$\mathbf{0.817 * \sqrt{\frac{Q_{Below\ Grade}}{k_p * \ \gamma * D}}}$$

Utilize standard design procedures for determining the required area of reinforcing steel.

A foundation design using Brom's Theory for granular soil is illustrated below:

Given:
Q = 10,700 lbf
M = 395,000 ft-lbs
γ = 110.0 lbs/ft^3
ϕ = 28.5 + 26/4 = 35°
D = 4.00 ft
N = 26 blows/ft

$K_p = \text{Tan}^2 (45 + 35/2) = 3.69$

$$L^3 - \frac{2 * 10{,}700 * L}{3.69 * \ 110 * 4} - \frac{2 * (395{,}000 + 0.5 * 10{,}700)}{3.69 * \ 110 * 4} = 0$$

$L^3 – 13.18*L – 493.2 = 0$

Solving for "L" we have,

L = 8.46 ft.

Adding ½ foot for the reveal yields,

L = 8.46 ft + 0.5 ft for reveal = 8.96 ft.

$$M_{max\,bending} = 395{,}0005 + 0.5 * 10{,}700 + \frac{0.545 * 10{,}700 * \sqrt{10{,}700}}{\sqrt{3.69 * 110 * 4}}$$

$M_{max\ bending}$ = 415,320 ft-lbs

$0.817 * \sqrt{\frac{10{,}700}{3.69 * 110 * 4}}$ = 2.10 ft Below Grade

Drilled Shafts - Compression and Uplift Capacity

When designing drilled shaft foundations for loads that consist of compression or uplift, the type of soil (cohesive or granular) will govern the design philosophy employed. The drilled shaft resistance is typically derived from the strength relationship of the cohesive or granular soil stratigraphy. Further, the depth of water will impact these design capacities.

Axial compressive loads applied to a drilled shaft are resisted by both skin friction and end bearing. Initially, as loads are applied, they are generally transferred as skin friction to the soil with increasing depth, and then redistributed from the top of the shaft down as loads increase. With increased axial compressive loads, the relative percentage of load carried at the base as end bearing increases. This load transfer mechanism is impacted by such factors as type of soil, depth of strata along shaft, base support material, water level, in situ stresses, construction methods employed, and relative size and shape of the shaft.

The generally accepted procedure for straight shafts in cohesive soils is described as the "Alpha" method and in cohesionless soils as the "K_O" method.

Cohesive (Undrained) Soil - Alpha Method

$$\mathbf{Q_T = Q_{SF} + Q_{EB}}$$

$$\mathbf{Q_{SF} = \pi * D \sum_{i=0}^{H} \frac{\alpha * c * L_i}{F.S.}}$$

$$\mathbf{Q_{EB} = \frac{\pi * D_B^2}{4} \frac{N_c * c}{F.S.}}$$

Where:

D = Diameter of shaft, in ft
DB = Diameter of shaft base, in ft
α= Alpha, shear strength reduction factor
c = Cohesion, un-drained shear strength (½ unconfined compressive strength), in psf
Li = Increments of shaft in different stratas, in ft
H = Total length of shaft, in ft
N_c = Bearing capacity factor for deep foundations
F.S. = Factor of safety

For drilled shaft foundations where the depth-to-diameter ratio exceeds approximately 4.0, the value for "Nc" for deep foundations may be assumed as equal to 9.0. At depth-to-diameter ratios less than 4.0, the "Nc" factor has to be adjusted toward shallow foundation design but will not be less than 5.14.

Because of the variability of soils, even with a comprehensive subsurface investigation, a factor of safety in compression of 2.0 to 3.0 is typically used for cohesive soils. A higher factor of safety in cohesive soils is justified to recognize the more long-term tendency of cohesive soils to experience settlement.

Cohesionless (Drained) Soil - K_O Method

$$\mathbf{Q_T = Q_{SF} + Q_{EB}}$$

$$\mathbf{Q_{SF} = \pi * D \sum_{i=0}^{H} \frac{K_o * \sigma_v * Tan(\emptyset)}{F.S.}}$$

$$\mathbf{Q_{EB} = \frac{\pi * D_B^2}{4} \frac{\gamma * H * N_q}{\gamma}}$$

Where:
D = Diameter of shaft, in ft
D_B = Diameter of shaft base, in ft
K_o = Coefficient of lateral earth pressure (generally between active and at-rest states and dependent upon shaft installation method and soil in situ density)
γ = Effective unit weight of soil, in pcf

σ_v = Effective vertical pressure at point of interest in psf
ϕ = Friction angle shaft/soil interaction (generally not in excess of angle of internal friction of soil strata)
H = Total length of shaft, in feet
N_q = Bearing capacity factor for deep foundations
F.S. = Factor of safety

The effective vertical stress in cohesionless soils for both the skin friction calculation and end bearing components are typically limited to their *critical depth*. The critical depth factor depends on soil density and generally varies between 10 to 20 times the shaft diameter. In theory, the crushing of sand grain particles and the resulting failure to achieve higher skin friction and end-bearing components occurs at depths where the shaft length exceeds these critical depth factors, and therefore the maximum effective stress has to be limited in the calculations.

Due to the relatively deep depth of drilled shafts, the bearing capacity factor Nq is generally discounted in the equation for end bearing and the relatively small percentage of overall capacity.

Consider total shaft settlement due to axial compressive loads. Settlement depends on the elastic shortening of the shaft and soil characteristics. Consolidation from uncompacted fills, landfills, organic layers, or collapsible or dispersive soils may result in unacceptable settlement or long-term negative skin-friction (down drag) of shafts. Rock formations should be evaluated to determine the mass formation properties such as weathering, joints and fractures, and clay filled cracks.

Belled shafts are frequently used where local soil conditions permit short-term stability of the bell. Where soils permit belled construction, typically cohesive soils and some shales, and total settlement is not anticipated to be excessive under higher loading, the base diameter may be used when determining the end bearing capacity. Typically, the length of the shaft occupied by the bell is discounted when determining the skin friction component.

A factor of safety in compression of 2.0 is typically used for cohesionless soils.

Uplift Capacity

For cohesive (undrained) soil the axial uplift capacity of straight shafts is derived from the skin friction or adhesion component of the shaft compressive capacity calculated as described by the cohesive equations previously covered. In addition, the effective dead weight of the shaft may be added (unfactored) to the resistance to uplift. This effective weight consists of the total weight of the shaft above the ground water level and the buoyant weight of the shaft below. Dependent upon the shaft depth-to-diameter ratio, and whether the shaft is belled, other failure surface concepts have been developed. For relatively shallow shafts or belled shafts, the methodology is to consider the shaft base to perform as an inverse bearing capacity failure without skin friction support, as either a cone-shaped or curved surface failure mode. The factor of safety in uplift of 1.5 is typically used for cohesive soils.

For cohesionless (drained) soil the axial uplift capacity of straight shafts is derived from the skin friction component of the shaft compressive capacity calculated by the "K_O" equations. A factor of safety in uplift of 1.5 is typically used for cohesionless soils. The effective weight of the shafts (total above the groundwater level and buoyant below) may be added as an unfactored component.

Spread Footings

Spread footings comprised of a vertical pier or wall seated on a square or rectangular slab located at some depth below grade have long been used in substation design. They are usually preferred for transformers, breakers, and other electrical equipment. They are economical where only a small quantity of foundations is required. They are reliable and easy to design. The installation time and costs for spread footings are more than for augered piers because of the excavation, forming, form stripping, backfilling, and compacting.

Compaction of backfill around spread footings should at least be equal to that of the undisturbed soil before the footing was installed. Spread footings should always be seated at a depth below the average frost penetration of the area. See Figure 9. The engineer should determine if the average frost depth will be sufficient for the prevailing conditions. Deeper footings may be warranted.

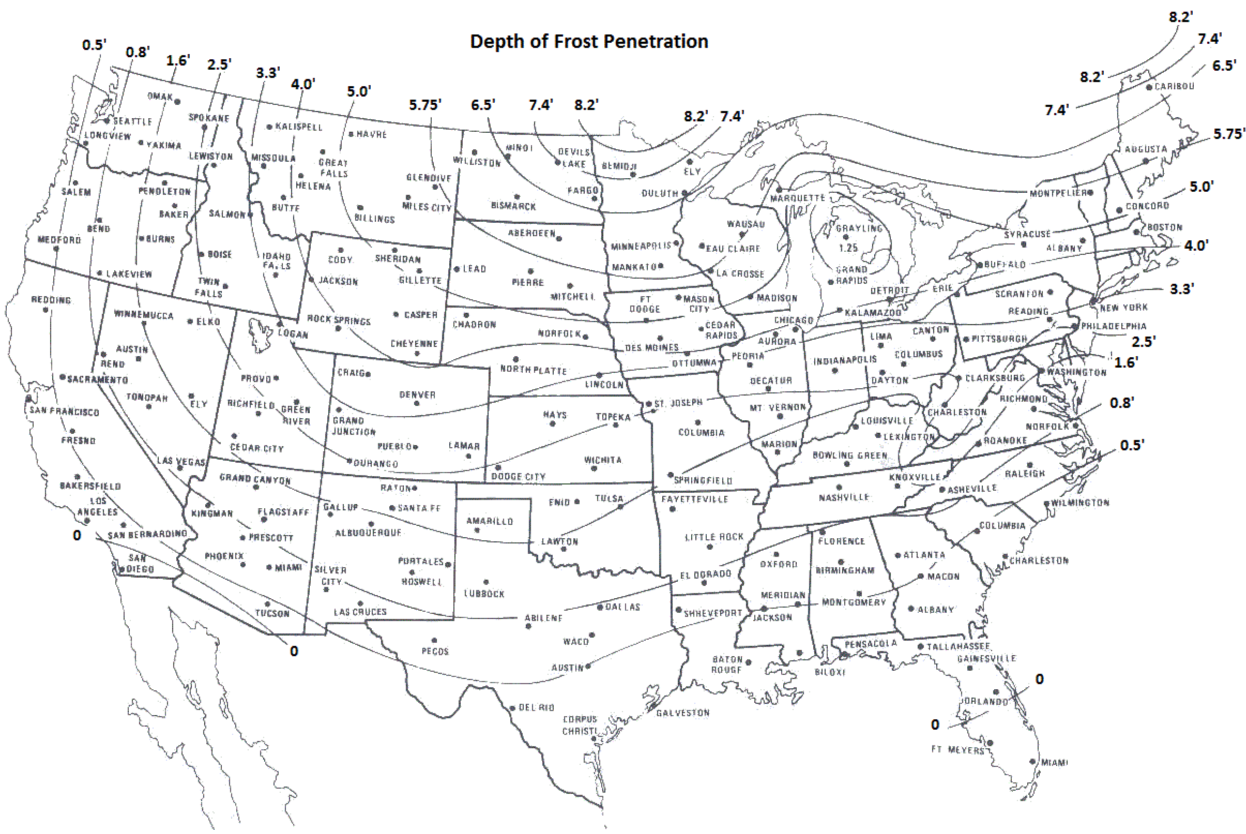


Figure 9

In designing a spread footing for only downward loads, divide the total net load by the allowable soil-bearing capacity and obtain the area of the footing required. In granular soils the water table location may have significant effect on the allowable soil capacity.

$$\mathbf{A} = \frac{\textbf{Total Net Load}}{\mathbf{P}}$$

Where:
A = Area of footing base, in ft^2
Total Net Load = LL + DL – Wt of displaced soil
LL = Maximum weight or force exerted on footing by equipment or structure
DL = Weight of foundation
P = Allowable soil bearing pressure, in psf

For the design of a spread footing for uplift only, the ultimate net uplift force (based on design loads times the appropriate *overload factor* (OLF) should be exceeded by the weight of the foundation and the weight of the soil that rests directly on the slab. Several sizes may be selected before one is obtained that will result in the desired safety factor, 1.5 minimum. In granular soils, the water table location may reduce the weight of the soil being relied upon for uplift resistance.

A spread footing subject to overturning may be designed as follows (See Figure 10):

Assume:

1. All resistance to overturning is furnished by the vertical load, weight of the concrete footing, and the weight of the soil block above the footing, the sum of which equals N.
2. The footing is rigid and tips about edge A.

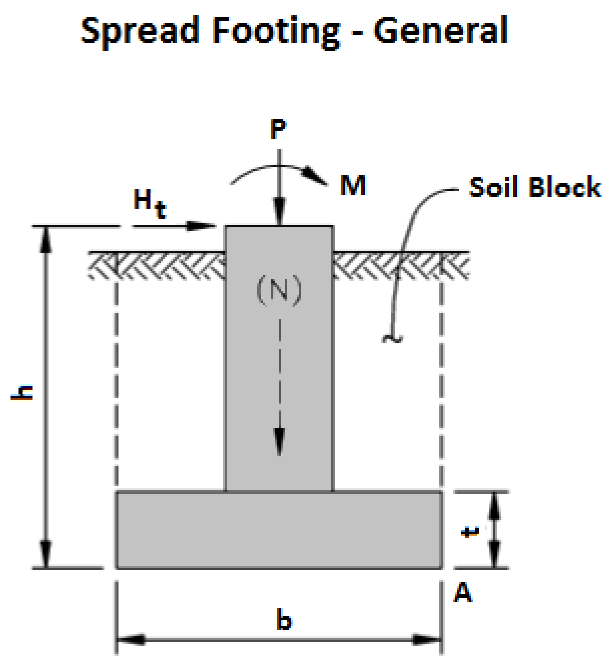


Figure 10

Overturning Moment,

$$M_{ot} = M + H_t h$$

Resisting Moment,

$$MR = \frac{Nb}{2}$$

$$F.S. = \frac{M_g}{M_{ot}}$$

(Between 1.5 and 2.0 is adequate for external stability)

$$\mathbf{e} = \frac{\mathbf{M_{OT}}}{\mathbf{N}}$$

$$\textbf{If e} < \frac{\mathbf{b}}{\mathbf{6}} \qquad \text{Then,} \qquad \mathbf{p} = \frac{\mathbf{N}}{\mathbf{A}} \pm \frac{\mathbf{M_{OT}}}{\mathbf{S}}$$

$$\textbf{If} = \ \mathbf{e} > \frac{\mathbf{b}}{\mathbf{6}} \qquad \text{Then,} \qquad \mathbf{p} = \frac{\mathbf{2N}}{\mathbf{3 * d * (\frac{b}{2} - e)}}$$

$$\mathbf{S} = \frac{\mathbf{db^2}}{\mathbf{6}}$$

Where:
e = Eccentricity measured from the centerline, in ft
M_{ot} = Overturning moment, in ft-lb
N = All resistance to overturning is furnished by the vertical load, weight of the concrete footing, and the weight of the soil block above the footing.
p = Actual soil pressure, in psf
A = Area of the footing, in ft^2
S = Section modulus of the bottom of the footing about the axis which the moment is acting
d = Width of footer, in ft
b = Length of footer, in ft

Figure 11 shows a Spread Footing with e < b/6

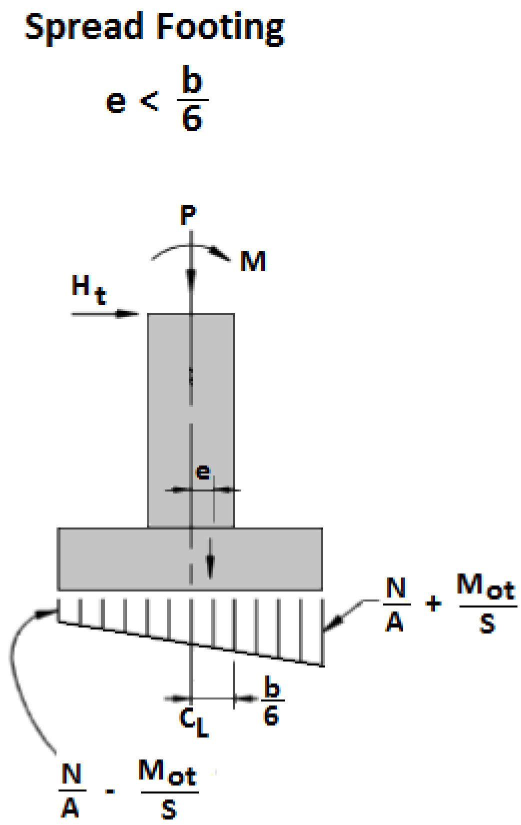


Figure 11

Figure 12 shows a spread footing with, e> b/6

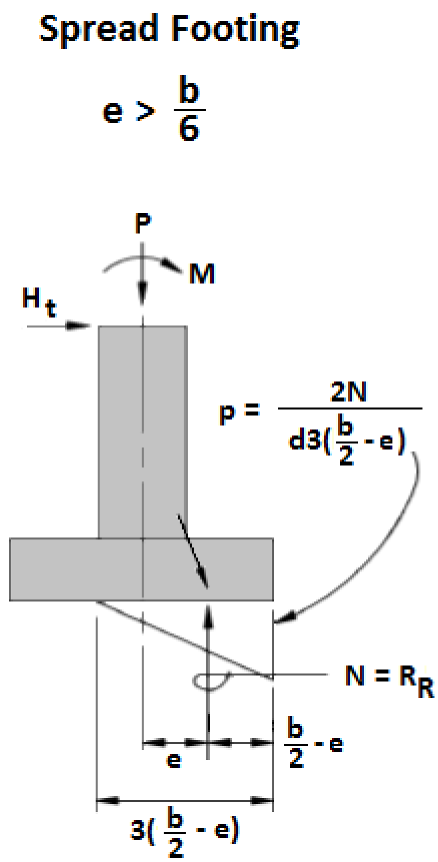


Figure 12

Moments and shears are calculated for the concrete design of the slab and pier, and conventional concrete design is employed to complete the design:

$$\mathbf{M_Z = \left(p - \frac{l * p}{3 * x}\right) * \frac{l^2}{2} + \left(\frac{l * p}{3 * x}\right) * \frac{l^2}{3}}$$

$$\mathbf{V = (p - \frac{l * p}{3 * x})^{l} + \left(\frac{l * p}{3 * x}\right) * (\frac{l}{2})}$$

Where
Mz = Bending moment about point z per unit width, in ft-lbs/ft
= Distance from toe of footer to bending point
x = Distance from toe of footer to the resultant reaction R_R
p = See Figure 12.

l = See Figure 13.

Figures 13 and 14 show a spread footing elevation view and a spread footing plan view.

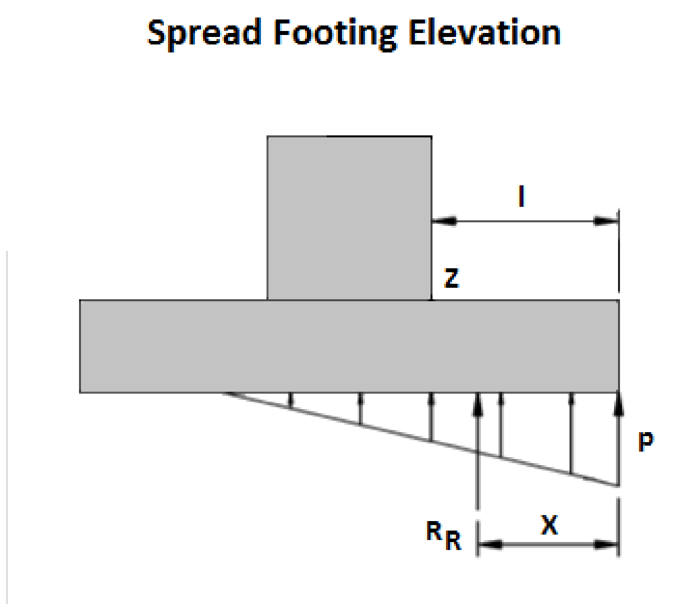


Figure 13

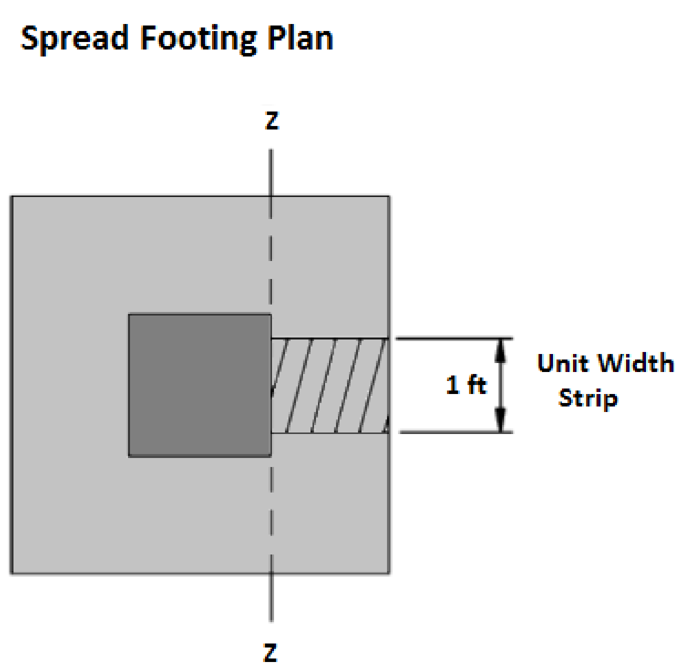


Figure 14

Here is a design example for a spreading footing. See Figure 15.

Given:

- Allowable bearing capacity of soil = 3,000 lbs/ft^2
- M = 29,500 ft-lbs
- Ht = 1,012 lbf
- P = 2500 lbf
- γ = 100 lbs/ft^3 (assumed weight of soil)
- Assume the water table is very deep and is not a factor.

Weight of pier = (2.0 ft)^2	* 5.0 ft	* 150 lbs/ft^3	=	3,000 lbs
Weight of slab = (6.5 ft)^2	* 1.0 ft	* 150 lbs/ft^3	=	6,340 lbs
Weight of soil = (6.5^2 – 2.0^2)	* 4.25	* 100 lbs/ft^3	=	16,256 lbs
Weight of structure & equipment			=	2,500 lbs
			Total	28,096 lbs

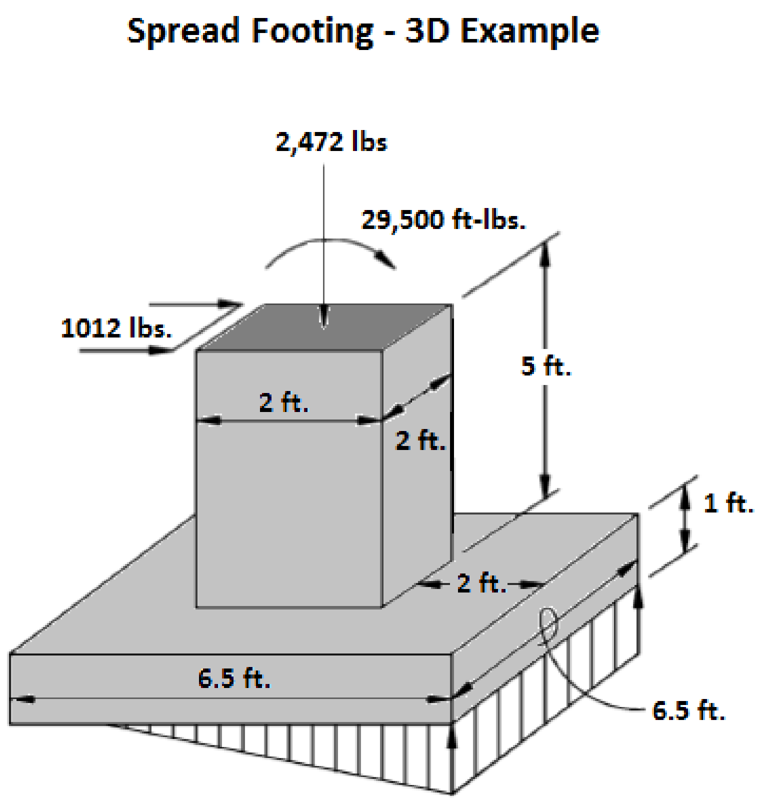


Figure 15

Check for Overturning:

M_{OT} = 29,500 + 6.0 ft * 1,012 = 35,572 ft-lbs

M_R =6.5/2 * 28,096 lbs = 91,312 ft-lbs

F.S. = 93,312 / 35,572 = 2.6

Since F.S. = 2.6 is greater than 1.5 it is okay.

Locate Resultant Force,

$$e = \frac{35{,}572}{28{,}096} = 1.27 \text{ ft}$$

b/6 = 6.5 / 6.0 = 1.08
e>b/6 since 1.27 is greater than 1.08.

So,

$$p = \frac{2}{3} * \frac{28{,}096}{(3.25 \text{ ft} - 1.27 \text{ ft}) * 6.5 \text{ ft}} = 1{,}455 \text{ lbs/ft}^2$$

$1{,}455 \text{ lbs/ft}^2 < 3{,}000 \text{ lbs/ft}^2$ so okay.

Determine Bending Moment in Slab for use in calculating reinforcing steel:

$$M_Z = \left(p - \frac{lp}{3x}\right) * \frac{l^2}{2} + \left(\frac{lp}{3x}\right) * \frac{l^2}{3}$$

$$M = \left(1{,}455 - \left(\frac{2 * 1455}{3 * (3.25 - 1.27)}\right)\right) * \frac{2^2}{2} + \left(\frac{2 * 1{,}455}{3 * (3.25 - 1.27)}\right) * \frac{2^2}{3}$$

M = 2,585 ft-lbs

Determine Bending Moment in Pier for use in calculating reinforcing steel. Assume that the pier is a vertical cantilever beam. Figure 16 shows an elevation view of the spread footing design.

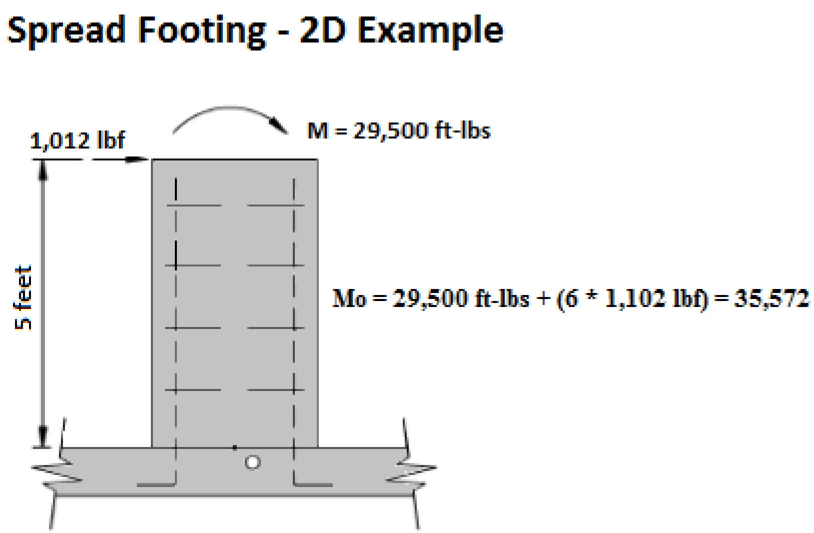


Figure 16

Slabs on Grade

Slabs on grade are sometimes used as foundations for miscellaneous equipment supports, switchgear, breakers, and power transformers. Slabs on grade should be used with caution where there is a chance of frost heave. This may cause problems with equipment that has rigid bus connections or in some other way may result in an operational malfunction of the equipment. Slabs on grade may be satisfactory in frost-prone climates if the subgrade is essentially granular and well drained.

An important part of the installation of a slab on grade is the preparation of the subgrade. The soil should be thoroughly mixed and compacted to provide a nearly homogeneous, firm bearing surface. Proper preparation may help prevent objectionable settlement. Slabs usually vary in thickness between 12 and 24 inches depending on the various design parameters. The slab should bear on the prepared subgrade and not on site stone or stone in oil retention sumps.

The types of loads (compression and overturning) that are typically present in spread footings are also present in slab-on-grade foundations for equipment. Compression loads include the equipment weight and, in soil pressure calculations, the foundation weight. In the case of transformer foundations, the equipment weight plus the weight of the contained insulating oil has to be included. For transformers, jacking loads have to be also be considered. The jacking loads are considered by the structural engineer in a check for local punching failure through the slab. Jacking loads are “point” loads generated as hydraulic jacks are used to raise and level the transformer on the slab. There are typically four jacking points, one in each corner of the transformer.

Overturning loads occur when horizontal loads act on the equipment at the top of the slab on grade. While overturning moments in a spread footing are transferred to the base slab through the concrete pedestal, overturning loads on a slab on grade are transferred through the base of the equipment. Wind loads, seismic loads, and horizontal dynamic loads are included in the list of loads that would contribute to overturning. Horizontal dynamic loads might include the circuit-tripping load from an oil-filled circuit breaker. Also, equipment eccentricity can contribute to overturning. This is a case where the center of gravity of the equipment is not in line with the centerline of the slab.

As with spread footings, slab-on-grade foundations have to be designed to not exceed the allowable soil pressure for the site. The allowable soil bearing pressure is site specific and should be determined by a geotechnical engineer following a subsurface investigation. In the absence of subsurface information, a reasonable range can be taken between 1,000 and 1,500 psf. This, however, may or may not be a conservative range for the given site. In addition to keeping the soil pressure below the slab under an allowable limit, an adequate safety factor against overturning has to be maintained. The approach is similar to that for spread footings.

Slab on Grade – Design Example

The following is an example of a slab on grade design.

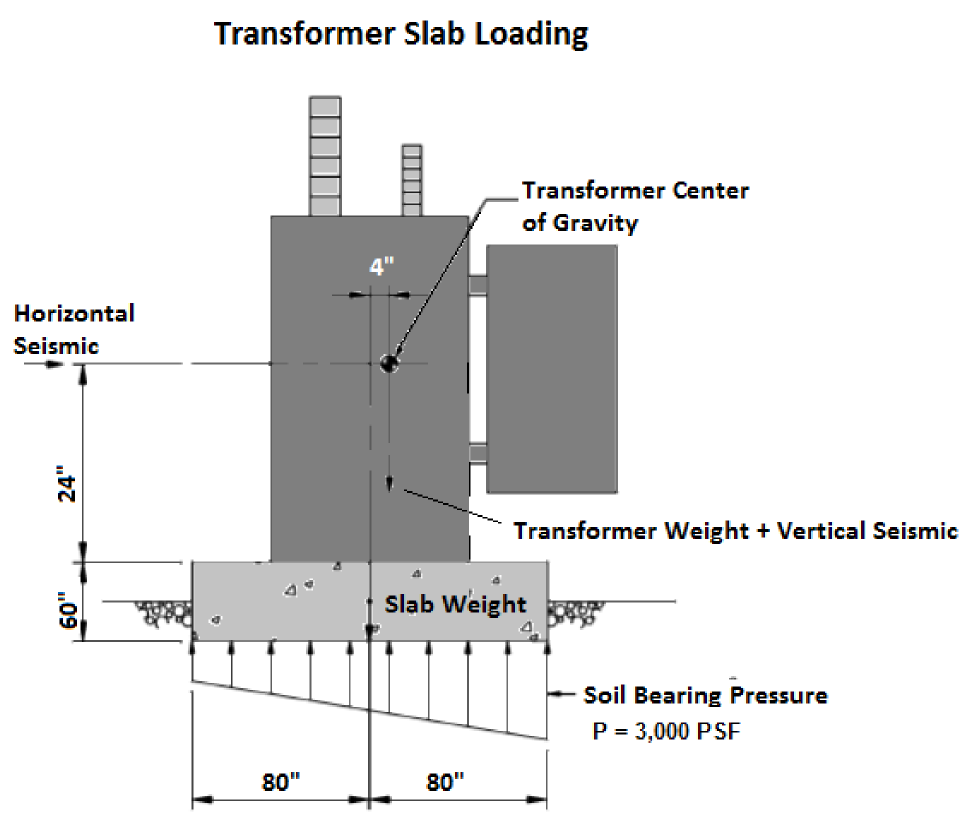


Figure 17

Check the soil bearing pressure and the factor of safety against overturning for a transformer slab based on the following assumptions:

161/13.2 kV transformer weight	245,000 lbs
Insulating Oil	128,000 lbs
Total transformer weight	373,000 lbs

- Transformer base dimensions: 19 ft x 10.5 ft
- Foundation size: 21 ft long x 13 ft wide x 2 ft thick
- The center of gravity of the transformer is 5.0 feet above the top of the foundation.
- The slab is 2.0 feet thick.
- The center of gravity of the transformer is 4 inches off the longitudinal centerline of the foundation.

Weight of foundation = 21 * 13 * 2 * 150 lbs/ft^3 = 81,900 lbs
Total vertical load, 373,000 + 81,900 = 454,900 lbs
Given allowable soil bearing pressure, P, based on soil report = 3000 lbs/ft^2

Calculate overturning moment Mot.

For a transformer in a seismically active area, seismic load usually controls rather than wind load because of the transformer's relatively large mass. For this example, assume the transformer is in seismic Zone 2A and that it has been determined that seismic loads control.

The horizontal seismic load would be calculated as follows:
Based on IEEE Std. C57.114, "IEEE Seismic Guide for Power Transformers and Reactors," for a Zone 2A region, where the weight of the transformer, W_{tr}, is 373,000 lbs,

Horizontal Load: 0.2 x W_{tr} = 74,600 lbs
Vertical Load: 2/3(0.2 x W_{tr}) = 49,700 lbs

Calculate the overturning moment due to the seismic load:

Horizontal Load: 74,600 * (5.0 ft + 2.0 ft)	= 522,200 ft-lbs
Vertical Load: 49,700 * 4"/12'	= 16,600 ft-lbs
Total seismic overturning moment	= 538,800 ft-lbs

Calculate the overturning moment due to equipment eccentricity:

Overturning moment (eccentricity) = 373,000 * 4/12 = 124,333 ft-lbs

Calculate the total overturning moment:

M_{ot} = 538,800 + 124,333 = 663,133 ft-lbs

Calculate the soil bearing pressure:

Eccentricity, e = Mot/N

e = 663,133 / 454,900 =1.45 ft

b/6 = 13 ft / 6 = 2.2 ft

e < b/6; therefore, p = N/A ± Mot/S

Slab area, A = 21 ft * 13 ft = 273 ft^2

Slab section modulus, S = 21' * 13^2 / 6 = 592 ft^3

Soil bearing pressure, p = 454,900 / 273 ± 651,267 / 592 = 1,666 lbs/ft^2 ± 1,100 lbs/ft^2

p_{max} = 2,766 lbs/ft^2 < 3000 lbs/ft^2 Okay.

Calculate the safety factor against overturning (has to be greater than 1.5):

M_{ot} = 663,133 ft-lbs

Calculate the moment that resists overturning (M_R):

Weight of transformer = 373,000 lbs
Weight of foundation = 81,900 lbs

M_R = (373,000 * 6.23 ft) + (81,900 * 6.5 ft) = 2,856,140 ft-lbs

F.S. = M_R/M_{ot} = 2,856,140 / 663,133 = 4.3 > 1.5 Okay

Foundation reinforcing steel is designed based on the American Concrete Institute Building Code Requirements for Reinforced Concrete (ACI 318). Although reinforcing steel design is beyond the scope of this chapter, a certain minimum level of reinforcing has to be maintained. ACI 318 specifies that a steel area of at least 0.0018 times the gross concrete area has to be provided. This is provided to prevent shrinkage cracks from forming as the concrete dries. For the foundation in the example above, the minimum area of reinforcing (A_s) for a unit width strip of 1 foot would be:

A_s = 0.25 * 1.00 ft * 2.0 ft = 0.012 in^2/ft

One combination that would satisfy the above minimum requirement would be:

#5 bars @ 12 inches on center, top and bottom, each way.

The cross-sectional area of a #5 bar = 0.31 in^2

For #5 top and bottom multiply by 2 bars:

A_s = 2 * 0.31 * 1 ft/ft/12" = 0.62 in^2 > 1.67 in^2 Okay.

Oil Pollution

40 CFR, Chapter 1, Parts 110 and 112, requires the containment of potential oil spills for any facility that contains oil stored underground in excess of 42,000 gallons or a capacity stored above ground in excess of 1320 gallons, or any single container with a capacity greater than 660 gallons. The regulations further require that when a situation as described above exists in a substation, and an oil spill could reasonably be expected to discharge into or upon navigable waters, that a "Spill Prevention, Control, and Countermeasure" plan (SPCC plan) has to be prepared. Detailed information concerning SPCC plan requirements and preparation are contained in 40 CFR 112.

If oil pollution abatement is necessary, the degree of reliability that is desired has to be decided. The primary function of all systems is to prevent oil from reaching prohibited areas, including the ground water table and any navigable waters. A determination has to be made as to whether the system will be self-operating or should be monitored periodically or seasonally.

<u>Basic Retention System</u>

This system should include an impervious, lined, open or stone-filled sump area around the oil containment vessel (transformer). Usually stone 2 to 3½ inches is desirable in the sump area to provide operators and maintenance personnel easy access to and around the transformer. The size and gradation of the stone affects the percentage of voids available to store oil. Stone of the size mentioned above may provide 25 to 40 percent voids. Perforated pipe placed in the bottom of the sump will convey, by gravity flow, water and oil to an underground storage tank. The tank has to have a sump pump to periodically pump out the water that has collected from rainfall. The pump may be regulated to cut on and off by a float valve or pressure switches. The transformer also should have a low-oil-level alarm that deactivates the sump pump.

At the cutoff position, there should be at least 6 inches of water covering the bottom of the tank. This enables small or slow oil leaks to be stored on top of the water. The storage tank should be designed to retain all the oil in one transformer between cutoff water level and inlet pipe. The oil is removed from the tank by pump trucks. The system is costly but reasonably reliable. Mechanical failure of the sump pump is a disadvantage. A variation of the approach discussed above is the utilization of a special oil-sensing control unit that works with a submersible drainage pump. The pump is placed in a vault outside the sump as described above. The pump discharges rainwater from the system, but is deactivated by the control unit when oil is detected.

Oil Separator Tank

This system is feasible only where there is sufficient gradient for gravity discharge from the underground tank. See Figure 18. The oil or water is collected from the transformer area as it was in the basic retention system. It is discharged into an oil separator tank. The principle upon which this system operates is that oil is lighter than water and floats upon it.

The oil separator tank should be designed to contain all the oil in one transformer should a major rupture occur. This system allows the water to continuously pass through but retains the oil. The oil retained in the tank has to be pumped into a tank truck for disposal. This system is costly but quite reliable. he oil separator tank principle may be applied to above-grade diked basins where freezing temperatures are not prevalent.

Modification to the above-grade separator system may be considered when regular inspection is anticipated. It could consist of a transformer area that is lined and diked with impervious material. A short piece of 4-inch pipe passes through the dike and contains a shutoff valve that may be left open for continuous drainage and closed in an emergency when alarms occur or inspection indicates.

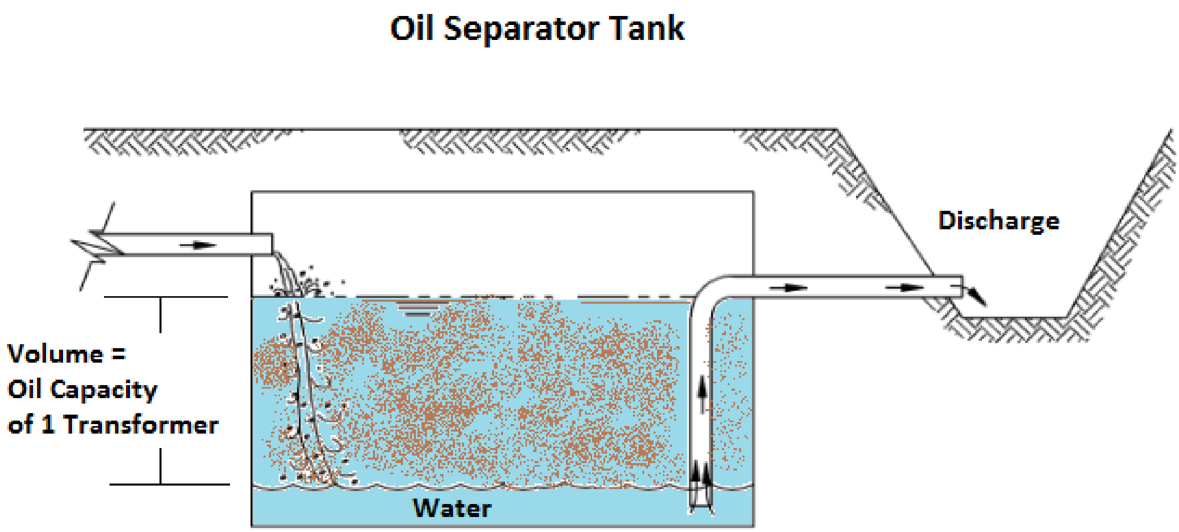


Figure 18

An alternative approach is to leave the valve closed and then open it as necessary to drain the collected water at periodic inspections. This system is relatively economical but requires a greater level of maintenance. Each solution to the oil abatement problem is not without its own problems. These should be evaluated along with system costs when deciding on the most desired system at each substation. The ultimate goal is to make sure all oil within the substation remains on site.

Chapter 16
Control Houses

As substations increase in voltage, size, and complexity, the necessity for supplemental equipment such as relays, meters, controls, batteries, communications equipment, and low-voltage distribution equipment also increases. For small distribution substations, this equipment can usually be contained in weatherproof enclosures or control cabinets. For larger substations, separate equipment housing is necessary.

A control house provides a weatherproof and, if required, environmentally controlled enclosure for supplemental substation equipment. Additional space can be provided for workshops, equipment testing and repair, storage areas, and lavatory facilities. Medium- and low-voltage switchgear can also be contained within control houses, or this equipment may be contained within weatherproof enclosures dedicated to that purpose.

Control House Construction

This chapter discusses general aspects of the control house construction. It does not attempt to cover all details of construction.

Foundation

The control house foundation typically consists of a spread footing with either masonry blocks or cast-inplace walls. The footing is designed for an allowable bearing capacity based on soil data. If soil data is not available, a maximum bearing of 1000 psf can be used. The footings are installed below frost depth and in accordance with local building codes and practices.

Drilled piers are an alternative to spread footings. Drilled piers are especially applicable for pre-engineered metal buildings with structural supporting bases that can rest directly on the piers without requiring a concrete floor slab. Soil data is necessary for determining the required depth, diameter, and reinforcing of the piers.

Damp-proofing of foundation walls is desirable, especially if concrete block is used. If a basement level is constructed, damp-proofing should be provided. Footing drains are usually provided when a basement level is constructed.

All foundation walls should be insulated with a 2-inch thickness of rigid insulation for energy conservation. It is preferable to install the insulation on the inside of the walls, although the outside is acceptable.

Floor

The control house floor is typically a floating concrete slab 5 to 6 inches thick reinforced with welded wire fabric, deformed steel bars, or a combination of both. The finished floor elevation is usually 4 to 8 inches above the finished grade outside the control house.

The base beneath the floor slab should be 4 inches of compacted sand or gravel, thoroughly mixed and compacted sand or gravel, or thoroughly mixed and compacted natural soil. 0.006-inch thick plastic film vapor barrier should be installed between the floor slab and the base.

The method for cable routing in the control house has to be considered before finalization of the floor slab design. Cable trenches can be formed into the floor slab, or false floors can be installed providing access to large areas below the finished floor.

Superstructure

The control house superstructure should be constructed from fire-resistant, low-maintenance building materials. Most control houses presently being designed and constructed are of the pre-engineered metal or masonry block type.

The pre-engineered metal building is the easiest to procure and erect. The manufacturer can design and fabricate the required building components when given the building size; wind, snow, and ice loads; and any special requirements such as additional roof loads for suspended cable trays or other equipment. Masonry buildings constructed of block masonry are most economical when masonry module dimensions are used to size the building and the building openings. Decorative block can be used as an inexpensive method to improve external appearance. Block cores should be filled with vermiculite or equivalent insulation.

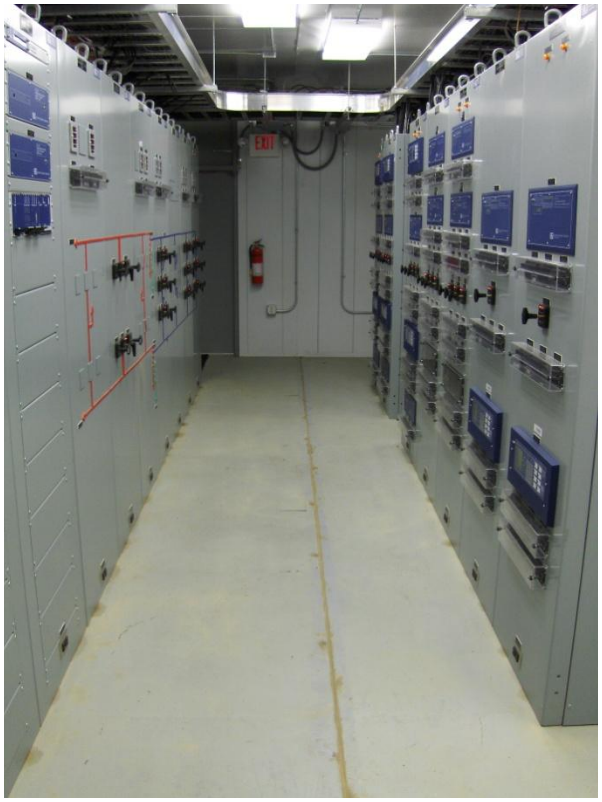
Two types of roof systems are commonly used for masonry buildings: precast, prestressed concrete panels; and steel joists and steel decks. A sloping roof is recommended for both systems and can be obtained by pitching the roof deck or installing tapered roof insulation. The roof membrane has to be compatible with the slope. For the slopes of one inch per foot and less, built-up pitch and slab is commonly used. For greater slopes, gravel is used.

The control house should be equipped with at least one double door, possibly with a removable transom, conveniently located to facilitate equipment entry and removal. In certain circumstances a second exit needs to be installed in the control house. If the plan of the room or space and the character and arrangement of equipment are such that an accident would be likely to close or make inaccessible a single exit, a second exit must be provided. The doors should include locking devices, a method to prevent water from entering, and adequate weatherstripping and hardware to permit a rapid exit from the control house.

The battery area needs to be adequately ventilated, either by a natural or powered ventilation system, to limit accumulation of hydrogen gas to less than an explosive mixture. A powered ventilation system needs to be annunciated to indicate ventilation failure. Portable or stationary water facilities or a neutralizing agent for rinsing eyes and skin in the battery area in addition to proper eye protection and clothing should be provided as well as adequate fire-extinguishing equipment in the control house.

Windows can be provided, if desired, in office and lavatory areas. Battery rooms and control and metering areas do not need windows. Consider adequate methods for building insulation. These methods include use of insulated wall panels, ceiling insulation, storm doors, and windows, and weatherstripping around all openings.

Metal buildings are shop painted and require only minor field touch-up after erection. Masonry buildings may be left unpainted or may be painted with portland cement or latex paint. Tint all prime coats to match the finished coat.



Control House Layout

<u>Control and Relay Panels</u>

Most relaying, metering, and control equipment is mounted on fabricated control and relay panels installed within the control house. A variety of panel types is available to suit individual requirements. A typical control house relay panel is shown in the photo on the right.

Single vertical panels can be used, particularly for distribution circuits where space requirements are minimal. The relaying, metering, and control equipment can all be mounted on one panel, allocating a separate panel for each circuit. In some instances, two circuits may share the same panel.

Double or duplex panels are commonly used for higher voltage circuits, necessitating additional space for equipment mounting. Normally, these panels are arranged in two parallel rows with the panel backs facing each other. In this configuration, operating, instrumentation, and control equipment for a circuit is installed on the front of one panel, and the corresponding relaying equipment for the same circuit is installed on the front of the panel directly to the rear. In some instances, two circuits may share the same control and relaying panels.

Some equipment such as static relaying systems and communications equipment is available mounted in racks. Consequently, separate relay and/or control panels are not required for this equipment. Modern SCADA and substation automation schemes may require space for installation of a PC with monitor and keyboard, as well as programmable logic controllers and data highway interface modules. This equipment can often be rack-mounted or installed in control panels, as appropriate.

The trend is toward more compact equipment arrangements that often reduce overall control house size. Individual three-phase microprocessor relays can replace three single-phase electromechanical relays and associated voltage, current, and power meters, all in one case. Compact relay and programmable logic controller designs can be mounted on 19-inch racks.

To facilitate operation, panels are located in an arrangement that conforms as closely as possible to the actual equipment and circuit layout in the substation yard. To assist in circuit location and operation, mimic buses are sometimes used on the control panels, particularly for large complex substations. The mimic buses identify the bus and circuit arrangements. Mimic buses may be implemented on screens viewed from a computer monitor. When practical, position meters at eye level and switches at a convenient operating level. Locate recording meters for ease of viewing and chart replacement. Locate relays beginning at the tops of the relay panels and working downward. Relays with glass covers should not be located within 12 inches of the floor to avoid inadvertent breaking of the glass. Locate operating switches at convenient heights near the center of the boards. Require nameplates for all devices. Provide ample space for relay installation, removal, operation, and testing.

Panel construction can include removable front plates for device mounting. Panels may also include 19-inch rack mounting facilities. Many of the newer relays and items of accessory equipment are designed to fit into 19-inch racks. Cover plates may cover space reserved for future use. In this way, only a new predrilled plate is required when changing out a device or modifying the configuration. Cutting, drilling, or covering openings in the panels is eliminated.

Panel wiring is accomplished on the backs of the panels. Devices are interconnected and wired to terminal blocks, as required, for operation and connection to devices on other panels. Panels can include small sections perpendicular to the main section at each end for installation of terminal blocks, fuse blocks, or small auxiliary devices. Cable connections from the equipment in the substation yard can be made directly to terminal blocks mounted on the panels or to strategically placed terminal cabinets. Interconnections between the terminal cabinets and the panels can then be made with single conductor wire.

Anchor panels to the floor in such a way as to facilitate relocation to coincide with yard equipment and circuit relocations. Panel arrangement in the control house should permit ready accessibility to the backs of the panels.

Some vendors of pre-engineered buildings can provide completely wired and tested control and relay panels and auxiliary AC/DC power systems as part of the building package. In this case, custom-designed relay and control schematics are submitted to the building vendor. The building vendor fabricates the panels, provides the relays and controls, wires the panels, and tests the complete installation. In this way, the entire panel line-up can be witness-tested in the factory. The complete building system is shipped to the site, fully tested. Only the external wiring from the building to the outdoor equipment has to be field-installed.

DC Equipment

Substation DC equipment located in the control house normally consists of the battery, battery charger, monitoring and control devices, and distribution panelboard. The battery should be located in a separate room where practical. If the battery cannot be located in a separate room, it should be located so that electrical switching devices and receptacles are not in the immediate vicinity, ventilation is adequate to prevent gas accumulation, and live parts are protected from accidental contact. The battery charger, monitoring and control devices, and distribution panelboard are normally located in the control and relay room to facilitate cable routing and equipment maintenance.

AC Equipment

A low-voltage AC system is provided in the substation for lighting; convenience outlets; heating, ventilating, and air conditioning (HVAC) equipment; and miscellaneous control functions. Strategically locate convenience outlets throughout the control house to provide adequate accessibility. Provide the workshop and testing area with a high-capacity AC source and a three-phase source. An AC distribution panelboard located inside the control house is used to supply the indoor lights, convenience outlets, HVAC equipment, and other devices. For greater reliability, two separate sources may be provided for the AC system service. These sources are often fed through a manual or automatic transfer switch so that AC system power can be restored if one source fails.

Cableways

Cable routing can be accomplished by using any of several methods. Cable trenches are formed into the concrete floor slab and are covered with metal plates. The covers should be flush with the finished floor when in place. The sizes and locations of the cable trenches are based on the quantities of cables and locations of panels and equipment to be interconnected. Usually, a cable trench is located adjacent to the backs of the control and relay panels to facilitate panel interconnections. With duplex panels, it may be desirable to use the entire space between the front and rear panels as cable trench, depending on circuit quantities.

False floors are useful when large open areas are desirable for cable routing. Lightweight removable floor panels installed on adjustable pedestals are positioned in areas requiring extensive cable interconnections or where future plans dictate a large amount of cable rerouting. The top of the removable panels should be flush with the finished floor.

When cables are mounted under false floors, establish routes and paths in which cables should be routed. This will allow the separation of circuits as required to maintain system reliability based on duplicate circuits. If circuits in one area are damaged, other undamaged circuits in the other parts of the building are likely to keep the substation in service.

Conduits can be used for cable routing in floors, along walls, and for cable entrance into the control house. Conduits are available in plastic, aluminum, and steel. Each of these types may be used in control houses for wire containment to convenience outlets, lighting fixtures, and other control house auxiliary power equipment. Plastic conduit is easily installed and is available in a variety of sizes. Take adequate physical and thermal precautions when using plastic conduit to ensure safe operation.

Metallic conduits of aluminum and steel are widely used as control house cableways. Intermediate- and heavy-walled steel conduit provide excellent physical protection. The installed costs, however, may be relatively high because of the extensive labor required for installation. The installed cost of rigid aluminum conduit may be somewhat less than that for steel. A lower installed cost may be realized by using thin-walled steel conduit (i.e., electrical metallic tubing) since it is less expensive and easier to install.

Wireways are sheet-metal troughs used for routing groups of power circuits around a control house to feed various branch circuits. Conduit is used between the wireway and the devices. Wireway offers the advantage of laying rather than pulling the cable into position and the ability to change or reroute circuits easily. Wireway is available with hinged or removable covers in a variety of lengths and sizes. Select and install wireway in accordance with the National Electrical Code.

Cable trays can be used for overhead routing of cables to and between ontrol and relay panels. Expanded metal or ladder-type trays provide the best facilities for conductors entering and leaving the trays. An advantage of cable trays is the ability to lay rather than pull in the conductors. Suspended cable trays, however, prevent extensive use of this technique because of support locations. A large variety of types, sizes, and fittings is available to suit individual requirements. Cable tray should be selected and installed in accordance with the NEC and NEMA Standards.

Cable Entrance

Control and power cables are brought into the control house through windows, sleeves, or cable pits. The windows are square or rectangular openings, usually through the foundation wall but possibly above grade. The window openings enable many cables to be pulled without interference. To protect the cables during pulling, the windows should have smooth surfaces and beveled or rounded edges. After cable pulling, split sleeves can be installed around the cables and grouted into place. Occasionally, the windows are left open to facilitate future cable installation. Heat loss through these openings should be considered. Provide additional windows for installation of future cables. The windows can be constructed and bricked up to be opened when required.

Cable sleeves can be used above or below grade. The sleeves are usually cast into place during construction of the foundation wall or installed during construction of the superstructure. Pitch the sleeves to drain out of the building. Provide covers over the cables. Install spare sleeves during initial construction.

Cable pits may be cast-in-place concrete or masonry openings through the control house foundation to permit access to the inside at floor level. Install a cover over the pit and provide a means to drain water.

Lighting

Fluorescent lamps are generally used for lighting in control houses. The trend is toward energy-saving lamps and energy-saving electromagnetic or electronic ballasts. Install lighting to eliminate, as much as possible, reflection and glare from meters, relays, and monitoring screens.

An emergency DC-operated incandescent system is recommended for most control houses. This system can be operated in case of failure of the AC system. It can be operated from battery-pack units or from the station battery system.

Control House HVAC Systems

To maintain the functions and accuracy of electrical equipment installed in the control house, HVAC systems may be desirable.

In areas requiring heat only, unit electric space heaters are positioned throughout the control house for balanced heating. If both heating and cooling are required, electric heat pumps can be used. Several small units, or one large unit with a duct system for air distribution, can be used. Supplemental electric resistance heating coils may be required for heating in colder areas.

Baseboard radiation heating units can be used in rooms not reached by the main heating system. These rooms include offices, lavatories, and storage rooms. The battery room is sometimes left unheated; however, maintenance of battery temperature close to 32C will prolong the life and capacity of most battery systems.

Temperature control levels may vary because of several requirements. Operating ranges of equipment have to be considered as well as economics. It is recommended that consideration be given to a dual control. Most stations will be unattended and, therefore, a normal personal comfort level is not required. However, for maintenance reasons, comfort levels are necessary.

The system then would normally maintain a minimum level based on equipment requirements with controls designed so that one or more additional units are used to raise the temperature to a comfort level suitable for maintenance personnel. In smaller control houses, this may be accomplished during maintenance periods by raising the thermostat controlling a small system.

If the control house is to be heated only, it is usually desirable to install power ventilation equipment for air circulation. Size the system for three to five air changes per hour. Place power-operated, thermostatically controlled roof ventilators and manually operated wall louvers to achieve good air circulation. Position wall louvers so that equipment does not interfere with air circulation. Provide fusible links to close the louvers in case of fire.

It is advisable to provide ventilation that will maintain a positive pressure within the control house at all times to prevent dust from entering through doors and other openings, and prevent accumulation of combustible gases. If control house air conditioning is used to provide positive air pressure, then the vent should remain open and fan should run continuously. This also applies if the unit is a heat pump. The isolated battery room should be equipped with a gravity roof ventilator to remove corrosive and combustible gases. Do not use power-operated roof ventilators.

Control House Plumbing

Control houses may require plumbing for stationary eyewash facilities. Additionally, very large, major locations may warrant a shower, lavatory, drinking fountain, and maintenance sink.

A water supply, when required, may be obtained from an existing system or a private well on the substation site. A well should be capable of supplying 5 gpm at a minimum pressure of 25 psi. Most substations with toilet facilities will require septic tank and drain field systems. These systems are designed and installed in accordance with local codes and regulations, obtainable through county health departments.

Separation between a water well and a septic tank and drain field should be in accordance with local ordinances. In the absence of such information, provide a minimum of 50 feet.

Communications

A commercial telephone is usually installed in the control house for external communications. Additionally, system telephones or voice channels over carrier systems may be used for system communications.

Larger installations may include substation automation systems or SCADA for remote control and monitoring of substation equipment.

Summary

This course has covered design factors related to the substation site. The objective of site work design for a substation yard is to provide an easily accessible, dry, maintenance-free area for the installation and operation of electrical substation equipment and structures.

The course also covered the design issues for substation foundations. Foundation design primarily depends on the in-place density and strength/strain properties of the soil on or in which foundations are located. The course reviewed the general considerations for the design and construction of a substation control house.

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