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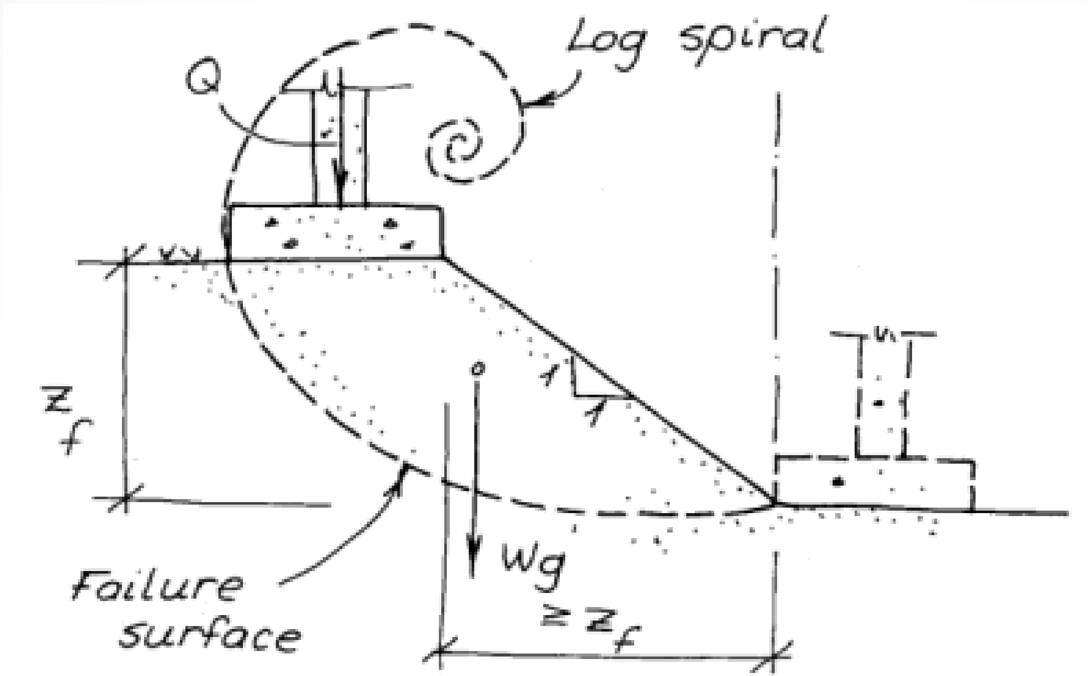
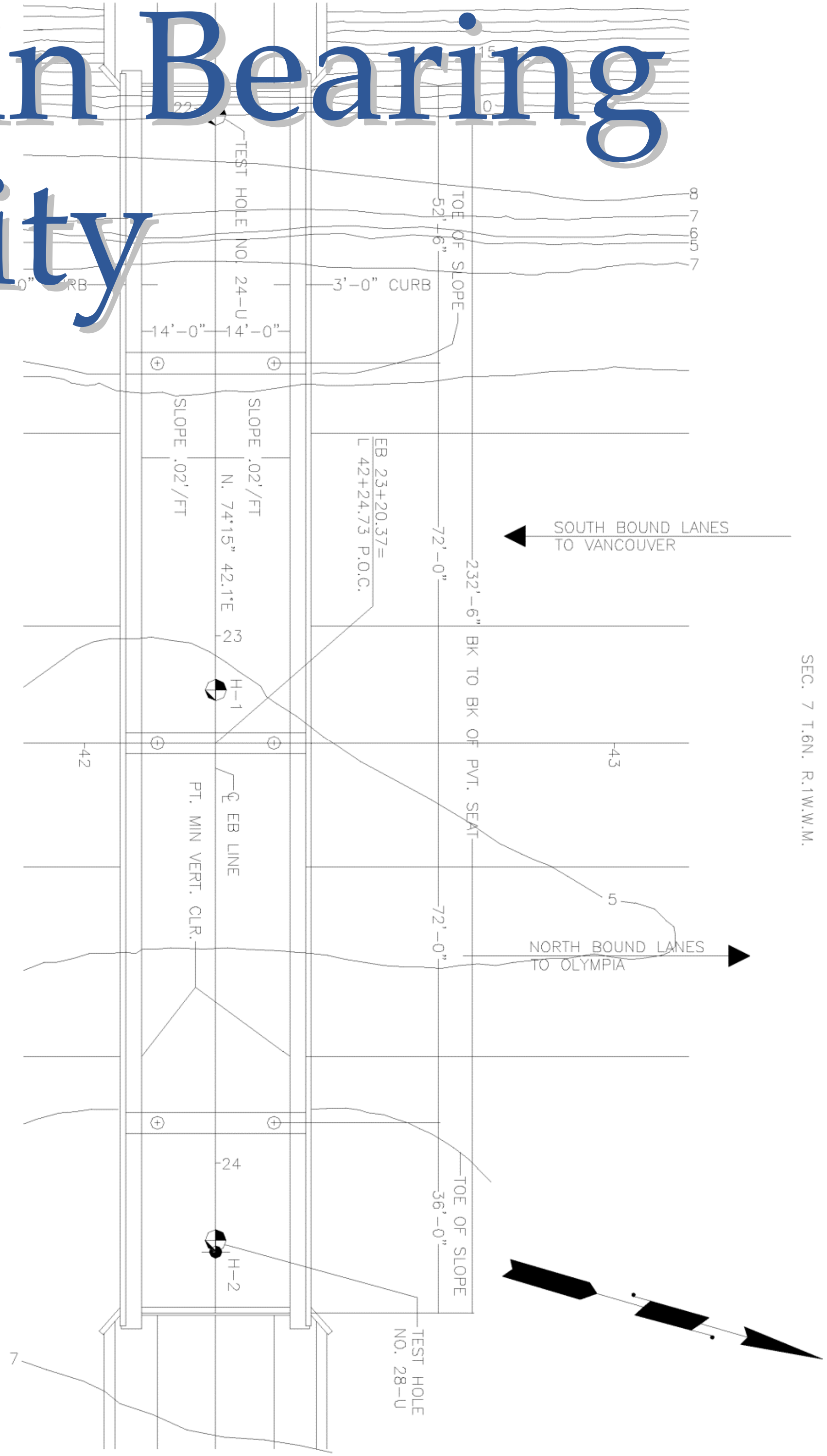


ENCE 4610
Foundation Analysis and Design

Lecture 4
Bearing Capacity
Other Topics

Other Topics in Bearing Capacity

- Bearing Capacity from Field Tests
  - SPT
  - CPT
- Bearing Capacity for Foundations on Top of a Slope
- Presumptive Bearing Capacities for Soil and Rock
- Foundations on Rock



Use of SPT and CPT Methods to Determine Bearing Capacity

- Approach I: Use SPT and CPT correlations (such as we discussed in 3610) and determine soil properties (γ , ϕ , c) and then apply to bearing capacity equations
  - We can use this for both SPT and CPT tests
  - For SPT tests, it is the principal way we will use
- Approach II: Use a "direct" approach
  - For CPT methods, we also have the option of using the CPT results to directly estimate the bearing capacity of a foundation
  - Fellenius gives an extensive review of these methods
  - DM 7.02 features one of these methods

Correction of SPT Results

The measured N-value is the number of blows required to drive the split spoon sampler a distance of 300 mm. The efficiency of the system can be obtained by comparing the kinetic energy, KE, (i.e., KE=½mv^2), with the potential energy, PE, of the system, (i.e., PE=mgh). The energy ratio (ER) is defined as KE/PE. For routine engineering practice in the United States, correlations for engineering properties are based on SPT N values measured based on a system which is 60 percent efficient, i.e., ER=60 percent. The N values corresponding to 60 percent efficiency are termed N_{60}. Numerous correction factors to the measured N-value are necessary because of energy inefficiencies and procedural variation in practice. When all factors are applied to the field recorded N-value (N_{meas}), the corrected value is calculated as:

$$N_{60} = N_{meas} C_E C_B C_S C_R \qquad \text{(Equation 3)}$$

where correction factors are presented in table 9 and include the effects of energy (C_E), borehole

the correction term for energy, i.e., C_E, vary over a relatively wide range. For this reason, accurate estimates of C_E are more important than estimates of the other correction factors. More accurate estimates of C_E should be evaluated by directly measuring the energy ratio (ER) of a particular SPT setup according to procedures in ASTM D 4633. Commercially available equipment can be used to perform this calibration. Hammer systems used for standard penetration testing should be periodically calibrated using the procedures outlined in ASTM D 4633.

SPT Efficiency Correction Factors (without overburden correction)

Table 9. Corrections to the SPT (after Skempton, 1986).

Factor	Equipment Variable	Term	Correction
Energy Ratio	Donut Hammer Safety Hammer Automatic Hammer	$C_E = ER/60$	0.5 to 1.0[(1)] 0.7 to 1.2[(1)] 0.8 to 1.5[(1)]
Borehole Diameter	65 to 115 mm 150 mm 200 mm	C_B	1.0 1.05 1.15
Sampling method	Standard sampler Non-standard sampler	C_S	1.0 1.1 to 1.3
Rod Length	3 to 4 m 4 to 6 m 6 to 10 m 10 to >30 m	C_R	0.75 0.85 0.95 1.0

[1] Values presented are for guidance only. Actual ER values should be measured per ASTM D 4633

SPT Correction Example (without overburden)

- Given
  - SPT reading 20m below ground surface
  - Readings: 10 – 15 – 12
  - Using Automatic Hammer, 80% efficiency
  - Borehole diameter = 100 mm
  - Standard Sampler
  - Rod length at least distance below ground surface
- Find
  - Corrected N_{60} SPT reading

Solution

N-value = sum of last two 6" blow counts = 15 + 12 = 27

Correction Factors

E_f = 80, for automatic hammer (efficiency of the hammer=80%), so C_E=80/60=1.333

All other correction factors C_R=C_S=C_B=1

N_{60} = (27)(1.333)(1)(1)(1) = 36 blows/ft (or blows/300 mm)

Correction for Overburden

Since N-values of similar materials increase with increasing effective overburden stress, the corrected blowcount (N_{60}) is often normalized to 1-atmosphere (or about 100 kPa) effective overburden stress using overburden normalization schemes. The normalized corrected blowcount is referred to as $(N_1)_{60}$, and is equal to:

$$(N_1)_{60} = C_N \, N_{60} \qquad \text{(Equation 4)}$$

where C_N is the stress normalization parameter calculated as:

$$C_N = (P_a/\sigma_{vo}')^n \qquad \text{(Equation 5)}$$

where P_a is atmospheric pressure in the same units as σ_{vo}', and n is a stress exponent typically equal to 1 in clays (e.g., Olsen, 1997; Mayne & Kemper, 1988) and 0.5 to 0.6 in sands (e.g., Seed et al., 1983; Liao & Whitman, 1986; Olsen, 1997). Figure 11 illustrates a correlation for C_N that is used for sands. There exist several soil specific correlations for C_N, as reported in the literature. C_N may vary slightly from the general values identified previously, depending upon the specific correlation.

SPT Correction for Overburden

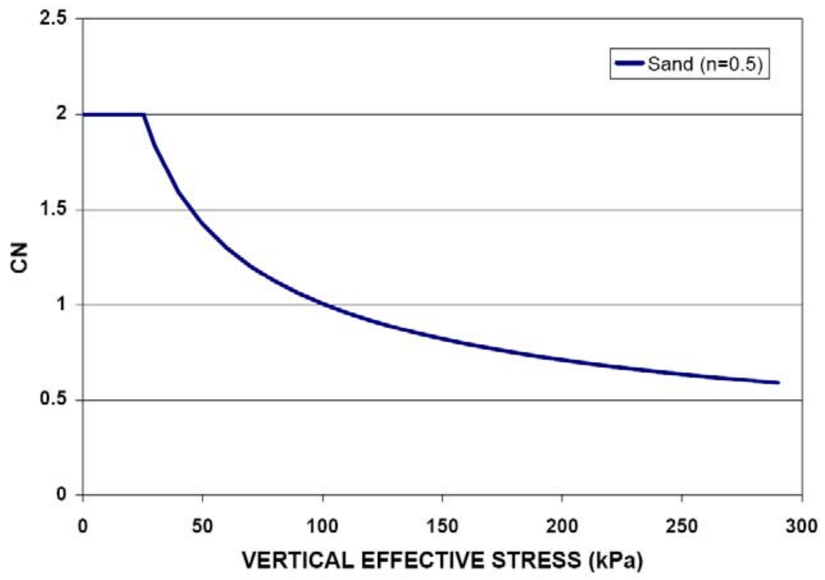


Figure 11. Stress normalization parameter, C_N, for sands.

- Applied after the other correction factors
- Only applied to an N_{60} SPT result
- Example
  - $N_{60} = 20$
  - $\sigma'_z = 2.5$ ksf

$$
\begin{aligned}
(N_1)_{60} &= N_{cor} = C_N N_{60} \\
C_N &= \sqrt{\frac{2}{\sigma'_z}} \leq 2 \; (U.S.Units,\ ksf) \\
C_N &= \sqrt{\frac{100}{\sigma'_z}} \leq 2 \; (SI\,Units,\ kPa)
\end{aligned}
$$

$$
\begin{aligned}
C_N &= \sqrt{\frac{2}{\sigma'_z}} = \sqrt{\frac{2}{2.5}} = 0.89 \leq 2 \; (U.S.Units,\ ksf) \\
(N_1)_{60} &= C_N N_{60} = (0.89)(20) = 17.9\,{}^{blows}/{}_{ft} \Rightarrow 18\,{}^{blows}/{}_{ft}
\end{aligned}
$$

Relative Density from SPT Test

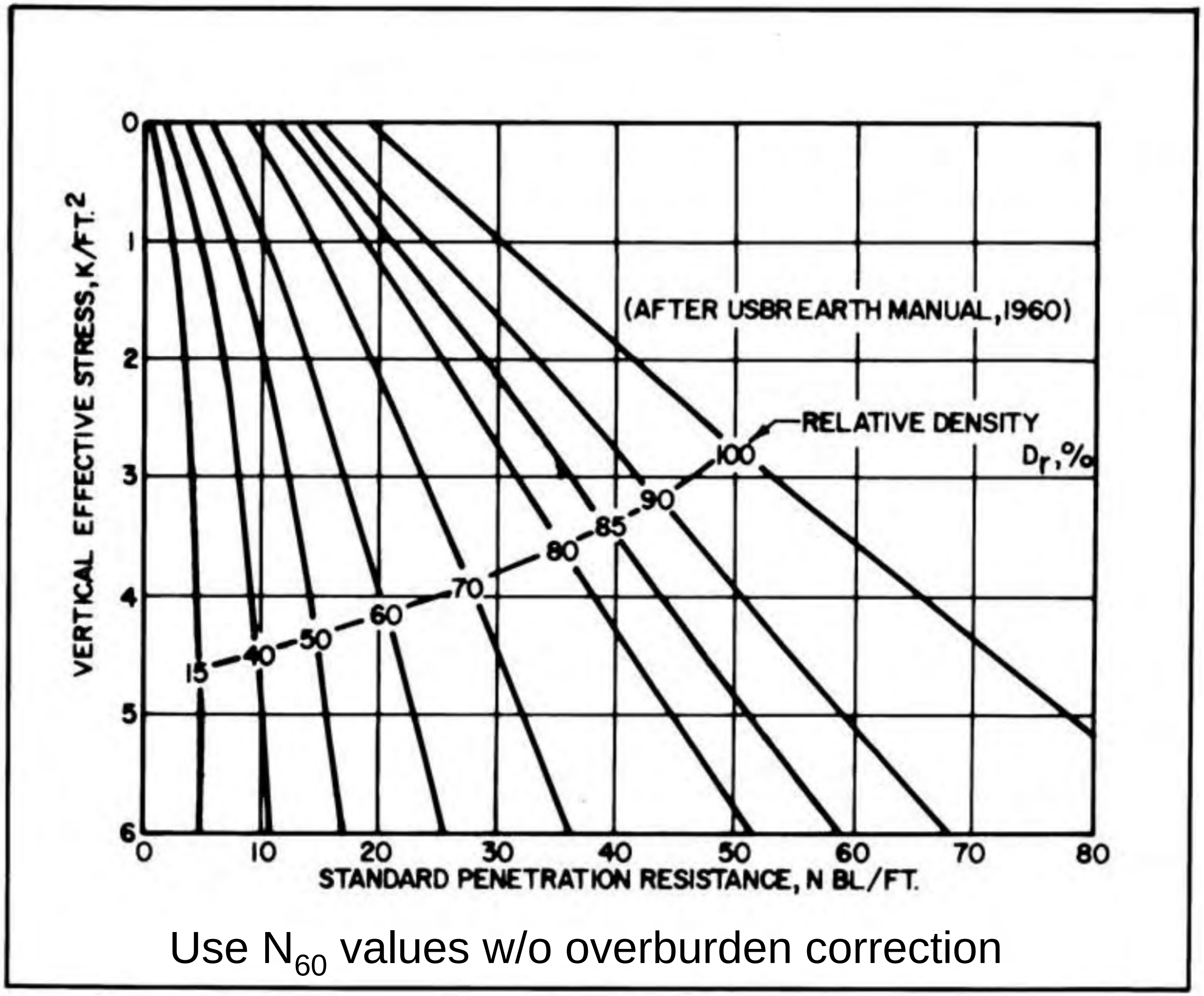


FIGURE 3
Correlations Between Relative Density and Standard Penetration Resistance in Accordance with Gibbs and Holtz

Properties from Relative Density and Classification

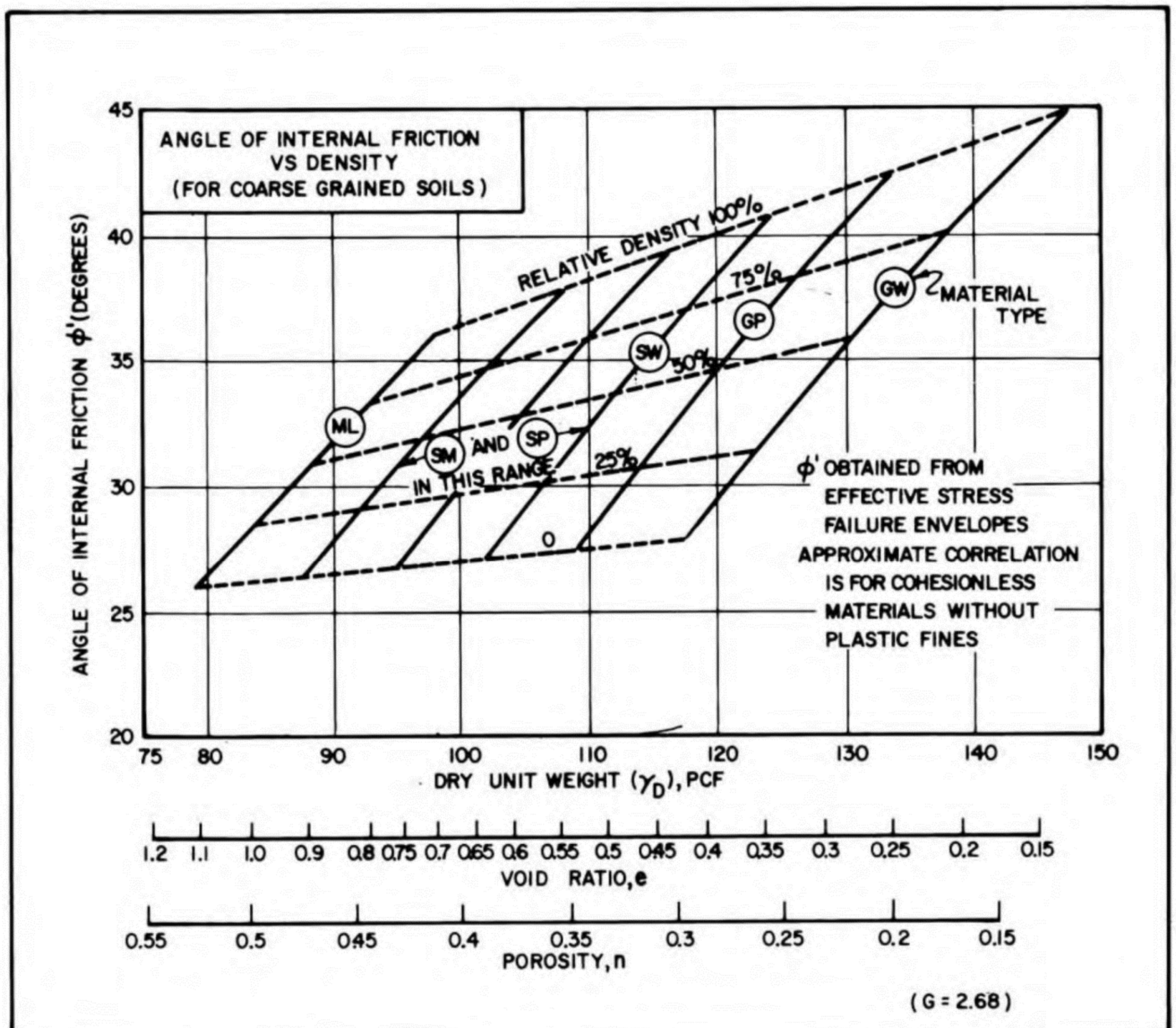


FIGURE 7
Correlations of Strength Characteristics for Granular Soils

Table 8-1

Estimation of friction angle of cohesionless soils from Standard Penetration Tests (after AASHTO, 2004 with 2006 Interims; FHWA, 2002c)

Description	Very Loose	Loose	Medium	Dense	Very Dense
Corrected SPT $N1_{60}$	0	4	10	30	50
Approximate ϕ, degrees*	25 – 30	27 – 32	30 – 35	35 – 40	38 – 43
Approximate moist unit weight, (γ) pcf*	70 – 100	90 – 115	110 – 130	120 – 140	130 – 150

* Use larger values for granular material with 5% or less fine sand and silt.

Note: Correlations may be unreliable in gravelly soils due to sampling difficulties with split-spoon sampler as discussed in Chapter 3.

TABLE 4-7 EMPIRICAL VALUES FOR UNCONFINED COMPRESSIVE STRENGTH (q_u) AND CONSISTENCY OF COHESIVE SOILS BASED ON UNCORRECTED N

(after Bowles, 1977)

Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
q_u, kPa (ksf)	0 – 24 (0 – 0.5)	24 – 48 (0.5 – 1.0)	48 – 96 (1.0 – 2.0)	96 – 192 (2.0 – 4.0)	192 – 384 (4.0 – 8.0)	384+ (8.0+)
Standard Penetration N value	0 - 2	2 - 4	4 – 8	8 - 16	16 - 32	32+
γ (saturated), kN/m^3 (lb/ft^3)	15.8 - 18.8 (100 – 120)	15.8 - 18.8 (100 – 120)	17.3 - 20.4 (110 – 130)	18.8 - 22.0 (120 – 140)	18.8 - 22.0 (120 – 140)	18.8 - 22.0 (120 – 140)

The undrained shear strength is 1/2 of the unconfined compressive strength.

Correlations are unreliable. Use for preliminary estimates only.

Bearing Capacity for Foundation at Top of a Slope

8.4.3.6 Sloping Ground Surface

Placement of footings on or adjacent to slopes requires that the designer perform calculations to ensure that both the bearing capacity and the overall slope stability are acceptable. The bearing capacity equation should include corrections recommended by AASHTO as adapted from NAVFAC (1986b) to design the footings. Calculation of overall (global) stability is discussed in Chapter 6.

For sloping ground surface, Equation 8-6 is modified to include terms N_{cq} and $N_{\gamma q}$ that replace the N_c and N_γ terms. The modified version is given by Equation 8-10. There is no surcharge term in Equation 8-10 because the surcharge effect on the slope side of the footing is ignored.

$$q_{ult} = c(N_{cq})s_c b_c + 0.5\gamma B_f (N_{\gamma q}) C_{w\gamma} s_\gamma b_\gamma \qquad \text{8-10}$$

Charts are provided in Figure 8-18 to determine N_{cq} and $N_{\gamma q}$ for footings on (Figure 8-18a) or close to (Figure 8-18d) slopes for cohesive ($\phi = 0^\circ$) and cohesionless ($c = 0$) soils. As indicated in Figure 8-18d, the bearing capacity is independent of the slope angle if the footing is located beyond a distance, 'b,' of two to six times the foundation width, i.e., the situation is identical to the case of horizontal ground surface.

Other forms of Equation 8-10 are available for cohesive soils ($\phi = 0^\circ$). However, because footings located on or near slopes consisting of cohesive soils, they are likely to have design limitations due to either settlement or slope stability, or both, the presentation of these equations is omitted here. The reader is referred to NAVFAC (1986a, 1986b) for discussions of these equations and their applications and limitations.

Equation 8-10, which includes the width term for cohesionless soils, is useful in designing footings constructed within bridge approach fills. In this case, obtain $N_{\gamma q}$ from Figure 8-18(c) or 8-18(f) and then compute the ultimate bearing capacity by using Equation 8-10.

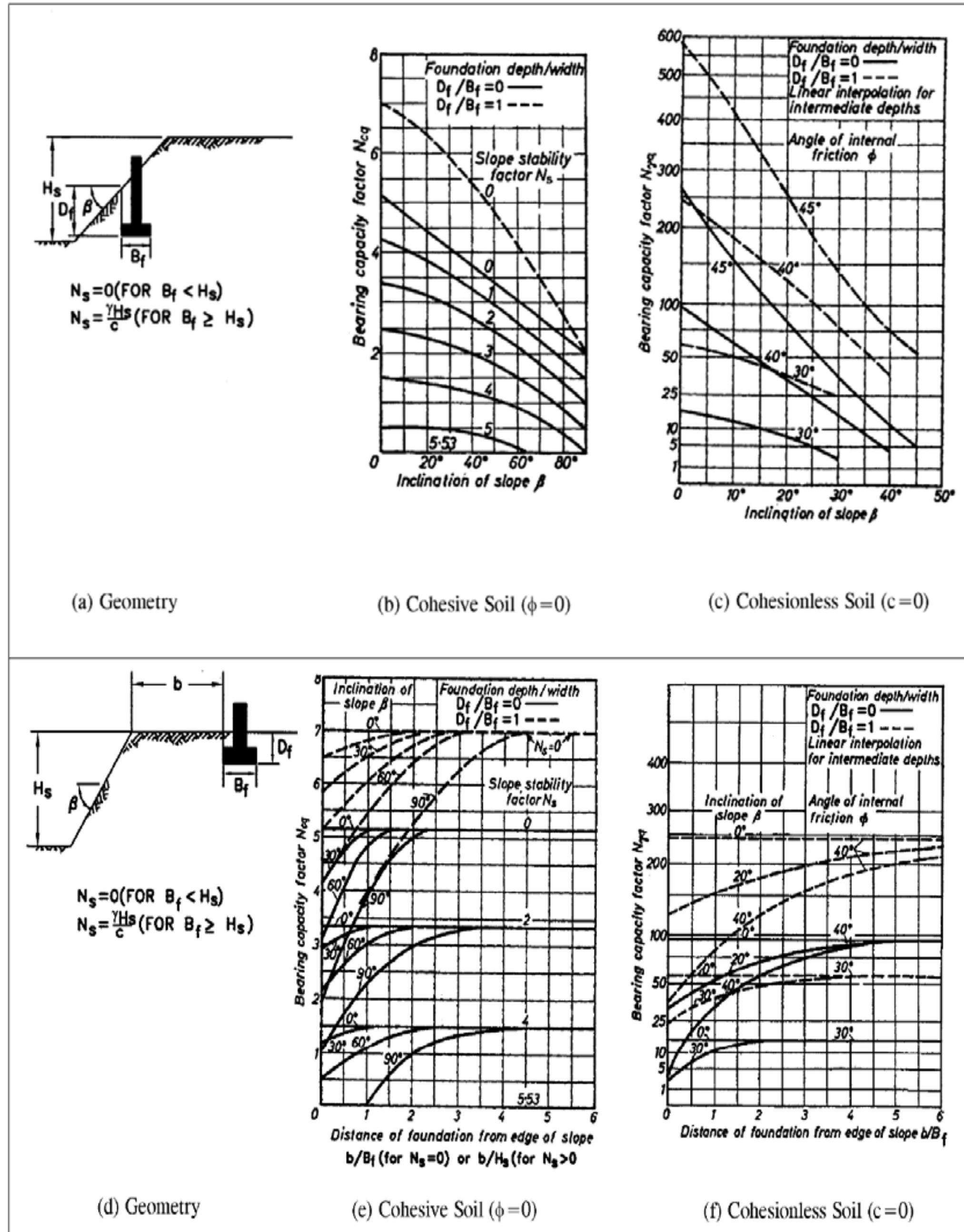


Figure 8-18. Modified bearing capacity factors for continuous footing on sloping ground (after Meyerhof, 1957, from AASHTO, 2004 with 2006 Interims)

Example of Footings on Slopes

- Given
  - Strip footing to be constructed on top of the slope
  - Soil properties: c = 75 kPa, γ = 18.5 kN/m^3, water table very deep
  - H = 8 m, B = 3 m, D = 1.5 m, b = 2 m, Slope Angle = 30 deg.
- Find
  - Ultimate Bearing Capacity of Footing, using solution method of previous slide
- Solution
  - B < H since 3 m < 8 m
  - Obtain N_{cq} from Figure 4b for Case I with N_o = 0
  - D/B = 1.5/3 = 0.5
  - b/B = 2/3 = 0.667
- Solution
  - N_{cq} for D/B = 0 and Slope Angle of 30 deg. = 4.9
  - N_{cq} for D/B = 1 and Slope Angle of 30 deg. = 6.4
  - Linearly interpolating, N_{cq} = (6.4+4.9)/2 = 5.7
  - $N_{\gamma q}$ = 1 since the soil is purely cohesive
  - B/2 = D
  - Shape factors are unity because it is a continuous footing; water table and embedment factors are like wise not considered
  - q_{ult} = (75)(5.7)+(1.5)(18.5) = 455.25 kPa

Required Footing Setbacks

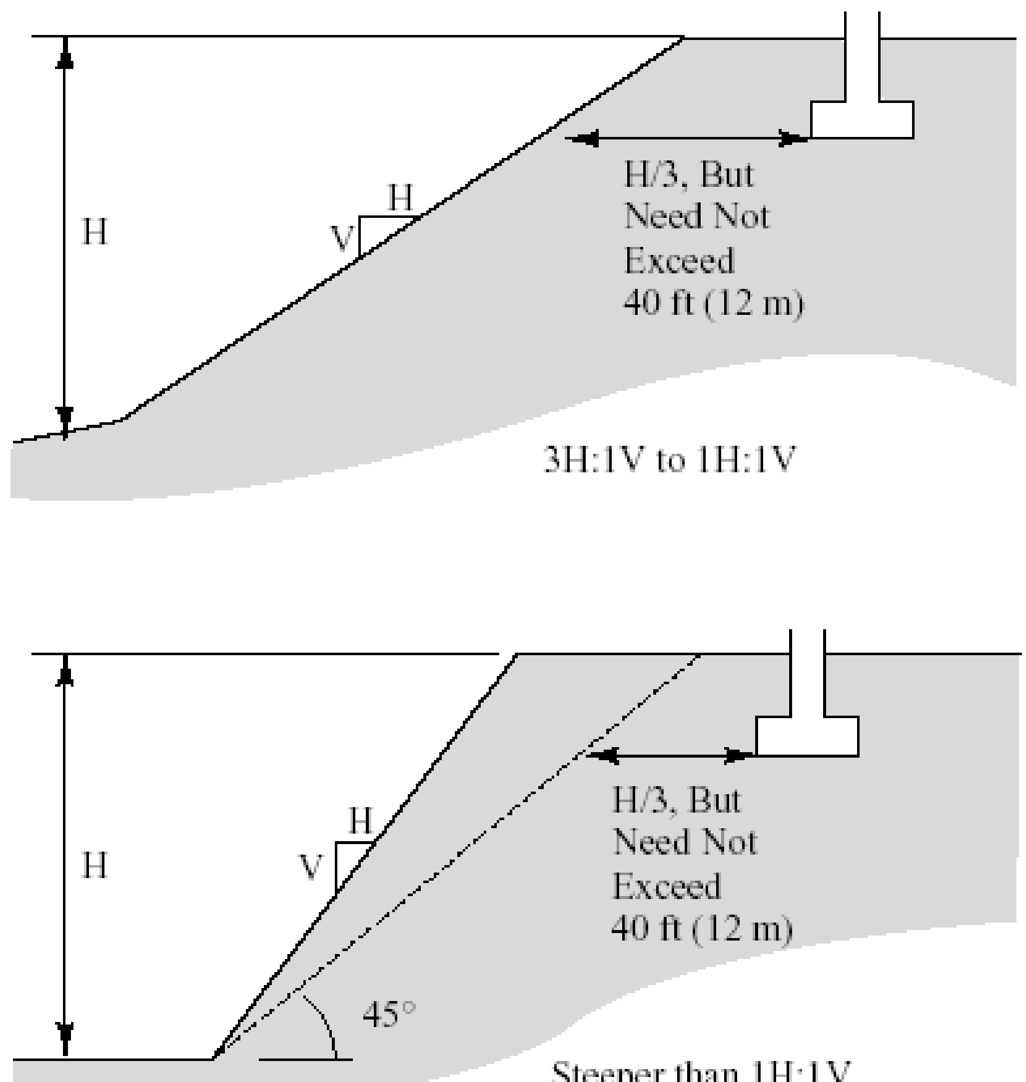


Figure 8.14 Footing setback as required by the Uniform Building Code [1806.5] and the International Building Code [1805.3] for slopes steeper than 3 horizontal to 1 vertical. The horizontal distance from the footing to the face of the slope should be at least *H*/3, but need not exceed 40 ft (12 m). For slopes that are steeper than 1 horizontal to 1 vertical, this setback distance should be measured from a line that extends from the toe of the slope at an angle of 45°. (Adapted from the 1997 edition of the *Uniform Building Code*, © 1997, with the permission of the publisher, the International Conference of Building Officials and the 2000 edition of the International Building Code).

Other Notes on Bearing Capacity Factors

- Two ways to handle B' and L' values when computing shape factors (which are a function of B/L):
  - AASHTO (2002) guidelines recommend calculating the shape factors, s, by using the effective footing dimensions, B′ and L'.
  - However, the original references (e.g., Vesić, 1975) do not specifically recommend using the effective dimensions to calculate the shape factors. Since the geotechnical engineer typically does not have knowledge of the loads causing eccentricity, use full footing dimensions be used to calculate the shape factors for use in computation of ultimate bearing capacity.
  - Either is acceptable for problems in this class. In practice, which one you would use would depend upon a) the project (a highway project would tend to use AASHTO recommendations) and how well the location of the loads was known.
- Bowles (1996) also recommends that the shape and load inclination factors (s and i) should not be combined.
- In certain loading configurations, the designer should be careful in using inclination factors together with shape factors that have been adjusted for eccentricity (Perloff and Baron, 1976). The effect of the inclined loads may already be reflected in the computation of the eccentricity. Thus an overly conservative design may result.

Presumptive Bearing Capacity

8.4.8 Presumptive Bearing Capacities

Many building codes include provisions that arbitrarily limit the amount of loading that may be applied on various classes of soils by structures subject to code regulations. These limiting loads are generally based on bearing pressures that have been observed to result in acceptable settlements. The implication is that on the basis of experience alone it may be presumed that each designated class of soil will safely support the loads indicated without the structure undergoing excessive settlements. Such values listed in codes or in the technical literature are termed presumptive bearing capacities.

8.4.8.1 Presumptive Bearing Capacity in Soil

The use of presumptive bearing capacities for shallow foundations bearing in soils is not recommended for final design of shallow foundations for transportation structures, especially bridges. Guesses about the geology and nature of a site and the application of a presumptive value from generalizations in codes or in the technical literature are not a substitute for an adequate site-specific subsurface investigation and laboratory testing program. As an exception, presumptive bearing values are sometimes used for the preliminary evaluation of shallow foundation feasibility and estimation of footing dimensions for preliminary constructability or cost evaluations.

8.4.8.2 Presumptive Bearing Capacity in Rock

Footings on intact sound rock that is stronger and less compressible than concrete are generally stable and do not require extensive study of the strength and compressibility characteristics of the rock. However, site investigations are still required to confirm the consistency and extent of rock formations beneath a shallow foundation.

Allowable bearing capacities for footings on relatively uniform and sound rock surfaces are documented in applicable building codes and engineering manuals. Many different definitions for sound rock are available. **In simple terms, however, "sound rock" can generally be defined as a rock mass that does not disintegrate after exposure to air or water and whose discontinuities are unweathered, closed or tight, i.e., less than about 1/8 in (3 mm) wide and spaced no closer than 3 ft (1 m) apart.** Table 8-8 presents allowable bearing pressures for intact rock recommended in selected local building codes (Goodman, 1989). These values were developed based on experience in sound rock formations, with the intention of satisfying both bearing capacity and settlement criteria in order to provide a satisfactory factor of safety. However, the use of presumptive values may lead to overly conservative and costly foundations. In such cases, most codes allow for a variance if the request is supported by an engineering report. Site-specific investigation and analysis is strongly encouraged.

In areas where building codes are not available or applicable, other recommended presumptive bearing values, such as those listed in Table 8-9, may be used to determine the allowable bearing pressure for sound rock. For footings designed by using these published values, the elastic settlements are generally less than 0.5 in (13 mm). Where the rock is reasonably sound, but fractured, the presumptive values listed in Tables 8-8 and 8-9 should be reduced by limiting the bearing pressures to tolerable settlements based on settlement analyses. Most building codes also provide reduced recommended bearing pressures to account for the degree of fracturing.

Peck, *et al.* (1974) presented an empirical correlation of presumptive allowable bearing pressure with Rock Quality Designation (RQD), as shown in Table 8-10. If the recommended value of allowable bearing pressure exceeds the unconfined compressive strength of the rock or allowable stress of concrete, the allowable bearing pressure should be taken as the lower of the two values. Although the suggested bearing values of Peck, *et al.* (1974) are substantially greater than most of the other published values and ignore the effects of rock type and conditions of discontinuities, they provide a useful guide for an upper-bound estimation as well as an empirical relationship between allowable bearing values and the intensity of fracturing and jointing (Table 8-10). Note that with a slight increase of the degree of fracturing of the rock mass, for example when the RQD value drops from 100 percent to 90 percent, the recommended bearing capacity value is reduced drastically from 600 ksf (29 MPa) to 400 ksf (19 MPa).

In no instance should the allowable bearing capacity exceed the allowable stress of the concrete used in the structural foundation. Furthermore, Peck, *et al.* (1974) also suggest that the average RQD for the bearing rock within a depth of the footing width (B_f) below the base of the footing should be used if the RQD values within the depth are relatively uniform. If rock within a depth of $0.5B_f$ is of poorer quality, the RQD of the poorer quality rock should be used to determine the allowable bearing capacity.

Presumptive Bearing Capacity on Rock

Table 8-8

Allowable bearing pressures for fresh rock of various types (Goodman, 1989)

Rock Type	Age	Location	Allowable Bearing Pressure tsf (MPa)
Massively bedded limestone[5]		U.K.[6]	80 (3.8)
Dolomite	L. Paleoz.	Chicago	100 (4.8)
Dolomite	L. Paleoz.	Detroit	20-200 (1.0 – 9.6)
Limestone	U. Paleoz.	Kansas City	20-120 (0.5 – 5.8)
Limestone	U. Paleoz.	St. Louis	50-100 (2.4 – 4.8)
Mica schist	Pre-Camb.	Washington	20-40 (0.5 – 1.9)
Mica schist	Pre-Camb.	Philadelphia	60-80 (2.9 – 3.8)
Manhattan schist	Pre-Camb.	New York	120 (5.8)
Fordham gneiss	Pre-Camb.	New York	120 (5.8)
Schist and slate	-	U.K.[6]	10-25 (0.5 – 1.2)
Argillite	Pre-Camb.	Cambridge, MA	10-25 (0.5 – 1.2)
Newark shale	Triassic	Philadelphia	10-25 (0.5 – 1.2)
Hard, cemented shale	-	U.K.[6]	40 (1.9)
Eagleford shale	Cretaceous	Dallas	13-40 (0.6 – 1.9)
Clay shale	-	U.K.[6]	20 (1.0)
Pierre shale	Cretaceous	Denver	20-60 (1.0 – 2.9)
Fox Hills sandstone	Tertiary	Denver	20-60 (1.0 – 2.9)
Solid chalk	Cretaceous	U.K.[6]	13 (0.6)
Austin chalk	Cretaceous	Dallas	30-100 (1.4 – 4.8)
Friable sandstone and claystone	Tertiary	Oakland	8-20 (0.4 – 1.0)
Friable sandstone (Pico formation)	Quaternary	Los Angeles	10-20 (0.5 – 1.0)

Notes:

[1] According to typical building codes; reduce values accordingly to account for weathering or unrepresentative fracturing

[2] Values from Thorburn (1966) and Woodward, Gardner and Greer (1972).

[3] When a range is given, it relates to usual range in rock conditions.

[4] Sound rock that rings when struck and does not disintegrate. Cracks are unweathered and open less than 10 mm.

[5] Thickness of beds greater than 3 ft (1 m), joint spacing greater than 2 mm; unconfined compressive strength greater than 160 tsf (7.7 MPa) (for a 4 in (100 mm) cube).

[6] Institution of Civil Engineers Code of Practice 4.

Table 8-9

Presumptive values of allowable bearing pressures for spread foundations on rock (modified after NAVFAC, 1986a, AASHTO 2004 with 2006 Interims)

Type of Bearing Material	Consistency In Place	Allowable Bearing Pressure tsf (MPa)	
		Range	Recommended Value for Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Hard, sound rock	120-200 (5.8 - 9.6)	160 (7.7)
Foliated metamorphic rock: Slate, schist (sound condition allows minor cracks)	Medium-hard, sound rock	60-80 (2.9-3.8)	70 (3.4)
Sedimentary rock; hard cemented shales, siltstone, sandstone, limestone without cavities	Medium-hard, sound rock	30-50 (1.4-2.4)	40 (1.9)
Weathered or broken bedrock of any kind except highly argillaceous rock (shale). RQD less than 25	Soft rock	16-24 (0.8-1.2)	20 (1)
Compacted shale or other highly argillaceous rock in sound condition	Soft rock	16-24 (0.8-1.2)	20 (1)

Notes:

1. For preliminary analysis or in the absence of strength tests, design and proportion shallow foundations to distribute their loads by using presumptive values of allowable bearing pressure given in this table. Modify the nominal value of allowable bearing pressure for special conditions described in notes 2 through 8.
2. The maximum bearing pressure beneath the footing produced by eccentric loads that include dead plus normal live load plus permanent lateral loads shall not exceed the above nominal bearing pressure.
3. Bearing pressures up to one-third in excess of the nominal bearing values are permitted for transient live load from wind or earthquake. If overload from wind or earthquake exceeds one-third of nominal bearing pressures, increase allowable bearing pressures by one-third of nominal value.
4. Extend footings on soft rock to a minimum depth of 1.5 in (40 mm) below adjacent ground surface or surface of adjacent floor, whichever elevation is the lowest.
5. For footings on soft rock, increase allowable bearing pressures by 5 percent of the nominal values for each 1 ft (300 mm) of depth below the minimum depth specified in Note 4.
6. Apply the nominal bearing pressures of the three categories of hard or medium hard rock shown above where the base of the foundation lies on rock surface. Where the foundation extends below the rock surface, increase the allowable bearing pressure by 10 percent of the nominal values for each additional 1ft (300 mm) of depth extending below the surface.
7. For footings smaller than 3 ft (1 m) in the least lateral dimension, the allowable bearing pressure shall be the nominal bearing pressure multiplied by the least lateral dimension.
8. If the above-recommended nominal bearing pressure exceeds the unconfined compressive strength of intact specimen, the allowable pressure equals the unconfined compressive strength.

Table 8-10

Suggested values of allowable bearing capacity (Peck, *et al.*, 1974)

RQD (%)	Rock Mass Quality	Allowable Pressure ksf (MPa)
100	Excellent	600 (29)
90	Good	400 (19)
75	Fair	240 (12)
50	Poor	130 (6)
25	Very Poor	60 (3)
0	Soil-like	20 (1)

Questions?