Technical Supplement 14Q

Abutment Design for Small Bridges

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Cover photo: Design of abutments for small bridges requires geotechnical analysis.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Technical Supplement 14Q

Abutment Design for Small Bridges

Purpose

This technical supplement presents a procedure for determining the ultimate and allowable bearing capacity for shallow strip footings adjacent to slopes. Additional guidelines related to scour protection and layout are also included. Structural design of bridges and footings is beyond the scope of this document

NRCS use of bridges

Bridges are installed in a variety of U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) applications including farm and rural access roads, livestock crossings, Emergency Watershed Protection (EWP) work, and recreation facilities (figs. TS14Q–1 and TS14Q–2). Bridges installed under NRCS programs are generally single-span, single-lane structures and use various simple structural systems including rail cars, steel I-beams, and timber stringers. Timber decking is often used to provide the driving surface. The entire bridge structure is normally supported by simple strip footings of timber or concrete on either abutment. The procedure presented in this technical supplement is appropriate for the design of abutments for the relatively small bridges typically constructed in NRCS work.

Small bridges may also be used to replace existing culverts that act as barriers to fish passage (figs. TS14Q–3 and TS14Q–4).

Bearing capacity

Flat ground formula

The bearing capacity of strip footings adjacent to slopes is an extension of the classical theory of bearing capacity for footings on flat ground.

The bearing capacity of a soil foundation is provided by the strength of the soil to resist shearing (sliding) along induced failure zones under and adjacent to the footing. Consequently, bearing capacity is a function of the soil's shear strength, cohesion, and frictional resistance. Further information on the selection of shear strength parameters for bearing capacity determination is included later in this technical supplement.

Bearing capacity formulas for footings on flat ground have been proposed by Terzaghi (1943) and others and have the following general form (fig. TS14Q–5 for definition sketch):

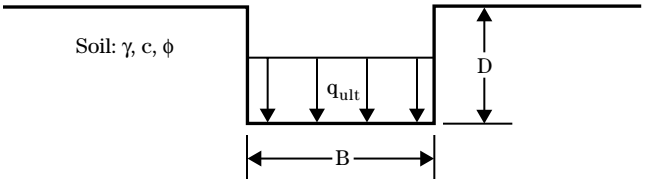
Figure TS14Q–1 Farm bridge with steel I-beam structural members and timber strip footing (*Photo courtesy of Ben Doerge*)

Figure TS14Q–2 Flood-damaged bridge in EWP program (*Photo courtesy of Ben Doerge*)

Figure TS14Q–3 Forest road culvert acting as fish passage barrier (*Photo courtesy of Christi Fisher*)

Figure TS14Q–4 Culvert replaced by bridge *(Photo courtesy of Christi Fisher)*

Figure TS14Q–5 Definition sketch—strip footing on flat ground



$$q_{ult} = c \times N_c + q \times N_q + \frac{1}{2} \times \gamma \times B \times N_\gamma$$

(eq. TS14Q–1)

where:

q_{ult} = ultimate bearing capacity (F/L^2)
c = cohesion (F/L^2)
q = γD = surcharge weight (F/L^2)
D = depth of footing below grade (L)
γ = unit weight of soil (F/L^3)
B = footing width (L)
N_c, N_q, and N_γ = bearing capacity factors related to cohesion, surcharge, and friction, respectively (unitless)

The bearing capacity factors, N_c, N_q, and N_γ, are all functions of the soil's angle of internal friction, ϕ (phi). Tables of bearing capacity factors for the flat ground case are given in tables TS14Q–1 and TS14Q–2 (Bowles 1996). In the classical treatment of bearing capacity, the foundation is assumed to consist of a single, homogeneous soil. The basic bearing capacity formula has been refined by the inclusion of modification factors to account for such variables as footing shape, inclination, eccentricity, and water table effects. See U.S. Army Corps of Engineers (USACE) EM–1110–1–1905 (1992a) and Bowles (1996) for further description of these factors.

Allowable bearing capacity

The allowable bearing capacity is determined by applying an appropriate factor of safety to the ultimate bearing capacity, q_{ult}, as determined from the bearing capacity formula. A factor of safety of 3.0 is often used with dead and live loads (USACE EM–1110–1–1905, U.S. Department of Navy Naval Facilities Engineering Command (NAVFAC) DM 7.2; American Association of State Highway and Transportation (AASHTO) Standard Specifications for Highway Bridges). Settlement of footings should also be checked to verify that excessive displacements will not occur. The treatment of bearing capacity under seismic loading is beyond the scope of this document.

Sloping ground methods

When the ground surface on one side of the footing is sloping, as in the case of a bridge abutment adjacent to a stream channel, the bearing capacity is reduced,

Table TS14Q–1 Terzaghi (1943) bearing capacity factors

ϕ, deg.	N_c	N_q	N_γ
0	5.7	1.0	0.0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
35	57.8	41.4	42.4
40	95.7	81.3	100.4

Table TS14Q–2 Meyerhof (1957) and Hansen (1970) bearing capacity factors

ϕ, deg.	N_c	N_q	N_γ	
			(Meyerhof)	(Hansen)
0	5.14	1.0	0.0	0.0
5	6.49	1.6	0.1	0.1
10	8.34	2.5	0.4	0.4
15	10.97	3.9	1.1	1.2
20	14.83	6.4	2.9	2.9
25	20.71	10.7	6.8	6.8
30	30.13	18.4	15.7	15.1
35	46.10	33.3	37.2	33.9
40	40.25	64.1	93.6	79.4

compared to the flat ground case. The reduction is the result of the loss of some of the available shear resistance on the sloping side due to the shortening of the failure surface and decrease in the total weight acting on the failure surface. The bearing capacity for footings adjacent to slopes is a function of the same five variables as for footings on flat ground, with two additional variables: slope angle (β) and distance from the shoulder of the slope to the corner of the footing on the slope side (b) (fig. TS14Q–6).

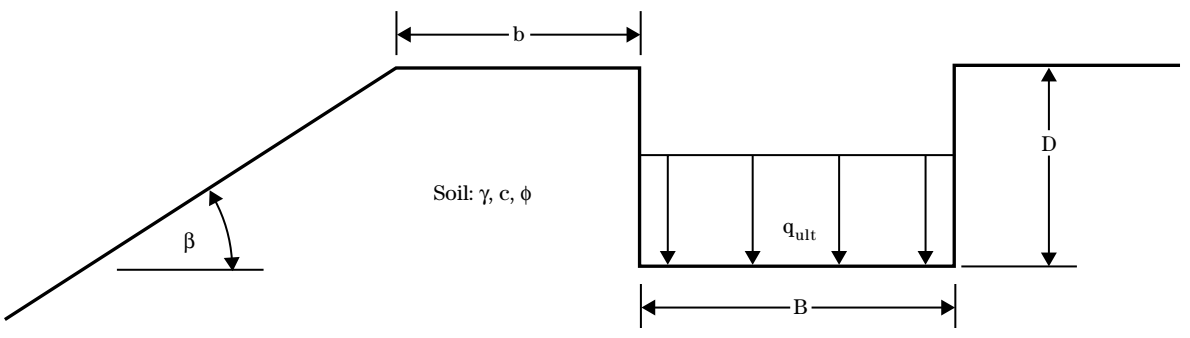
The reduction in bearing capacity for footings located adjacent to slopes has been studied extensively, using both theoretical and physical models. The effect of the slope is typically accounted for by modifying the bearing capacity factors in the traditional bearing capacity equation for flat ground. As in the classical analysis for the flat ground case, all methods for sloping ground are based on the assumption of a homogeneous foundation. A surprisingly wide variation in results may be observed between the various methods. The findings of tests on actual physical models are useful in assessing the validity of the various theoretical models.

The Meyerhof method is selected for the purposes of this technical supplement due to its long-term acceptance within the profession and its relative simplicity. Example calculations are given later.

Meyerhof method

The Meyerhof (1957) method is based on the theory of plastic equilibrium. It is the oldest method for analyzing footings adjacent to slopes and has enjoyed wide use. For example, this method is cited in NAVFAC DM 7.2

Figure TS14Q–6 Definition sketch—strip footing adjacent to a slope



and in the AASHTO Standard Specification for Highway Bridges.

In the Meyerhof (1957) method, the bearing capacity formula takes the following form:

$$q_{ult} = c \times N_{cq} + \frac{1}{2} \times \gamma \times B \times N_{\gamma q} \qquad \text{(eq. TS14Q–2)}$$

The terms N_{cq} and $N_{\gamma q}$ are modified bearing capacity factors in which the effect of surcharge is included. These factors are determined from design charts reproduced in NAVFAC DM 7.2 (fig. TS14Q–7). Procedures for analyzing rectangular, square, and circular footings, as well as water table effects, are also presented in these charts. With the Meyerhof (1957) method, foundations of both cohesive and cohesionless soils may be analyzed, and the footing may be located either on or at the top of the slope. Values of $N_{\gamma q}$ for cohesionless soils are also given in tabular form in table TS14Q–3 and for cohesive soils in table TS14Q–4.

Other methods

Many other methods exist for estimating the bearing capacity of strip footings adjacent to slopes. Bearing capacity may also be estimated using the principles of limit equilibrium or finite element analysis. The designer may wish to use more than one method and compare the results before making a final design decision. It is recommended that the bearing capacity for the flat ground case also be computed for reference as an upper bound.

Selection of shear strength parameters

Soil shear strength parameters (ϕ and c) are selected based on the assumption that footing loads may be applied rapidly. If the foundation soils may become saturated at any time during the life of the structure, then the design shear strength parameters should be selected accordingly.

For cohesive soils that develop excess pore pressure when loaded rapidly (CH, MH, and CL soils), undrained shear parameters are used. The cohesion (c) is taken to be the undrained shear strength (s_u) of the soil in a saturated condition, and an angle of internal friction (ϕ) of zero is used. The undrained shear strength may be determined from the unconsolidated-undrained (UU) shear test, the unconfined compression (q_u) test, or the vane shear test. The undrained shear strength may also be estimated from field tests or index properties. See table TS14Q–5 to estimate the undrained shear strength of saturated fine-grained soils from consistency, standard penetration test (blow count), and liquidity index.

For cohesionless, free-draining soils that are able to dissipate excess pore pressure rapidly (SW, SP, GW, GP, SM, GM, and nonplastic ML soils), the effective shear strength parameters (ϕ' and c') are used. The effective cohesion parameter (c') for cohesionless soils is either zero or very small and is normally neglected. The effective angle of internal friction (ϕ') may be determined from shear tests or may be estimated from generalized charts and tables. See table TS14Q–6 (USACE 1992a) and figure TS14Q–8 (NAVFAC 1982b) for charts to estimate the effective angle of internal friction for sands and gravels.

A qualified soils engineer should be consulted when selecting shear strength parameters for design.

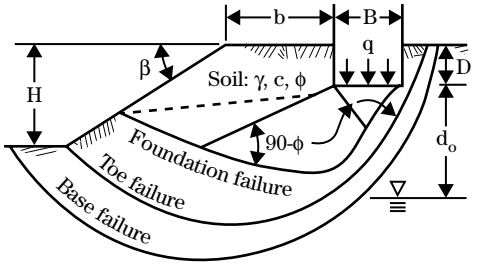
Other considerations

Bridge abutments consisting of soil or other erodible material must be protected against the effects of scour. Either the footing should be embedded below the maximum anticipated scour depth or adequate scour protection must be provided. The embedment approach is generally not applicable with the shallow strip footings normally used with NRCS-designed bridges. Therefore, scour protection, such as rock riprap or gabions, should be provided, as necessary, to prevent erosion of the slope and possible undermining of the footings.

The U.S. Forest Service recommends that bridges with shallow strip footings be used primarily in stream channels that are straight and stable, have low scour potential, and will not accumulate significant debris or ice. Any additional antiscour measures needed to ensure the long-term integrity of the bridge abutments should be incorporated into the design (McClelland 1999).

Figure TS14Q–7 Meyerhof method design charts: ultimate bearing capacity for shallow footing placed on or near a slope

Case I: Continuous footing at top of slope



Water at $d_o \geq B$

$$q_{ult} = cN_{cq} + \gamma_T \frac{B}{2} N_{\gamma q} \quad (a)$$

Water at ground surface

$$q_{ult} = cN_{cq} + \gamma_{sub} \frac{B}{2} N_{\gamma q} \quad (b)$$

If $B \leq H$:
Obtain N_{cq} from figure 4b for case I with N_o=0.
Interpolate for values of $0 < D/B < 1$
Interpolate q_{u1t} between eq (a) and (b) for water at intermediate level between ground surface and d_o=B.

If $B > H$:
Obtain N_{cq} from figure 4b for case I with stability number

$$N_o = \frac{\gamma H}{c}$$

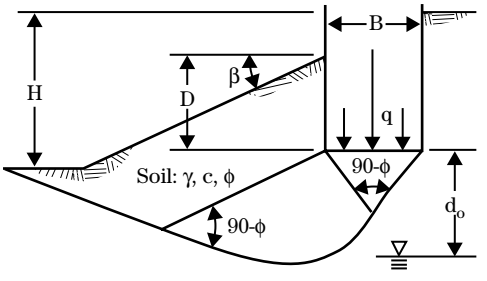
Interpolate for values $0 < D/B < 1$ for $0 < N_o < 1$. If $N_o \geq 1$, stability of slope controls ultimate bearing pressure.
Interpolate q_{u1t} between eq (a) and (b) for water intermediate level between ground surface and d_o = B. for water at ground surface and sudden drawdown: Substitute ϕ' for ϕ in eq (b)

$$\phi' = \tan^{-1}\left(\frac{\gamma_{sub}}{\gamma_T}\tan\phi\right)$$

Cohesive soil ($\phi = 0$)
Substitute in eq (a) and (b) D for B/2 and $N_{\gamma q} = 1$.
Rectangular, square, or circular footing:

$$q_{u1t} = \left[q_{u1t} \begin{array}{l}\text{for continuous footing}\\ \text{as given above}\end{array} \right] \times \left[\frac{q_{u1t}\text{ for finite footing}}{q_{u1t}\text{ for continuous footing}} \right]$$

Case II: Continuous footings on slope



Same criteria as for case I except that N_{cq} and $N_{\gamma q}$ are obtained from diagrams for case II

Figure TS14Q–7 Meyerhof method design charts: bearing capacity for shallow footing placed on or near a slope—Continued

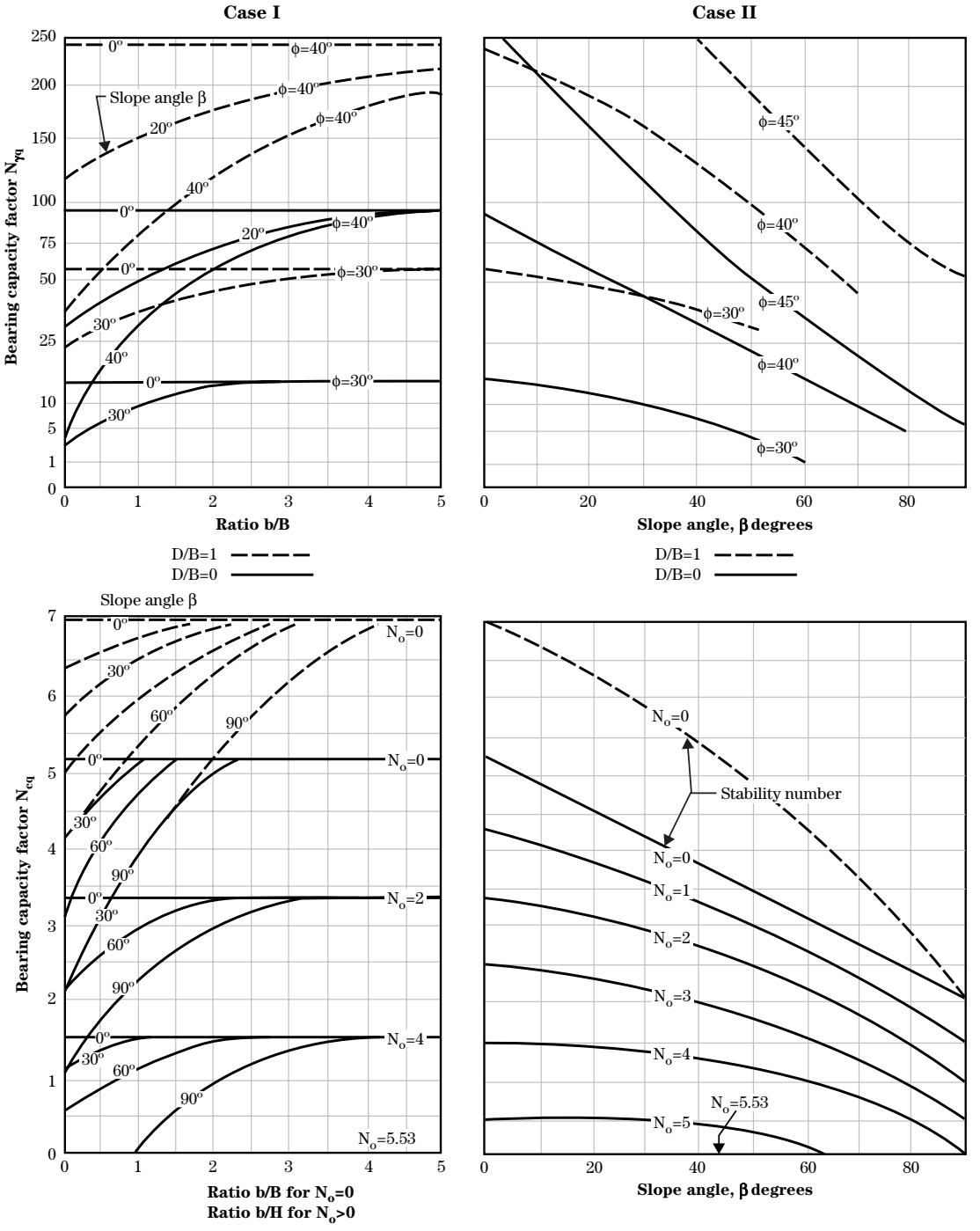


Table TS14Q–3 Meyerhof method design table (cohesionless soils)

Meyerhof method—cohesionless soils				Bearing capacity factors near slopes									
				$N_{\gamma q}$									
				b/B									
ϕ, deg	**D/B**	**β, deg**	**Z**	**0.00**	**0.25**	**0.50**	**0.75**	**1.00**	**1.50**	**2.00**	**3.00**	**4.00**	**6.00**
30	0	**0**		15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
		18.4	3H:1V	7.0	8.3	9.8	11.3	12.2	13.8	14.4	15.0	15.0	15.0
		21.8	2.5H:1V	5.6	7.0	8.8	10.6	11.7	13.5	14.3	15.0	15.0	15.0
		26.6	2H:1V	3.5	5.2	7.5	9.7	11.0	13.2	14.1	15.0	15.0	15.0
		30		2.0	4.0	6.5	9.0	10.5	13.0	14.0	15.0	15.0	15.0
	1	**0**		57.0	57.0	57.0	57.0	57.0	57.0	57.0	57.0	57.0	57.0
		18.4	3H:1V	36.1	39.2	41.7	43.5	46.0	49.0	52.1	54.5	56.4	57.0
		21.8	2.5H:1V	32.3	35.9	38.8	41.0	43.9	47.6	51.2	54.1	56.3	57.0
		26.6	2H:1V	26.9	31.3	34.8	37.5	41.0	45.5	49.9	53.5	56.1	57.0
		30		23.0	28.0	32.0	35.0	39.0	44.0	49.0	53.0	56.0	57.0
40	0	**0**		92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0
		18.4	3H:1V	36.8	41.4	46.0	51.5	56.1	64.4	71.8	82.8	87.4	92.0
		20		32.0	37.0	42.0	48.0	53.0	62.0	70.0	82.0	87.0	92.0
		21.8	2.5H:1V	29.4	34.7	39.9	46.0	51.2	60.7	68.8	81.4	86.8	92.0
		26.6	2H:1V	22.4	28.4	34.4	40.7	46.4	57.1	65.7	79.7	86.3	92.0
		33.7	1.5H:1V	12.0	19.2	26.2	32.9	39.3	51.7	61.1	77.2	85.6	92.0
		40		2.8	11.0	19.0	26.0	33.0	47.0	57.0	75.0	85.0	92.0
	1	**0**		240.0	240.0	240.0	240.0	240.0	240.0	240.0	240.0	240.0	240.0
		18.4	3H:1V	133.3	140.6	148.0	154.4	159.0	171.9	182.0	200.4	213.3	230.8
		20		124.0	132.0	140.0	147.0	152.0	166.0	177.0	197.0	211.0	230.0
		21.8	2.5H:1V	116.2	124.4	132.7	140.1	145.7	160.5	172.2	193.4	208.6	228.6
		26.6	2H:1V	95.3	104.3	113.3	121.6	128.9	145.9	159.5	183.8	202.1	224.7
		33.7	1.5H:1V	64.4	74.5	84.5	94.3	104.1	124.2	140.7	169.6	192.5	219.0
		40		37.0	48.0	59.0	70.0	82.0	105.0	124.0	157.0	184.0	214.0

Notes:
1. Bold values of β and the associated $N_{\gamma q}$ values are read directly from the Meyerhof charts. Other values are interpolated.
2. Intermediate values of β, $N_{\gamma q}$ values may be determined by linear interpolation.
3. To calculate ultimate bearing capacity: $q_{ult} = 0.5\phi B N_{\gamma q}$

Table TS14Q–4 Meyerhof method design table (cohesive soils)

Meyerhof method—cohesive soils (ϕ = 0)				Bearing capacity factors near slopes									
				N_{cq}									
				b/B or b/H									
D/B	**N_s**	**β, deg**	**Z**	**0**	**0.5**	**1**	**1.5**	**2**	**2.5**	**3**	**3.5**	**4**	**4.5**
0	0	**0**		5.14	5.14	5.14	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		18.4	3H:1V	4.55	4.90	5.12	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		21.8	2.5H:1V	4.44	4.86	5.12	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		26.6	2H:1V	4.29	4.79	5.11	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		30		4.18	4.75	5.11	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		33.7	1.5H:1V	4.04	4.66	5.07	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		60		3.08	4.06	4.82	5.12	5.14	5.14	5.14	5.14	5.14	5.14
		90		1.93	3.00	3.90	4.58	5.00	5.14	5.14	5.14	5.14	5.14
	2	**0**		3.33	3.33	3.33	3.33	3.33	3.33	3.33	3.33	3.33	3.33
		18.4	3H:1V	3.08	3.23	3.32	3.33	3.33	3.33	3.33	3.33	3.33	3.33
		21.8	2.5H:1V	3.03	3.21	3.32	3.33	3.33	3.33	3.33	3.33	3.33	3.33
		26.6	2H:1V	2.97	3.18	3.32	3.33	3.33	3.33	3.33	3.33	3.33	3.33
		30		2.92	3.16	3.32	3.33	3.33	3.33	3.33	3.33	3.33	3.33
		33.7	1.5H:1V	2.83	3.09	3.28	3.32	3.33	3.33	3.33	3.33	3.33	3.33
		60		2.16	2.62	3.00	3.22	3.32	3.33	3.33	3.33	3.33	3.33
		90		1.04	1.71	2.28	2.65	2.97	3.14	3.27	3.33	3.33	3.33
	4	**0**		1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
		18.4	3H:1V	1.32	1.43	1.49	1.50	1.50	1.50	1.50	1.50	1.50	1.50
		21.8	2.5H:1V	1.28	1.42	1.49	1.50	1.50	1.50	1.50	1.50	1.50	1.50
		26.6	2H:1V	1.23	1.40	1.48	1.50	1.50	1.50	1.50	1.50	1.50	1.50
		30		1.20	1.39	1.48	1.50	1.50	1.50	1.50	1.50	1.50	1.50
		33.7	1.5H:1V	1.12	1.32	1.43	1.48	1.49	1.50	1.50	1.50	1.50	1.50
		60		0.52	0.83	1.10	1.30	1.42	1.50	1.50	1.50	1.50	1.50
		90				0.03	0.60	0.98	1.21	1.33	1.41	1.48	1.50
1	0	**0**		7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00
		15		6.50	6.68	6.82	6.94	7.00	7.00	7.00	7.00	7.00	7.00
		18.4	3H:1V	6.35	6.57	6.75	6.90	6.98	7.00	7.00	7.00	7.00	7.00
		21.8	2.5H:1V	6.21	6.46	6.68	6.86	6.96	7.00	7.00	7.00	7.00	7.00
		26.6	2H:1V	6.01	6.31	6.59	6.81	6.94	7.00	7.00	7.00	7.00	7.00
		30		5.86	6.20	6.52	6.77	6.92	7.00	7.00	7.00	7.00	7.00
		33.7	1.5H:1V	5.68	6.06	6.40	6.69	6.87	6.98	7.00	7.00	7.00	7.00
		45	1H:1V	5.14	5.62	6.05	6.43	6.73	6.93	7.00	7.00	7.00	7.00
		60		4.11	4.80	5.44	5.95	6.41	6.74	6.98	7.00	7.00	7.00
		90				4.00	4.67	5.27	5.75	6.25	6.63	6.88	7.00

Notes:
1. Bold values of β and the associated N_{cq} values are read directly from the Meyerhof charts. Other values are interpolated.
2. Intermediate values of β, N_{cq} values may be determined by linear interpolation.
3. N_s = stability factor of slope = $\gamma H/c$
   where:
   γ = unit weight of soil (lb/ft^3)
   H = vertical height of slope (ft)
   c = cohesion (or undrained shear strength) of soil (lb/ft^2)
4. To calculate ultimate bearing capacity: $q_{ult} = cN_{cq}$

Table TS14Q–5 Undrained shear strength values for saturated cohesive soils

Consistency description	s_u 1/ (lb/ft²)	Thumb penetration/consistency	LI 2/	N_{60} (SPT) 3/ (blows/ft)
Very soft	0–250	Thumb penetrates >1 in, extruded between fingers	≥1.0	<2
Soft	250–500	Thumb penetrates 1 in, molded by light finger pressure	1.0–0.67	2–4
Medium	500–1,000	Thumb penetrates ¼ in, molded by strong finger pressure	0.67–0.33	4–8
Stiff	1,000–2,000	Indented by thumb, but not penetrated	0.33–0	8–15
Very stiff	2,000–4,000	Not indented by thumb, but indented by thumbnail	≤0	15–30
Hard	>4,000	Not indented by thumbnail	<0	>30

1/ s_u = undrained shear strength of soil

2/ LI = liquidity index = $\frac{w_{sat} - PL}{PI} = \frac{w_{sat} - LL + PI}{PI}$

where:

w_{sat}, % = saturated water content at in situ density = $\left[\left(\frac{\gamma_w}{\gamma_d}\right) - \left(\frac{1}{G_s}\right)\right] \times 100\%$

γ_w = unit weight of water = 62.4 lb/ft³

γ_d = unit weight of soil at *in situ* density, lb/ft³

G_s = specific gravity of soil solids, unitless

3/ N_{60} = blows per foot by standard penetration test (SPT), corrected for overburden pressure

Table TS14Q–6 ϕ' values for sands

Angle of internal friction of sands, ϕ'

a. Relative density and gradation (Schmertmann 1978)

Relative density D_r, percent	Fine grained		Medium grained		Coarse grained	
	Uniform	Well graded	Uniform	Well graded	Uniform	Well graded
40	34	36	36	38	38	41
60	36	38	38	41	41	43
80	39	41	41	43	43	44
100	42	43	43	44	44	46

b. Relative density and initial *in situ* soil tests

Soil type	Relative density D_r percent	Standard penetration resistance N_{60} (Terzaghi and Peck 1967)	Cone penetration resistance q_c, ksf (Meyerhof 1974a)	Friction angle ϕ', deg		
				Meyerhof (1974b)	Peck, Hanson, and Thornburn (1974)	Meyerhof (1974b)
Very loose	<20	<4	—	<30	<29	<30
Loose	20–40	4–10	0–100	30–35	29–30	30–35
Medium	40–60	10–30	100–300	35–38	30–36	35–40
Dense	60–80	30–50	300–500	38–41	36–41	40–45
Very dense	>80	>50	500–800	41–44	>41	>45

(a) ASTM D653 defines relative density as the ratio of the difference in void ratio of a cohesionless soil in the loosest state at any given void ratio to the difference between the void ratios in the loosest and in the densest states. A very loose sand has a relative density of 0 percent and 100 percent in the densest possible state. Extremely loose honeycombed sands may have a negative relative density.

(b) Relative density may be calculated using standard test methods ASTM D4254 and the void ratio of the *in situ* cohensionless soil,

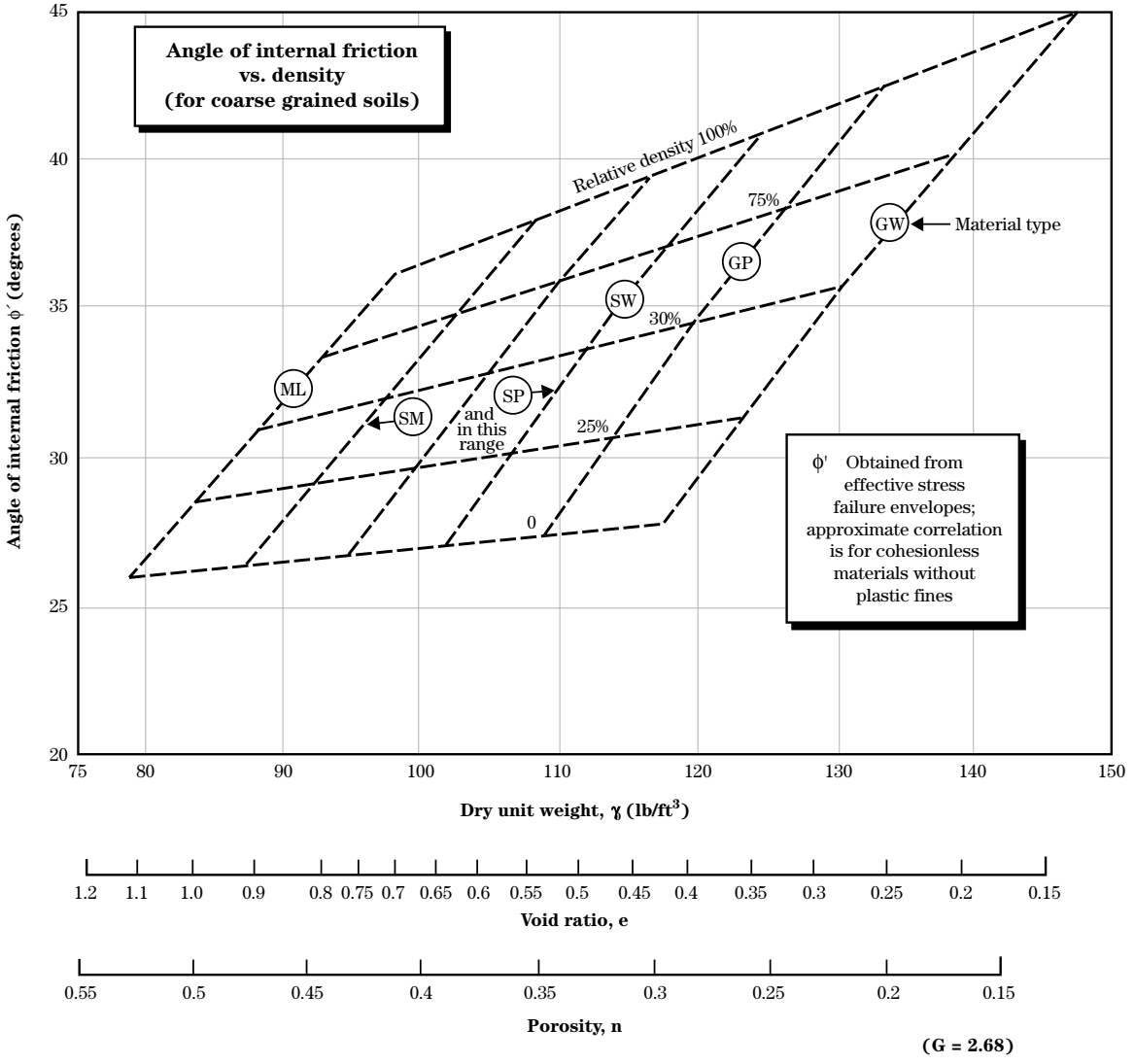
$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

$$e = \frac{G}{\gamma_d}\gamma_w - 1$$

where:

e_{min} = reference void ratio of a soil at the maximum density
e_{max} = reference void ratio of a soil at the minimum density
G = specific gravity
γ_d = dry density, kips/ft^3
γ_w = unit weight of water, 0.0625 kips/ft^3

Figure TS14Q–8 ϕ´ values for coarse grained soils



Example problem 1: Strip footing adjacent to a slope

Given: The abutment shown in figure TS14Q–9.

Find: Ultimate and allowable bearing capacity, q_{ult} and $q_{allowable}$, respectively.

Solution: Use the Meyerhof (1957) method to estimate the ultimate and allowable bearing capacity.

Compute $\frac{b}{B} = \frac{D}{B} = \frac{3\text{ ft}}{3\text{ ft}} = 1$

Since the soil is cohesionless (c = 0), the N_{cq} term in equation TS14Q–2 may be neglected.

Determine $N_{\gamma q}$ by interpolation from table TS14Q–3.

For $\phi = 30^\circ$, $N_{\gamma q} = 41.0$, and for $\phi = 40^\circ$, $N_{\gamma q} = 128.9$

By interpolation, for $\phi = 35^\circ$, $N_{\gamma q} = 85$

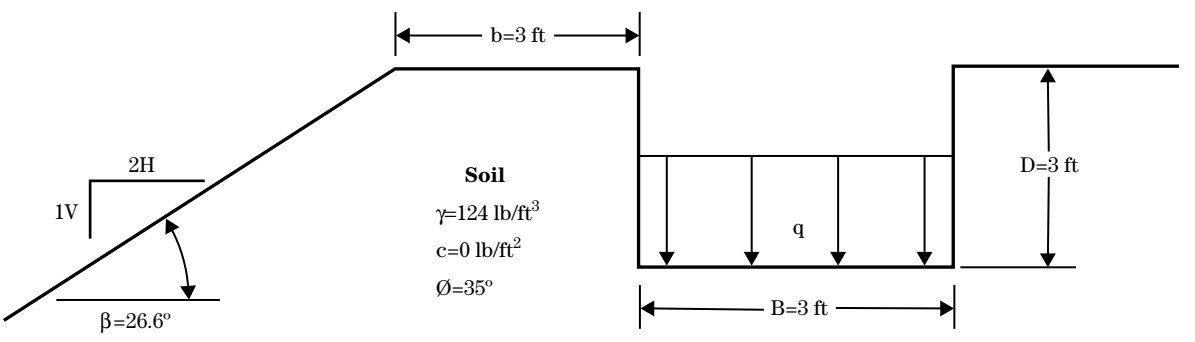
Solve for ultimate bearing capacity, q_{ult}, using equation TS14Q–2:

$$\begin{aligned} q_{ult} &= \frac{1}{2} \times \gamma \times B \times N_{\gamma q} \\ &= (0.5) \times \left(124\text{ lb/ft}^3\right) \times (3\text{ ft}) \times (85) \\ &= 15{,}810\text{ lb/ft}^2 \\ &= 15.8\text{ k/ft}^2 \end{aligned}$$

Applying a factor of safety (FS) of 3.0 to determine the allowable bearing capacity:

$$\begin{aligned} q_{allowable} &= \frac{q_{ult}}{FS} \\ &= \frac{\left(15.8\text{ k/ft}^2\right)}{3} \\ &= 5.3\text{ k/ft}^2 \end{aligned}$$

Figure TS14Q–9 Problem schematic for example problem 1—strip footing adjacent to a slope



Example problem 2: Surface load adjacent to slope

Given: The abutment shown in figure TS14Q–10.

Find: Ultimate and allowable bearing capacity, q_{ult} and $q_{allowable}$, respectively.

Solution: Use the Meyerhof (1957) method to estimate the ultimate and allowable bearing capacity.

Compute

$$\frac{b}{B} = \frac{3\text{ ft}}{3\text{ ft}} = 1$$

and

$$\frac{D}{B} = \frac{0\text{ ft}}{3\text{ ft}} = 0$$

Since the soil is purely cohesive ($\phi = 0$), the $N_{\gamma q}$ term in equation TS14Q–2 may be neglected.

$$N_s = \left(100\text{ lb/ft}^3\right) \times \frac{\left(10\text{ ft}\right)}{\left(500\text{ lb/ft}^2\right)}$$

$$= 3.0$$

Since $N_s > 0$, compute b/H = (3 ft)/(10 ft) = 0.33

Determine N_{cq} by interpolation from table TS14Q–4.

b/H	N_{cq}
0.50	3.18
0.33	3.11
0.00	2.97

So, by equation TS14Q–2, $q_{ult} = cN_c = (500\text{ lb/ft}^2)\times(3.11) = 1{,}560\text{ lb/ft}^2$

and

$q_{allowable} = q_{ult}/FS = (1{,}560\text{ lb/ft}^2)\div(3.0) = 520\text{ lb/ft}^2$

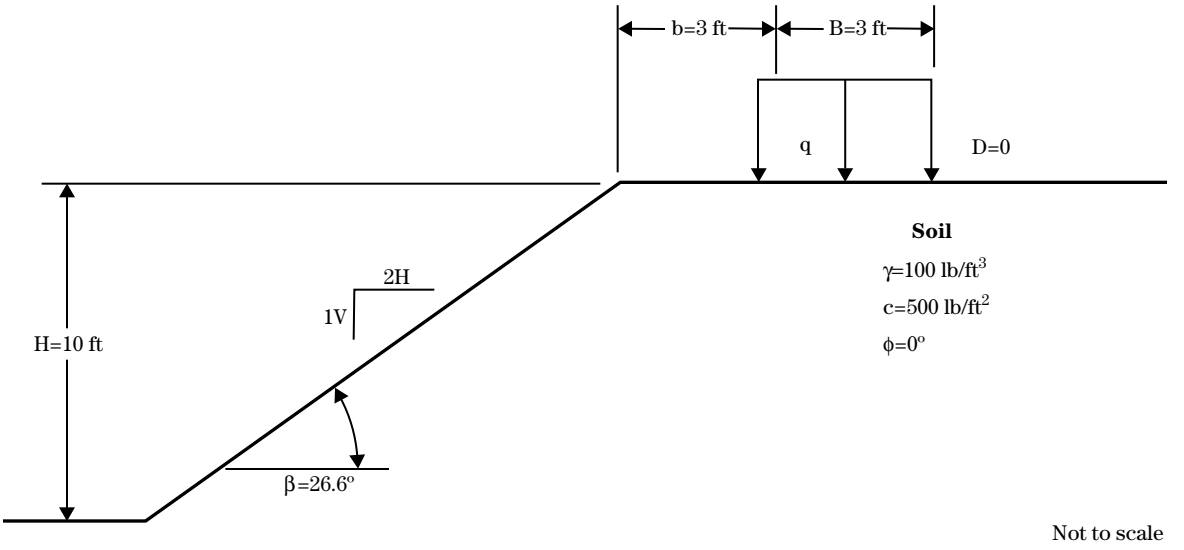



Figure TS14Q–10 Problem schematic for example problem 2—surface load adjacent to a slope