NCHRP 24-31

LRFD DESIGN SPECIFICATIONS FOR SHALLOW FOUNDATIONS

Final Report
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APPENDIX G
BIAS CALCULATION EXAMPLES

Prepared for
National Cooperative Highway Research Program
Transportation Research Board
National Research Council

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G.1.1 Given Data: Footings in Granular Soils: FOTID #35 in UML-GTR ShalFound07

The tested footing data is from the source Briaud and Gibbens (1994). The soil profile is given in Table G1-1, and the reported soil parameters are listed in Table G1-2. Figure G1-1 shows the observed SPT-N counts for the subsurface. Further data about FotID #35 are:

- Footing dimension: $L \times B$ = 39in×39in = 3.25ft×3.25ft
- Embedment depth: D_f = 28in = 2.33ft
- Footing thickness: 46in
- Depth of groundwater table is 16.0ft > 7.21ft (=1.5B + D_f), hence there is no effect of GWT.
- The average relative density of the soil layer to a depth of 2B below the footing base is about 50%.

Table G1-1. Soil profile

Depth (ft)	Soil Description
11.5	medium dense tan silty fine Sand
23.0	medium dense silty Sand w/ clay and gravel
36.1	medium dense silty Sand to sandy clay w/gravel
108.3	very hard dark Clay

Table G1-2. Reported soil unit weight and soil friction angle of the subsoil

(a)

Depth (ft)	Unit wt (pcf)
1.0	116.59
3.0	120.42
4.9	119.78
6.9	116.59
9.8	117.23
11.8	124.88
15.7	122.97
19.7	121.05
24.6	126.15
29.5	110.43

(b)

Depth (ft)	ϕ_f (deg)
2.0	33.2
3.9	33.9
5.9	33.6
7.9	29.2
9.8	29.4
12.1	27.0
14.1	31.1

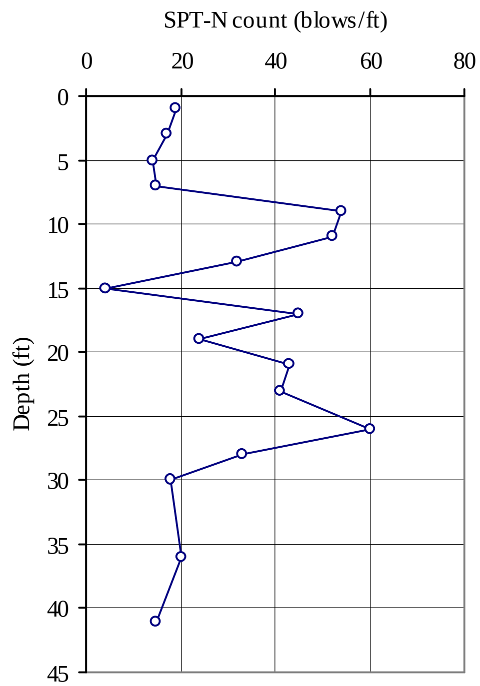

Figure G1-1. SPT-N counts of the subsurface

G.1.2 Interpreted Measured Failure Load

Considering the average relative density of the soil below the footing, the failure of the footing in local shear failure mode can be expected. In the load-settlement curve for the footing presented in Figure G1-2, it can be observed that the minimum slope starts at a load of 13.94tsf (S_e/B = 7.8%). Hence, using the Minimum Slope criterion (Vesić, 1963), the interpreted failure (ultimate) load capacity of the footing is $q_{u,\mathrm{meas}}$ = 13.94tsf (1335kPa).

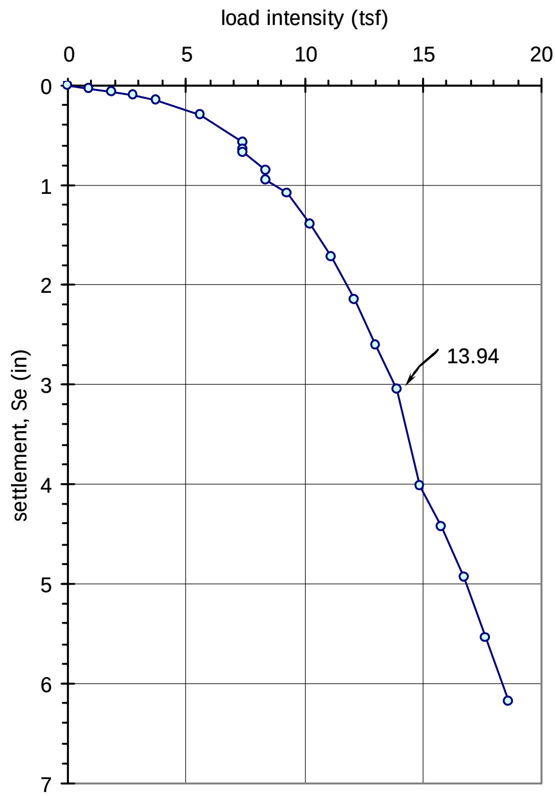

Figure G1-2. Load-settlement curve for FotID #35 footing

G.1.3 Ultimate Bearing Capacity (Vesic, 1975 and AASHTO, 2007)

The bearing capacity q_u of the footing is given by equation (34)

$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + \frac{1}{2}\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma \tag{34}$$

where $q = \sum_{D_f} (\gamma_i D_i)$ and $B' = B - 2e_B$, e_B being load eccentricity along width B. For this example, cohesion c = 0, hence only the terms with subscripts q and γ are considered. Also, $e_B = 0$, hence, $B' = B$ and $L' = L$.

The soil parameters for the bearing capacity calculation are taken as the weighted average of the parameters of each layer, usually considered up to a depth of 2B below footing base, i.e., the influence depth = 2B + D_f = 8.83ft below ground level.

Here, the average (weighted) of soil friction angle to a depth 2B below footing base is

$$\phi_f = \frac{(3.9-2.33)\times 33.9 + (5.9-3.9)\times 33.6 + (7.9-5.9)\times 29.2 + (8.83-7.9)\times 29.4}{(8.83-2.33)} = 31.72^\circ$$

Similarly, the average (weighted) of soil unit weight to a depth 2B below footing base is

$$\gamma = \frac{(3.0-2.33)\times120.42+(4.9-3.0)\times119.78+(6.9-4.9)\times116.59+(8.83-6.9)\times117.23}{(8.83-2.33)}$$

$$= 118.11\text{pcf}$$

Bearing capacity factors (equations (21) and (29)):

$$N_q = \exp(\pi \tan\phi_f)\cdot\tan^2(45+0.5\phi_f)$$

$$= \exp(3.1416\times\tan 31.72)\cdot\tan^2(45+0.5\times31.72) = 22.43$$

$$N_\gamma = 2(N_q+1)\cdot\tan\phi_f = 2(22.43+1)\cdot\tan(31.72) = 28.97$$

Shape factors:

$$s_q = 1+\frac{B'}{L'}\cdot\tan\phi_f = 1+\tan(31.72) = 1.618$$

$$s_\gamma = 1-0.4\frac{B'}{L'} = 0.6$$

Depth factors:

Here, $D_f / B' = 28/39 = 0.718 < 1.0$. Hence,

$$d_q = 1+2\tan\phi_f(1-\sin\phi_f)^2(D_f / B')$$

$$= 1+2\tan(31.72)\cdot(1-\sin 31.72)^2\times0.718 = 1.199$$

$$d_\gamma = 1.0$$

Bearing capacity:

$$q = \sum_{D_f}(\gamma_i D_i) = 116.59\times1.0+120.42(2.33-1.0) = 277.15\text{psf}$$

$$q_{u,\text{calc}} = qN_q s_q d_q + \frac{1}{2}\gamma B' N_\gamma s_\gamma d_\gamma$$

$$= 277.15\times22.43\times1.618\times1.199+0.5\times118.11\times3.25\times28.97\times0.6\times1.0$$

$$= 12059.85+3336.11 \quad \text{(psf)}$$

$$= 15.40\text{ksf} = 7.70\text{tsf}$$

G.1.4 Bias in the Bearing Capacity

The bias, defined as the ratio of measured to calculated bearing capacities, for the current footing is:

$$\lambda = \frac{q_{u,\text{meas}}}{q_{u,\text{calc}}} = \frac{13.94}{7.70} = 1.81$$

G.2 BIAS DETERMINATION FOR BEARING CAPACITY OF A FOOTING UNDER VERTICAL ECCENTRIC LOADING

G.2.1 Given Data: Footings in Granular Soils: FOTID #471 in UML-GTR ShalFound07

The tested footing data is from the source Perau (1995) (PeB1.6). The soil profile and the reported soil parameters are given in Table G2-1. Further data about FotID #471 are as follows:

- Footing dimension: $L \times B$ = 3.54in×3.54in (0.09m×0.09m)
- Embedment depth: D_f = 0in
- Groundwater table is not present.
- Depth of test pit = 11.4in (0.29m)
- The average relative density of the soil layer is 84.5%.
- Load eccentricity along the footing width = e_B = 0.91in (0.023m)

Table G2-1. Soil profile

Depth (ft)	Soil Description	Unit Wt (pcf)	ϕ_f (deg)
0.95	medium to coarse Sand, dense to very dense	110.73	44.93

G.2.2 Interpreted Measured Failure Load

In the load-settlement curve for the footing presented in Figure G2-1 for the load test carried out, it can be observed that the minimum slope starts at a load of about 150.0lbs ($S_e/B \approx 8\%$). Hence, using the Minimum Slope criterion (Vesić, 1963), the interpreted failure (ultimate) load capacity of the footing is $Q_{u,\text{meas}}$ = 150.0lbs.

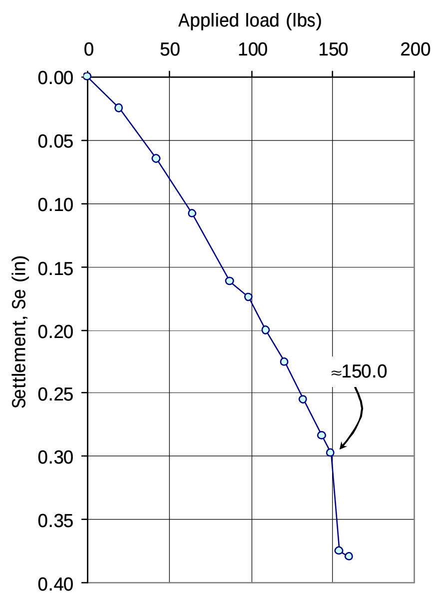

Figure G2-1. Load-settlement curve for FotID #471 footing

G.2.3 Ultimate Bearing Capacity (Vesić, 1975 and AASHTO, 2007)

The bearing capacity q_u of the footing is given by

$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + \frac{1}{2}\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma \tag{34}$$

where $q = \sum_{D_f} (\gamma_i D_i)$ and $B' = B - 2e_B$. For this example, cohesion c = 0, hence only the terms with subscripts q and γ are considered. Here, e_B = 0.91in, hence, B' = 1.73in (= 0.09 − 2 × 0.023 = 0.044m) and $L' = L$. Since the subsoil is homogeneous dense sand, the soil parameters are taken as reported in Table G2-1.

Bearing capacity factors:

$$N_q = \exp(\pi \tan\phi_f)\cdot \tan^2(45+0.5\phi_f)$$
$$= \exp(3.1416\times \tan 44.93)\cdot \tan^2(45+0.5\times 44.93) = 133.47$$
$$N_\gamma = 2(N_q+1)\cdot \tan\phi_f = 2(133.47+1)\cdot \tan(44.93) = 268.32$$

Shape factors:

$$s_q = 1+\frac{B'}{L'}\cdot \tan\phi_f = 1+\frac{1.73}{3.54}\tan(44.93) = 1.50$$
$$s_\gamma = 1-0.4\frac{B'}{L'} = 1-0.4\frac{1.73}{3.54} = 0.80$$

Depth factors:

Here, $D_f / B' = 0$. Hence, the term with subscript q in the BC equation is zero and $d_\gamma = 1.0$.

Bearing capacity:

$$q_{u,\text{calc}} = qN_q s_q d_q + \frac{1}{2}\gamma B' N_\gamma s_\gamma d_\gamma$$
$$= 0.0 + 0.5\times 110.73\times (1.73/12)\times 268.32\times 0.80\times 1.0$$
$$= 0.0 + 1714.0 \quad \text{(psf)}$$
$$= 1.714\text{ksf}$$

i.e.,

$$Q_{u,calc} = 1714.0\times(1.73\times 3.54)/144 = 73.0\text{lbs}$$

G.2.4 Bias in the Bearing Capacity

The bias, defined as the ratio of measured to calculated bearing capacities, for the current footing is:

$$\lambda = \frac{Q_{u,\text{meas}}}{Q_{u,\text{calc}}} = \frac{150.0}{73.0} = 2.06$$

G.3 BIAS DETERMINATION FOR BEARING CAPACITY OF A FOOTING UNDER INCLINED CENTRIC LOADING

G.3.1 Given Data: Footings in Granular Soils: FOTID #547 in UML-GTR ShalFound07

The tested footing data is from the source Gottardi (1992) (GoD6.3). The soil profile and the reported soil parameters are given in Table G3-1. Further data about FotID #547 are as follows:

- Footing dimension: $L \times B$ = 19.70in×3.94in (0.50m×0.10m)
- Embedment depth: D_f = 0in
- Groundwater table is not present.
- Depth of test pit = 1.0ft (0.3m)
- The average relative density of the soil layer is 86.0%.
- Load inclination to the vertical = δ = 6.25°; load applied in radial load path at 90° to the longitudinal side, i.e., θ = 90°.

Table G3-1. Soil profile

Depth (ft)	Soil Description	Unit Wt (pcf)	ϕ_f (deg)
1.0	Dense Adige Sand	102.13	44.84

G.3.2 Interpreted Measured Failure Load

The load-displacement curves obtained from the load test of the footing is presented in Figure G3-1. In the vertical load vs. settlement curve, it can be observed that the slope of the curve changes from positive to negative when the applied vertical component of the inclined load is 2.16kips, meaning failure takes place at this point. Since the load has been applied in the radial load path, the corresponding horizontal component at this failure point is given by:

$$F_{3,ult} = F_{1,ult} \times \tan\delta = 2.16 \times \tan(6.25) = 0.24\text{kips}$$

Upon examination of the horizontal load vs. horizontal displacement curve, it can be seen that the abrupt change in slope occurred when the horizontal component of the inclined load is about 0.24kips. This suggests that the footing bearing capacity failure observed in both horizontal and vertical load-displacements curves coincide. Hence, as concluded in Chapter 3, interpretation of the failure load form only the vertical load vs. settlement curve suffices. Thus, using the Minimum Slope criterion (Vesić, 1963), the interpreted failure (ultimate) load capacity of the footing is established as $Q_{u,\text{meas}}$ = 2.16kips.

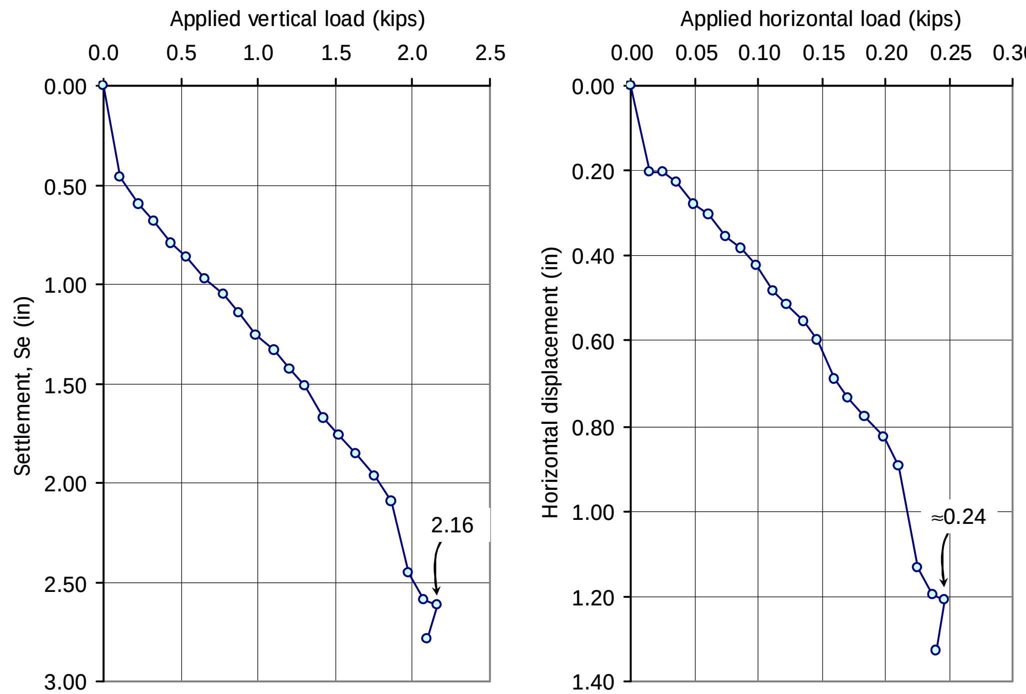

Figure G3-1. Load-displacement curves for loads and displacements in vertical and horizontal directions for FotID #547 footing, respectively.

G.3.3 Ultimate Bearing Capacity (Vesić, 1975 and AASHTO, 2007)

The bearing capacity q_u of the footing is given by

$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + \frac{1}{2}\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma \tag{34}$$

where $q = \sum_{D_f}(\gamma_i D_i)$. For this example, $D_f = 0$ and cohesion $c = 0$, hence only the term with subscript γ is considered. $B' = B - 2e_B = B$ since $e_B = 0$. Since the subsoil is homogeneous dense sand, the soil parameters are taken as reported in Table G3-1.

Bearing capacity factors:

$$N_q = \exp(\pi \tan\phi_f)\cdot\tan^2(45+0.5\phi_f)$$

$$= \exp(3.1416\times\tan 44.84)\cdot\tan^2(45+0.5\times 44.84) = 131.49$$

$$N_\gamma = 2(N_q+1)\cdot\tan\phi_f = 2(131.49+1)\cdot\tan(44.84) = 263.51$$

Shape factors:

$$s_\gamma = 1-0.4\frac{B'}{L'} = 1-0.4\frac{3.94}{19.7} = 0.92$$

Depth factors:

$$d_\gamma = 1.0$$

Load inclination factors:

Since θ = 90°,

$$n = \frac{2 + B'/L'}{1 + B'/L'} \cdot 1.0 = 1.833$$

$$i_\gamma = \left(1 - \frac{F_3}{F_1}\right)^{n+1} = \left(1 - \tan\delta\right)^{n+1} = \left(1 - \tan(6.25)\right)^{(1.833+1)} = 0.720$$

Bearing capacity:

$$\begin{aligned} q_{u,\mathrm{calc}} &= qN_q s_q d_q + \frac{1}{2}\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma \\ &= 0.0 + 0.5 \times 102.13 \times (3.94/12) \times 263.5 \times 0.92 \times 1.0 \times 0.720 \\ &= 0.0 + 2926.4 \quad \text{(psf)} \\ &= 2926.4\text{psf} \end{aligned}$$

i.e.,

$$Q_{u,calc} = 2926.4 \times (19.7 \times 3.94)/144 \times 10^{-3}\,\text{(kips)} = 1.58\text{kips}$$

G.3.4 Bias in the Bearing Capacity

The bias, defined as the ratio of measured to calculated bearing capacities, for the current footing is:

$$\lambda = \frac{Q_{u,\mathrm{meas}}}{Q_{u,\mathrm{calc}}} = \frac{2.16}{1.58} = 1.36$$

G.4 BIAS DETERMINATION FOR BEARING CAPACITY OF A FOOTING UNDER INCLINED ECCENTRIC LOADING

G.4.1 Given data: Footings in Granular Soils: FOTID #504 in UML-GTR ShalFound07

The tested footing data is from the source Perau (1995) (PeE1.12). The soil profile and the reported soil parameters are given in Table G4-1. Further data about FotID #504 are as follows:

- Footing dimension: $L \times B$ = 3.54in×3.54in (0.09m×0.09m)
- Embedment depth: D_f = 0in
- Groundwater table is not present.
- Depth of test pit = 11.4in (0.29m)
- The average relative density of the soil layer is 89.7%.
- Inclined load applied in a step-like load path at 90° to the longitudinal side, i.e., $\theta = 90°$.
- 1-way load eccentricity along the footing width, e_B = 0.59in (0.015m) generating positive moment (refer to Chapter 3 for sign conventions).

Table G4-1. Soil profile

Depth (ft)	Soil Description	Unit Wt (pcf)	ϕ_f (deg)
0.95	medium to coarse Sand, dense to very dense	110.41	44.74

G.4.2 Interpreted Measured Failure Load

The load-displacement curves obtained from the load test of the footing is presented in Figure G4-1. In the vertical load vs. settlement curve (left), it can be observed that the curve changes abruptly when the applied vertical component of the inclined load is 172.4lbs, meaning failure takes place at this point. Hence, the vertical component of the ultimate load $F_{1,ult}$ (= $Q_{u,meas}$) is 172.4lbs. Similar failure load can be identified in the horizontal load vs. horizontal displacement curve (right). The horizontal component of the applied inclined load thus identified is $F_{3,ult}$ = 10.8lbs. Since the load has been applied in a step-like load path, the angle of load inclination at failure is given by:

$$\delta = \arctan\left(\frac{F_{3,ult}}{F_{1,ult}}\right) = \arctan\left(\frac{10.8}{172.4}\right) = 3.6°$$

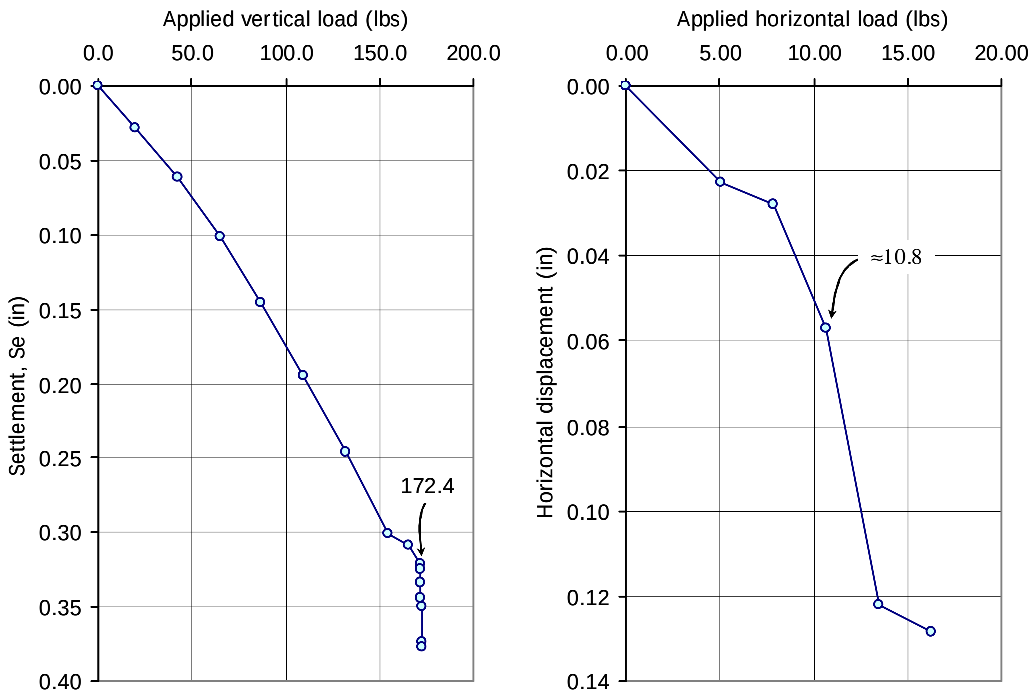

Figure G4-1. Load-displacement curves for loads and displacements in vertical and horizontal directions for FotID #504 footing, respectively.

G.4.3 Ultimate Bearing Capacity (Vesic, 1975 and AASHTO, 2007)

The bearing capacity q_u of the footing is given by

$$q_u = cN_c s_c d_c i_c + qN_q s_q d_q i_q + \frac{1}{2}\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma \tag{34}$$

where $q = \sum_{D_f}(\gamma_i D_i)$. For this example, $D_f = 0$ and cohesion $c = 0$, hence only the term with subscript γ is considered.

Effective width, $B' = B - 2e_B = 2.36\text{in}$ (=0.09 – 2×0.015 = 0.06m)

Since the subsoil is homogeneous dense sand, the soil parameters are taken as reported in Table G4-1.

Bearing capacity factors:

$$N_q = \exp(\pi \tan\phi_f)\cdot\tan^2(45+0.5\phi_f)$$
$$= \exp(3.1416\times\tan 44.75)\cdot\tan^2(45+0.5\times 44.75) = 129.64$$
$$N_\gamma = 2(N_q+1)\cdot\tan\phi_f = 2(129.64+1)\cdot\tan(44.75) = 259.00$$

Shape factors:

$$s_\gamma = 1 - 0.4\frac{B'}{L'} = 1 - 0.4\frac{2.36}{3.54} = 0.733$$

Depth factors:

$$d_\gamma = 1.0$$

Load inclination factors:

Since θ = 90°,

$$n = \frac{2 + B'/L'}{1 + B'/L'} \cdot 1.0 = 1.60$$

$$i_\gamma = \left(1 - \frac{F_3}{F_1}\right)^{n+1} = \left(1 - \frac{F_{3,ult}}{F_{1,ult}}\right)^{(n+1)} = \left(1 - \frac{10.8}{172.4}\right)^{(1.60+1)} = 0.845$$

Bearing capacity:

$$\begin{aligned} q_{u,\text{calc}} &= qN_q s_q d_q + \frac{1}{2}\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma \\ &= 0.0 + 0.5 \times 110.41 \times (2.36/12) \times 259.0 \times 0.733 \times 1.0 \times 0.845 \\ &= 0.0 + 1741.7 \quad \text{(psf)} \\ &= 1741.7\text{psf} \end{aligned}$$

i.e.,

$$Q_{u,calc} = 1741.7 \times (3.54 \times 2.36)/144 = 101.05\text{lbs}$$

G.4.4 Bias in the Bearing Capacity

The bias, defined as the ratio of measured to calculated bearing capacities, for the current footing is:

$$\lambda = \frac{Q_{u,\text{meas}}}{Q_{u,\text{calc}}} = \frac{172.4}{101.05} = 1.71$$

G.5 BIAS DETERMINATION FOR BEARING CAPACITY OF A FOOTING ON ROCK USING GOODMAN'S (1989) METHOD FOR PLATE LOAD TEST DATA

G.5.1 Given Data: UML-GTR RockFound07 Database Table: E-3 of Appendix E

- Database Case No.: 122
- Type of Load Test: Plate Load Test
- Rock Description: Sandstone
- Interpreted Foundation Capacity (q_{L2}): 334.17 ksf
- Rock Properties: Friction angle (ϕ) = 30^{o} Uniaxial compressive strength (q_u) = 83.54 ksf
- Discontinuity Spacing: Fractured

Using Equation (77):

$$N_f = \tan^2\left(45 + f/2\right) \quad (77)$$

where ϕ = internal friction angle

Substituting ϕ into equation (77):

$$N_f = \tan^2\left(45 + 30/2\right) = 3$$

Using equation (79):

$$q_{ult} = q_u\left(N_f + 1\right) \quad (79)$$

where q_u = uniaxial compressive strength of the intact rock

Substituting q_u and N_ϕ values into equation (77):

$$q_{ult} = 83.54\left(3+1\right) = 334.17 ksf$$

The bias of Goodman's (1989) method in case no. 122:

$$l = \frac{measured\ capacity}{calculated\ capacity} = \frac{q_{L2}}{q_{ult}} = \frac{334.17}{334.17} = 1.00$$

G.6 BIAS DETERMINATION FOR BEARING CAPACITY OF A FOOTING ON ROCK USING GOODMAN'S (1989) METHOD FOR ROCK SOCKET LOAD TEST DATA

G.6.1 Given Data: UML-GTR RockFound07 Database Table: E-3 of Appendix E

- Database Case No.: 9
- Type of Load Test: Rock Socket
- Rock Description: Fractured clay-shale
- Interpreted Foundation Capacity (q_{L2}): 114.87 ksf
- Rock Properties: Friction angle (ϕ) = 23.5°
- Uniaxial compressive strength (q_u) = 29.66 ksf
- Discontinuity Spacing: Fractured

Using equation (77):

$$N_f = \tan^2\left(45 + f\!\!\left/\!2\right.\right) \tag{77}$$

where ϕ = internal friction angle

Substituting ϕ into equation (77):

$$N_f = \tan^2\left(45 + 23.5\!\left/\!2\right.\right) = 2.33$$

Using equation (79):

$$q_{ult} = q_u\left(N_f + 1\right) \tag{79}$$

where q_u = uniaxial compressive strength of the intact rock

Substituting q_u and N_ϕ values into equation (79):

$$q_{ult} = 29.66\left(2.33 + 1\right) = 98.65 ksf$$

The bias of Goodman's (1989) method in case no. 9:

$$l = \frac{measured\ capacity}{calculated\ capacity} = \frac{q_{L2}}{q_{ult}} = \frac{114.87}{98.65} = 1.16$$

G.7 BIAS DETERMINATION FOR BEARING CAPACITY OF A FOOTING ON ROCK USING CARTER AND KULHAWY (1988) METHOD FOR

G.7.1 Given Data: UML/GTR RockFound07 Database Table: E-2 of Appendix E

- Database Case No.: 122
- Type of Load Test: Plate Load Test
- Rock Description: Fractured sandstone
- Rock Quality: Good
- Interpreted Foundation Capacity (q_{L2}): 334.17 ksf
- Uniaxial Compressive Strength (q_u): 83.54 ksf
- Rock Type: C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage – ***sandstone and quartzite*** (see Table 2-25 (AASHTO, 2007 Table 10.4.6.4-4)
- Strength Parameters of the Rockmass: m = 1.231 and s = 0.00293 Table 2-25, (AASHTO, 2007 Table 10.4.6.4-4)

Using Equation (82):

$$q_{ult} = \left(m + \sqrt{s}\right) q_u \tag{82}$$

where q_u = uniaxial compressive strength of the intact rock
s and m = empirically determined strength parameters for the rockmass, which are somewhat analogous to c and ϕ of the Mohr-Coulomb failure criterion

Substituting q_u, m and s values into Equation (82):

$$q_{ult} = \left(1.231 + \sqrt{0.00293}\right) 83.54 = 107.36 \boldsymbol{ksf}$$

The bias of Carter and Kulhawy's (1988) method in case no. 122:

$$l = \frac{measured\ capacity}{calculated\ capacity} = \frac{q_{L2}}{q_{ult}} = \frac{334.17}{107.36} = 3.11$$

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