NCHRP 24-31

LRFD DESIGN SPECIFICATIONS FOR SHALLOW FOUNDATIONS

Final Report
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APPENDIX H
DESIGN EXAMPLES

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H.1 EXAMPLE 1: BRIDGE PIER ON NATURAL SOIL DEPOSITS – GEC6-EXAMPLE 1

H.1.1 Subsurface Condition

The subsurface conditions given in Example C1 of FHWA Geotechnical Engineering Circular No. 6 (GEC6), Appendix C (Kimmerling, 2002) are summarized in Table H-1. The groundwater table is at a depth of 30.0ft below the ground surface and the soil unit weight is assumed to be 125pcf for all the layers. The soil friction angles are calculated using the correlation with SPT blow counts proposed by Peck, Hanson and Thornburn (PHT) as modified by Kulhawy and Mayne (1990). This calculation is compatible with the methodology used in developing the resistance factors. The footing is to be cast in-situ on the silty sand layer.

TABLE H-1. Soil parameters – Example 1 (GEC6-Example 1)

Depth (ft)	SPT
2.5	6
5.0	7
7.5	18
10.1	20
12.6	22
15.1	42
20.0	38
24.9	47
29.9	33
34.8	45
39.7	49
44.6	42
49.5	37

Layer #	Depth (ft)	Soil Description	g (pcf)	f_f (deg)
1	7.55	Lean Clay	124.9	not needed
2	14.4	Silty Sand	124.9	34.5
3a	30.0	Well-graded Sand above GW	124.9	37.5
3b	39.7	Well-graded Sand below GW	62.4	36.0
4	49.5	Clean, uniform Sand	62.4	35.0

* Groundwater table present at a depth of 30.0ft

H.1.2 Loads, Load Combinations and Limit States

The loading from the structure at the footing base is presented in Table H-2. The notations for the loadings and the sign conventions used in the calculation follow Figure 120 of Chapter 5, hence the moments M_y and M_z in Figure H-1 correspond to M_3 and M_2, respectively, and vertical load P to F_1. F_2 is the horizontal loading along the transverse direction of the bridge (along y-axis). It should be noted that all load components are one-way inclined (across the bridge) and two-way eccentric. In addition to the loadings given in Table H-2, the weight of the footing and the soil above the footing have been considered as a vertical-centric load of 519.2 kips (1154.02 kN).

Table H-3 includes the investigated load combinations and the resultant characteristic loading as well as the resultant load inclination F_2/F_1 and the eccentricity in both directions; $e_2 = e_L$ and $e_3 = e_B$ for the different load combinations. Here, $M_2 = M_z$ and $M_3 = M_y$ and for the square footing B = L (see Figure 120 of Chapter 5 and Figure H-1). Table H-4 summarizes the load factors for the strength limit state applied to the bearing capacity calculations (Table H-4.1) and the sliding calculations (Table H-4.2).

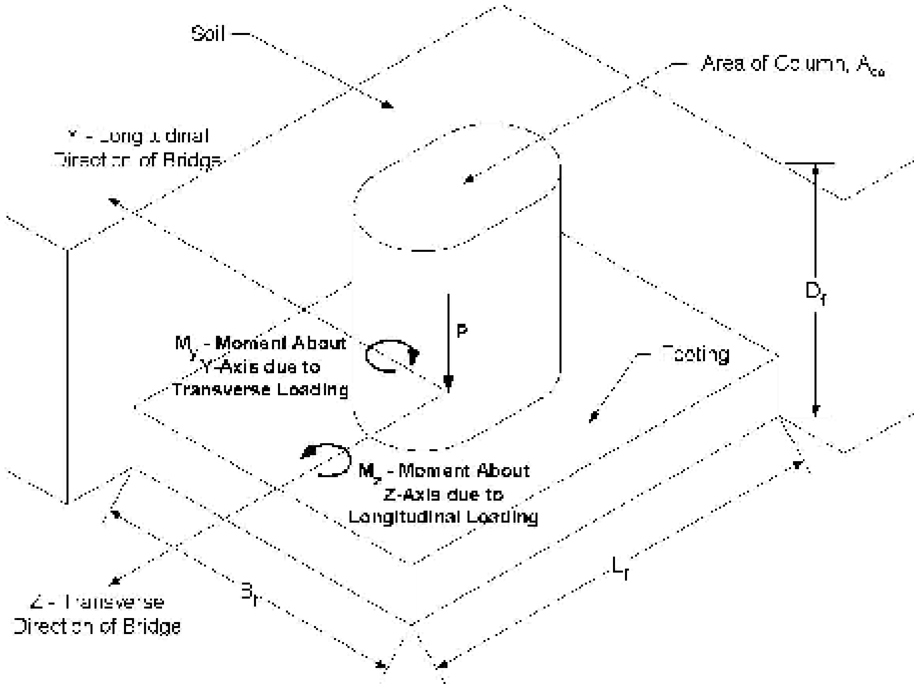

Figure H-1. Geometry of interior bridge pier founded on spread footing – example 1 (GEC6-Example 1)

TABLE H-2. Load at the column base of the bridge pier

Load at Column Base	F_1 kips (kN)	F_2 kips (kN)	M_2 kip-ft (kNm)	M_3 kip-ft (kNm)
dead load (DL)	1438.7 (6400.0)	37.5 (167.0)	155.6 (211.0)	551.5 (748.0)
live load (LL)	375.4 (1670.0)	9.4 (42.0)	301.6 (409.0)	144.9 (196.5)
impact (IM) (neglected)	70.8 (315.0)	1.8 (8.0)	56.8 (77.0)	27.3 (37.0)
wind on structure (WS)	198.7 (884.0)	11.0 (49.0)	65.6 (89.0)	166.6 (226.0)
wind on live load (WL)	4.0 (18.0)	0.9 (4.0)	5.2 (7.0)	19.2 (26.0)
earthquake (EQ)	375.6 (1671.0)	180.7 (804.0)	1235.1 (1675.0)	4089.3 (5546.0)

TABLE H-3. Load combinations and resultant characteristic (unfactored) loading

Load Combinations	F_1 kips (kN)	F_2 kips (kN)	M_2 kips-ft (kNm)	M_3 kips-ft (kNm)	F_2/F_1	$e_L = M_3/F_1$ ft (m)	$e_B = M_2/F_1$ ft (m)
Service-I: DL+LL+WS+WL	2137.2 (9507.2)	51.2 (227.7)	482.0 (653.7)	765.6 (1038.3)	0.024	0.358 (0.109)	0.226 (0.069)
Strength-I: DL+LL	2073.6 (9224.0)	47.0 (209.0)	457.2 (620.0)	696.4 (944.5)	0.023	0.335 (0.102)	0.220 (0.067)
Extreme-I: DL+EQ	2073.8 (9225.0)	218.3 (971.0)	1390.6 (1886.0)	4640.8 (6294.0)	0.105	2.237 (0.682)	0.669 (0.204)

TABLE H-4.1 Load factors used for the bearing capacity strength limit state

Load Combination Limit State	DL	DW	EH	LL, IM, CE, BR, PL, LS, EL	WA	WS	WL	FR	EQ
Strength-I	1.25	1.5	1.5	1.75	1	0.0	0	1	0
Strength-II	1.25	1.5	1.5	1.35	1	0.0	0	1	0
Strength-III	1.25	1.5	1.5	0.00	1	1.4	0	1	0
Strength-IV	1.25	1.5	1.5	0.00	1	0.0	0	1	0
DC ONLY	1.50	1.5	1.5						
Strength-V	1.25	1.5	1.5	1.35	1	0.4	1	1	0
Extreme Event-I	1.25	1.5	1.5	γ_{EQ}	1	0.0	0	1	1
Extreme Event-II	1.25	1.5	1.5	0.50	1	0.0	0	1	0
Service-I	1.00	1.0	1.0	1.00	1	0.3	1	1	0
Service-II	1.00	1.0	1.0	1.30	1	0.0	0	1	0
Service-III	1.00	1.0	1.0	0.80	1	0.0	0	1	0

γ_{EQ} shall be determined on project-specific basis γ_{EQ} =0 or 1 (0 in the example)

TABLE H-4.2 Load factors used for the sliding strength limit state

Load Combination Limit State	DL	DW	EH	LL, IM, CE, BR, PL, LS, EL	WA	WS	WL	FR	EQ
Strength-I	0.9	0.65	1.5	1.75	1	0.0	0	1	0
Strength-II	0.9	0.65	1.5	1.35	1	0.0	0	1	0
Strength-III	0.9	0.65	1.5	0.00	1	1.4	0	1	0
Strength-IV	0.9	0.65	1.5	0.00	1	0.0	0	1	0
DC ONLY	1.5	1.50	1.5						
Strength-V	0.9	0.65	1.5	1.35	1	0.4	1	1	0
Extreme Event-I	0.9	0.65	1.5	γ_{EQ}	1	0.0	0	1	1
Extreme Event-II	0.9	0.65	1.5	0.50	1	0.0	0	1	0
Service-I	1.0	1.00	1.0	1.00	1	0.3	1	1	0
Service-II	1.0	1.00	1.0	1.30	1	0.0	0	1	0
Service-III	1.0	1.00	1.0	0.80	1	0.0	0	1	0

γ_{EQ} shall be determined on project-specific basis γ_{EQ} =0 or 1 (0 in the example)

The calculation of the bearing resistance and the sliding resistance is based on the characteristic load components (i.e., load eccentricity and inclination are obtained from unfactored load components) as given in Table H-3. However, for stability analysis, the design load components are required, which are summarized in Table H-5. The loads for Service-I are not factored (except for WS component), whereas, those for Strength-I and Extreme-I are factored by the load factors specified in Section 3 of the AASHTO (2007) specifications and provided in Tables H-4.1 and H-4.2. As the different limiting conditions make use of different factors (e.g. increased vertical loading for bearing capacity evaluation and decreased vertical loading for friction resistance in sliding evaluation), Table H-5 was divided to represent the loading of both limiting strength states corresponding to the factors presented in Tables H-4.1 and H-4.2. It should be noted that as the lower limit of the dead load is used for the vertical load utilized in sliding analysis, the lateral load is reduced as well. In this design example, only Service-I and Strength-I limit states have been considered to determine the design footing width.

TABLE H-5.1. Load combinations and resultant design (factored) loading required for bearing resistance

Load Combinations	F_1 kip (kN)	F_2 kip (kN)	M_2 kip-ft (kNm)	M_3 kip-ft (kNm)
Service-I: DL+LL+WS+WL	2137.2 (9507.2)	51.2 (227.7)	482.0 (653.7)	765.6 (1038.3)
Strength-I: DL+LL	2779.7 (12365.0)	63.4 (282.3)	722.2 (979.5)	943.0 (1278.9)
Extreme-I: DL+EQ	2498.3 (11113.5)	227.7 (1012.8)	1429.5 (1938.8)	4778.7 (6481.0)

TABLE H-5.2. Load combinations and resultant design (factored) loading required for sliding resistance

Load Combinations	F_1 kip (kN)	F_2 kip (kN)	M_2 kip-ft (kNm)	M_3 kip-ft (kNm)
Service-I: DL+LL+WS+WL	2137.2 (9507.2)	51.2 (227.7)	482.0 (653.7)	765.6 (1038.3)
Strength-I: DL+LL	2185.3 (9721.1)	50.2 (223.8)	667.8 (905.7)	749.9 (1017.1)
Extreme-I: DL+EQ	1904.0 (8469.6)	214.5 (954.3)	1375.1 (1864.9)	4585.7 (6219.2)

H.1.3 Soil Parameter Estimation

The soil friction angle ϕ_f has been estimated using the correlation proposed by Peck, Hanson and Thornburn as modified by Kulhawy and Mayne (1990), Equation (H-1), based on the corrected SPT values $(N1)_{60}$ at the layer mid-heights.

$$\phi_f \approx 54 - 27.6034 \cdot \exp\left(-0.014 (N1)_{60}\right) \tag{H-1}$$

Table H-6 shows the friction angles estimated using the correlation. For layer# 2.1, for example: overburden at layer mid-height, $\sigma_v = \left(7.5 + (10.1 - 7.5)/2\right) \times 124.9 = 1099.12$psf =0.550tsf

And,

$$(N1)_{60} = N_{60}\sqrt{1/\sigma_v} = 20 \times \sqrt{1/0.55} = 26.98$$

$$\phi_f = 54 - 27.6034 \exp\left(-0.014 \times 26.98\right) = 35.08°$$

Table H-6. Estimation of soil friction angle from SPT N counts

Layer #	Depth (ft)	N_{60}	Layer mid-height overburden σ_v (tsf)	Corrected $(N1)_{60}$ (Liao and Whitmann 1986)	ϕ_f (deg) (PHT 1990)
1.1	2.5	6	0.079	21.37	lean clay
1.2	5.0	7	0.235	14.43	
1.3	7.5	18	0.392	28.75	
2.1	10.1	20	0.550	26.98	35.08
2.2	12.6	22	0.707	26.16	34.86
2.3	15.1	42	0.864	45.19	39.34
3a.1	20.0	38	1.095	36.31	37.40
3a.2	24.9	47	1.402	39.69	38.16
3a.3	29.9	33	1.709	25.24	34.61
3b.1	34.8	45	1.939	32.31	36.44
3b.2	39.7	49	2.093	33.87	36.82
4.1	44.6	42	2.246	28.02	35.35
4.2	49.5	37	2.400	23.88	34.24

The required soil parameters have been taken as the weighted average of the parameters of each layer to a depth of $2B + D_f$., considered as the influence depth from the ground level.
E.g.
For footing width of B = 4.9ft placed at an embedment depth of 7.5ft:
The depth of influence for bearing capacity calculation is $2B + D_f = 17.4$ft. Hence,

$$\text{average } \phi_f = \frac{(10.1-7.5)\,35.08 + (12.6-10.1)\,34.86 + (15.1-12.6)\,39.34 + (17.4-15.1)\,37.40}{17.4-7.5} = 36.64$$

The average soil friction angle thus obtained hence varies according to the footing width.

H.1.4 Nominal Bearing Resistances at the Limit State

H.1.4.1 Footing Information: Embedment and Shape

The bearing resistances of square footings with widths 2.95ft to 20.70ft have been calculated. Since the soft lean clay is present at a shallow depth, underlain by stiffer sand layers, the footing has been considered to rest on the second soil layer, on silty sand, at an embedment depth of 7.55ft from the ground surface.

From Table H-3, the load eccentricities along the footing width and footing length are, respectively, $e_B = 0.220$ft and $e_L = 0.335$ft. Hence, for a trial footing width of, say, 4.9ft, the effective width $B' = B - 2e_B = 4.9 - 2\times0.220 = 4.48$ft and the effective length $L' = L - 2e_L = 4.9 - 2\times0.335 = 4.25$ft.

H.1.4.2 Bearing Capacity Factors

The bearing resistances have been calculated for Strength-I limit state according to AASHTO (2007) (equation 10.6.3.1.2), Equations (95) through (99) in the draft Final Report, with depth modification factor as mentioned in Table 28.

For cohesionless soils, $c = 0$, hence the bearing capacity factors required are given by Equations (H-2) and (H-3).

$$N_q = \exp\left(\pi \tan\phi_f\right)\cdot\tan^2\left(\frac{\pi}{4}+\frac{\phi_f}{2}\right) \quad \text{(H-2)}$$

$$N_\gamma = 2\left(N_q+1\right)\cdot\tan\phi_f \quad \text{(H-3)}$$

For B = 4.9ft, the average ϕ_f has been obtained as 36.64°. Hence

$$N_q = \exp\{\pi \tan(36.6)\}\tan^2(45+36.6/2) = 41.00\text{ , and}$$
$$N_\gamma = 2\,(41.0+1)\tan(36.6) = 62.46$$

H.1.4.3 Bearing Capacity Modification Factors

Shape factors:

$$s_q = 1+\tan\phi_f\,(B'/L') = 1+\tan(36.6)\cdot(4.48/4.25) = 1.784 \quad \text{(H-4a)}$$

$$s_\gamma = 1-0.4(B'/L') = 1-0.4(4.48/4.25) = 0.578 \quad \text{(H-4b)}$$

Depth factors:
For the current example, due to the presence of lean clay layer, the depth factor d_q is taken as 1.0.

Load inclination factors:
The bearing capacity modification factors for load inclination are given by Equations (H-6).

$$i_q = \left(1-\frac{H}{\left(V+A'\cdot c'\cdot\cot\phi_f\right)}\right)^n \quad \text{(H-6a)}$$

$$i_\gamma = \left(1-\frac{H}{\left(V+A'\cdot c'\cdot\cot\phi_f\right)}\right)^{n+1} \quad \text{(H-6b)}$$

where H and V are the horizontal and vertical components of the applied inclined load P (unfactored), A' is the effective area of footing, c' is soil cohesion; and

$$n = \left[\frac{\left(2+L'/B'\right)}{\left(1+L'/B'\right)}\right]\cos^2\theta + \left[\frac{\left(2+B'/L'\right)}{\left(1+B'/L'\right)}\right]\sin^2\theta \quad \text{(H-6c)}$$

where θ is the projected direction of load in the plane of the footing, measured from the side of length L in degrees; L' and B' are effective length and width.
Here the projected direction of the inclined load in the plane of the footing θ = 0°. Hence

$$n = \frac{2 + 4.25/4.48}{1 + 4.25/4.48} = 1.513$$

Then

$$i_q = \left(1 - \frac{47.0}{2073.6 + 0}\right)^{1.513} = 0.9659 \text{ and}$$

$$i_g = \left(1 - \frac{47.0}{2073.6 + 0}\right)^{(1.513+1)} = 0.9440$$

H.1.4.4 Modified Bearing Capacity Factors

$$N_{qm} = N_q s_q d_q i_q = 41.0 \times 1.784 \times 1.0 \times 0.9659 = 70.64 \text{ and}$$
$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 62.46 \times 0.578 \times 0.9440 = 34.10$$

H.1.4.5 Groundwater Table Modification Factors

For B = 4.9ft, the groundwater table is below the depth of 1.5B from the footing base as well as the footing embedment D_f;

$$1.5B + D_f = 1.5 \times 4.9 + 7.5 = 14.85\text{ft} < 29.9\text{ft (GWT)}$$

Therefore, the soil unit weights γ_1 and γ_2 are equal to γ. When $1.5B + D_f > GWT$, the soil unit weight below the footing base is taken as:

$$\gamma_2 = \gamma \times \left[1 - \frac{\gamma_w}{\gamma}\left(1 - \frac{D_w - D_f}{1.5B}\right)\right]$$

H.1.4.6 Bearing Capacity

The nominal (unfactored) bearing resistance of the footing of width 4.9ft calculated using the bearing capacity equation given in AASHTO (2007) is thus

$$\begin{aligned} q_u &= cN_{cm} + \gamma_1 D_f N_{qm} + 0.5\gamma_2 B' N_{\gamma m} \\ &= 0 + 124.9 \times 7.55 \times 70.64 + 0.5 \times 124.9 \times 4.25 \times 34.10 = 75.61\text{ksf} \end{aligned}$$

Table H-7 presents values of the average soil parameters, the bearing capacity factors and their modification factors and the calculated bearing capacity for footing widths 2.95ft to 20.67ft.

Table H-7. Detailed bearing capacity calculation for Example 1.

Soil parameters and GWT:

γ (pcf)	124.9
D_w (ft)	29.9

Footing information:

B/L	1.00	
D_f (ft)	7.55	
depth factor, d_q	1.00	(taken as 1.0 because of lean clay layer present till the depth of D_f from GL)
depth factor, d_γ	1.00	(Vesic 1975)

Load eccentricity and inclination:

inclination, H/V	0.023	(H along transverse dir.)
eccenticity, e_L	0.336	
eccenticity, e_B	0.221	

B (ft)	B'	L'	2B+D_f	avg ϕ_f	N_q	N_γ	s_q	s_γ	pow n	i_q	i_γ	N_{qm}	$N_{\gamma m}$	1.5B+D_f	γ_1	γ_2	q_n (ksf)	Q_n (kips)
2.95	2.51	2.28	13.5	35.6	35.90	52.84	1.788	0.560	1.524	0.9657	0.9438	62.00	27.90	12.0	124.9	124.9	62.40	357.4
3.94	3.50	3.27	15.4	36.5	40.01	60.59	1.791	0.572	1.517	0.9658	0.9439	69.20	32.70	13.5	124.9	124.9	71.88	820.4
4.92	4.48	4.25	17.4	36.6	40.99	62.46	1.784	0.578	1.513	0.9659	0.9440	70.64	34.10	14.9	124.9	124.9	75.61	1439.5
5.91	5.46	5.23	19.4	36.8	41.66	63.75	1.780	0.582	1.511	0.9660	0.9441	71.63	35.05	16.4	124.9	124.9	78.96	2258.0
6.89	6.45	6.22	21.3	36.9	42.54	65.46	1.780	0.585	1.509	0.9660	0.9441	73.14	36.16	17.9	124.9	124.9	82.96	3326.3
7.87	7.43	7.20	23.3	37.1	43.40	67.13	1.780	0.587	1.508	0.9660	0.9441	74.64	37.21	19.4	124.9	124.9	87.07	4660.8
8.86	8.42	8.19	25.3	37.1	43.71	67.72	1.779	0.589	1.507	0.9661	0.9442	75.11	37.64	20.8	124.9	124.9	90.02	6202.5
9.84	9.40	9.17	27.2	36.9	42.30	64.99	1.769	0.590	1.506	0.9661	0.9442	72.30	36.20	22.3	124.9	124.9	88.86	7661.1
10.83	10.39	10.15	29.2	36.7	41.19	62.85	1.762	0.591	1.506	0.9661	0.9442	70.10	35.06	23.8	124.9	124.9	88.29	9311.1
11.81	11.37	11.14	31.2	36.6	40.81	62.13	1.758	0.592	1.505	0.9661	0.9442	69.33	34.71	25.3	124.9	124.9	89.47	11331.5
12.80	12.35	12.12	33.1	36.6	40.74	62.00	1.757	0.592	1.505	0.9661	0.9442	69.15	34.68	26.7	124.9	124.9	91.41	13690.9
13.78	13.34	13.11	35.1	36.6	40.71	61.93	1.755	0.593	1.504	0.9661	0.9442	69.04	34.67	28.2	124.9	124.9	93.44	16336.1
14.76	14.32	14.09	37.1	36.6	40.79	62.09	1.755	0.593	1.504	0.9661	0.9442	69.16	34.79	29.7	124.9	124.9	95.78	19331.5
15.75	15.31	15.08	39.0	36.6	40.86	62.22	1.755	0.594	1.504	0.9661	0.9442	69.26	34.89	31.2	124.9	121.4	97.20	22430.0
16.73	16.29	16.06	41.0	36.6	40.62	61.76	1.753	0.594	1.504	0.9661	0.9442	68.78	34.66	32.6	124.9	117.9	97.64	25545.0
17.72	17.28	17.04	43.0	36.5	40.27	61.09	1.750	0.595	1.503	0.9661	0.9442	68.09	34.30	34.1	124.9	114.9	97.73	28777.4
18.70	18.26	18.03	44.9	36.4	39.91	60.40	1.748	0.595	1.503	0.9661	0.9442	67.38	33.93	35.6	124.9	112.1	97.78	32188.3
19.68	19.24	19.01	46.9	36.3	39.35	59.34	1.744	0.595	1.503	0.9661	0.9442	66.31	33.35	37.1	124.9	109.6	97.24	35576.6
20.67	20.23	20.00	48.9	36.2	38.86	58.40	1.741	0.595	1.503	0.9661	0.9443	65.36	32.83	38.5	124.9	107.4	96.83	39170.0

H.1.5 Allowable Bearing Resistance at the Limit State

H.1.5.1 Overview

The allowable bearing resistances for a Service limit state of allowable settlement of 1.5inches have been obtained using the AASHTO (2007) method (equation 10.6.2.4.2-1), Schmertmann (1978), and Hough (1959) settlement calculation methods.

1. Influence depth:
   The influence depth below the footing base for all settlement calculations has been calculated as given in Table H-8 below.

Table H-8. Influence depth below footing base for different footing shapes

L/*B* ratio	Influence depth below footing base
$0 < L/B \le 5$	2*B*
$5 < L/B < 10$	3*B*
$L/B \ge 10$	4*B*

2. Corrected SPT-N value and E_s from correlation with $(N1)_{60}$ at each layer mid-height:

Table H-9. Corrected SPT $(N1)_{60}$ values at mid-layer depths, their correlations with Young's modulus of elasticity E_s and values of E_s for each layer defined

Layer #	Depth (ft)	N_{60}	Mid-layer Overburden s_v (tsf)	$(N1)_{60}$ (Liao and Whitmann 1996)	E_s from $(N1)_{60}$ (AASHTO 2007) (tsf)	E_s (tsf)
1.1	2.5	6	0.079	21.37		lean clay
1.2	5.0	7	0.235	14.43		
1.3	7.5	18	0.392	28.75		
2.1	10.1	20	0.550	26.98	$7(N1)_{60}$	188.85
2.2	12.6	22	0.707	26.16	$7(N1)_{60}$	183.13
2.3	15.1	42	0.864	45.19	$7(N1)_{60}$	316.34
3a.1	20.0	38	1.095	36.31	$7(N1)_{60}$	254.20
3a.2	24.9	47	1.402	39.69	$7(N1)_{60}$	277.85
3a.3	29.9	33	1.709	25.24	$7(N1)_{60}$	176.70
3b.1	34.8	45	1.939	32.31	$7(N1)_{60}$	226.20
3b.2	39.7	49	2.093	33.87	$7(N1)_{60}$	237.10
4.1	44.6	42	2.246	28.02	$7(N1)_{60}$	196.16
4.2	49.5	37	2.400	23.88	$7(N1)_{60}$	167.19

H.1.5.2 AASHTO (2007) Method

AASHTO method uses half-space elastic solution to estimate the settlement under the footing, given by Equation (H-7) below.

$$S_e = \frac{q(1-\nu^2)\sqrt{A}}{E_s\beta_z} \qquad \text{(H-7)}$$

where q is the applied vertical stress on the footing with base area A, ν and E_s are Poisson's ratio and modulus of elasticity of underlying soil layer, respectively, β_z is the elastic shape and rigidity factor (Table 10.6.2.4.2-1, AASHTO 2007). The elastic shape and rigidity factor is interpolated for the intermediate L/B ratios.
Here, Poisson's ratio ν has been taken 0.3 and β_z = 1.08 (square and rigid footing)

1. Weighted average mean soil parameters:
   For a square footing of B = 4.9ft, the depth of influence for settlement calculation is 2B + D_f = 17.4ft. Hence,

$$\text{average } E_s = \frac{(10.1\text{-}7.5)\ 188.9\ +\ (12.6\text{-}10.1)\ 183.1\ +\ (15.1\text{-}12.6)\ 316.3\ +\ (17.4\text{-}15.1)\ 254.2}{17.4-7.5} = 234.8\text{tsf}$$

2. Load required to develop settlement of 1.5inches:

$$q = \frac{S_e E_s \beta_z}{(1-\nu^2)\sqrt{A}} = \frac{(1.5/12)\times 234.8\times 1.08}{(1-0.3^2)\sqrt{4.9\times 4.9}} = 7.1\text{tsf}$$

Thus, it is estimated from the AASHTO method that a load of 7.1tsf on the footing produces a settlement of 1.5inches. The load required to produce a settlement of 1.5in for other footing sizes can be obtained in similar fashion.

H.1.5.3 Schmertmann et al. (1978) Method

The settlement is estimated using the following equation:

$$S_e = C_1 C_2\, \Delta q \sum_{i=1}^{n} \left(\frac{I_z}{E_s}\right)_i \Delta z_i \qquad \text{(H-8)}$$

where,

- S_e = settlement (ft)
- i = ith layer below the footing base
- Δz_i = thickness of individual layer (ft)
- n = number of soil layers below the footing base up to the influence depth
- Δq = net applied pressure = $q - q_0$
  - q = applied footing stress (tsf)
  - q_0 = effective stress at footing depth
  - s'_{vp} = initial effective vertical pressure at the depth z_p where I_{zp} occurs
- C_1 = depth correction factor = $1.0 - 0.5(q_0/\Delta q) \geq 0.5$
- C_2 = creep correction factor = $1.0 + 0.2\log(10t)$
- t = time for creep calculation in years, and
- I_z = strain influence factor, given as follows
  For a square (axisymmetric) footing:

I_z = 0.1 at depth = 0
I_z = I_{zp} at depth = z_p = 0.5B
I_z = 0.0 at depth = D = 2.0B

For a strip footing with L/B = 10:

I_z = 0.2 at depth = 0
I_z = I_{zp} at depth = z_p = 1.0B
I_z = 0.0 at depth = D = 4.0B

For footings with 1 < L/B < 10:

At depth = 0, I_z is interpolated value between 0.1 and 0.2
z_p is interpolated between 0.5B and 1.0B, and
influence depth at which I_z = 0 is interpolated between 2.0B and 4.0B
The maximum value of I_z at depth z_p is given by:

$$I_{zp} = 0.5 + 0.1\sqrt{\frac{\Delta q}{s'_{vp}}} \tag{H-9}$$

1. Sub-division of subsurface layers:
   For simplicity and automation, the soil layer considered below the footing base has been divided into six layers irrespective of the size of footing as illustrated in Figure H-2. Here, L/B = 1. Hence z_p = 0.5B = 2.45ft and D = 2.0B = 9.8ft.

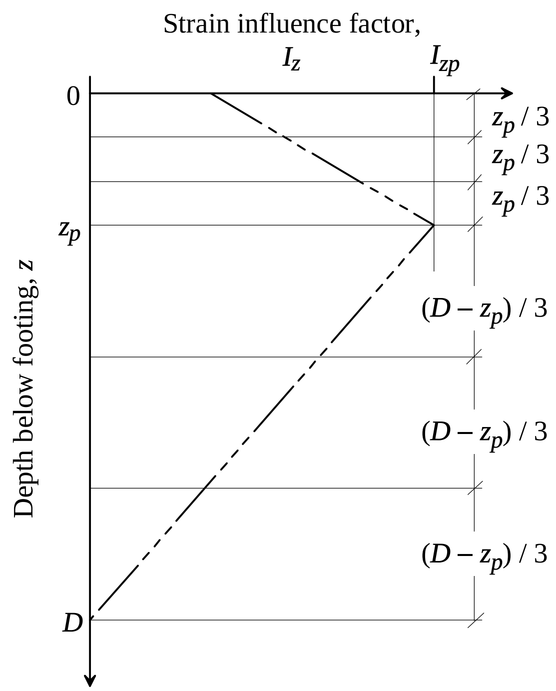

Figure H-2. Subsurface layer division for Schmertmann (1978) method

2. Effective stresses and maximum strain influence factor:
   Effective stress at footing depth, $q_0 = \gamma D_f$ = 124.9 × 7.55 = 943.0psf = 0.4715tsf
   $I_z = I_{zp}$ at the depth of 0.5B+D_f = 0.5×4.9 + 7.55 = 10.0ft from the ground level.

Initial stress at which I_{zp} occurs (=10.0ft) is

$\sigma'_{vp} = \sum \gamma_i \Delta z_i$

$= 124.9\times2.5 + 124.9\times(5.0–2.5) + 124.9\times(7.5–5.0) + 124.9\times(10.0–7.5)$

$= 1249\text{psf} = 0.6245\text{tsf}$

3. Assumption of a load for settlement prediction:
Since I_z and C_1 are functions of the applied load on the footing, an iteration process is necessary to obtain the required load q to produce a prescribed settlement S_e (1.5inches here).
For the start, let q = 3.0tsf. Then,

$$\Delta q = 3.0 - 0.4715 = 2.53\text{tsf} \text{ , and}$$

$$I_{zp} = 0.5 + 0.1\sqrt{\frac{2.53}{0.6245}} = 0.701$$

$$C_1 = 1.0 - 0.5(0.4715/2.53) = 0.9068 > 0.5$$

4. Strain influence factor I_z at mid-height of each of the subdivided layer:
Let the depth of layer mid-height from the footing base be $D_{zi}\times B$. Then
For $D_{zi} < z_p/B$, I_{zi} can be interpolated as:

$$I_{zi} = I_{zp} - \left(\frac{0.5 - D_{zi}}{0.5 - 0}\right)\left(I_{zp} - 0.1\right)$$

And, for $D_{zi} \geq z_p/B$, I_{zi} can be interpolated as:

$$I_{zi} = I_{zp} - \left(\frac{D_{zi} - 0.5}{2.0 - 0.5}\right)\left(I_{zp} - 0\right)$$

For layer #1, $D_{z1} = 0.5\times(0.5/3) = 0.0833$

$$I_{z1} = I_{zp} – 2\ (0.5–0.0833)\times(I_{zp}– 0.1) = 0.701 – (–0.5010) = 0.2002$$

Similarly, For layer #4, $D_{z4} = 0.5 + 0.5\times(2.0 – 0.5)/3 = 0.75$

$$I_{z4} = I_{zp} – (0.75–0.5)\ I_{zp} / (1.5) = I_{zp}\ (0.8333) = 0.5843$$

The values of I_{zi} for other soil layers, calculated in similar fashion, are shown in the detail calculations.

5. E_s for each sub-divided layer:
The Young’s modulus of elasticity has been taken as the weighted average of each soil layer present in the subsurface below the footing base, which has been subdivided as shown in Figure H-2.
For layer #1 to #3, E_s = 188.85tsf (since z_p < 10.1ft from the ground surface, ref. Table H-9).
For layer #4, the depth ranges from 10.0ft (= z_p+ D_f) to 12.5ft (= z_p+ $(D–z_p)/3$ + D_f).
Hence, the weighted average of E_s, considered to be at the mid-height of layer 4, is obtained from Table H-9 as:

$$\text{avg } E_s = \frac{188.85(10.1-10.0)+183.13(12.5-10.1)}{(12.5-10.0)} = 183.30\text{tsf}$$

and so on for other layers.

6. Detailed calculations:
After the sum of $(I_z / E_s)\times\Delta z$ is obtained, the resulting settlement can be calculated using Equation (H-8). The detailed calculation is shown below. The calculation is repeated with trial applied loads *q* until the required settlement is obtained.

B (ft) =	4.9
From GL, z_p (ft) =	10.0
From GL, D (ft) =	17.4
σ'_{vp} (tsf) =	0.6245
q_0 (tsf) =	0.4715

Trail 1:

Let q (tsf) =	3.00
Then Δq =	2.53
I_{zp} =	0.7012
C_1 =	0.9068
C_2 =	1.0000

Subdivided Layer #	Depth below GL (ft)	Depth below footing base (ft)	Layer thickness Δz (ft)	Mid-height depth below footing base D_z (ft)	Strain influence factor, I_z	Average E_s (tsf)	I_z/ E_s * Δz
1	8.4	0.817	0.817	0.408	0.2002	188.85	0.000866
2	9.2	1.633	0.817	1.225	0.4006	188.85	0.001732
3	10.0	2.450	0.817	2.042	0.6010	188.85	0.002599
4	12.5	4.900	2.450	3.675	0.5843	183.30	0.007810
5	14.9	7.350	2.450	6.125	0.3506	308.28	0.002786
6	17.4	9.800	2.450	8.575	0.1169	259.06	0.001105
						sum:	0.016899

S_e (in) = 0.465

Example of a next trial:

Let q (tsf) =	7.04
Then Δq =	6.57
I_{zp} =	0.8243
C_1 =	0.9641
C_2 =	1.0000

Subdivided Layer #	Depth below GL (ft)	Depth below footing base (ft)	Layer thickness Δz (ft)	Mid-height depth below footing base D_z (ft)	Strain influence factor, I_z	Average E_s (tsf)	$I_z / E_s * \Delta z$
1	8.4	0.817	0.817	0.408	0.2207	188.85	0.000954
2	9.2	1.633	0.817	1.225	0.4621	188.85	0.001998
3	10.0	2.450	0.817	2.042	0.7036	188.85	0.003042
4	12.5	4.900	2.450	3.675	0.6869	183.30	0.009181
5	14.9	7.350	2.450	6.125	0.4121	308.28	0.003275
6	17.4	9.800	2.450	8.575	0.1374	259.06	0.001299
						sum:	0.019751

S_e (in) = 1.500

Hence, for a footing of width 4.9ft, a load of 7.0tsf is estimated to produce a settlement of 1.5in using Schmertmann (1978) method. The load required to produce a settlement of 1.5in for other footing sizes can be obtained in similar fashion.

H.1.5.4 Hough (1959) Method

The settlement below a footing is estimated as:

$$S_e = \sum_{i=1}^{n} \frac{1}{C'_i} \Delta z_i \ln\left(\frac{\sigma_{v0i} + \Delta\sigma_{vi}}{\sigma_{v0i}}\right) \tag{H-10}$$

where C'_i is bearing capacity index obtained based on corrected $(N1)_{60}$ value from Figure H-3 (= $(1 + e_0) / C_c$; e_0 is initial void ratio and C_c is virgin compressibility index); Δz_i is the layer thickness of layer *i*, σ_{v0i} is initial effective overburden pressure and $\Delta\sigma_{vi}$ change in effective vertical stress both at mid-height of layer *i*, *n* is the number of layers present within the influence depth below the footing base.

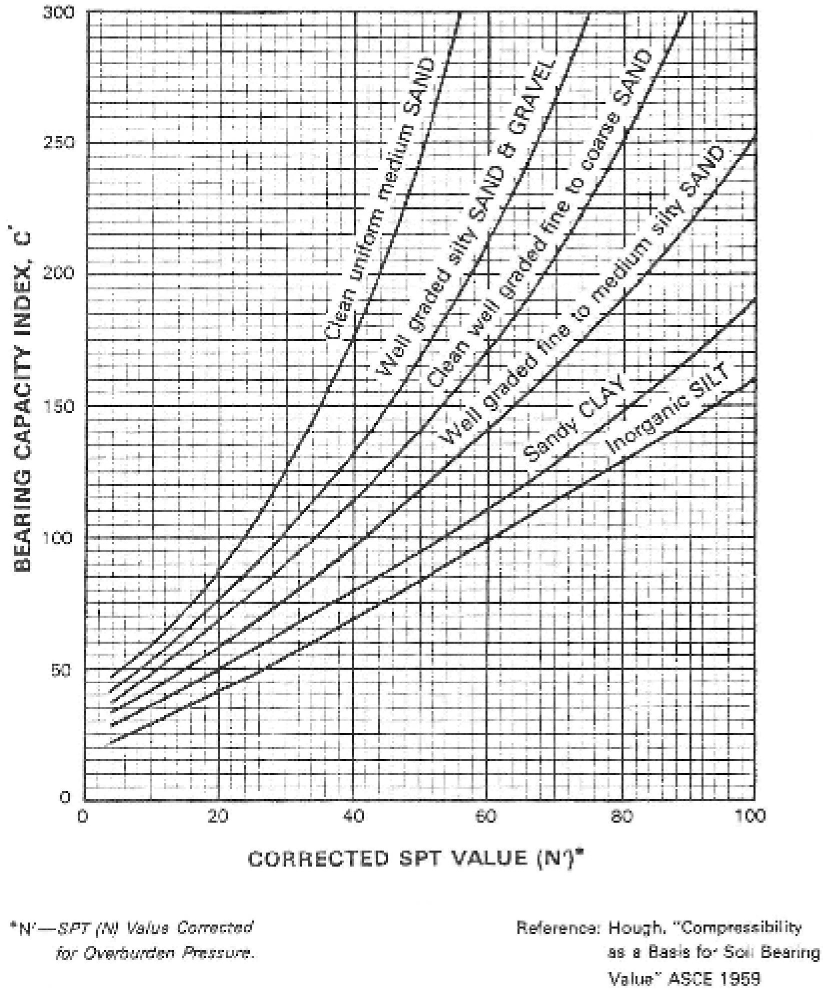

Figure H-3. Bearing capacity index versus corrected SPT value (= $(N1)_{60}$)
(Cheney and Chassie 1982, modified from Hough 1959)

1. Bearing capacity index C′ based on corrected SPT value at layer mid-height:
   In the calculation presented here, the value of C′ has taken from digitized and fitted curves of Figure H-3 for automation. The curve fittings are listed in Table H-10 below.

Table H-10. Bearing capacity index from corrected SPT values based on Figure H-3

Soil description	Fitted curve from Figure H-3
Clean uniform med Sand	$C' = 0.0746(N1)_{60}^2 + 0.1313(N1)_{60} + 51.157$
Well graded silty Sand and Gravel	$C' = 0.0335(N1)_{60}^2 + 0.8276(N1)_{60} + 42.86$
Clean well-graded fine to coarse Sand	$C' = 0.0002(N1)_{60}^3 - 0.01(N1)_{60}^2 + 2.1694(N1)_{60} + 27.145$
Well-graded fine to medium silty Sand	$C' = 0.009(N1)_{60}^2 + 1.3134(N1)_{60} + 28.052$
Sandy Clay	$C' = 0.0052(N1)_{60}^2 + 1.1066(N1)_{60} + 24.928$
Inorganic Silt	$C' = 0.0022(N1)_{60}^2 + 1.2166(N1)_{60} + 16.49$

A soil layer has been taken as the layer for each SPT observation present, e.g. layer numbers 2.1 and 2.2 shown in Table H-11 are two layers.

Table H-11. Bearing capacity index C¢ for each soil layer

Layer #	Depth (ft)	N_{60}	Mid-layer overburden σ_{v0} (tsf)	$(N1)_{60}$ (Liao and Whitmann 1996)	Soil description	BC index C'
1.1	2.5	6	0.079	21.37	Lean clay	
1.2	5.0	7	0.235	14.43		
1.3	7.5	18	0.392	28.75		
2.1	10.1	20	0.550	26.98	silty sand	70.0
2.2	12.6	22	0.707	26.16		68.6
2.3	15.1	42	0.864	45.19		105.8
3a.1	20.0	38	1.095	36.31	well graded sand (taken as fine to med.)	87.6
3a.2	24.9	47	1.402	39.69		94.4
3a.3	29.9	33	1.709	25.24		66.9
3b.1	34.8	45	1.939	32.31		79.9
3b.2	39.7	49	2.093	33.87		82.9
4.1	44.6	42	2.246	28.02	clean uniform sand	113.4
4.2	49.5	37	2.400	23.88		96.8

2. Increase in stress at each layer mid-height:
   The increase in stress at layer mid-height is obtained using 2:1 method of stress distribution. This method approximates the vertical stress $\Delta\sigma_v$ at a depth z which is caused by a footing of dimension $L\times B$ loaded with a force Q as the following.

$$\Delta\sigma_v = \frac{Q}{(B+z)(L+z)} \tag{H-11}$$

3. Settlement in each layer and total settlement:
   The influence depth has been taken according to Table H-8. For a square footing of B = 4.9ft placed at and embedment depth D_f of 7.55ft, influence depth from the ground surface is 17.35ft. Further, as $\Delta\sigma_v$ is directly related to the applied load Q, it is easier to estimate the required load to produce a prescribed settlement of 1.5in by hit and trial. For the start, trial 1, an applied vertical stress of 3tsf is assumed at the footing base. The detailed calculations using Equations (H-10) and (H-11) and the bearing capacity index C′ from Table H-11 are presented below.

B (ft) = 4.9
L (ft) = 4.9
Depth of influence from GL (ft) = $2B + D_f$ = 17.35

Trial 1:

Let $\boxed{q \text{ (tsf)} = 3.00}$
Then,
applied load (ton) = 72.03

Layer #	Depth (ft)	Layer thickness Δz (ft)	Depth of layer mid-height from footing base z (ft)	Initial effective vertical stress at layer mid-height, σ_{v0} (tsf)	Increase in pressure at layer mid-height $\Delta\sigma_v$ (tsf)	Bearing capacity index C'	Settlement in each layer ΔH (in)
1.1	2.5						
1.2	5.0						
1.3	7.5						
2.1	10.1	2.5	1.3	0.550	1.899	70.04	0.2809
2.2	12.6	2.5	3.8	0.707	0.955	68.57	0.1641
2.3	15.1	2.5	6.3	0.864	0.575	105.79	0.0627
3a.1	17.4	2.3	8.7	1.013	0.391	90.48	0.0425
						sum:	0.550

$\boxed{S_e \text{ (in)} = 0.550}$

Example of a next trial:

Let $\boxed{q \text{ (tsf)} = 23.40}$
Then,
applied load (ton) = 561.83

Layer #	Depth (ft)	Layer thickness Δz (ft)	Depth of layer mid-height from footing base, z (ft)	Initial effective vertical stress at layer mid-height, σ_{v0} (tsf)	Increase in pressure at layer mid-height, $\Delta\sigma_v$ (tsf)	Bearing capacity index C'	Settlement in each layer ΔH (in)
1.1	2.5						
1.2	5.0						
1.3	7.5						
2.1	10.1	2.5	1.3	0.550	14.811	70.04	0.6261
2.2	12.6	2.5	3.8	0.707	7.448	68.57	0.4695
2.3	15.1	2.5	6.3	0.864	4.483	105.79	0.2238
3a.1	17.4	2.3	8.7	1.013	3.051	90.48	0.1807
						sum:	1.500

$\boxed{S_e \text{ (in)} = 1.500}$

For a footing of width 4.9ft, a load of 16.35tsf is estimated to produce a settlement of 1.5in using Hough (1959) method. The load required to produce a settlement of 1.5in for other footing sizes can be estimated in a similar fashion.

H.1.6 Resistance Factors

The footing will be constructed on the in-situ soil stratum (natural soil condition) without replacing the silty sand layer with an engineering fill. Hence the resistance factors to be used are the ones given for *natural deposited granular soil condition*. The newly proposed factors developed in this research for Strength-I design corresponding to the inclined-eccentric positive eccentricity loading condition are as shown in Table H-12, which varies according to the average soil friction angle of the granular material considered. It can be seen that the resistance factor is expected to be essentially 0.40, as $\phi = 0.35$ is applicable for either very small or large footings. The resistance factor in the current AASHTO (2007) specification is given as $\phi = 0.45$ and has been presented here for comparison. No resistance factors exist in the current specifications for Service limit state, hence, the estimated load required to produce a settlement of 1.5in has been left unfactored.

Table H-12 Average soil friction angle and recommended resistance factor variation according to the footing size (thereby the influence depth)

B (ft)	Average f_f (deg)	Recommended f	B (ft)	Average f_f (deg)	Recommended f
2.95	35.60	0.35	12.80	36.60	0.40
3.94	36.45	0.35	13.78	36.59	0.40
4.92	36.64	0.40	14.76	36.61	0.40
5.91	36.77	0.40	15.75	36.62	0.40
6.89	36.93	0.40	16.73	36.57	0.40
7.87	37.09	0.40	17.72	36.51	0.35
8.86	37.14	0.40	18.70	36.44	0.35
9.84	36.89	0.40	19.68	36.33	0.35
10.83	36.68	0.40	20.67	36.23	0.35
11.81	36.61	0.40			

H.1.7 Design Footing Width

The load eccentricities according to Table H-3 are: for the Strength-I limit state across the footing length $e_L = 0.335$ ft and across the footing width $e_B = 0.220$ ft, and for the Service-I limit state: $e_L = 0.358$ ft and $e_B = 0.226$ ft. The maximum load eccentricity for design is hence 0.358 ft. Hence the minimum admissible footing width is B = 2.15ft (= $e_B \times 6$ = 0.358ft×6). At this stage, the footing is designed for Strength-I and Service-I vertical loads. The maximum vertical factored load for Strength-I limit state (bearing resistance - see Table H-5.1) is 2780 kips, and the vertical unfactored load is 2140 kips for Service-I limit state (bearing resistance and sliding resistance see Table H-5.1 and H-5.2).

Figure H-4 presents the unfactored as well as the factored bearing resistances for different effective footing widths. The bearing loading intensities (stresses) are plotted in the upper figure, whereas the lower ones present the bearing loads. The footing width refers to the effective width for both bearing capacity and settlement analyses. While the settlement analyses were carried out for the geometrical (full) foundation width, in the presentation of Figure H-4, the width was transformed to be the effective width.

Applying the aforementioned vertical loads to the corresponding limit states in Figure H-4 results with the following: (a) the unfactored strength limit states is satisfied by a footing size of 6.75×6.75ft (full geometrical size = 6 + 2×0.36), and (b) the unfactored allowable bearing resistance obtained using Hough (1959) method results in footing dimensions of 16.25 ft ×16.25 ft and using Schmertmann (1978) method results in dimensions of 19.50 ft×19.50 ft, whereas, AASHTO (2007) method results with a much larger footing. The original example (FHWA GEC – Example 1) resulted with a full geometrical foundation size of 16.5×16.5ft based on the Hough method, which is close to the foundation size obtained here. For the factored Strength-I bearing resistance, a square footing of 9.75ft side meets the requirement of all resistance factors criteria, whereas, to meet the factored Service-I bearing resistances demand, a footing larger than the relations presented in the figure (effective width of 21.0 ft) is necessary. Extrapolating the trend in Service-I bearing resistance, a square footing of about 50.0ft side is required.

The conclusions from Figure H-4 are, therefore:

1. Based on strength limit state alone; the following foundation sizes are sufficient:
   - Strength limit state $\phi = 0.35$ to 0.40: 9.75ft×9.75 ft
   - Strength limit state $\phi = 0.45$: 9.25×9.25 ft
2. Based on unfactored serviceability limit state (current AASHTO); a minimum footing size of 16.25×16.25ft is required.

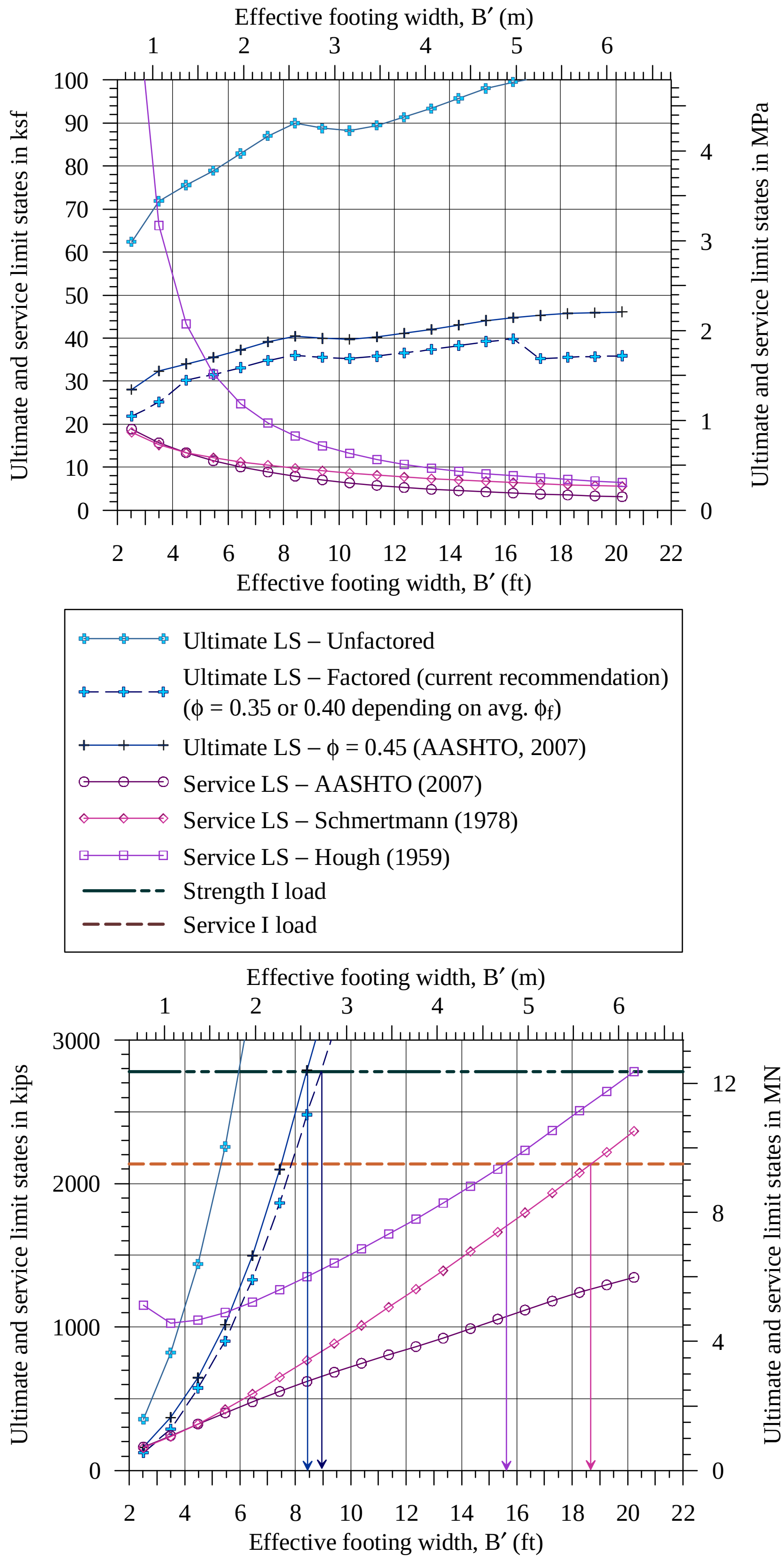

Figure H-4. Variation of bearing resistances for Strength-I and Service-I limit states with effective footing width for Example 1

H.1.8 Sliding Resistance

The footing is poured on site; hence, the recommended resistance factors for cast in-place footings to be used for sliding resistance are $\phi_\tau = 0.40$ when lateral load due to at-rest earth pressure is acting and $\phi_\tau = 0.45$ when lateral load due to active earth pressure is acting. The relation between soil-footing friction angle δ_s and soil friction angle is $\tan\delta_s = 0.91\tan\phi_f$ for cast in-place footings. Hence, for $\phi_f = 34.5°$, nominal sliding resistance, $F_{2\tau} = F_1 \times \tan(\delta_s) = F_1 \times 0.91\tan(34.5)$. The minimum factored vertical load for the designed footing width for Strength-I and Service-I load for sliding (Table H-5.2) is 2137.2kips (Service-I), for which the lateral load is 51.2kips. That is, Factored sliding resistance, $\phi_\tau F_{2\tau} = 0.40 \times 2137.2 \times 0.91\tan(34.5) = 534.7$kips > 51.2kips. Hence the designed footing is safe in sliding.

H.1.9 Discussions and Conclusions

It can be seen from Figure H-4 that Service limit states govern the footing dimension in this design example. While the ultimate limit state (Strength I) can be satisfied with a foundation size of 9.75×9.75ft (considering all possibilities), the serviceability limit state requires a foundation size of at least 16.25×16.25ft. The AASTHO (2007) method gives the most conservative estimate of the allowable load for the given allowable 1.5inch settlement. Schmertmann (1978) method gives allowable loads comparable to those obtained using the AASHTO method for smaller footings, while it gives the allowable loads closer to those obtained using Hough (1959) method as the footing width increases. For that reason, one can conclude that: (i) the recommended new strength limit state factors would not affect this design example as it is controlled by the service limit, and (ii) if resistance factors were to be applied to the service limit, for the given example and a limit settlement of 1.5inch the foundation size would increase significantly.

H.2 EXAMPLE 2: BILLERICA BRIDGE, CENTRAL PIER ON GRAVEL FILL

H.2.1 General Information

The central pier and the east abutment of the Billerica B-12-025 (2004) bridge are analyzed in examples 2 and 3, respectively. Billerica bridge B-12-025 (2004) is a 2-span, continuous medium length span (CM-M); skewed structure with a skew angle of 20°-13′-32″. The bridge dimensions and the footing dimensions used are:

Bridge:	
Span lengths	112.8ft-112.8ft (34.4m-34.4m)
Span width	49.0ft (14.93m)
Foundations:	
East Abutment	width = 12.5ft (3.8m); length = 61.65ft (18.79m); average height of abutment from abutment footing base = 23.4ft (7.14m); footing thickness = 2.95ft (0.9m); abutment wingwall – acute side = 42.45ft (12.94m), obtuse side = 41.34ft (12.60m)
Central Pier	width = 13.12ft (4.0m); length = 52.4ft (15.96m); thickness = 3.28ft (1.0m); given maximum bearing pressure = 37.6ksf (1800kPa) for Strength LS (factored bearing pressure = 13.16ksf or 630kPa), and 6.27ksf (300kPa) for Service LS for allowable settlement of maximum 1.5inches (38mm)
West Abutment	width = 12.5ft (3.8m); length = 61.65ft (18.79m); height of abutment from abutment footing base = 23.4ft (7.14m); footing thickness = 2.95ft (0.9m); abutment wingwall –acute side = 36.2ft (11.04m), obtuse side = 30.85ft (9.40m)

H.2.2 Subsurface Condition

The subsurface at the central pier location consists (based on boring GB-22) approximately of 3ft of loose granular fill overlaying 5.5ft of very dense coarse sand and gravel overlaying a rock layer. The geotechnical report (URS, 2001) called for the replacement of the loose fill with gravel borrow material that would extend to the proposed footing elevation. As such, the foundation design follows the geotechnical report assuming the central pier to be founded on compacted gravel. The parameters provided for the gravel borrow in the geotechnical report are: bulk unit weight $\gamma\,(\gamma')$ = 120.0 pcf (63.65 pcf) (18.85/9.99 kN/m^3), internal friction angle of 38° and interfacial friction angle between the footing base and the soil $\delta_s = 29.7°$. The groundwater table is located at elevation 157.5 ft (48.0 m) and the foundation base is at elevation of 160.1ft (48.8m).

H.2.3 Loads, Load Combinations and Limit States

The different load components as provided are summarized in Table H-13. The weight of the footing and the soil above the footing has been considered as a vertical centric load of 519.2kips in addition to the vertical load component F_1.

Table H-14 summarizes the investigated load combinations and the resultant characteristic loading as well as the resultant load inclination $\sqrt{F_2^2 + F_3^2}/F_1$ and eccentricity in both directions e_2 and e_3 for the different load combinations (the directions and notations are as described in Figure 120 of Chapter 5). The calculation of the bearing resistance and the sliding resistance are based on the characteristic load components as given in Table H-14. The design load components required for the stability analysis with the load factors according to AASHTO Section 3 (2007) presented in Tables H-4.1 and H-4.2 are summarized in Table H-15.1 and H-15.2 for the bearing capacity and sliding Strength limit states, respectively. The Extreme-I C9 combination includes the highest moment and the highest horizontal loading together with a relatively small vertical load. In the other load combinations either the moments or the horizontal loads are relatively high.

TABLE H-13. Loading at footing base for Example 2 (Billerica Bridge, Central pier)

Load at Footing Base	F_1 kips (kN)	F_2 kips (kN)	F_3 kips (kN)	M_2 kip-ft (kNm)	M_3 kip-ft (kNm)
Weight of footing, columns and cap (F)	574.1 (2553.8)	0	0	0	0
Dead load (DL)	1675.4 (7452.9)	0	0	0	0
Live load and impact (LL+I) case I	500.6 (2226.9)	0	0	0	0
Live load and impact (LL+I) case II	370.8 (1649.6)	0	0	0	0
Live load and impact (LL+I) case III	500.8 (2227.9)	0	0	0	0
Wind on structure: 0° to z-dir. (W(0))	0	46.7 (207.8)	0	0	802.2 (1087.9)
Wind on structure: 30° to z-dir. (W(30))	0	42.4 (188.8)	2.9 (12.9)	49.8 (67.5)	728.8 (988.4)
Wind on structure: 60° to z-dir. (W(60))	0	22.4 (99.7)	11.9 (52.9)	204.3 (277.1)	385.0 (522.1)
Wind on live load: 0° to z-dir. (WL(0))	0	10.6 (47.1)	0	0	181.8 (246.6)
Wind on live load: 30° to z-dir. (WL(30))	0	9.6 (42.8)	0.7 (2.9)	11.3 (15.4)	165.2 (224.0)
Wind on live load: 60° to z-dir. (WL(60))	0	5.1 (22.6)	2.7 (12.0)	46.3 (62.8)	87.3 (118.4)
Lateral force (LF)	0	14.3 (63.5)	5.3 (23.4)	90.3 (122.5)	245.3 (332.7)
Earthquake (EQ1)	0	128.0 (569.5)	59.1 (262.9)	1014.8 (1376.3)	2198.8 (2982.1)
Earthquake (EQ2)	0	59.1 (262.9)	128.0 (569.5)	2198.8 (2982.1)	1014.8 (1376.3)

TABLE H-14. Load combinations and resultant characteristic (unfactored) loading for Example 2

Load Combi-nations	Load Components	F_1 kips (kN)	F_2 kips (kN)	F_3 kips (kN)	M_2 kips-ft (kNm)	M_3 kips-ft (kNm)	$\frac{\sqrt{F_2^2+F_3^2}}{F_1}$	$e_2 = M_3/F_1$ ft (m)	$e_3 = M_2/F_1$ ft (m)
C1	F+DL + (L+I(caseII))	2620.3 (11656.3)	0.0	0.0	0.0	0.0	0.000	0.000	0.000
C2	F+DL + (LL+I(caseIII))	2750.3 (12234.6)	0.0	0.0	0.0	0.0	0.000	0.000	0.000
C3	F+DL + (LL+I(caseII)) + W(0)	2620.3 (11656.3)	46.7 (207.8)	0.0	0.0	802.2 (1087.9)	0.018	0.305 (0.093)	0.000
C4	F+DL + (LL+I(caseII)) + W(60)	2620.3 (11656.3)	22.4 (99.7)	11.9 (52.9)	204.3 (277.1)	385.0 (522.1)	0.010	0.148 (0.045)	0.079 (0.024)
C5	F+DL + (LL+I(caseII)) + W(0) + WL(0)	2620.3 (11656.3)	57.3 (254.9)	0.0	0.0	984.0 (1334.6)	0.022	0.374 (0.114)	0.000
C6	F+DL + (LL+I(caseII)) + W(60) + WL(60)	2620.3 (11656.3)	27.5 (122.3)	14.6 (64.9)	250.6 (339.9)	472.3 (640.6)	0.012	0.180 (0.055)	0.095 (0.029)
C7	F+DL + (LL+I(caseII)) + W(0) + WL(0) + LF	2620.3 (11656.3)	71.6 (318.4)	5.3 (23.4)	90.3 (122.5)	1229.3 (1667.3)	0.027	0.469 (0.143)	0.036 (0.011)
C8	F+DL + (LL+I(caseII)) + W(60) + WL(60) + LF	2620.3 (11656.3)	41.8 (185.9)	19.9 (88.3)	341.0 (462.4)	717.6 (973.3)	0.018	0.272 (0.083)	0.131 (0.040)
C9	F+DL + (LL+I(caseII)) + EQ1	2620.3 (11656.3)	128.0 (569.5)	59.1 (262.9)	1014.8 (1376.3)	2198.8 (2982.1)	0.054	0.840 (0.256)	0.387 (0.118)
C10	F+DL + (LL+I(caseII)) + EQ2	2620.3 (11656.3)	59.1 (262.9)	128.0 (569.5)	2198.8 (2982.1)	1014.8 (1376.3)	0.054	0.387 (0.118)	0.840 (0.256)

TABLE H-15.1. Load combinations and resultant design (factored) loading required for bearing resistance

Load Combinations	F_1 kips(kN)	F_2 kips(kN)	F_3 kips(kN)	M_2 kip-ft (kNm)	M_3 kip-ft (kNm)
Service-I C1	2620.3 (11656.3)	0.00	0.0	0.0	0.0
Service-I C2	2750.3 (12234.6)	0.00	0.0	0.0	0.0
Service-I C3	2620.3 (11656.3)	14.0 (62.3)	0.0	0.0	240.7 (326.4)
Service-I C4	2620.3 (11656.3)	6.7 (29.9)	3.6 (15.9)	61.3 (83.1)	115.5 (156.6)
Service-I C5	2620.3 (11656.3)	24.6 (109.4)	0.0	0.0	422.5 (573.0)
Service-I C6	2620.3 (11656.3)	11.8 (52.5)	6.3 (27.9)	107.6 (146.0)	202.8 (275.1)
Strength-I C1	3460.8 (15395.2)	0.0	0.0	0.0	0.0
Strength-I C2	3688.3 (16407.2)	0.0	0.0	0.0	0.0
Strength-I C7	3460.8 (15395.2)	25.0 (111.2)	9.2(41.0)	158.1 (214.4)	429.3 (582.2)
Strength-V C5	3312.5 (14735.3)	29.3 (130.2)	0.0	0.0	502.7 (681.8)
Strength-V C6	3312.5 (14735.3)	14.1 (62.5)	7.5 (33.2)	128.1 (173.7)	241.3 (327.3)
Strength-V C7	3312.5 (14735.3)	48.6 (216.0)	7.1 (31.6)	122.0 (165.4)	833.9 (1130.9)
Strength-V C8	3312.5 (14735.3)	33.3 (148.3)	14.6 (64.8)	250.0 (339.1)	572.5 (776.4)
Extreme-I C9	3182.7 (14158.0)	128.0 (569.5)	59.1 (262.9)	1014.8 (1376.3)	2198.8 (2982.1)
Extreme-I C10	3182.7 (14158.0)	59.1 (262.9)	128.0 (569.5)	2198.8 (2982.1)	1014.8 (1376.3)

TABLE H-15.2. Load combinations and resultant design (factored) loading required for sliding resistance

Load Combinations	F_1 kips(kN)	F_2 kips(kN)	F_3 kips(kN)	M_2 kip-ft (kNm)	M_3 kip-ft (kNm)
Service-I C3	2620.3 (11656.3)	14.0 (62.3)	0.0	0.0	240.7 (326.4)
Service-I C4	2620.3 (11656.3)	6.7 (29.9)	3.6 (15.9)	61.3 (83.1)	115.5 (156.6)
Service-I C5	2620.3 (11656.3)	24.6 (109.4)	0.0	0.0	422.5 (573.0)
Service-I C6	2620.3 (11656.3)	11.8 (52.5)	6.3 (27.9)	107.6 (146.0)	202.8 (275.1)
Strength-I C7	2673.5 (11892.8)	25.0 (111.2)	9.2(41.0)	158.1 (214.4)	429.3 (582.2)
Strength-V C5	2525.2 (11233.0)	29.3 (130.2)	0.0	0.0	502.7 (681.8)
Strength-V C6	2525.2 (11233.0)	14.1 (62.5)	7.5 (33.2)	128.1 (173.7)	241.3 (327.3)
Strength-V C7	2525.2 (11233.0)	48.6 (216.0)	7.1 (31.6)	122.0 (165.4)	833.9 (1130.9)
Strength-V C8	2525.2 (11233.0)	33.3 (148.3)	14.6 (64.8)	250.0 (339.1)	572.5 (776.4)
Extreme-I C9	2395.4 (10655.6)	128.0 (569.5)	59.1 (262.9)	1014.8 (1376.3)	2198.8 (2982.1)
Extreme-I C10	2395.4 (10655.6)	59.1 (262.9)	128.0 (569.5)	2198.8 (2982.1)	1014.8 (1376.3)

H.2.4 Nominal Bearing Resistances at the Limit State

H.2.4.1 Footing Information:

The footing length is kept fixed at 52.4 ft, which is comparable to the bridge span width, and is assumed to have no embedment depth. The bearing resistances of footings with length fixed and widths varied from 2.95ft to 20.70ft have been calculated.

The load combinations considered for the bearing resistance estimation of the rectangular footings are Strength-I C7 and Strength-I C2 limit states according to AASHTO (2007) with an embedment depth equal to zero. The Strength-I C7 limit state has the 2-way load inclination and 2-way load eccentricity with the highest load inclination as well as the highest load eccentricity along the footing width among the possible load combinations considered, whereas, Strength-I C2 limit state has the highest unfactored as well as factored vertical-centric loading (Tables H-14 and H-15).

For Strength-I C7 LS, the maximum load eccentricities along the footing width and the footing length are 0.469ft and 0.034ft, respectively. Detailed calculations for an example footing of width B = 4.9ft are presented here. The effective footing dimensions for the C7 limit state are as follows

$$\text{Effective width } B' = B - 2e_2 = 4.9 - 2\times 0.469 = 3.98\text{ft}$$
$$\text{Effective length } L' = L - 2e_3 = 52.4 - 2\times 0.034 = 52.3\text{ft}$$

Here, the eccentricity ratios across the footing length (e_3/L, Table H-14) are very small, even for load combination C10 related to Extreme-I loading conditions (a maximum of 0.016). Hence, the effect of the load eccentricity across the footing length can be neglected for practical purposes for this example, however, the calculations have been presented using the effective length.

H.2.4.2 Bearing Capacity Factors

Since the average ϕ_f has been assumed to be 38.0°, the bearing capacity factors are as follows.

$$N_q = \exp\{\pi \tan(38.0)\} \tan^2(45+38.0/2) = 48.93 \text{ , and}$$
$$N_\gamma = 2\,(48.93+1)\tan(38.0) = 78.02$$

H.2.4.3 Bearing Capacity Modification Factors

Shape factors for Strength-I C7 LS:

$$s_q = 1+\tan\phi_f\,(B'/L') = 1+\tan(38)(3.98/52.29) = 1.060$$
$$s_\gamma = 1-0.4(B'/L') = 1-0.4(3.98/52.29) = 0.970$$

Depth factors:
The footing is assumed to be on the ground surface, i.e. $D_f = 0$. Hence, $d_q = 1.0$.

Load inclination factors for Strength-I C7 LS:
Here, the projected direction of the inclined load in the plane of the footing is given by

$$\theta = \tan^{-1}(F_3/F_2) = \tan^{-1}(5.3/71.6) = 4.233.$$

Hence

$$n = \left(\frac{2+52.29/3.98}{1+52.29/3.98}\right)\cos^2(4.233) + \left(\frac{2+3.98/52.29}{1+3.98/52.29}\right)\sin^2(4.233) = 1.075$$

Then

$$i_q = \left(1 - \frac{\sqrt{71.6^2+5.3^2}}{2620.3+0}\right)^{1.075} = 0.971 \text{and}$$

$$i_\gamma = \left(1 - \frac{\sqrt{71.6^2+5.3^2}}{2620.3+0}\right)^{(1.075+1)} = 0.944$$

H.2.4.4 Modified Bearing Capacity Factors for Strength-I C7 LS

$$N_{qm} = N_q s_q d_q i_q = 48.93 \times 1.060 \times 1.0 \times 0.971 = 50.32 \text{ and}$$
$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma = 78.02 \times 0.970 \times 0.944 = 71.41$$

H.2.4.5 Groundwater Table Modification Factors

Here, $D_f = 0.0 < D_w$ (=2.6ft). Hence,

$$\gamma_1 = \gamma$$

For B = 4.9ft, $1.5B+D_f = 4.9 + 0.0 = 4.9\text{ft} > 2.6\text{ft}$ (GWT). Therefore,

$$\gamma_2 = \gamma\left[1 - \frac{\gamma_w}{\gamma}\left(1 - \frac{D_w - D_f}{1.5B}\right)\right] = 120.1\left[1.0 - \frac{62.4}{120.1}\left(1 - \frac{2.6-0.0}{1.5\times 4.9}\right)\right] = 79.8\text{pcf}$$

H.2.4.6 Bearing Capacity for Strength-I C7 LS

Hence, the nominal (unfactored) bearing resistance of the footing of width 4.9ft calculated using the bearing capacity equation given in AASHTO (2007) is

$$\begin{aligned} q_u &= cN_{cm} + \gamma_1 D_f N_{qm} + 0.5\gamma_2 B' N_{\gamma m} \\ &= 0 + 0 + 0.5 \times 79.8 \times 3.98 \times 71.41 = 11.36\text{ksf} \end{aligned}$$

Table H-16(a) presents the details of the nominal bearing capacity calculation for Strength-I, combination C7 loading in which the load is 2-way inclined and 2-way eccentric, and Table H-16(b) presents the details for Strength-I combination C2 loading in which the load is vertical-centric for footing of length 52.4ft and widths varying from 2.95ft to 20.67ft.

Table H-16. Detailed bearing capacity calculation for Example 2

(a) Loading combination: Strength-I, combination C7 (2-way load inclination and 2-way eccentricity)

Soil parameters and GWT:

γ (pcf)	120.10
D_w (ft)	2.6

Footing information:

L (ft)	52.40	
D_f (ft)	0.00	
depth factor, d_q	1.00	(assumed to be on the ground surface)
depth factor, d_γ	1.00	(Vesic 1975)

Load eccentricity and inclination:

inclination, H/V	0.027	(H along transverse dir.)
eccenticity, e_L	0.034	
eccenticity, e_B	0.469	

B (ft)	B'	L'	2B+D_f	avg ϕ_f	N_q	N_γ	s_q	s_γ	pow n	i_q	i_γ	N_{qm}	$N_{\gamma m}$	1.5B+D_f	γ_1	γ_2	q_n (ksf)	Q_n (kips)
2.95	2.01	52.29	5.9	38.0	48.93	78.02	1.030	0.985	1.042	0.9715	0.9449	48.97	72.59	4.4	120.10	94.661	6.92	728.8
3.94	3.00	52.29	7.9	38.0	48.93	78.02	1.045	0.977	1.059	0.9710	0.9444	49.64	72.00	5.9	120.10	85.411	9.22	1445.5
4.92	3.98	52.29	9.8	38.0	48.93	78.02	1.060	0.970	1.075	0.9706	0.9440	50.32	71.41	7.4	120.10	79.861	11.36	2365.1
5.91	4.97	52.29	11.8	38.0	48.93	78.02	1.074	0.962	1.091	0.9702	0.9436	51.00	70.82	8.9	120.10	76.161	13.40	3479.3
6.89	5.95	52.29	13.8	38.0	48.93	78.02	1.089	0.954	1.106	0.9697	0.9432	51.67	70.24	10.3	120.10	73.518	15.37	4781.8
7.87	6.94	52.29	15.7	38.0	48.93	78.02	1.104	0.947	1.121	0.9693	0.9428	52.35	69.66	11.8	120.10	71.536	17.28	6266.8
8.86	7.92	52.29	17.7	38.0	48.93	78.02	1.118	0.939	1.135	0.9690	0.9424	53.02	69.08	13.3	120.10	69.994	19.15	7928.9
9.84	8.90	52.29	19.7	38.0	48.93	78.02	1.133	0.932	1.149	0.9686	0.9421	53.70	68.50	14.8	120.10	68.760	20.97	9762.8
10.83	9.89	52.29	21.7	38.0	48.93	78.02	1.148	0.924	1.163	0.9682	0.9417	54.38	67.92	16.2	120.10	67.751	22.75	11763.6
11.81	10.87	52.29	23.6	38.0	48.93	78.02	1.162	0.917	1.176	0.9679	0.9414	55.05	67.34	17.7	120.10	66.910	24.49	13926.1
12.80	11.86	52.29	25.6	38.0	48.93	78.02	1.177	0.909	1.188	0.9675	0.9410	55.73	66.76	19.2	120.10	66.199	26.20	16245.2
13.78	12.84	52.29	27.6	38.0	48.93	78.02	1.192	0.902	1.200	0.9672	0.9407	56.41	66.19	20.7	120.10	65.589	27.87	18716.1
14.76	13.83	52.29	29.5	38.0	48.93	78.02	1.207	0.894	1.212	0.9669	0.9404	57.09	65.62	22.1	120.10	65.060	29.51	21333.7
15.75	14.81	52.29	31.5	38.0	48.93	78.02	1.221	0.887	1.224	0.9666	0.9401	57.76	65.04	23.6	120.10	64.598	31.11	24093.1
16.73	15.79	52.29	33.5	38.0	48.93	78.02	1.236	0.879	1.235	0.9663	0.9398	58.44	64.47	25.1	120.10	64.190	32.68	26989.4
17.72	16.78	52.29	35.4	38.0	48.93	78.02	1.251	0.872	1.246	0.9660	0.9395	59.12	63.90	26.6	120.10	63.827	34.21	30017.6
18.70	17.76	52.29	37.4	38.0	48.93	78.02	1.265	0.864	1.256	0.9657	0.9393	59.80	63.33	28.1	120.10	63.502	35.72	33172.9
19.68	18.75	52.29	39.4	38.0	48.93	78.02	1.280	0.857	1.266	0.9654	0.9390	60.47	62.76	29.5	120.10	63.210	37.18	36450.4
20.67	19.73	52.29	41.3	38.0	48.93	78.02	1.295	0.849	1.276	0.9652	0.9387	61.15	62.19	31.0	120.10	62.946	38.62	39845.1

Table H-16 continued.

(b) Loading combination: Strength-I, combination C2 (vertical eccentric)

Soil parameters and GWT:

γ (pcf)	120.10
D_w (ft)	2.6

Footing information:

L (ft)	52.40	
D_f (ft)	0.00	
depth factor, d_q	1.00	(assumed to be on the ground surface)
depth factor, d_γ	1.00	(Vesic 1975)

Load eccentricity and inclination:

inclination, H/V	0.000	(H along transverse dir.)
eccenticity, e_L	0.000	
eccenticity, e_B	0.000	

B (ft)	B'	L'	$2B+D_f$	avg ϕ_f	N_q	N_γ	s_q	s_γ	pow n	i_q	i_γ	N_{qm}	$N_{\gamma m}$	$1.5B+D_f$	γ_1	γ_2	q_n (ksf)	Q_n (kips)
2.95	2.95	52.36	5.9	38.0	48.93	78.02	1.044	0.977	1.058	1.0000	1.0000	51.09	76.26	4.4	120.10	94.661	10.66	1647.9
3.94	3.94	52.36	7.9	38.0	48.93	78.02	1.059	0.970	1.075	1.0000	1.0000	51.81	75.68	5.9	120.10	85.411	12.72	2622.9
4.92	4.92	52.36	9.8	38.0	48.93	78.02	1.073	0.962	1.090	1.0000	1.0000	52.53	75.09	7.4	120.10	79.861	14.76	3802.3
5.91	5.91	52.36	11.8	38.0	48.93	78.02	1.088	0.955	1.106	1.0000	1.0000	53.25	74.50	8.9	120.10	76.161	16.75	5180.8
6.89	6.89	52.36	13.8	38.0	48.93	78.02	1.103	0.947	1.120	1.0000	1.0000	53.96	73.92	10.3	120.10	73.518	18.72	6753.4
7.87	7.87	52.36	15.7	38.0	48.93	78.02	1.117	0.940	1.135	1.0000	1.0000	54.68	73.33	11.8	120.10	71.536	20.65	8514.8
8.86	8.86	52.36	17.7	38.0	48.93	78.02	1.132	0.932	1.149	1.0000	1.0000	55.40	72.74	13.3	120.10	69.994	22.55	10459.9
9.84	9.84	52.36	19.7	38.0	48.93	78.02	1.147	0.925	1.162	1.0000	1.0000	56.12	72.16	14.8	120.10	68.760	24.42	12583.7
10.83	10.83	52.36	21.7	38.0	48.93	78.02	1.162	0.917	1.175	1.0000	1.0000	56.84	71.57	16.2	120.10	67.751	26.25	14880.8
11.81	11.81	52.36	23.6	38.0	48.93	78.02	1.176	0.910	1.187	1.0000	1.0000	57.56	70.98	17.7	120.10	66.910	28.05	17346.2
12.80	12.80	52.36	25.6	38.0	48.93	78.02	1.191	0.902	1.200	1.0000	1.0000	58.28	70.40	19.2	120.10	66.199	29.81	19974.8
13.78	13.78	52.36	27.6	38.0	48.93	78.02	1.206	0.895	1.211	1.0000	1.0000	58.99	69.81	20.7	120.10	65.589	31.55	22761.3
14.76	14.76	52.36	29.5	38.0	48.93	78.02	1.220	0.887	1.223	1.0000	1.0000	59.71	69.22	22.1	120.10	65.060	33.25	25700.6
15.75	15.75	52.36	31.5	38.0	48.93	78.02	1.235	0.880	1.234	1.0000	1.0000	60.43	68.64	23.6	120.10	64.598	34.91	28787.7
16.73	16.73	52.36	33.5	38.0	48.93	78.02	1.250	0.872	1.245	1.0000	1.0000	61.15	68.05	25.1	120.10	64.190	36.54	32017.3
17.72	17.72	52.36	35.4	38.0	48.93	78.02	1.264	0.865	1.255	1.0000	1.0000	61.87	67.46	26.6	120.10	63.827	38.14	35384.3
18.70	18.70	52.36	37.4	38.0	48.93	78.02	1.279	0.857	1.266	1.0000	1.0000	62.59	66.88	28.1	120.10	63.502	39.71	38883.5
19.68	19.68	52.36	39.4	38.0	48.93	78.02	1.294	0.850	1.276	1.0000	1.0000	63.31	66.29	29.5	120.10	63.210	41.24	42509.8
20.67	20.67	52.36	41.3	38.0	48.93	78.02	1.308	0.842	1.285	1.0000	1.0000	64.02	65.70	31.0	120.10	62.946	42.74	46258.1

H.2.5 Allowable Bearing Resistances at the Limit State

H.2.5.1 Overview

The allowable bearing resistances for a Service-I limit state of an allowable settlement of 1.5inches have been obtained using the AASHTO (2007) method (equation 10.6.2.4.2-1), Schmertmann et al. (1978) and Hough (1959) settlement calculation methods.

1. Influence depth:
   For a footing of width L×B = 52.4ft×4.9ft, L/B > 10, therefore, the influence depth below the footing base for settlement calculations is 19.6ft (= 4×4.9ft) (Table H-13).

2. Corrected SPT-N value and E_s from correlation with $(N1)_{60}$:
   The corrected SPT-N value has been assumed to be at the mid-height of the influence depth below the footing base. It has been estimated using the correlation of soil friction angle ϕ_f and $(N1)_{60}$ as:

$$(N1)_{60} = \ln\Big((54-\phi_f)/27.6034\Big)\Big/(-0.014) \approx 39$$

   Hence for gravel, the Young's modulus E_s has been estimated using the following modified correlation given in AASHTO (2007) (Table C10.4.6.3-1)

$$E_s = 0.167(N1)_{60} \text{ ksi } = 12\times(N1)_{60} \text{ tsf } = 468\text{tsf}$$

H.2.5.2 AASHTO (2007) Method

The variation of the elastic shape and rigidity factor β_z with L/B ratio is given in Table H-17 for rigid footings (Table 10.6.2.4.2-1, AASHTO 2007). For the intermediate *L*/*B* ratios, β_z needs to be interpolated as is presented in Table H-18.

Table H-17. Rigidity factor for rigid base footings (AASHTO 2007)

L/B	Rigidity Factor b$_z$
1	1.08
2	1.10
3	1.15
5	1.24
≥ 10	1.41

Table H-18. Interpolated rigidity factors for trial footing widths with a constant length L

L (ft)	B (ft)	L/B	Rigidity Factor b_z
52.4	2.95	17.73	1.410
52.4	3.94	13.30	1.410
52.4	4.92	10.64	1.410
52.4	5.91	8.87	1.371
52.4	6.89	7.60	1.328
52.4	7.87	6.65	1.296
52.4	8.86	5.91	1.271
52.4	9.84	5.32	1.251
52.4	10.83	4.84	1.233
52.4	11.81	4.43	1.215
52.4	12.80	4.09	1.199
52.4	13.78	3.80	1.186
52.4	14.76	3.55	1.175
52.4	15.75	3.33	1.165
52.4	16.73	3.13	1.156
52.4	17.72	2.96	1.148
52.4	18.70	2.80	1.140
52.4	19.68	2.66	1.133
52.4	20.67	2.53	1.127

Here, the Poisson's ratio ν of 0.3 has been taken for the gravel subsurface. Load required to develop settlement of 1.5inches:

$$q = \frac{S_e E_s \beta_z}{(1-\nu^2)\sqrt{A}} = \frac{(1.5/12)\times 468\times 1.41}{(1-0.3^2)\sqrt{4.9\times 52.4}} = 5.65\text{tsf}$$

Thus, it is estimated from the AASHTO method that a load of 5.65tsf on the footing produces a settlement of 1.5inches. The load required to produce a settlement of 1.5in for other footing sizes can be obtained in the similar fashion.

H.2.5.3 Schmertmann (1978) Method

Here, L/B = 52.4/4.92 = 10.6 >10.0. Hence,

I_z = 0.2 at depth = 0

$I_z = I_{zp}$ at depth = z_p = 1.0B = 4.9ft

I_z = 0.0 at depth = D = 4.0B = 19.6ft

The maximum value of I_z at depth z_p is given by:

$$I_{zp} = 0.5 + 0.1\sqrt{\frac{\Delta q}{s'_{vp}}}$$

1. Sub-division of subsurface layers:

For simplicity and automation, the soil layer considered below the footing base has been divided into six layers irrespective of the size of footing, as has been illustrated in Figure H-2.

2. Effective stresses and maximum strain influence factor:
   Effective stress at footing depth, $q_0 = \gamma D_f = 0.0$
   $I_z = I_{zp}$ at the depth of $1.0B+D_f = 1.0\times4.9 + 0.0 = 4.9$ft from the ground level.
   Initial stress at which I_{zp} occurs (=4.9ft) is

$$\sigma'_{vp} = \sum \gamma_i \Delta z_i = 120.1\times4.9 - 62.4(4.9 - 2.6) = 444.97\text{psf} = 0.2225\text{tsf}$$

3. Assumption of a load for settlement prediction:
   Since I_z and C_1 are functions of the applied load on the footing, an iteration process is necessary to obtain the required load q to produce a prescribed settlement S_e (1.5inches here). For the start, let q = 3.0tsf. Then,

$$\Delta q = q - q_0 = 3.0 - 0.0 = 3.0\text{tsf} \text{ , and}$$

$$I_{zp} = 0.5 + 0.1\sqrt{\frac{3.0}{0.222}} = 0.867$$

$$C_1 = 1.0 - 0.5(0.0/3.0) = 1.0$$

4. Strain influence factor Iz at mid-height of each of the subdivided layer:
   Let the depth of layer mid-height from the footing base be $D_{zi}\times B$. Then For $D_{zi} < z_p/B$, I_{zi} can be interpolated as:

$$I_{zi} = I_{zp} - \left(\frac{1.0 - D_{zi}}{1.0 - 0}\right)\left(I_{zp} - 0.2\right)$$

   And, for $D_{zi} \geq z_p/B$, I_{zi} can be interpolated as:

$$I_{zi} = I_{zp} - \left(\frac{D_{zi} - 1.0}{4.0 - 1.0}\right)\left(I_{zp} - 0\right)$$

   For layer #1, $D_{z1} = 0.5\times(1.0/3) = 0.1667$

$$I_{z1} = I_{zp} - (1.0–0.1667)/(1.0)\times(I_{zp}– 0.2) = 0.867 - (0.556) = 0.311$$

   Similarly, For layer #4, $D_{z4} = 1.0 + 0.5\times(4.0 - 1.0)/3 = 1.50$

$$I_{z4} = I_{zp} - (1.5–1.0)\ I_{zp}/(3.0) = I_{zp}\ (0.8333) = 0.722$$

   The values of I_{zi} for other soil layers, calculated in similar fashion, are shown in the detailed calculations.

5. Es for each sub-divided layer:
   The Young's modulus of elasticity has been considered to be a constant of 468.0tsf throughout the soil layer up to the influence depth.

6. Detailed calculations:

After the sum of $(I_z / E_s)\times\Delta z$ is obtained, the resulting settlement can be calculated using Equation (H-8). The detailed calculation is shown below. The calculation is repeated with trial applied loads *q* until the required settlement is obtained.

B (ft) =	4.9
From GL, z_p (ft) =	4.9
From GL, D (ft) =	19.6
σ'_{vp} (tsf) =	0.222
q_0 (tsf) =	0.00

Trail 1:

Let q (tsf) =	3.00
Then Δq =	3.00
I_{zp} =	0.8672
C_1 =	1.0000
C_2 =	1.0000

Subdivided Layer #	Depth below GL (ft)	Depth below footing base (ft)	Layer thickness Δz (ft)	Mid-height depth below footing base D_z (ft)	Strain influence factor, I_z	Average E_s (tsf)	$I_z / E_s * \Delta z$
1	1.6	1.633	1.633	0.817	0.3112	468.00	0.001086
2	3.3	3.267	1.633	2.450	0.5336	468.00	0.001862
3	4.9	4.900	1.633	4.083	0.7560	468.00	0.002638
4	9.8	9.800	4.900	7.350	0.7227	468.00	0.007566
5	14.7	14.700	4.900	12.250	0.4336	468.00	0.004540
6	19.6	19.600	4.900	17.150	0.1445	468.00	0.001513
						sum:	0.019206

S_e (in) = 0.691

Example of a next trial:

Let q (tsf) =	5.66
Then Δq =	5.66
I_{zp} =	1.0044
C_1 =	1.0000
C_2 =	1.0000

Subdivided Layer #	Depth below GL (ft)	Depth below footing base (ft)	Layer thickness Δz (ft)	Mid-height depth below footing base D_z (ft)	Strain influence factor, I_z	Average E_s (tsf)	$I_z / E_s * \Delta z$
1	1.6	1.633	1.633	0.817	0.3341	468.00	0.001166
2	3.3	3.267	1.633	2.450	0.6022	468.00	0.002102
3	4.9	4.900	1.633	4.083	0.8703	468.00	0.003037
4	9.8	9.800	4.900	7.350	0.8370	468.00	0.008763
5	14.7	14.700	4.900	12.250	0.5022	468.00	0.005258
6	19.6	19.600	4.900	17.150	0.1674	468.00	0.001753
						sum:	0.022079

S_e (in) = 1.500

Hence, for a footing of width 4.9ft, a load of 5.66tsf is estimated to produce a settlement of 1.5in using Schmertmann (1978) method. The load required to produce a settlement of 1.5in for other footing sizes can be obtained in the similar fashion.

H.2.5.4 Hough (1959) Method

1. Bearing capacity index C′ based on corrected SPT value at layer mid-height:
   In the calculation presented here, the value of C′ has taken from digitized and fitted curves of Figure H-3 for automation. The curve fittings are listed in Table H-10.
   For well graded silty Sand and Gravel,

   $$C' = 0.0335(N1)_{60}^{2} + 0.8276(N1)_{60} + 42.86 = 0.0335\times 39^{2} + 0.8276\times 39 + 42.86 = 126.090$$

2. Increase in stress at each layer mid-height:
   The increase in stress at layer mid-height is obtained using 2:1 method of stress distribution. This method approximates the vertical stress $\Delta\sigma_v$ at a depth z which is caused by a footing of dimension $L\times B$ loaded with a force Q as the following.

   $$\Delta\sigma_v = \frac{Q}{(B+z)(L+z)} = q\cdot\frac{BL}{(B+z)(L+z)}$$

3. Estimation of load required:
   Since the layer is assumed to be of homogeneous gravel borrow of unit weight 120.1pcf, the load required for the stated settlement of 1.5in can be calculated by rearrangement of Equation (H-10), without the need for iteration.
   Layer thickness = depth of influence below footing base = Δz = 19.6ft
   Layer mid-height depth from footing base $z = \Delta z/2$ = 9.8ft
   Initial effective overburden pressure at layer mid-height,

   $$\sigma_{v0} = 120.1\times 9.8 - 62.4(9.8 - 2.6) = 727.7\text{psf} = 0.364\text{tsf}$$

   Equation (H-10) can be arranged as follows to estimate the load required, q.

   $$\frac{S_e C'}{\Delta z} = \log_{10}\left(\frac{\sigma_{v0}+\Delta\sigma_v}{\sigma_{v0}}\right) \Rightarrow \Delta\sigma_v = \sigma_{v0}\left(10^{\left(\frac{S_e C'}{\Delta z}\right)} - 1\right)$$

   Hence,

   $$q = \frac{(B+z)(L+z)}{BL}\cdot\sigma_{v0}\left(10^{\left(\frac{S_e C'}{\Delta z}\right)} - 1\right)$$
   $$= \frac{(4.9+9.8)(52.4+9.8)}{4.9\times 52.4}(0.364)\left(10^{(1.5/12\times 126.0/19.6} - 1\right) = 6.95\text{tsf}$$

For a footing of width 4.9ft, a load of 6.95tsf is estimated to produce a settlement of 1.5in using Hough (1959) method. The load required to produce a settlement of 1.5in for other footing sizes can be estimated in the similar fashion.

H.2.6 Resistance Factors

The footing for the central pier is to be constructed on site, resting on a gravel fill, hence in a controlled soil condition for which soil friction angle is assumed to be 38°. The resistance factors, recommended in this study, to be used for Strength-I corresponding to the C2 loading combination is 0.70, while that corresponding to the C7 loading combination is 0.45 (positive eccentricity). The AASHTO (2007) specification recommends $\phi = 0.45$. No resistance factors exist in the current specifications for the Service limit state, hence, the estimated load required to produce a settlement of 1.5in has been left unfactored.

H.2.7 Design Footing Width

The maximum load eccentricity of 0.47ft across the footing width, according to Table H-14, is caused by the load combination C7 for both Strength-I and Service-I load conditions. In addition, the eccentricity ratios across the footing length (e_3/L) are very small even for the load combination C10 related to Extreme-I loading conditions (a maximum of 0.016), hence, the effect of the load eccentricity across the footing length can be neglected for all practical purposes for this example. The maximum load eccentricity for design is thus taken as along the footing width only with a rounded-off value of $e_2 = 0.50$ft. Hence, the minimum admissible footing width is B = 3.0ft ($=e_B \times 6 = 0.50\text{ft} \times 6$), considering the limiting eccentricity ratio e_B/B of 1/6.

The maximum factored vertical load from Strength-I load is 3688.3 kips (corresponding to Strength-I C2), whereas, that for Service-I load is 2750.3 kips (corresponding to Service-I C2) (refer to Table H-15.1), whereas, the factored vertical load from Strength-I C7 is 3460.8kips.

Figures H-5 and H-6 present the unfactored and factored bearing resistances for Strength-I loading, respectively, for different effective footing widths. The bearing load intensities (stresses) are plotted in the upper figures, whereas the lower ones present the bearing loads. The footing width refers to the effective width for both bearing capacity and settlement analyses. While the settlement analyses were carried out for the geometrical (full) foundation width, in the presentation of Figures H-5 and H-6, the width was transformed to be the effective width.

Applying the aforementioned corresponding vertical loads for the limit states in Figures H-5 and H-6, the following results are obtained: (a) for unfactored Service limit state (current AASHTO specifications), a footing width of 3.0ft (minimum admissible width) is required according to Hough (1959) method, 4.3ft is required according to Schmertmann (1978) method and 4.5ft is required according to AASHTO (2007) method, and (b) for factored Strength-I limit state, the minimum required footing width is 6.0ft when Strength-I C2 loading is considered, 8.9 ft when Strength-I C7 loading is considered. The recommended resistance factor in this study for Strength-I C7 loading is $\phi = 0.45$, which corresponds to the current AASHTO (2007) specifications recommendation, thus the minimum footing width required as per the AASHTO(2007) specifications is also 8.9ft.

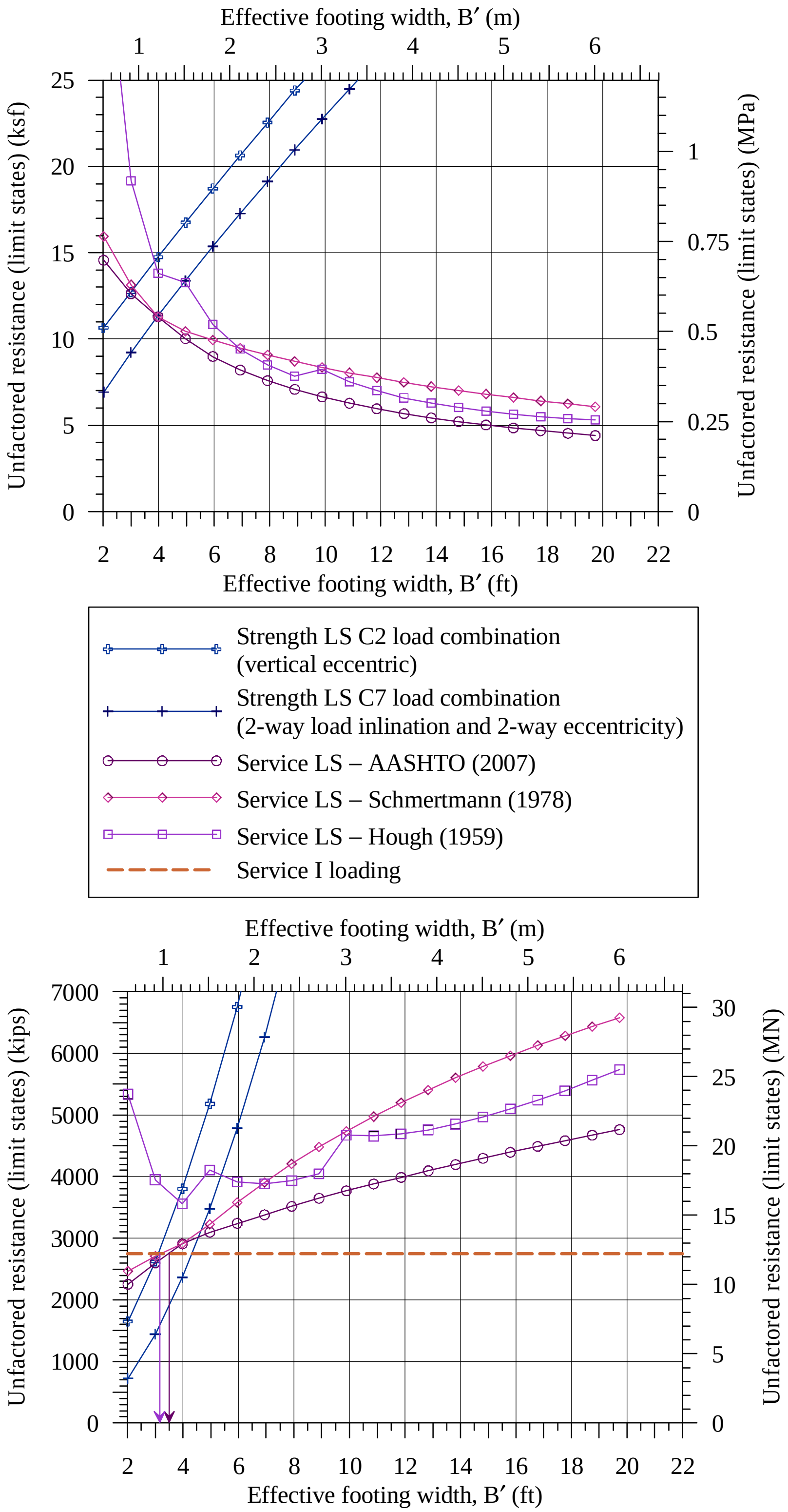

Figure H-5. Variation of unfactored bearing resistance for Strength-I and Service-I limit states with effective footing width for Example 2

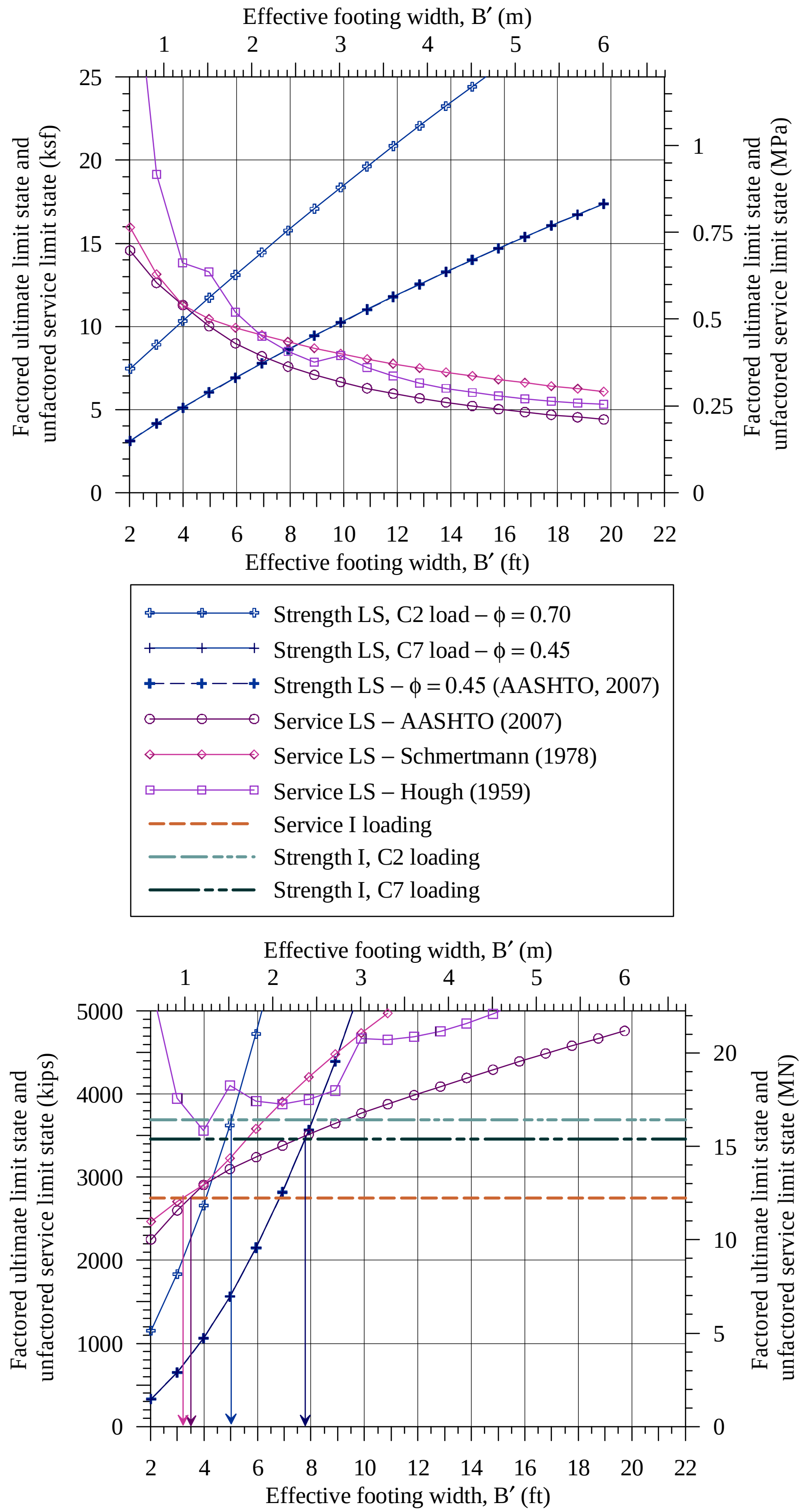

Figure H-6 Variation of factored bearing resistance for Strength-I and unfactored resistance for Service-I limit state with effective footing width for Example 2

The conclusions possible from Figures H-5 and H-6 are therefore:

1. Based on the strength limit state alone; the following foundation sizes (full geometric) are sufficient:
   - Strength limit state $\phi = 0.45$: 8.9ft×52.4 ft
   - Strength limit state $\phi = 0.70$: 6.0ft×52.4 ft
2. Based on the unfactored serviceability limit state (current AASHTO): a footing size of 4.5ft×52.4ft is required.

It can be noted that the Strength (C7) limit state determines the design in this design example.

H.2.8 Sliding Resistance

The interfacial friction angle between the footing base and the gravel borrow fill is given as $\delta_s = 29.7°$ in the geotechnical report. This interfacial friction angle is conservative compared to the one recommended in this study. For $\phi_f = 38°$, the interfacial friction angle obtained from the recommended relation is

$$\tan(\delta_s) = 0.91 \tan(38) \Rightarrow \delta_s = 35.4°$$

But here, nominal sliding resistance has been calculated as follows:

$$F_{2\tau} = F_1 \times \tan(\delta_s) = F_1 \times \tan(29.7)$$

The minimum factored vertical load shown in Table H-15.2 for the designed footing width for Strength-I and Service-I load against sliding is 2620.3kips (Service-I C5), for which the maximum lateral load is 24.6kips. Though in the case of the central pier no lateral force is caused by the earth pressure, the recommended resistance factors is taken as the minimum of the recommended in this study for cast in-situ footings, i.e. $\phi_\tau = 0.40$. Hence,

$$\text{Factored sliding resistance, } \phi_\tau F_{2\tau} = 0.40 \times 2620.3 \times \tan(29.7) = 597.8\text{kips} > 24.6\text{kips}$$

The designed footing is safe in sliding for the Service-I C5 loading. For other load combinations, e.g. Strength-I C7, though the resultant lateral load is larger ($=(25.0^2+9.2^2)^{0.5} = 26.6$kips), the vertical load is larger (=2673.5kips) too. The factored sliding resistance for this vertical load is 610.0kips, which is much larger than the lateral load of 26.6kips. Therefore, the designed footing is safe in sliding.

H.2.9 Discussions and Conclusions

It is seen from Figures H-5 and H-6 that the Strength limit states govern the footing dimension in this design example. The Strength limit states are satisfied with a full geometric foundation width of 8.9ft (considering a maximum eccentricity of 0.50ft). The unfactored Service limit state requires a foundation width of at least 4.5ft. The footing widths for both of these limit states are smaller than the actually constructed footing, of width 13.1ft. This could be due to the difference in the settlement estimation methods used in the design reference and this study; the reference uses settlement estimation method described in Peck et al. (1974), which has not been used in this study.

H.3 EXAMPLE 3: BILLERICA BRIDGE, EAST ABUTMENT ON GRAVEL FILL

H.3.1 Subsurface Condition

For general information regarding the Billerica Bridge, please refer to section 5.4.1. The subsurface at the east abutment location (based on boring GB-21) consists of 9inch of asphalt overlaying approximately 7.8ft of dense granular fill and then 4.0ft of very dense coarse sand and gravel overlaying a rock layer. The geotechnical report (URS, 2001) called for the replacement of the fill with gravel borrow material that would extend to the proposed footing elevation. As such, the foundation design follows the geotechnical report assuming the east abutment to be founded on the top of a compacted gravel layer, as for the central pier presented in Example 2. The parameters provided for the gravel borrow in the geotechnical report are: bulk unit weight γ (γ') = 120.0 pcf (63.65 pcf) (18.85/9.99 kN/m³), internal friction angle ϕ_f = 38° and interfacial friction angle between the footing base and the soil δ_s = 29.7°. The unit weight of the soil backfill of the abutment is taken as 124.9 pcf (19.6 kN/m³). The groundwater table is located at elevation 157.5 ft (48.0 m) and the foundation base is at elevation of 166.7ft (50.8m).

H.3.2 Loads, Load Combinations and Limit States

The load components as given in the reference are summarized in Table H-19. The loads are provided in units of force per unit foundation length referring to the abutment length of 61.65ft (across the bridge). The dead load includes the weights of superstructure and abutment as well as the soil backfill. The investigated load combinations and the resultant characteristic loading as well as the eccentricity e_2 (refer to Figure 120 of Chapter 5 for load notations and directions) for the different load combinations are summarized in Table H-20. The design load components required for the stability analysis, which are the factored characteristic loadings with load factors according to AASHTO Section 3 (2007) (presented in Tables H-4.1 and H-4.2), are summarized in Tables H-21.1 and H-21.2 for the bearing capacity and sliding strength limit states, respectively. Only Service-I and Strength-I limit states have been used here for the determination of the design footing width. Due to the large magnitude of earth pressures at the abutment, the lateral loads and the eccentricities are markedly higher than those presented in the central pier analysis in example 2.

TABLE H-19. Loading at footing base for Example 3

Load at footing base	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
dead load (DL)	35.80 (522.36)	0.0	-37.16 (-165.32)
live load (LL)	4.40 (64.23)	1.61 (23.56)	23.24 (103.38)
earth pressure (E)	0.0	9.61 (140.20)	90.14 (400.97)
wind on structure (W)	0.0	0.19 (2.73)	3.25 (14.44)
wind on live load (WL)	0.0	0.04 (0.61)	0.73 (3.23)
lateral force (LF)	0.0	0.13 (1.94)	2.31 (10.26)
temperature effects (RST)	0.0	0.59 (8.64)	10.28 (45.71)
earthquake (EQ)	0.0	3.97 (57.87)	68.82 (306.13)

TABLE H-20. Load combinations and resultant characteristic (unfactored) loading for Example 3

Load combi-nations	Load components	F_1 kips/ft (kN/m)	F_2 kips/ft (kN/m)	M_3 kips-ft/ft (kNm/m)	F_2/F_1	$e_2 = M_3/F_1$ ft (m)
C1	DL + LL + E	40.2 (586.6)	11.2 (163.8)	76.2 (339.0)	0.279	1.896 (0.578)
C2	DL + E + W	35.8 (522.4)	9.8 (142.9)	56.2 (250.1)	0.274	1.571 (0.479)
C3	DL + LL + E + W + WL + LF	40.2 (586.6)	11.6 (169.0)	82.5 (367.0)	0.288	2.053 (0.626)
C4	DL + LL + E + RST	40.2 (586.59)	11.8 (172.4)	86.5 (384.7)	0.294	2.152 (0.656)
C5	DL + E + W + RST	35.8 (522.4)	10.4 (151.6)	66.5 (295.8)	0.290	1.856 (0.566)
C6	DL + LL + E + W + WL + LF + RST	40.2 (586.6)	12.2 (177.7)	92.8 (412.7)	0.303	2.309 (0.704)
C7	DL + LL + E + EQ	40.2 (586.6)	20.8 (304.0)	145.0 (645.2)	0.518	3.608 (1.100)

TABLE H-21.1. Load combinations and resultant design (factored) loading for bearing resistance

Limit state load combinations	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
Service-I C1	40.2 (586.6)	11.2 (163.8)	76.2 (339.0)
Service-I C2	35.8 (522.4)	9.7 (141.0)	53.9 (240.0)
Service-I C3	40.2 (586.6)	11.5 (167.1)	80.2 (356.9)
Service-I C4	40.2 (586.6)	11.9 (174.1)	88.5 (393.9)
Service-I C5	35.8 (522.4)	10.4 (151.4)	66.3 (294.8)
Service-I C6	40.2 (586.6)	12.2 (177.5)	92.6 (411.7)
Strength-I C1	52.5 (765.4)	17.2 (251.5)	129.4 (575.7)
Strength-I C4	52.5 (765.4)	17.9 (261.9)	141.8 (630.6)
Strength-V C2	50.7 (739.7)	16.9 (246.4)	125.3 (557.2)
Strength-V C3	50.7 (739.7)	16.6 (242.1)	120.1 (534.4)
Strength-V C5	44.8 (653.0)	15.2 (221.8)	102.4 (455.4)
Strength-V C6	50.7 (739.7)	17.6 (256.8)	137.6 (612.1)
Extreme-I C7	49.2 (717.2)	20.0 (291.7)	180.8 (804.3)

$\gamma_{EQ} = 1.0$

TABLE H-21.2. Load combinations and resultant design (factored) loading for sliding resistance

Limit state load combinations	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
Service-I C1	40.2 (586.6)	11.2 (163.8)	76.2 (339.0)
Service-I C2	35.8 (522.4)	9.7 (141.0)	53.9 (240.0)
Service-I C3	40.2 (586.6)	11.5 (167.1)	80.2 (356.9)
Service-I C4	40.2 (586.6)	11.9 (174.1)	88.5 (393.9)
Service-I C5	35.8 (522.4)	10.4 (151.4)	66.3 (294.8)
Service-I C6	40.2 (586.6)	12.2 (177.5)	92.6 (411.7)
Strength-I C1	39.9 (582.5)	17.2 (251.5)	142.4 (633.6)
Strength-I C4	39.9 (582.5)	17.9 (261.9)	154.8 (688.4)
Strength-V C2	38.2 (556.8)	16.9 (246.4)	138.3 (615.1)
Strength-V C3	38.2 (556.8)	16.6 (242.1)	133.1 (592.2)
Strength-V C5	32.2 (470.1)	15.2 (221.8)	115.4 (513.3)
Strength-V C6	38.2 (556.8)	17.6 (256.8)	150.6 (669.9)
Extreme-I C7	36.6 (534.4)	20.0 (291.7)	193.8 (862.2)

H.3.3 Nominal and Allowable Bearing Resistances at the Limit States

The bearing resistances of rectangular footings with widths of 2.95ft to 20.70ft, with the footing length kept fixed at 61.65ft according to the length of the abutment, have been calculated for Strength-I C4 limit state, according to AASHTO (2007) equation 10.6.3.1.2 with embedment depth equal to zero (note: the results are presented in the following sections as effective widths). The allowable bearing resistance for a Service-I limit state for a settlement of 1.5inches has been obtained using the AASHTO (2007) settlement calculation method (equation 10.6.2.4.2-1), Schmertmann (1978) and Hough (1959) settlement calculation methods.

From Table H-20, the Strength-I C4 loading on the footing produces one-way eccentricity of e_2 = 2.15ft along the footing width along with one-way inclination. Hence for an example footing width of, say, B = 4.9ft, the effective footing width is $B' = 4.9 - 2\times2.15 = 0.6$ft.

The footing for the abutment is placed on a gravel borrow fill compacted to result in an internal friction of 38°. The recommended resistance factor for Strength-I C4 load in/on *controlled soil condition* with soil of $\phi_f = 38°$ is $\phi = 0.45$, which coincides with that recommended in AASHTO (2007) specifications. No resistance factors exist in the current specifications for the service limit state, hence, the estimated load required to produce a settlement of 1.5in has been left unfactored.

H.3.4 Design Footing Width

The maximum load eccentricity corresponding to Strength-I loading is 2.15ft, produced by C4 load combination, whereas, for Service-I loading is 2.31ft produced by C6 load combination, along the footing width in both limit states, according to Table H-20. Hence, the minimum foundation width required for the limiting eccentricity is B = 13.86ft (=2.31ft×6). The maximum vertical factored load for Strength-I limit state (bearing resistance; Table H-21.1) is 52.5kip/ft and the vertical unfactored load for Service-I limit state is 40.2kips/ft.

Figures H-7 and H-8 present the unfactored and factored bearing resistances for different effective footing widths. The bearing load intensities (stresses) are plotted in the upper figures, whereas the lower ones present the bearing loads per unit length of the foundation to be compatible with the load representation. The footing width refers to the effective width for both bearing capacity and settlement analyses. While the settlement analyses were carried out for the geometrical (full) foundation width, in the presentation of Figures H-7 and H-8, the width was transformed to be the effective width.

Figure H-7 shows the variation of unfactored bearing capacities with effective footing width for different Strength limit states as well as Service limit state estimated using AASHTO (2007) method. It can be seen that the unfactored load combination C7, which is related to the Extreme-I event (Tables H-20 and H-21.1), is the dominant load combination for design. But the current discussions are limited to the Strength-I limit state load combinations, namely C1 and C4, since the resistance factors have been developed only for the Strength-I limit states. Figure H-8 shows the variation of factored bearing capacities with effective footing width for Service-I and Strength-I (C4) loadings.

Applying the aforementioned vertical loads for the corresponding limit states in Figure H-8, the following results are obtained: (a) the minimum footing width required for Strength-I loading is 15.44ft when the recommended resistance factor of $\phi = 0.45$ is applied, and (b) the minimum effective width required for Service-I LS, can be taken as the minimum admissible footing width for the limiting eccentricity corresponding to a full width 13.86ft (which corresponds to $B' = 9.25$ft in Figures H-7 and H-8).

The conclusions possible from Figures H-7 and H-8 are therefore:

1. Based on strength limit state alone, the following foundation size (full geometry) is sufficient:
   a. Strength limit state $\phi = 0.45$: 15.5 ft×61.65 ft
2. Based on unfactored serviceability limit state (current AASHTO), all admissible footings for limiting eccentricity are safe: 13.9ft×61.65 ft

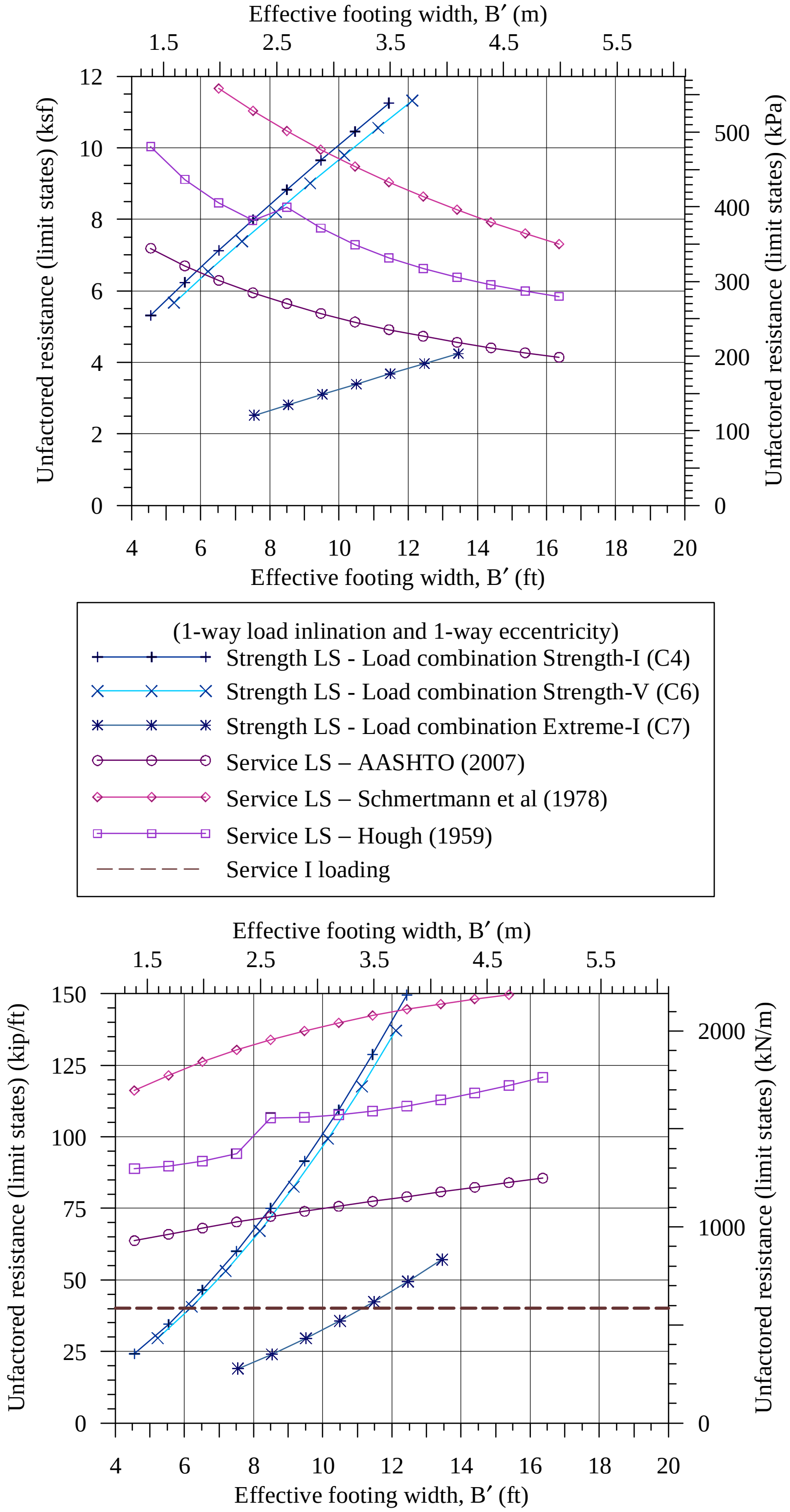

Figure H-7. Variation of unfactored bearing resistance for Strength and Service-I limit states with effective footing width for Example 3; loads are expressed per unit length of the foundation (L = 61.65ft)

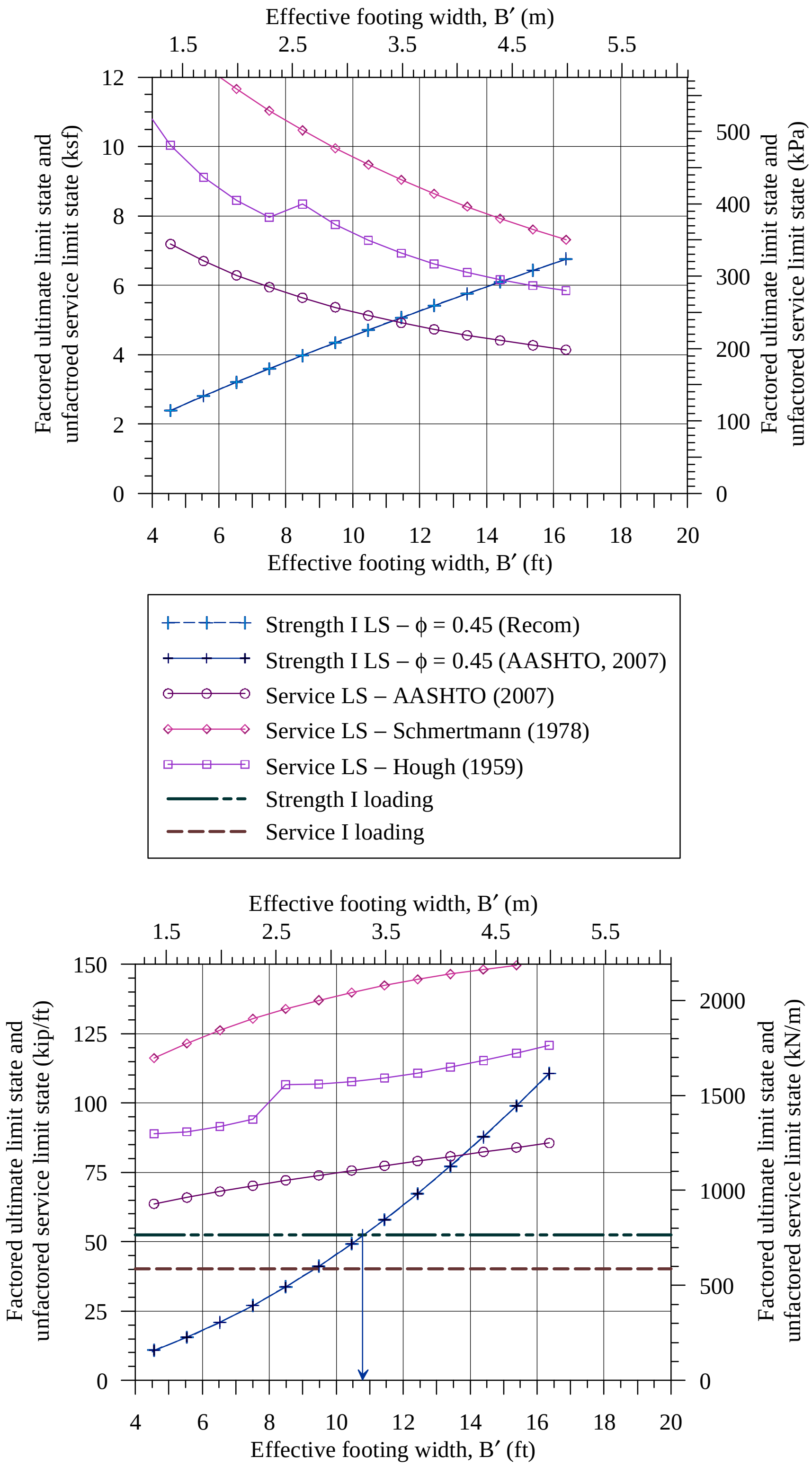

Figure H-8. Variation of factored bearing resistance for Strength-I C4 loading combination and unfactored Service-I limit states with effective footing width for Example 3; loads are expressed per unit length of the foundation (L = 61.65ft)

H.3.5 Sliding Resistance

The interfacial friction angle between the footing base and the gravel borrow fill is given as $\delta_s = 29.7°$ in the geotechnical report. This interfacial friction angle is conservative compared to the one recommended in this study. For $\phi_f = 38°$, the interfacial friction angle obtained from the recommended relation in this study is as follows, which has been used only for the purpose of comparison:

$$\tan(\delta_s) = 0.91\tan(38) \quad \Rightarrow \quad \delta_s = 35.4°$$

The recommended resistance factor for cast in-situ footings when at-rest earth pressure is acting is $\phi_\tau = 0.40$ and that when active earth pressure is acting is $\phi_\tau = 0.45$. The current AASHTO (2007) specification recommends $\phi_\tau = 0.80$. Here, the lateral earth load considered during the design process is related to the at-rest earth pressure. For the back-fill with soil friction angle $\phi_f = 38°$, the ratio of the lateral active earth pressure coefficient to the lateral at-rest earth pressure coefficient, $K_a / K_0 = 1 / (1 + \sin\phi_f) = 1/1.616$, assuming Rankine's active earth pressure and at-rest earth pressure for normally consolidated cohessionless sand.

For the designed footing, the minimum factored vertical load and the corresponding lateral loads under Strength-I and Service-I loadings, thereby, the factored sliding resistance in each case are as follows.

Service-I LS:

At-rest earth pressure:

The minimum vertical load = 35.8kips/ft, and the corresponding maximum lateral load = 10.4kips/ft (Table H-21.2)

$\delta_s = 29.7°$: $\phi_\tau F_{2\tau} = 0.40 \times 35.8 \times \tan(29.7) = 8.2\text{kips/ft} < 10.4\text{kips/ft}$

$\delta_s = 35.4°$: $\phi_\tau F_{2\tau} = 0.40 \times 35.8 \times \tan(35.4) = 10.2\text{kips/ft} < 10.4\text{kips/ft}$

Active earth pressure:

The corresponding lateral load involving active earth pressure is (Tables H-19 and H-21.2)

$$F_{2Ea} = 10.4 - 9.61 + \left(\frac{1}{1.616} \times 9.61\right) = 6.74\text{kips/ft}.$$

$\delta_s = 29.7°$: $\phi_\tau F_{2Ea} = 0.45 \times 35.8 \times \tan(29.7) = 9.2\text{kips/ft} > 6.74\text{kips/ft}$

$\delta_s = 35.4°$: $\phi_\tau F_{2Ea} = 0.45 \times 35.8 \times \tan(35.4) = 11.4\text{kips/ft} > 6.74\text{kips/ft}$

Current AASHTO:

$\delta_s = 29.7°$: $\phi_\tau F_{2\tau} = 0.80 \times 35.8 \times \tan(29.7) = 16.3\text{kips/ft} > 10.4\text{kips/ft}$

$\delta_s = 35.4°$: $\phi_\tau F_{2\tau} = 0.80 \times 35.8 \times \tan(35.4) = 20.4\text{kips/ft} > 10.4\text{kips/ft}$

Strength I LS:

At-rest earth pressure:

The minimum vertical load = 39.9kips/ft, and the corresponding maximum lateral load = 17.9kips/ft (Table H-21.2)

$\delta_s = 29.7°$: $\phi_\tau F_{2\tau} = 0.40 \times 39.9 \times \tan(29.7) = 9.10\text{kips/ft} < 17.9\text{kips/ft}$

$\delta_s = 35.4°$: $\phi_\tau F_{2\tau} = 0.40 \times 39.9 \times \tan(35.4) = 11.34\text{kips/ft} < 17.9\text{kips/ft}$

Active earth pressure:

The corresponding lateral load involving factored active earth pressure is (load factors given in Table H-4.2)

$$\gamma_i F_{2Ea} = 17.9 - 1.5\times 9.61 + 1.5\times\left(\frac{1}{1.616}\times 9.61\right) = 12.41\text{kips/ft} .$$

$\delta_s = 29.7°$: $\phi_\tau F_{2Ea} = 0.45 \times 39.9\times\tan(29.7) = 10.2\text{kips/ft} < 12.4\text{kips/ft}$

$\delta_s = 35.4°$: $\phi_\tau F_{2Ea} = 0.45 \times 39.9\times\tan(35.4) = 12.8\text{kips/ft} > 12.4\text{kips/ft}$

Current AASHTO:

$\delta_s = 29.7°$: $\phi_\tau F_{2\tau} = 0.80 \times 39.9\times\tan(29.7) = 18.2\text{kips/ft} > 17.9\text{kips/ft}$

$\delta_s = 35.4°$: $\phi_\tau F_{2\tau} = 0.80 \times 39.9\times\tan(35.4) = 22.7\text{kips/ft} > 17.9\text{kips/ft}$

This shows that the sliding resistance factors recommended in this study result in footings larger than the designed footing for design against sliding failure due to lateral loads involving at-rest earth pressure whether the soil-footing interfacial friction angle recommended in the geotechnical report is used or the one obtained from the relation between ϕ_f and δ_s in this study is used. However, the application of the resistance factors in the current AASHTO (2007) specifications shows that the designed footing is safe in sliding failure. Unlike for bridge pier designs, in the bridge abutment designs, design against sliding failure is critical as the lateral forces from the back-fill earth pressure is constantly acting on the abutment footing. This result shows that it is desirable to further study the sliding resistance uncertainty, and consequently the resistance factors recommended in the present study

H.3.6 Discussions and Conclusions

The design footing width required for the nominal and allowable bearing resistances at the limit states is found to be at least 15.5ft when $\phi = 0.45$ is used considering a maximum load eccentricity of 2.31ft (refer to Table H-21.1). It can be noted here (as well as from Figure H-8) that the Strength limit state dominates the design footing width for all footings with minimum admissible width for the limiting eccentricity. In addition, the foundation widths for the Strength-I loadings, factored with 0.45, as well as the Service-I loadings are greater than the actual bridge abutment designed width of 12.5ft. This special case strongly emphasizes the importance of careful design under large load inclinations for which the serviceability does not necessarily control the foundation dimensions.

H.4 EXAMPLE 4: INTEGRAL BRIDGE ABUTMENT ON STRUCTURAL FILL – GEC6–EXAMPLE 2

H.4.1 Subsurface Conditions

The subsurface conditions and the abutment geometry given in Example B2 of FHWA Geotechnical Engineering Circular No. 6 (GEC6), Appendix B (Kimmerling, 2002) is shown in Figure H-9, and the soil parameters are summarized in Table H-22. The groundwater table is located at 42.0 ft (12.81 m) below the surface of the proposed bridge approach elevation. The abutment is placed on structural fill of well graded silty sand and gravel that is 15.0ft (4.57m) deep below the footing base. The fill forms a slope with a grade of 2H:1V at a distance of 1.5 times the width of the proposed footing from the slope.

The soil friction angles are calculated using the correlation with SPT blow counts proposed by Peck, Hanson and Thornburn (PHT) as modified by Kulhawy and Mayne (1990). This calculation of the soil friction angle is compatible with the methodology used in developing the resistance factors. The footing is poured on site, hence, the base friction angle δ_s is assumed to be equal to the soil friction angle ϕ_f.

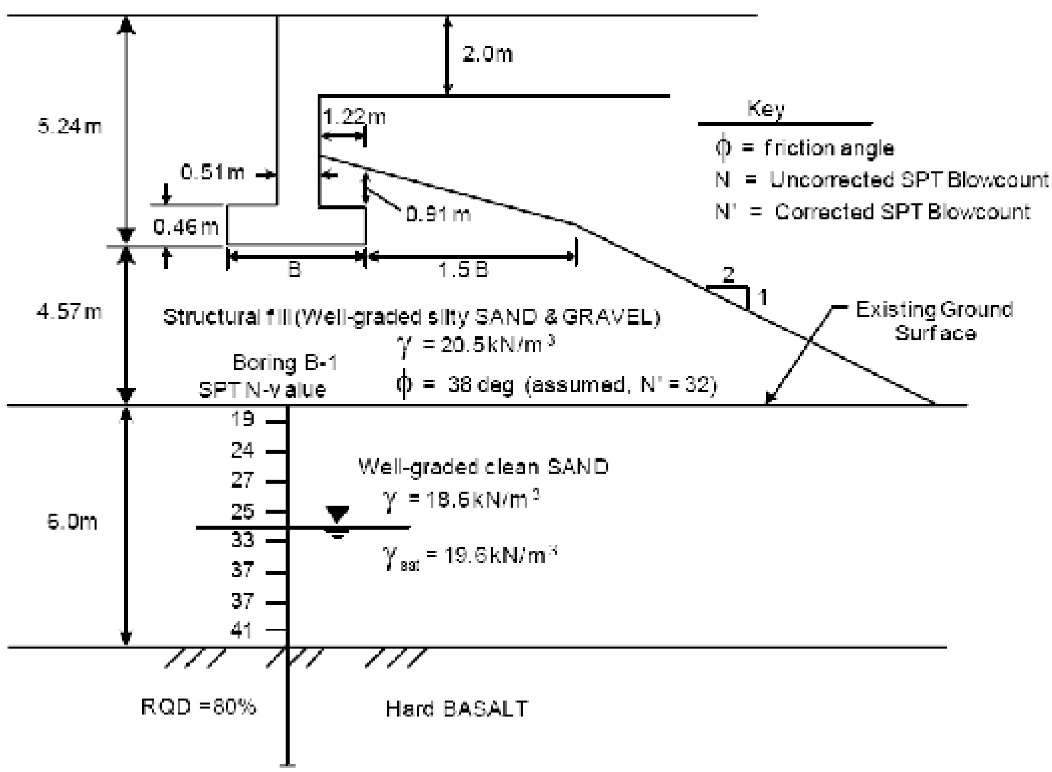

Figure H-9 Geometry and soil conditions of integral bridge abutment – Example 4 (1m » 3.3ft)

TABLE H-22. Soil parameters

Layer #	Thickness below footing ft (m)	Soil Type	g / g' pcf (kN/m³)	f_f [°]
1	15.0 (4.6)	Structural fill (sand & gravel)	130.6 (20.5)	38.0
2a	9.8 (3.0)	Sand above groundwater	118.5/61.2 (18.6/9.6)	39.3
2b	9.8 (3.0)	Sand below groundwater	118.5/61.2 (18.6/9.6)	38.3
3	∞	Basalt	-	-

H.4.2 Loads, Load Combinations and Limit States

Table H-23 presents the loadings from the bridge structure at the footing base that are given as load per unit length of the foundation being an abutment 82.0ft (25.0m) long (across the bridge). The notations and directions of which correspond to those presented in Figure 120 of Chapter 5. Table H-24 summarizes the investigated load combinations and the resultant characteristic loading as well as the load eccentricity for different load combinations. The design load components required for the stability analysis are the factored characteristic loadings with load factors presented in Tables H-4.1 and H-4.2 (according to AASHTO specifications, 2007) for the bearing and the sliding resistances, respectively. Only the Service-I and Strength-I load conditions are checked for this example. These design loadings are presented in Tables H-25.1 and H-25.2, respectively.

TABLE H-23. Load components at footing base for Example 4

Load component	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
dead load of components (DL)	11.60 (169.22)	2.92 (42.61)	-4.15 (-18.44)
dead load of wearing surfaces (DW)	1.37 (19.99)	1.14 (16.64)	-2.84 (-12.62)
vehicular live load (LL)	3.33 (48.60)	1.14 (16.64)	-6.81 (-30.28)
vehicular braking forces (BR)	0.04 (0.58)	0.25 (3.65)	-1.22 (-5.44)
earth pressure at rest (E)	0.0	-7.60 (-110.92)	-43.55 (-193.74)
earth pressure from live loads (EL)	0.0	-1.74 (-25.40)	-14.96 (-66.55)
dead weight of stem	3.10 (45.30)	0.0	-0.25 (-1.13)
dead weight of footing	2.22 (32.43)	0.0	0.0
weight of soil over toe	1.49 (21.76)	0.0	-4.23 (-18.82)
weight of soil over heal	8.15 (118.98)	0.0	46.72 (207.82)

TABLE H-24. Load combinations and resultant characteristic (unfactored) loading

Load combinations	Load components	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_{center} kip-ft/ft (kNm/m)	F_2/F_1	$e_2 = M_3/F_1$ ft (m)
C1	EG+DL+DW+E +LL+EL+BR	31.3 (456.9)	3.5 (50.5)	31.3 (139.2)	0.111	1.000 (0.305)
C2	EG+DL+DW+E	27.9 (407.7)	3.5 (51.7)	8.3 (36.9)	0.127	0.298 (0.091)

TABLE H-25.1. Load combinations and resultant design (factored) loading for bearing resistance

Limit state load combinations	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
Service-I C1	31.3 (456.9)	3.5 (50.5)	31.3 (139.2)
Strength-I C1	41.2 (600.67)	5.9 (86.13)	62.2 (276.74)
Strength-I C2	35.3 (514.61)	6.0 (88.16)	63.8 (283.65)

TABLE H-25.2. Load combinations and resultant design (factored) loading for sliding resistance

Limit state load combinations	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
Service-I C1	31.3 (456.9)	3.5 (50.5)	31.3 (139.2)
Strength-I C1	30.7 (448.0)	7.9 (115.2)	73.1 (325.3)
Strength-I C2	24.8 (361.9)	8.0 (117.2)	40.4 (179.5)

γ_{EQ} assumed to be 0.0 in this example

H.4.3 Nominal and Allowable Bearing Resistances at the Limit States

The bearing resistances of rectangular footings with widths of 2.95 ft to 20.70 ft have been calculated for Strength-I limit states (C1 and C2 loads) as well as for the Service-I limit state taking an embedment depth equal to 4.5ft (note: the results are presented in the following sections as effective width). The footing length corresponds to the fixed length of the abutment and is kept fixed at 82.0 ft. The bearing resistances have been calculated according to Figure 10.6.3.1.2c-2 of AASHTO (2007) (Section 10) to account for the effect of the slope. The allowable bearing resistance for a Service-I limit state of allowable settlement of 1.5inches has been obtained using the AASHTO (2007) settlement calculation method (equation 10.6.2.4.2-1), Schmertmann (1978) and Hough (1959) settlement calculation methods.

The footing for the abutment is placed on a structural fill, which is to be compacted to result in an internal friction of 38°. The resulting average soil friction angle to the depth of influence for different footing sizes are obtained as within 38±0.5°, or 38°. Hence, the recommended resistance factor for the Strength-I loads (both C1 and C2 produce one-way inclined and one-way-eccentric loading) in/on *controlled soil condition* is taken as $\phi = 0.45$ for positive eccentricity for soil of $\phi_f = 38°$. This resistance factor coincides with that recommended in AASHTO (2007) specifications. No resistance factors exist in the current specifications for the service limit state, hence, the estimated load required to produce a settlement of 1.5in has been left unfactored.

H.4.4 Design Footing Width

From Table H-24, it can be seen that the maximum load eccentricity along the footing width is 1.0ft produced by the C1 load combination. Hence, the minimum foundation width admissible by the limiting eccentricity to B/6 is B = 6.0ft (=1.0ft×6). The maximum vertical Strength-I loading is 41.2kips/ft while the maximum Service-I loading is 31.3kips/ft (Table H-25.1).

Figures H-10 and H-11 present the unfactored and factored bearing resistances for different effective footing widths. The bearing load intensities (stresses) are plotted in the upper figures, whereas the lower ones present the bearing loads per unit length of the foundation to be compatible with the load presentation. The footing width refers to the effective width for both bearing capacity and settlement analyses. While the settlement analyses were carried out for the geometrical (full) foundation width, in the presentation of Figures H-10 and H-11, the width was transformed to be the effective width.

Figure H-10 shows the variation of the unfactored bearing capacities with effective footing width for different Strength limit states as well as the Service limit states estimated. The unfactored load combination C1 causes a larger load eccentricity and lower load inclination, while C2 causes a higher load inclination but a smaller load eccentricity with a smaller vertical load component (Table H-24). From the figure, it is seen that the difference of the bearing resistances for both these load combinations is, however, very small. Hence, Strength-I C1, which has higher vertical loading, has been considered. Figure H-11 shows the variation of factored bearing capacities with effective footing width for Service-I and Strength-I C1 loadings.

Applying the aforementioned vertical loads for the corresponding limit states in Figure H-11, the following results are obtained: all footing widths larger than the minimum admissible width satisfy the Strength-I as well as the Service-I load requirements.

The conclusions possible from Figures H-10 and H-11 are therefore:

1. Based on strength limit state alone, the following foundation size (full geometry) is sufficient: Strength limit state $\phi = 0.45$: 6.0ft×82.0 ft
2. Based on the unfactored service limit state (current AASHTO) also, all the footing sizes admissible by limiting eccentricity are safe: 6.0 ft×82.0 ft

These footing widths are smaller than the designed width of 9.84ft in GEC6, which uses Hough (1959) method for the settlement calculation. This discrepancy can arise due to the way the soil parameters are evaluated and considered settlement estimation.

For comparison, when the limiting eccentricity of B/4 is taken instead of B/6 used here, the minimum admissible footing dimension is 4.0ft×82.0ft. Then referring to Figure H-11, the minimum admissible footing size still governs the footing design. Hence, the choice of the limiting eccentricity totally governs the design in this example.

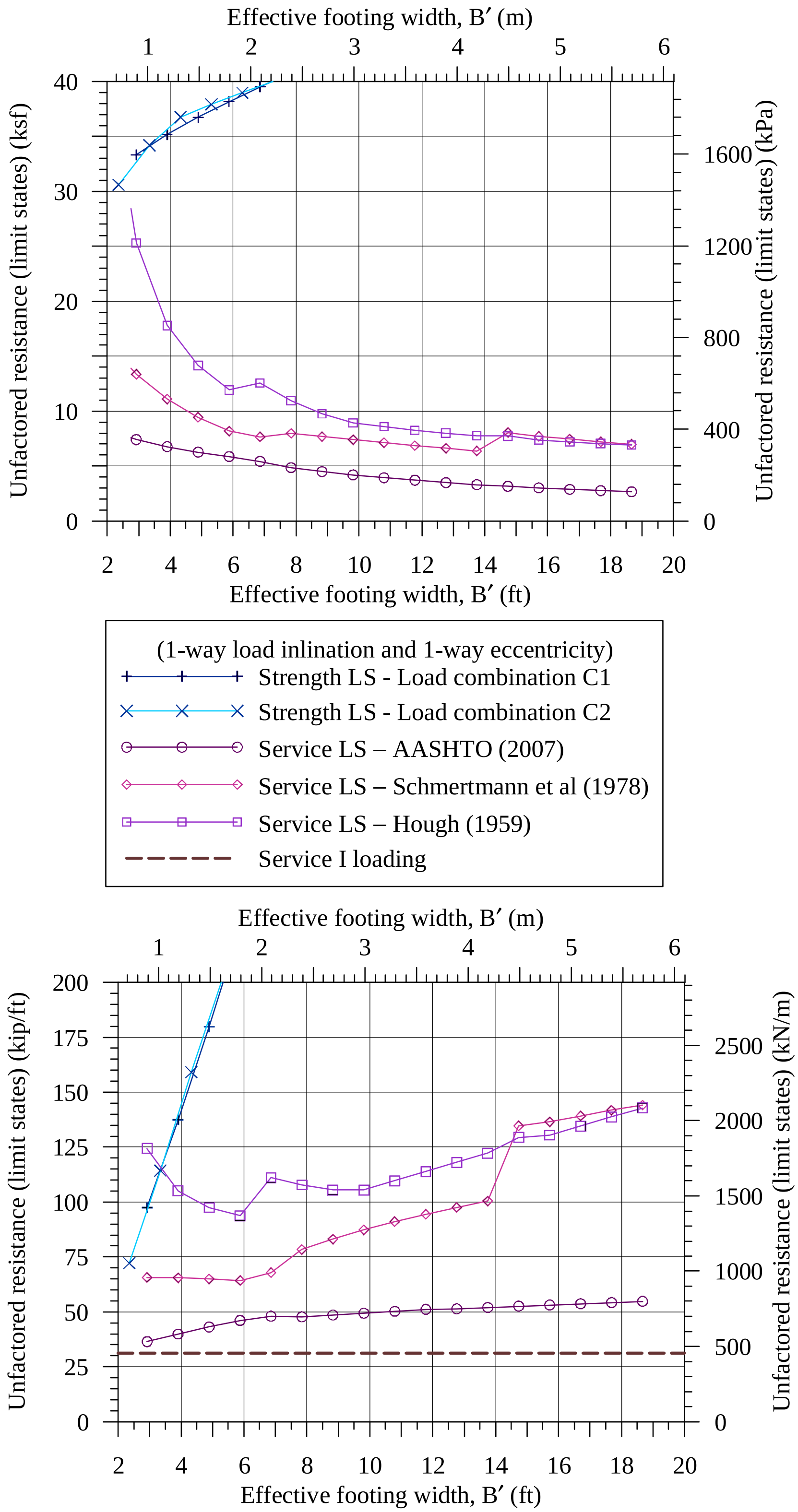

Figure H-10. Variation of unfactored bearing resistance for Strength-I and Service-I limit states with effective footing width for Example 4.

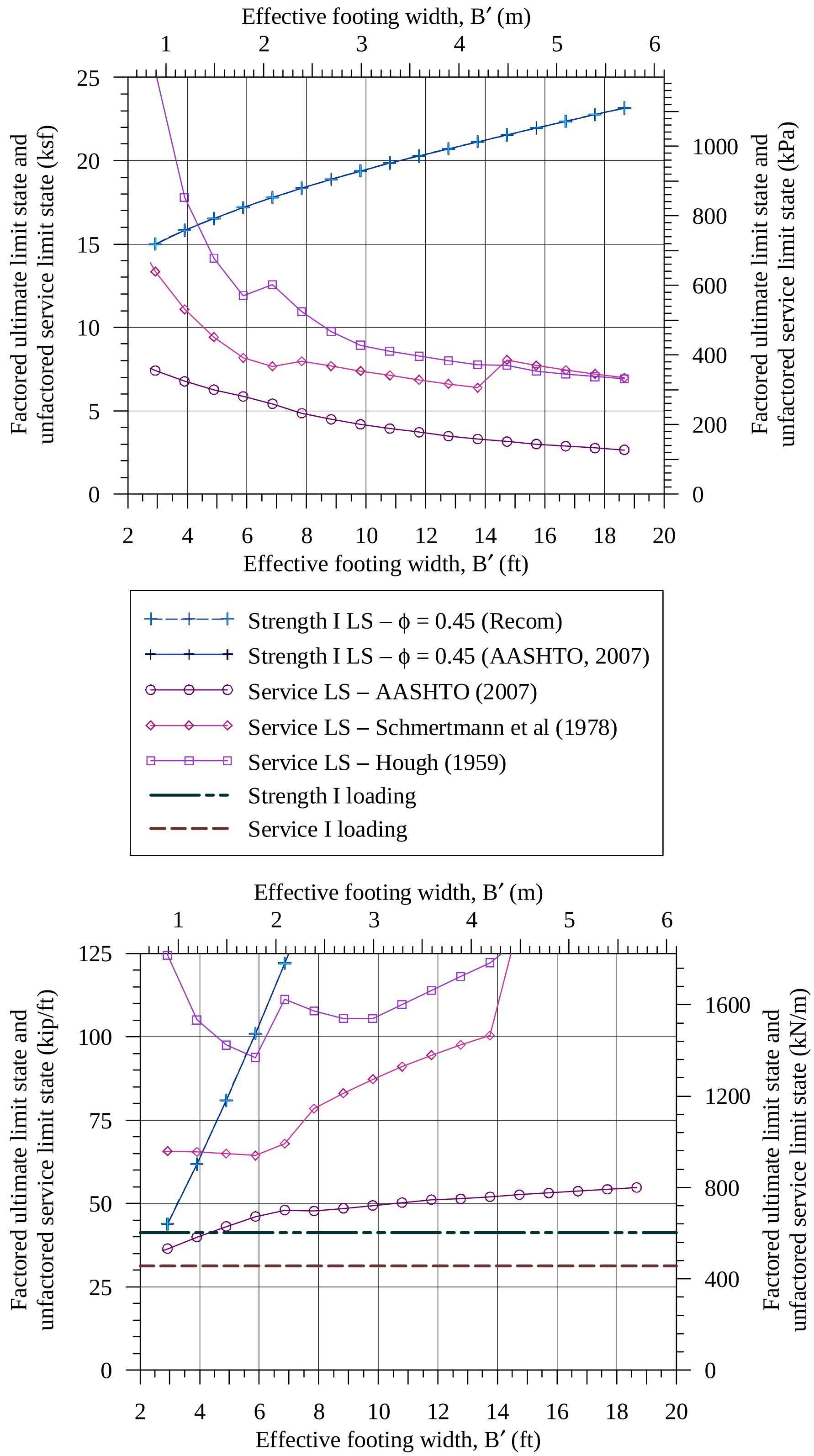

Figure H-11. Variation of factored bearing resistance for Strength-I C1 and unfactored Service-I limit states with effective footing width for Example 4.

H.4.5 Sliding Resistance

The footing is poured on site; the recommended resistance factor for cast in-place footings when at-rest earth pressure is acting is $\phi_\tau = 0.40$ and that when active earth pressure is acting is $\phi_\tau = 0.45$, while the current AASHTO (2007) specification recommends $\phi_\tau = 0.80$.

Here, the lateral earth load considered during the design process is related to the at-rest earth pressure. For the back-fill with soil friction angle $\phi_f = 38°$, the ratio of the lateral active earth pressure coefficient to the lateral at-rest earth pressure coefficient, $K_a / K_0 = 1/(1 + \sin\phi_f) = 1/1.616$, assuming Rankine's active earth pressure and at-rest earth pressure for normally consolidated cohessionless sand. Also, for $\phi_f = 38°$, the interfacial friction angle obtained from the recommended relation in this study is as follows:

$$\tan(\delta_s) = 0.91\tan(38) \quad \Rightarrow \; \delta_s = 35.4°$$

For the designed footing, the minimum factored vertical load and the corresponding lateral loads under Strength-I and Service-I loadings, thereby, the factored sliding resistance in each case are as follows.

Service-I LS:

At-rest earth pressure:

The minimum vertical load = 31.3kips/ft and the corresponding maximum total lateral load = 3.5kips/ft (Table H-25.2) when at-rest earth pressure is acting. Hence,

Factored sliding resistance $\phi_\tau F_{2E0} = 0.40 \times 31.3 \times \tan(35.4) = 8.90\text{kips/ft} > 3.5\text{kips/ft}$

Active earth pressure:

The corresponding lateral load involving active earth pressure is (Tables H-23 and H-25.2)

$$F_{2Ea} = 3.5 - (-7.60) + \left(\frac{1}{1.616} \times (-7.60)\right) = 6.40\text{kips/ft} .$$

Factored sliding resistance $\phi_\tau F_{2Ea} = 0.45 \times 31.3 \times \tan(35.4) = 10.01\text{kips/ft} > 6.40\text{kips/ft}$

Current AASHTO:

Factored sliding resistance $\phi_\tau F_{2\tau} = 0.80 \times 31.3 \times \tan(35.4)$
$= 17.8\text{kips/ft} > 3.5\text{kips/ft} > 6.40\text{kips/ft}$

Strength I LS:

At-rest earth pressure:

The minimum vertical load = 30.7kips/ft, and the corresponding maximum lateral load = 7.9kips/ft (Table H-25.2) when active earth pressure is acting. Hence,

Factored sliding resistance $\phi_\tau F_{2E0} = 0.40 \times 30.7 \times \tan(35.4) = 8.73\text{kips/ft} > 7.9\text{kips/ft}$

Active earth pressure:

The corresponding lateral load involving active earth pressure is (Tables H-23 and H-25.2)

$$F_{2Ea} = 7.9 - 1.5 \times (-7.60) + 1.5 \times \left(\frac{1}{1.616} \times (-7.60)\right) = 12.25\text{kips/ft} .$$

Factored sliding resistance $\phi_\tau F_{2Ea}$ = 0.45 × 30.7×tan(35.4) = 9.82kips/ft < 12.25kips/ft

Current AASHTO:

Factored sliding resistance $\phi_\tau F_{2\tau}$ = 0.80 × 30.7×tan(35.4)
= 17.45kips/ft > 7.9kips/ft > 12.25kips/ft

This shows that the sliding resistance factors recommended in this study result in footings larger than the designed footing for design against sliding failure due to lateral loads involving at-rest as well as active earth pressures, except when unfactored lateral active earth pressure is considered. Since the design of abutment footings against sliding is critical, further study on the application of the resistance factors for sliding is necessary.

H.4.6 Discussions and Conclusions

The limiting eccentricity governs the footing design in the example. From Figures H-10 and H-11 it is seen that for both the Strength-I and Service-I limit states, the footing dimension required is that of the minimum admissible size for limiting eccentricity. When the limiting eccentricity of B/6 is used, a footing of 6.0ft×82.0ft fulfills the requirements for Strength-I and Service-I limit states. When the limiting eccentricity of B/4 is used, a footing of 4.0ft×82.0ft fulfills the requirements for both the limit states. A footing of 6.0ft×82.0ft may be recommended for this example.

H.5 EXAMPLE 5: STUB SEAT-TYPE BRIDGE ABUTMENT ON STRUCTURAL FILL – GEC6-EXAMPLE 3

H.5.1 Subsurface Conditions

The subsurface conditions and the abutment geometry given in Example C2 of FHWA Geotechnical Engineering Circular No. 6 (GEC6), Appendix C (Kimmerling, 2002) shown in Figure H-12, and the soil parameters are summarized in Table H-26. The groundwater table is located 51.9 ft (15.81 m) below the surface of the proposed bridge approach elevation. The abutment is placed on structural fill of well graded silty sand and gravel which is 15.0ft (4.57m) deep below the footing base. The fill forms a slope with a grade of 2H:1V at a distance of 1.5 times the width of the proposed footing from the slope.

The soil friction angles are calculated using the correlation with SPT blow counts proposed by Peck, Hanson and Thornburn (PHT) as modified by Kulhawy and Mayne (1990). As also mentioned in the Example 1 presented here, this calculation of soil friction angle is compatible with the methodology used in developing the resistance factors. The footing is poured on site, hence, the base friction angle δ_s is assumed to be equal to the soil friction angle ϕ_f.

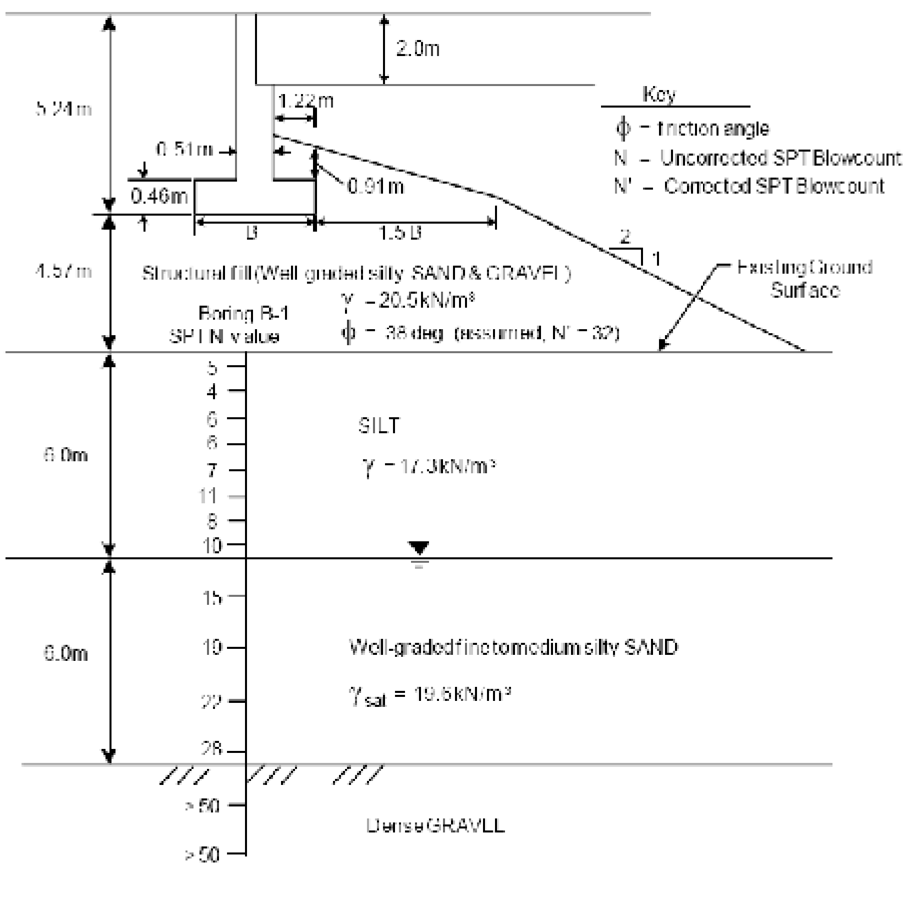

Figure H-12. Geometry and soil conditions of stub seat-type abutment for Example 5 (1m»3.3ft).

TABLE H-26. Soil parameters

Layer #	Thickness below footing ft (m)	Soil type	g psf (kN/m^3)	f_f [°]
1	15.0 (4.6)	Sand and Gravel (fill)	130.6 (20.50)	38.00
2	19.7 (6.0)	Silt	110.2 (17.30)	30.11
3	19.7 (6.0)	silty Sand below groundwater	124.9 (19.60)	31.54
4	∞	Gravel, dense	-	

H.5.2 Loads, Load Components and Limit States

The loadings from the bridge structure at the footing base are given for per unit length of the foundation in Table H-27, the notations and directions of which correspond to those presented in Figure 120 of Chapter 5. The moment M_3 refers to the moment at the center of the footing and counter-clockwise moments are taken positive. Table H-28 summarizes the investigated load combinations and the resultant characteristic loading as well as the load eccentricity for different load combinations. The design load components required for the stability analysis are the factored characteristic loadings with load factors presented in Tables H-4.1 and H-4.2 (according to AASHTO specifications, 2007) for the bearing and the sliding resistances, respectively. Only the Service-I and Strength-I load conditions are checked for this example. These design loadings are presented in Tables H-29.1 and H-29.2, respectively.

TABLE H-27. Loading at footing base

Load components	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
dead load of components (DL)	14.35 (209.32)	0.00	-9.88 (-43.95)
vehicular live load (LL)	4.22 (61.52)	0.00	-2.90 (-12.91)
dead load on wearing surfaces (DW)	1.22 (17.84)	0.00	-0.84 (-3.74)
shear loads from bearing pads (V)	0.00	2.87 (41.88)	-30.50 (-135.68)
active earth pressure from soil fill (E)	0.00	4.79 (69.86)	-27.43 (-122.02)
earth pressure from live loads (EL)	0.00	1.10 (16.00)	-9.42 (-41.92)
dead weight of stem (EG)	3.93 (57.29)	0.00	-1.61 (-7.16)
dead weight of footing (EG)	2.37 (34.59)	0.00	0.00
weight of soil over toe (EG)	Neglected		
weight of soil over heal (EG)	9.44 (137.72)	0.00	26.78 (119.13)

TABLE H-28. Load combinations and resultant characteristic (unfactored) loading

Load combinations	Load components	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)	F_2/F_1	$e_2 = M_3/F_1$ ft (m)
C1:	EG+DL+DW+E	31.3 (456.8)	4.8 (69.9)	13.0 (57.8)	0.153	0.413 (0.126)
C2:	EG+DL+DW+E+V	31.3 (456.8)	7.7 (111.7)	43.5 (193.4)	0.245	1.387 (0.423)
C3:	EG+DL+DW+E+LL+EL	35.5 (518.3)	5.9 (85.9)	25.3 (112.6)	0.166	0.712 (0.217)
C4:	EG+DL+DW+E+LL+EL+V	35.5 (518.3)	8.8 (127.7)	55.8 (248.3)	0.246	1.571 (0.479)

TABLE H-29.1. Load combinations and resultant design (factored) loading for bearing resistance

Load combinations	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
Service-I C2	31.3 (456.8)	7.7 (111.7)	43.5 (193.4)
Strength-I C1	39.4 (575.4)	7.2 (104.8)	23.3 (103.6)
Strength-I C2	39.4 (575.4)	10.6 (155.0)	59.9 (266.4)
Strength-V C3	42.9 (625.7)	9.1 (132.8)	51.2 (227.6)
Strength-V C4	42.9 (625.7)	12.5 (183.0)	87.8 (390.4)

TABLE H-29.2. Load combinations and resultant design (factored) loading for sliding resistance

Load Combinations	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
Service-I C2	31.3 (456.8)	7.7 (111.7)	43.5 (193.4)
Strength-I C1	27.9 (406.6)	7.2 (104.8)	27.9 (124.2)
Strength-I C2	27.9 (406.6)	10.6 (155.0)	64.5 (287.1)
Strength-V C3	36.8 (537.2)	9.1 (132.8)	47.0 (209.0)
Strength-V C4	36.8 (537.2)	12.5 (183.0)	83.6 (371.8)

H.5.3 Nominal and Allowable Bearing Resistances at Limit States

The bearing resistances of rectangular footings with widths of 2.95 ft to 20.70 ft have been calculated for Strength-I limit states for the C2 load combination, as well as for the Service-I limit state taking embedment depth equal to 4.5ft (note: the results are presented in the following sections as effective width). The footing length corresponds to the fixed length of the abutment and is kept fixed at 82.0 ft. The bearing resistances have been calculated according to Figure 10.6.3.1.2c-2 of AASHTO (2007) (Section 10) to account for the effect of the slope. The allowable bearing resistance for a Service-I limit state of an allowable settlement of 1.5inches has been obtained using the AASHTO (2007) settlement calculation method (equation 10.6.2.4.2-1), Schmertmann (1978) and Hough (1959) settlement calculation methods

The footing for the abutment is placed on gravel borrow fill, filled to a shallow depth. Table H-30 shows the variation of average soil friction angle of the soil strata below footing base, along with whether the subsurface is considered *controlled* or *natural* soil condition and the recommended resistance factors for bearing resistance. The soil condition has been taken as *natural*, if less than 50% of the influence depth below the footing base is gravel borrow fill, i.e., more than 50% of this is natural strata. The recommended resistance factors considered thus vary according to the average friction angle as well as the soil condition for different footing width, which ranges from 0.45 for smaller footings and 0.35 for larger footings. The current AASHTO (2007) specification recommends the use of ϕ = 0.45 for all footing sizes. No resistance factors exist in the current specifications for the service limit state, hence, the estimated load required to produce a settlement of 1.5in has been left unfactored.

Table H-30 Variation of average f_f and thereby the recommended resistance factors according to the footing width for the given subsurface conditions

B (ft)	Average f_f (°)	Soil Condition*	Recommended f	B (ft)	Average f_f (°)	Soil Condition*	Recommended f
2.95	38.00	Controlled	0.45	12.80	35.18	Controlled	0.40
3.94	38.00	Controlled	0.45	13.78	34.86	Controlled	0.40
4.92	38.00	Controlled	0.45	14.76	34.58	Controlled	0.40
5.91	38.00	Controlled	0.45	15.75	34.33	Natural	0.35
6.89	38.00	Controlled	0.45	16.73	34.11	Natural	0.35
7.87	37.70	Controlled	0.45	17.72	33.94	Natural	0.35
8.86	37.02	Controlled	0.45	18.70	33.85	Natural	0.35
9.84	36.45	Controlled	0.40	19.68	33.77	Natural	0.35
10.83	35.96	Controlled	0.40	20.67	33.69	Natural	0.35
11.81	35.54	Controlled	0.40				

* Soil condition taken as Natural when more than 50% of the subsurface strata within the influence depth below the footing base incorporates natural strata.

H.5.4 Design Footing Width

The largest load eccentricity caused by the load combinations related to Service-I and Strength-I loads, according to the characteristic loadings listed in Table H-28, is 1.39 ft from C2 combination (C3 and C4 combinations are applicable to Strength-V only, so, not considered at present). Hence, the minimum admissible footing due to limited eccentricity is of width B = 8.35ft (=1.39ft×6) considering the limiting eccentricity as B/6. The maximum vertical loading in Strength-I is 39.4kips/ft while the maximum in Service-I is 31.3kips/ft.

Figures H-13 and H-14 present the unfactored and factored bearing resistances for different effective footing widths. The bearing load intensities (stresses) are plotted in the upper figures, whereas the lower ones present the bearing loads per unit length of the foundation to be compatible with the load presentation. The footing width refers to the effective width for both bearing capacity and settlement analyses. While the settlement analyses were carried out for the geometrical (full) foundation width, in the presentation of Figures H-13 and H-14, the widths were transformed to be the effective widths.

Figure H-13 shows the variation of the unfactored bearing capacities with effective footing width for different Strength limit states as well as the Service limit state. The unfactored load combination C2 causes a larger load eccentricity as well as a larger load inclination compared to the combination C1 (Table H-28). Figure H-14 shows the variation of factored bearing capacities with effective footing width for Strength-I limit state for C2 load combination and unfactored Service-I limit state. It is to be noted that while the AASHTO (2007) method leads to lower allowable loads for 1.5inch settlement as the footing size increases, the allowable pressure (stress) decreases with the increase in the footing width. On the other hand, Schmertmann (1978) and Hough (1959) methods show an overall increase in the allowable pressures with an increase in the footing size. This difference is attributed by the fact that in AASHTO (2007) method the soil elastic modulus has been taken as the weighted average of all the soil strata to the influence depth from the footing base, whereas Schmertmann (1978) method estimate the settlement caused by each soil stratum using the average modulus for each stratum and Hough (1959) method uses bearing capacity index C′ based on empirical curves for different soil types.

Applying the aforementioned vertical loads for the corresponding limit states in Figure H-14, the following results are obtained: (a) the minimum footing width (full size) required for the Strength-I limit state is B = 6.4ft, which is smaller than the minimum admissible footing of width B = 8.35ft, and (b) the minimum effective footing widths required for Service-I loadings are smaller than 14.8ft when AASHO method of settlement estimation is used, while using Schmertmann (1978) and Hough (1959) methods result in a footing of minimum admissible width is sufficient.

The conclusions possible from Figures H-13 and H-14 are therefore:

1. Based on the strength limit state alone, the minimum admissible footing size (full geometry) is required:
    - Strength limit state ϕ = 0.45 to 0.35: 8.35ft×82.0 ft
    - Strength limit state ϕ = 0.45 (current AASHTO): 8.35ft×82.0 ft
2. Based on the unfactored serviceability limit state (current AASHTO): 8.35ft×82.0 ft is recommended

The footing dimensions obtained here for the factored service limit state provides a footing of a smaller dimension compared to 10.5ft obtained in GEC6. The discrepancy in the widths can arise from the differences in the way different soil parameters are considered and the settlement calculation methods used.

Further, for comparison, when the limiting eccentricity of B/4 is taken instead of B/6 used here, the minimum admissible footing dimension is 5.6ft×82.0ft. In this case, the Strength-I limit state governs the design as the Service-I limit state requires the minimum admissible footing size. In this sense, the choice of the limiting eccentricity governs which limit state, either Strength-I or Service-I, dominates the footing design in this example.

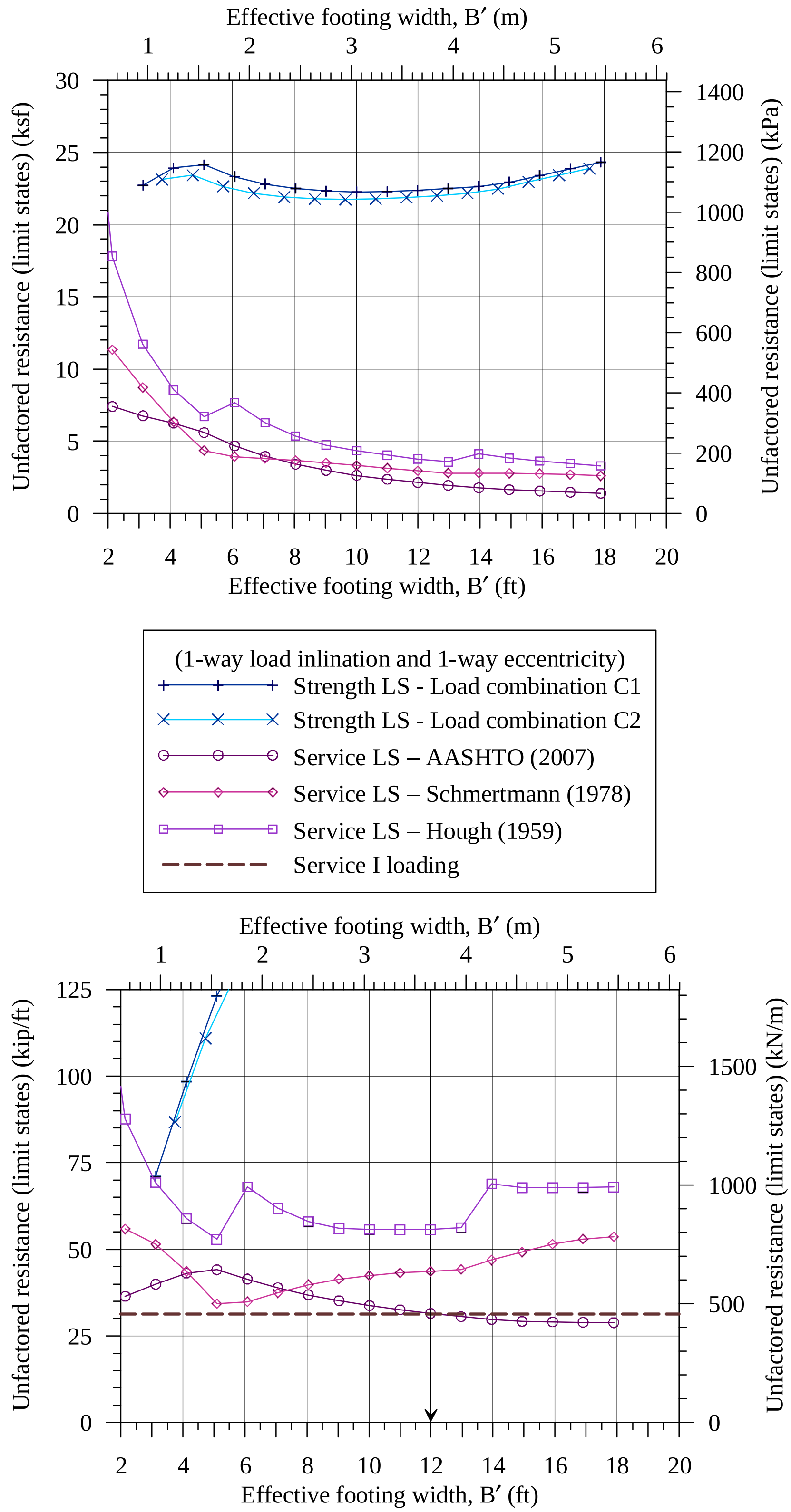

Figure H-13. Variation of unfactored bearing resistance for Strength-I and Service-I limit states with effective footing width for Example 5.

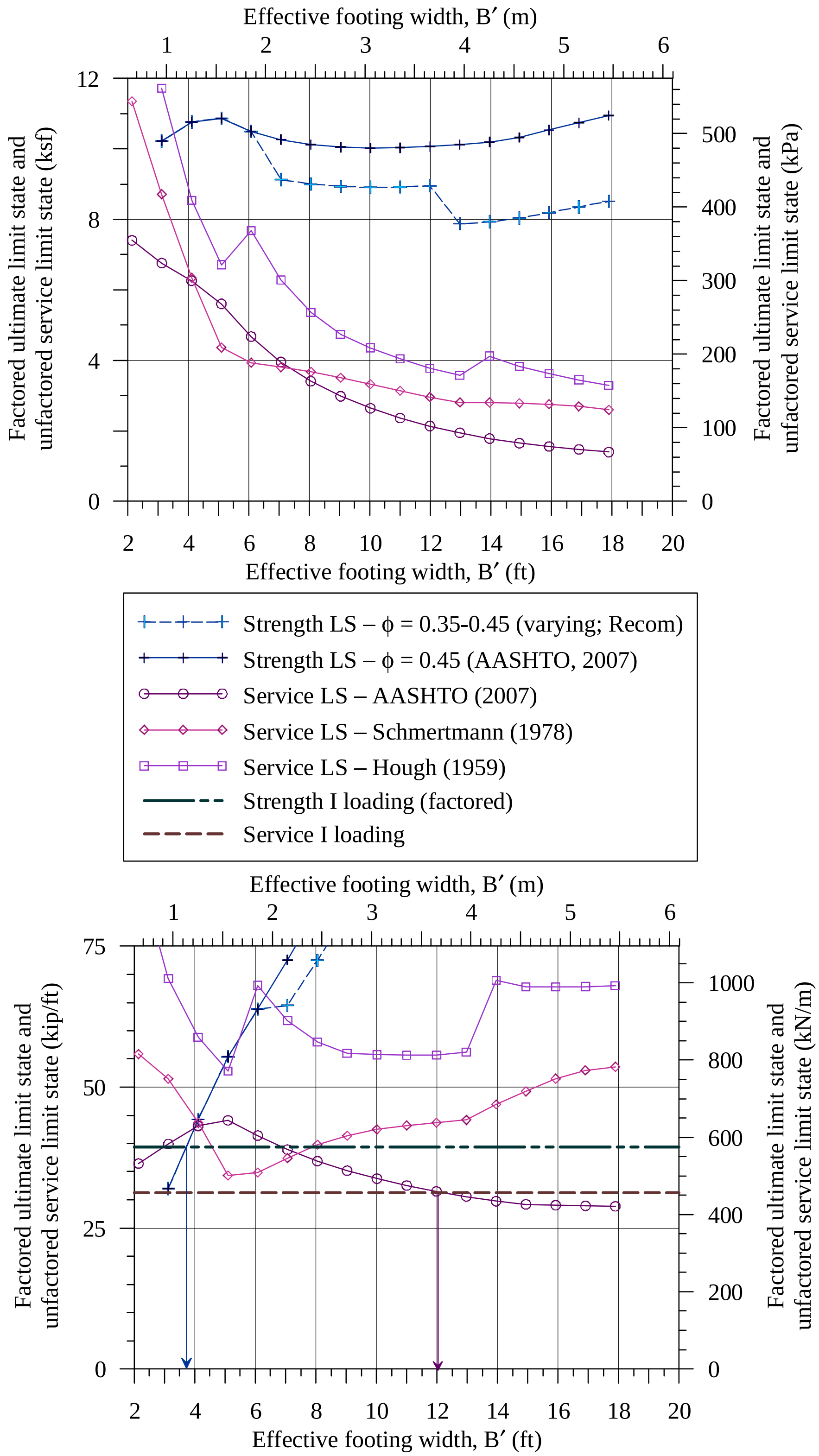

Figure H-14. Variation of factored bearing resistance for Strength-I C2 and unfactored Service-I limit states with effective footing width for Example 5.

H.5.5 Sliding Resistance

The footing is cast in-place; the recommended resistance factor for cast in-place footings when at-rest earth pressure is acting is $\phi_\tau = 0.40$ and that when active earth pressure is acting is $\phi_\tau = 0.45$, while the current AASHTO (2007) specification recommends $\phi_\tau = 0.80$.

Here, the lateral earth load considered during the design process is related to the active earth pressure. For the back-fill with soil friction angle $\phi_f = 38°$, the ratio of the lateral at-rest earth pressure coefficient to the lateral active earth pressure coefficient, $K_0 / K_a = (1 + \sin\phi_f) = 1.616$, assuming Rankine's active earth pressure and at-rest earth pressure for normally consolidated cohessionless sand.

Also, for $\phi_f = 38°$, the interfacial friction angle obtained from the recommended relation in this study is as follows, which has been used only for the purpose of comparison:

$$\tan(\delta_s) = 0.91\tan(38) \quad \Rightarrow \quad \delta_s = 35.4°$$

For the designed footing, the minimum factored vertical load and the corresponding lateral loads under Strength-I and Service-I loadings, thereby, the factored sliding resistance in each case are as follows.

Service-I LS:

At-rest earth pressure:

The minimum vertical load = 31.3kips/ft and the corresponding maximum total lateral load = 7.7kips/ft (Table H-29.2) when active earth pressure is acting. Hence, the corresponding maximum total lateral load when at-rest earth pressure is acting is (refer to Tables H-27 and H-29.2):

$$F_{2E0} = 7.7 - 4.79 + (1.616 \times 4.79) = 10.65\text{kips/ft}$$

Factored sliding resistance $\phi_\tau F_{2E0} = 0.40 \times 31.3 \times \tan(35.4) = 8.90\text{kips/ft} < 10.65\text{kips/ft}$

Active earth pressure:

Factored sliding resistance $\phi_\tau F_{2Ea} = 0.45 \times 31.3 \times \tan(35.4) = 10.01\text{kips/ft} > 7.70\text{kips/ft}$

Current AASHTO:

Factored sliding resistance $\phi_\tau F_{2\tau} = 0.80 \times 31.3 \times \tan(35.4)$
$= 17.8\text{kips/ft} > 10.65\text{kips/ft} > 7.70\text{kips/ft}$

Strength I LS:

At-rest earth pressure:

The minimum vertical load = 27.9kips/ft, and the corresponding maximum lateral load = 10.6kips/ft (Table H-29.2) when active earth pressure is acting. Hence, the corresponding maximum total lateral load when at-rest earth pressure is acting is (refer to Tables H-27 and H-29.2):

$$F_{2E0} = 10.6 - 1.5 \times 4.79 + 1.5 \times (1.616 \times 4.79) = 15.03\text{kips/ft}$$

Factored sliding resistance $\phi_\tau F_{2E0} = 0.40 \times 27.9 \times \tan(35.4) = 7.93\text{kips/ft} < 15.03\text{kips/ft}$

Active earth pressure:

Factored sliding resistance $\phi_\tau F_{2Ea} = 0.45 \times 27.9 \times \tan(35.4) = 8.92\text{kips/ft} < 10.60\text{kips/ft}$

Current AASHTO:

Factored sliding resistance $\phi_\tau F_{2\tau} = 0.80 \times 27.9 \times \tan(35.4)$

$= 15.86\text{kips/ft} > 15.03\text{kips/ft} > 10.60\text{kips/ft}$

This shows that the sliding resistance factors recommended in this study result in footings larger than the designed footing for design against sliding failure due to lateral loads involving at-rest as well as active earth pressures, except when unfactored lateral active earth pressure is considered. Since the design of abutment footings against sliding is critical, further study on the application of the resistance factors for sliding is necessary.

H.5.6 Discussions and Conclusions

From Figures H-13 and H-14, it is seen that the limiting eccentricity governs the footing design in this example when the limiting eccentricity is chosen as B/6. Further, within the range of the minimum admissible footing width, the recommended resistance factor is essentially $\phi = 0.45$, and for footing larger than this should be taken as $\phi = 0.40$ (Table H-30). A footing of size 8.35ft×82.0ft sufficiently fulfills the requirements for Strength-I and Service-I limit states.

If, however, the limiting eccentricity is chosen as B/4, the minimum footing dimension required for Strength-I limit state is 6.4ft×82.0ft, whereas, that required for the Service-I limit state is equal to the minimum admissible footing size of 5.6ft×82.0ft. Hence Strength-I limit stated governs the design if limiting eccentricity of B/4 is considered.

A footing of 8.35ft×82.0ft is recommended for design.

H.6 EXAMPLE 6: FULL HEIGHT BRIDGE ABUTMENT ON NATURAL SOIL - GEC6-EXAMPLE 4

H.6.1 Subsurface Conditions

The subsurface conditions and the abutment geometry given in Example B4 of FHWA Geotechnical Engineering Circular No. 6 (GEC6), Appendix C (Kimmerling, 2002) shown in Figure H-15, and the soil parameters are summarized in Table H-31. The groundwater table is located 14.75ft (4.5 m) below the surface of the ground surface. The abutment is placed in the natural soil of well graded sand of thickness 19.7 ft (6.0 m), which is underlain by shale. This is a special example in which the failure plane is assumed to be limited to the sand layer for nominal bearing resistance analysis, as the consideration of the shale layer would require a different method for which the nominal bearing resistance factor has not been calibrated in the current research study, hence, ignored. Further, the depth of influence zone is assumed to be limited to the sand layer (and the shale layer considered incompressible) for the allowable bearing resistance analysis.

The soil friction angles are calculated using the correlation with SPT blow counts proposed by Peck, Hanson and Thornburn (PHT) as modified by Kulhawy and Mayne (1990). As also mentioned in the Example 1 presented here, this calculation of soil friction angle is compatible with the methodology used in developing the resistance factors. The footing is poured on site, hence, the base friction angle δ_s is assumed to be equal to the soil friction angle ϕ_f.

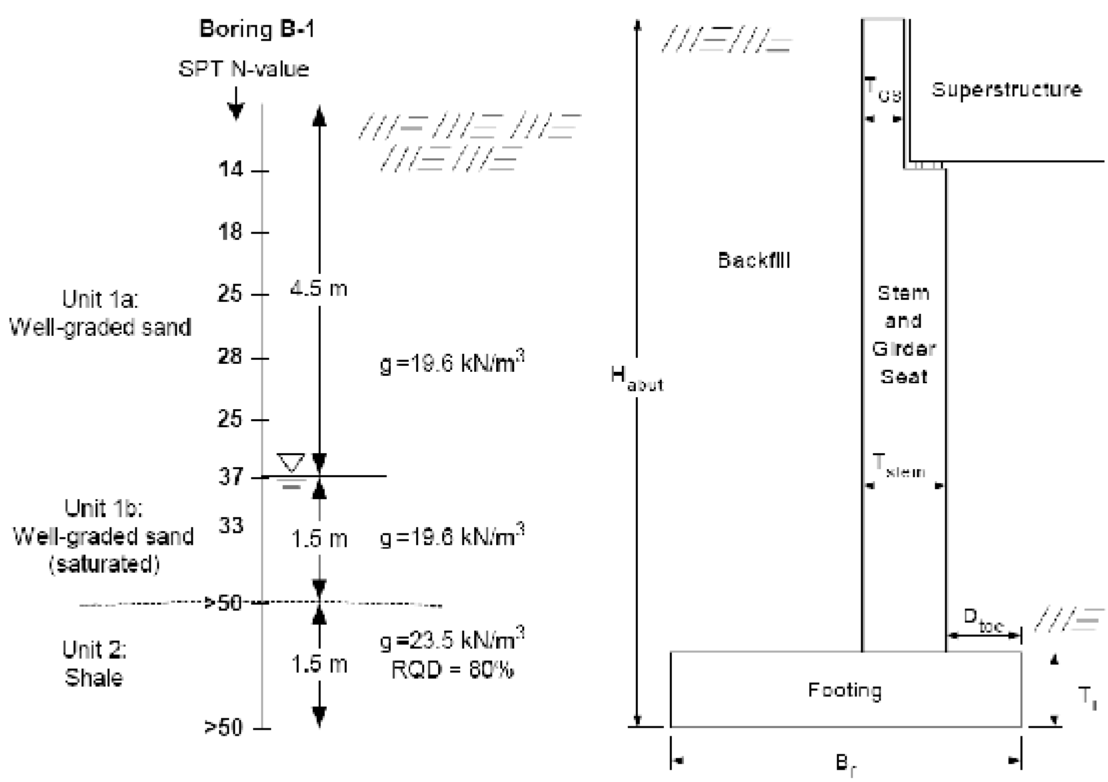

Figure H-15. Geometry and soil conditions of full height bridge abutment – Example 6

TABLE H-31. Soil parameters

Layer #	Thickness ft (m)	Soil type	g kip/ft^3 (kN/m^3)	f_f [°]
1a	14.8 (4.5)	Sand above groundwater	0.12 (19.60)	36.6
1b	4.9 (1.5)	Sand below groundwater	0.12 (19.60)	37.0
2	∞	Shale	0.15 (23.50)	-

H.6.2 Loads, Load Combinations and Limit States

The loadings from the bridge structure at the footing base are given for per unit length of the foundation in Table H-32, the notations and directions of which correspond to those presented in Figure 120 of Chapter 5. The moment M_3 refers to the moment at the center of the footing and counter-clockwise moments are taken positive. Table H-33 summarizes the investigated load combinations and the resultant characteristic loading as well as the load eccentricity for different load combinations. Note that the load combination C1 results in higher load inclination and lower load eccentricity as compared to C2 and vice-versa. The design load components required for the stability analysis are the factored characteristic loadings with load factors presented in Tables H-4.1 and H-4.2 (according to AASHTO specifications, 2007) for the bearing and the sliding resistances, respectively. Only the Service-I and Strength-I load conditions are checked for this example. These design loadings are presented in Tables H-34.1 and H-34.2, respectively.

TABLE H-32. Loading at footing base for Example 6

Load Component	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
dead load (D)	15.6 (227.0)	2.9 (41.9)	-120.1 (-534.2)
live load (L)	4.2 (61.6)	0.0	-15.5 (-68.8)
active earth pressure from soil fill (E)	0.0	11.5 (168.4)	-107.3 (-477.2)
dead weight of stem	9.3 (136.3)	0.0	-26.6 (-118.1)
dead weight of footing (EG)	7.5 (110.0)	0.0	0.0
weight of soil over toe (EG)	1.0 (14.5)	0.0	-6.5 (-28.8)
weight of soil over heal (EG)	23.1 (337.3)	0.0	84.7 (376.6)

TABLE H-33. Load combinations and resultant characteristic (unfactored) loading

Load combinations	Load components	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)	F_2/F_1	$e_2 = M_3/F_1$ ft (m)
C1	EG+D+E	56.5 (825.0)	14.4 (210.3)	175.7 (781.7)	0.255	3.109 (0.948)
C2	EG+D+E+L	60.8 (886.6)	14.4 (210.3)	191.2 (850.5)	0.237	3.146 (0.959)

TABLE H-34.1. Load combinations and resultant design (factored) loading for bearing resistance

Load Combinations	F_1 kips (kN)	F_2 kips (kN)	M_3 kip-ft (kNm)
Service-I C2: EG+D+E+L	60.8 (886.6)	14.4 (210.3)	191.2 (850.5)
Strength-I C1: EG+D+E	70.7 (1031.3)	20.9 (305.0)	246.5 (1096.5)
Strength-I C2: EG+D+E+L	78.1 (1139.1)	20.9 (305.0)	273.5 (1216.9)

TABLE H-34.2. Load combinations and resultant design (factored) loading for sliding resistance

Load Combinations	F_1 kips (kN)	F_2 kips (kN)	M_3 kip-ft (kNm)
Service-I C2: EG+D+E+L	60.8 (886.6)	14.4 (210.3)	191.2 (850.5)
Strength-I C1: EG+D+E	50.9 (742.5)	19.9 (290.4)	222.5 (989.9)
Strength-I C2: EG+D+E+L	58.3 (850.3)	19.9 (290.4)	249.6 (1110.3)

H.6.3 Nominal and Allowable Bearing Resistances at the Limit States

The bearing resistances of rectangular footings with widths of 2.95ft to 20.70ft have been calculated for Strength-I limit states for the C1 and C2 load combinations, as well as for Service-I limit state taking an embedment depth equal to 4.9ft (note: the results are presented in the following sections as effective width). The footing length corresponds to the fixed length of the abutment of 82.0ft. The bearing resistances have been calculated according to AASHTO (2007) (equation 10.6.3.1.2) and Equations 95 through 99 in the Final Draft Report. The allowable bearing resistance for a Service-I limit state of allowable settlement of 1.5inches has been obtained using the AASHTO (2007) settlement calculation method (equation 10.6.2.4.2-1), Schmertmann (1978) and Hough (1959) settlement calculation methods.

The footing for the abutment is placed on the natural soil stratum of well-graded sand. Table H-35 shows the variation of the soil friction angle of the soil strata below the footing base as well as the recommended resistance factors for bearing resistance according to the footing width chosen. The average soil friction angle has been calculated as the average of the soil to the influence depth, taken as 2B below the footing base. The resistance factors are for natural soil conditions, and their values change from 0.35 to 0.40 as the footing size increases. No resistance factors exist in the current specifications for the service limit state, hence, the estimated load required to produce a settlement of 1.5in has been left unfactored. For the shale layer, the Young's modulus has been taken as 204,480.0ksf according to Table C10.4.6.5-1 in AASHTO (2007) specifications for settlement evaluations.

Table H-35 Average soil friction angle and recommended resistance factor variation according to the footing size (thereby the influence depth) in natural soil condition

B (ft)	Average f_f (deg)	Recommended f	B (ft)	Average f_f (deg)	Recommended f
2.95	36.50	0.35	12.80	36.75	0.40
3.94	36.50	0.35	13.78	36.75	0.40
4.92	36.50	0.35	14.76	36.75	0.40
5.91	36.56	0.40	15.75	36.75	0.40
6.89	36.61	0.40	16.73	36.75	0.40
7.87	36.75	0.40	17.72	36.75	0.40
8.86	36.75	0.40	18.70	36.75	0.40
9.84	36.75	0.40	19.68	36.75	0.40
10.83	36.75	0.40	20.67	36.75	0.40
11.81	36.75	0.40			

H.6.4 Design Footing Width

The largest load eccentricity caused by the load combinations related to Service-I and Strength-I loads, according to the characteristic loadings listed in Table H-33, is 3.15ft from C2 combination. Hence, the minimum admissible footing due to limited eccentricity is of width B_{min} = 18.9ft (=3.15ft×6) considering the limiting eccentricity as B/6. The maximum vertical loading in Strength-I C1 is 50.9kips/ft and Strength-I C2 is 58.3kips/ft while that in Service-I is 60.8kips/ft.

Figures H-16 and H-17 present the unfactored and factored bearing resistances for different effective footing widths. The bearing load intensities (stresses) are plotted in the upper figures, whereas the lower ones present the bearing loads per unit length of the foundation to be compatible with the load presentation. The footing width refers to the effective width for both bearing capacity and settlement analyses. While the settlement analyses were carried out for the geometrical (full) foundation width, in the presentation of Figures H-16 and H-17, the widths were transformed to be the effective widths.

Figure H-16 shows the variation of the unfactored bearing capacities with effective footing width for the two Strength limit states as well as for the Service limit state. The unfactored load combination C2 causes a larger load eccentricity but a smaller load inclination compared to the combination C1 (Table H-33). Because the effect of the load inclination on the bearing resistance is greater, the load combination C2 provides a higher unfactored resistance than the load combination C1. Figure H-17 shows the variation of factored bearing capacities with effective footing width for Strength-I C2 loading and unfactored bearing capacities for Service-I loading.

Applying the aforementioned vertical loads for the corresponding limit states in Figures H-17, the following results are obtained: (a) the minimum footing width (full size) required for the Strength-I limit state is B = 13.6ft, which is smaller than the minimum admissible footing of width B_{min} = 18.9ft, and (b) the minimum effective footing widths required for Service-I loadings are B = 13.55ft ($<B_{min}$) when AASHTO (2007) method of settlement estimation is used, while using Schmertmann (1978) and Hough (1959) methods result in a footing of minimum admissible width.

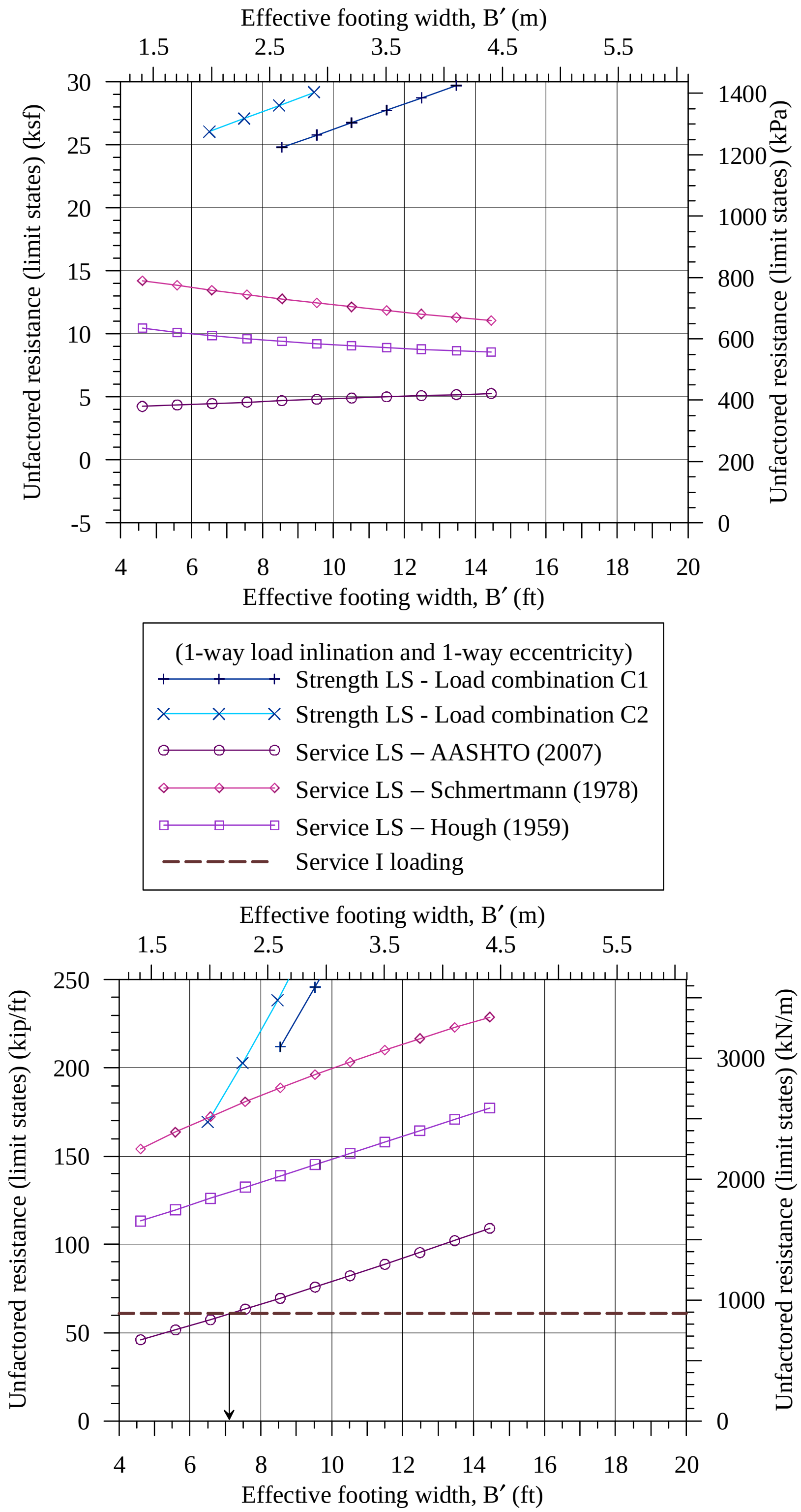

Figure H-16. Variation of unfactored bearing resistance for Strength-I and Service-I limit states with effective footing width for Example 6

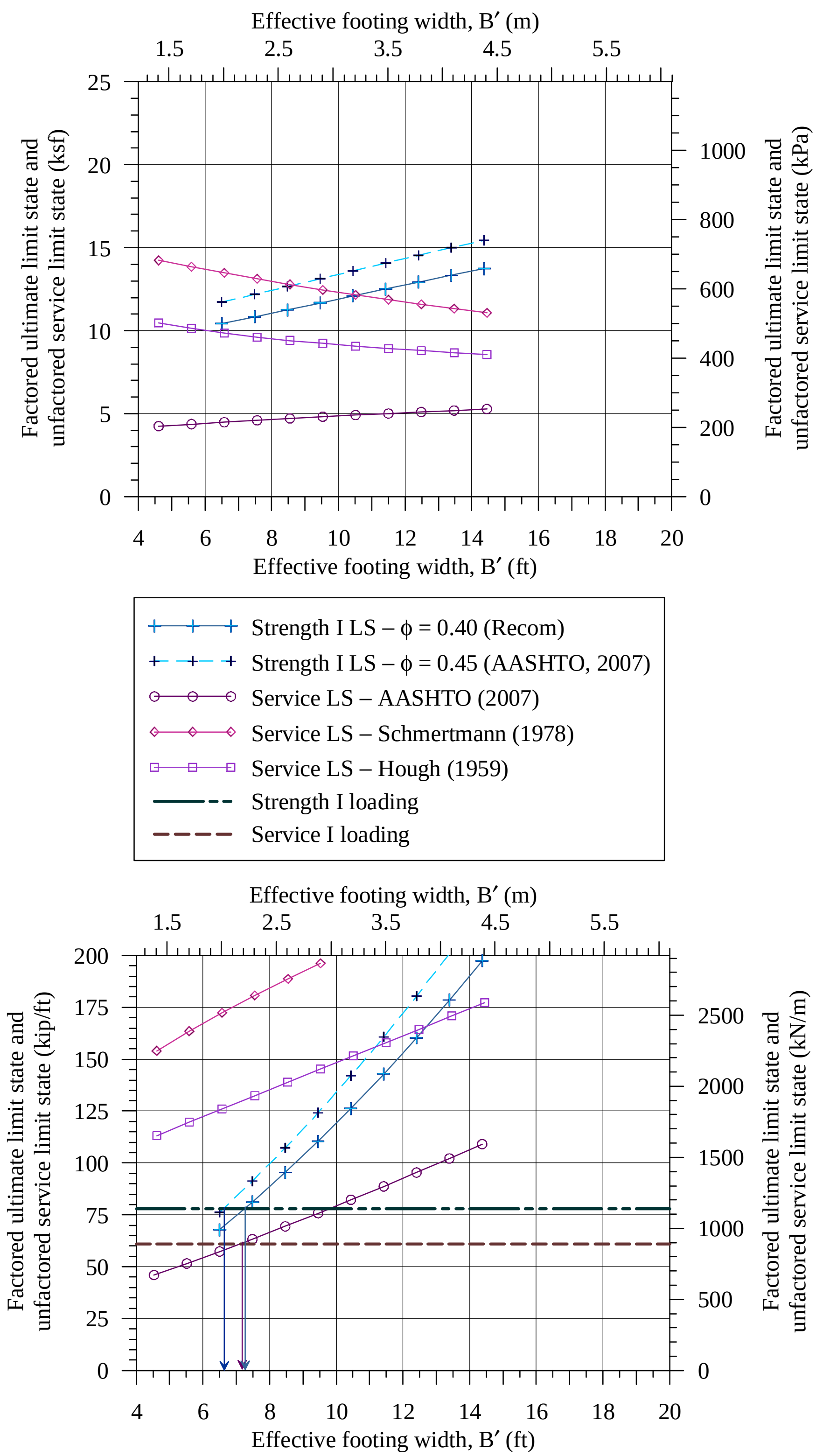

Figure H-17. Variation of factored bearing resistance for Strength-I C2 and unfactored Service-I limit states with effective footing width for Example 6

The conclusions possible from Figures H-16 and H-17 are therefore:

1. Based on the strength limit state alone, the minimum admissible footing size (full geometry is required:
    - Strength limit state ϕ = 0.35 to 0.40: 18.9 ft×82.0 ft
    - Strength limit state ϕ = 0.45 (current AASHTO): 18.9 ft×82.0 ft
2. Based on the unfactored serviceability limit state (current AASHTO), 18.9 ft×82.0 ft is required.

The footing dimensions obtained above for the factored service limit state provides a footing larger than that obtained in GEC6 (of width 17.1 ft). In this example, it is seen that the minimum allowable footing size decided according to the limiting eccentricity governs the design when e_B/B is taken as 1/6. In GEC6, the limiting eccentricity is taken as B/4, i.e. the minimum admissible width is 12.6ft. When the limiting e_B/B is taken as ¼, the footing dimension obtained for factored strength limit state is 13.6ft and those obtained for unfactored serviceability limit state is 13.55ft when AASHTO (2007) method is used and 12.6ft when Schmertmann (1978) and Hough (1959) methods are used. Therefore, the Strength-I limit sate governs the design when limiting eccentricity of B/4 is used. Though in GEC6, Hough (1959) method of settlement estimation has been used with limiting eccentricity of B/4, the discrepancy in the widths can arise from the differences in the way different soil parameters are considered and the settlement calculation methods used.

H.6.5 Sliding Resistance

The footing is cast in-place; the recommended resistance factor for cast in-place footings when at-rest earth pressure is acting is $\phi_\tau = 0.40$ and that when active earth pressure is acting is $\phi_\tau = 0.45$, while the current AASHTO (2007) specification recommends $\phi_\tau = 0.80$.

Here, the lateral earth load considered during the design process is related to the active earth pressure. The back-fill is well-graded silty sand and gravel for which soil friction angle $\phi_f = 38°$. The ratio of the lateral at-rest earth pressure coefficient to the lateral active earth pressure coefficient for the back-fill is $K_0 / K_a = (1 + \sin\phi_f) = 1.616$, assuming Rankine's active earth pressure and at-rest earth pressure for normally consolidated cohessionless sand.

The abutment rests on well-graded sand with $\phi_f = 36.5°$, therefore, the interfacial friction angle obtained from the recommended relation in this study is as follows:

$$\tan(\delta_s) = 0.91\tan(36.5) \quad \Rightarrow \quad \delta_s = 33.95°$$

For the designed footing, the minimum factored vertical load and the corresponding lateral loads under Strength-I and Service-I loadings, thereby, the factored sliding resistance in each case are as follows.

Service-I LS:

At-rest earth pressure:

The minimum vertical load = 56.5kips/ft and the corresponding maximum total lateral load = 14.4kips/ft (Table H-33, C1 combination) when active earth pressure is acting. Hence, the corresponding maximum total lateral load when at-rest earth pressure is acting is (refer to Tables H-32 and H-33):

$$F_{2E0} = 14.4 - 11.5 + (1.616 \times 11.5) = 21.48\text{kips/ft}\ .$$

Factored sliding resistance $\phi_\tau F_{2E0}$ = 0.40 × 56.5×tan(33.95) = 15.2kips/ft < 21.5kips/ft

Active earth pressure:

Factored sliding resistance $\phi_\tau F_{2Ea}$ = 0.45 × 56.5×tan(33.95) = 17.1kips/ft > 14.4kips/ft

Current AASHTO:

Factored sliding resistance $\phi_\tau F_{2\tau}$ = 0.80 × 56.5×tan(33.95)
= 30.4kips/ft > 21.5kips/ft > 14.4kips/ft

Strength I LS:

At-rest earth pressure:

The minimum vertical load = 50.9kips/ft, and the corresponding maximum lateral load = 19.9kips/ft (Table H-34.2) when active earth pressure is acting. Hence, the corresponding maximum total lateral load when at-rest earth pressure is acting is (refer to Tables H-32 and H-34.2):

$$F_{2E0} = 19.9 - 1.5\times 11.5 + 1.5\times(1.616\times 11.5) = 27.0\text{kips/ft}$$

Factored sliding resistance $\phi_\tau F_{2E0}$ = 0.40 × 56.5×tan(33.95) = 15.2kips/ft < 27.0kips/ft

Active earth pressure:

Factored sliding resistance $\phi_\tau F_{2Ea}$ = 0.45 × 56.5×tan(33.95) = 17.1kips/ft < 19.9kips/ft

Current AASHTO:

Factored sliding resistance $\phi_\tau F_{2\tau}$ = 0.80 × 56.5×tan(33.95)
= 30.4kips/ft > 27.0kips/ft > 19.9kips/ft

This shows that the sliding resistance factors recommended in this study result in footings larger than the designed footing for design against sliding failure due to lateral loads involving at-rest as well as active earth pressures, except when unfactored lateral active earth pressure is considered. Since the design of abutment footings against sliding is critical, further study on the application of the resistance factors for sliding is necessary.

H.6.6 Discussions and Conclusions

From Figures H-16 and H-17 it is seen that the limiting eccentricity governs the footing design in this example when the limiting eccentricity is chosen as B/6. Further, within the range of the minimum admissible footing width, the recommended resistance factor is essentially ϕ = 0.40 (Table H-35). A footing of size 18.9ft×82.0ft fulfills the requirements for Strength-I and Service-I limit states. If, however, the limiting eccentricity is chosen as B/4, the minimum footing dimension required for Strength-I limit state is 13.6ft×82.0ft, whereas, that required for the Service-I limit state is equal to the minimum admissible footing size of 12.6ft×82.0ft except when AASHTO (2007) method is used for service limit state estimation. Hence Strength-I limit stated governs the design if limiting eccentricity of B/4 is considered. A footing of 18.9ft×82.0ft may be recommended for design.

H.7 EXAMPLE 7: NEW MARLBOROUGH BRIDGE, SOUTH ABUTMENT ON ROCK

H.7.1 General Information

The south abutment of the New Marlborough Bridge N-08-013 (2005) is analyzed in example 7. The New Marlborough bridge N-08-013 is a simple, single-span and short length span (SS-S). The constructed bridge dimensions and footing dimensions are:

Bridge:	
Span length	38.5ft (11.73m)
Span width	32.2ft (9.81m)
Foundations:	
South Abutment	Width = 10.5 ft (3.2m); length = 38.4ft (11.71m); average height of abutment from abutment footing base = 9.0ft (2.75m); abutment wingwall –SE side = 20.5ft (6.25m), SW side = 17.2ft (5.25m)
North Abutment	Width = 10.5 ft (3.8m); length = 38.4ft (11.71m); average height of abutment from abutment footing base = 9.0ft (2.75m); abutment wingwall –NE side = 26.3ft (8.0m), NW side = 23.0ft (7.0m)

H.7.2 Subsurface Condition

The subsurface at the south abutment location based on boring B-1 consists of 6inch of asphalt and 6inch of road base overlaying approximately 9.4ft of dry, loose to medium dense fine sand, with inorganic silt, and trace of gravel overlaying a quartzite rock layer. The geotechnical report (Mass Highway, 1999) called for placing the footing on a horizontal leveled rock ledge excavated at least 6inch deep. Based on boring B-1, the quartzite rock has an RQD of 59% up to a depth of 20.6ft (6.28m) to which drilled sample has been obtained. The point load strength has been reported to be 2,700psi.

The parameters provided in the geotechnical report for the gravel borrow used in the backfill are: bulk unit weight $\gamma = 130.0$pcf (20.4kN/m³) and internal friction angle $\phi_f = 35°$. The groundwater table is located at elevation 851.7ft (259.6m) and the foundation base is at elevation of 856.3ft (261.0m), i.e. the GWT is 4.6ft (1.4m) below the footing base. Hence the backfill soil is assumed to be dry for the design purpose.

H.7.3 Loads, Load Combinations and Limit States

The provided load components are summarized in Table H-36. The loads are provided in units of force per unit foundation length referring to the abutment length of 38.4ft (across the bridge). The dead (DL) load includes the weight of the superstructure and the abutment, whereas the vertical pressure from the dead load of earth fill is EV and earth surcharge load is ES. The investigated load combination and the resultant characteristic loading as well as the eccentricity e_2 (refer to Figure 120 of Chapter 5 for load notations and directions) for the load combination considered is summarized in Table H-37. The loading produces one-way inclination and one-way eccentricity with a negative eccentricity (refer to Figure 69b in section 3.7). The design load components required for the stability analysis, which are the factored characteristic loadings with load factors according to AASHTO Section 3 (2008) (presented in Tables H-4.1 and H-4.2), are summarized in Tables H-38.1 and H-38.2 for the bearing capacity and sliding strength limit states, respectively. Only Service-I and Strength-I limit states will be used here for the design of the footing width. Settlement evaluation is excluded but should be considered even if it is less likely to control the design of a footing on rock.

TABLE H-36. Loading at footing base for Example 7

Load at footing base	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
dead load (DL)	15.2 (222.1)	0.0	-45.3 (-201.4)
live load (LL)	1.9 (28.0)	0.0	-4.2 (-18.7)
at-rest earth pressure (EH)	0.0	6.4 (93.6)	43.0 (191.1)
Vertical load of earth fill (EV)	8.2 (119.7)	0.0	-63.0 (-280.3)
earth surcharge (ES)	1.0 (14.1)	1.1 (16.0)	3.4 (15.0)
live load surcharge (LS)	1.5 (22.3)	1.7 (25.3)	-2.1 (-9.4)

TABLE H-37. Load combinations and resultant characteristic (unfactored) loading for Example 7

Load combi-nations	Load components	F_1 kips/ft (kN/m)	F_2 kips/ft (kN/m)	M_3 kips-ft/ft (kNm/m)	F_2/F_1	$e_2 = M_3/F_1$ ft (m)
C1	DL+LL+EH+EV+ES+LS	27.8 (406.2)	9.2 (134.9)	-68.3 (-303.6)	0.332	-7.383 (-2.250)

TABLE H-38.1. Load combinations and resultant design (factored) loading for bearing resistance

Limit state load combinations	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
Service-I C1	27.8 (405.2)	9.2 (134.9)	-68.3 (-303.6)
Strength-I C1	37.6 (548.4)	13.3 (194.6)	-89.6 (-398.8)

TABLE H-38.2. Load combinations and resultant design (factored) loading for sliding resistance

Limit state load combinations	F_1 kip/ft (kN/m)	F_2 kip/ft (kN/m)	M_3 kip-ft/ft (kNm/m)
Service-I C1	27.8 (405.2)	9.2 (134.9)	-68.3 (-303.6)
Strength-I C1	28.7 (418.2)	12.5 (182.6)	-54.3 (-241.4)

H.7.4 Estimation of Rock Parameters

Based on Table 11 (Deere, 1968) in section 1.8.2 of the report, the rockmass quality corresponding to the given RQD of 59% is Fair. Using this information and Table 10.4.6.4-3 in AASHTO (2008) (Table 17 in section 1.8.3 of the report), the RMR ranges from 41 to 60. From Table 10.4.6.4-4 in AASHTO (2008) (Table 19 in section 1.8.3 of the report), for fair quality quartzite rock, the material constants are; m = 0.275 and s = 0.00009, and the joint spacing range from 1ft to 3ft. From Table 12 (Bieniawski, 1978) in section 1.8.2 of the report, for the Fair rockmass with $41 \leq \text{RMR} \leq 60$, the internal friction angle of the rockmass lies between 25° and 35°. Hence a friction angle of 30° (hence $N_\phi = 0.30$) and a joint spacing of 2ft (average of the ranges) have been adopted, similarly to the way the uncertainty of the methods have been established for calibration.

Based on the correlation between the point load strength and unconfined compressive strength proposed by Prakoso (2002), the unconfined compressive strength of the quartzite rockmass has been taken as

$$q_u = 23.3 \times I_s = 23.3 \times 2700 = 62910\text{psi} = 9059\text{ksf}$$

where I_s is the point load strength.

The rockmass Young's modulus of elasticity, E_m, for the quartzite rock of RQD = 59% has been estimated based on the average Young's modulus of elasticity of intact quartzite E_i and the ratio of E_m to E_i given by O'Neill and Reese (1999) (Table 10.4.6.5-1 in AASTHO, 2008). The average Young's modulus of elasticity for intact quartzite has been taken as $E_i = 9.59 \times 10^{-3}$ ksi (Table C10.4.6.5-1 in AASHTO, 2008) and the E_m/E_i ratio for RQD = 59% is about 0.42. Hence, the rockmass modulus of elasticity, $E_m = 4.03 \times 10^{-3}$ ksi. Average value of Poisson's ratio for the rock has been taken as 0.14 (Table C10.4.6.5-2 in AASHTO, 2008).

H.7.5 Nominal and Allowable Bearing Resistances at the Limit States

The bearing resistances of rectangular footings with widths of 4.0ft to 14.0ft, with the footing length kept fixed at 38.4ft according to the length of the abutment, have been calculated for Strength-I C1 limit state. Carter and Kulhawy (1988) method and Goodman (1989) method for non-fractured rockmass have been used to estimate the nominal bearing capacities. The recommended resistance factor in the present study as well as that recommended in AASHTO (2008) have been applied and the resulting footing widths compared. The recommended resistance factor to be used with Carter and Kulhawy (1988) method for the range of RMR established is $\phi = 1.00$, and $\phi = 0.35$ when the RMR range is not considered, while that to be used with Goodman (1989) method for both the joint spacing s' and friction angle ϕ_f estimated from RQD is $\phi = 0.30$. The resistance factor in the current AASHTO (2008) specifications is $\phi = 0.45$, irrespective of the estimation method used.

For the footing on the quartzite rock with small Poisson's ratio and large E_m, the settlement can be expected to be very small, which is observed in the calculation of the allowable bearing resistance for the Service-I limit state using the AASHTO (2008) settlement calculation method (equation 10.6.2.4.4-3). No resistance factors had been yet established for the settlement evaluation.

H.7.6 Design Footing Width

The load eccentricity corresponding to the C1 loading is 7.38ft along the footing width, according to Table H-37. Hence, the minimum foundation width required for the limiting eccentricity is B_{min} = 44.3ft (=7.38ft×6) considering a limiting eccentricity of B/6, while B_{min} = 29.5ft (=7.38ft×4) considering a limiting eccentricity of B/4. The maximum vertical factored load for Strength-I limit state (bearing resistance; Table H-38.1) is 37.6kip/ft and the vertical unfactored load for Service-I limit state is 27.8kips/ft.

Figures H-18 and H-19 present the unfactored and factored bearing resistances for different footing widths for bearing resistances estimated using Carter and Kulhawy (1988) method and Goodman (1989) method, respectively. The bearing load intensities (stresses) are plotted in the upper figures, whereas the lower ones present the bearing loads per unit length of the foundation to be compatible with the load representation. Both the bearing capacity as well as the settlement analysis have been carried out for the full geometric foundation width.

Figure H-18 shows the variation of factored and unfactored bearing capacities with full footing width for Strength-I limit state estimated using Carter and Kulhawy (1988) method and Service-I limit state estimated using AASHTO (2008) method. The recommended resistance factor in the present study being 1.00 for the rock with 44 ≤ RMR ≤ 65, the unfactored (nominal) as well as the factored bearing resistance coincide in this example. Figure H-19 shows the variation of factored and unfactored bearing resistances with full footing width for Strength-I limit state estimated using Goodman (1989) method for non-fractured rocks.

Applying the aforementioned vertical loads for the corresponding limit states in Figures H-18 and H-19, the following results are obtained, irrespective of the method used for bearing resistance estimation: (a) while all the footing widths for which the bearing resistances are evaluated fulfill the Strength-I and Service-I limit state loading requirements, the abutment is subjected to inclined-eccentric loading; hence the recommended footing size has to be of the minimum admissible width for limiting eccentricity; (b) if the limiting eccentricity criterion is ignored because the resultant load eccentricity is negative eccentricity, the factored Strength-I limit state dominates the footing design, especially for footings with B ≤ 12ft; however, this cannot be less than the thickness of the abutment wall, which is 4.0ft.

The conclusions possible from Figures H-18 and H-19 are, therefore, that based on the strength as well as service limit states, the following foundation sizes (full geometry) are sufficient, if the limiting eccentricity criterion is not taken into consideration:

1. Carter and Kulhawy (1988) method:
   a. Strength limit state ϕ = 1.00 or 0.35: between 4.0ft×38.4ft and 44.3ft×38.4ft
   b. Strength limit state ϕ = 0.45: between 4.0ft×38.4ft and 44.3ft×38.4ft (current AASHTO)
2. Goodman (1989) method:
   a. Strength limit state ϕ = 0.30: between 4.0ft×38.4ft and 44.3ft×38.4ft
   b. Strength limit state ϕ = 0.45: between 4.0ft×38.4ft and 44.3ft×38.4ft (current AASHTO)

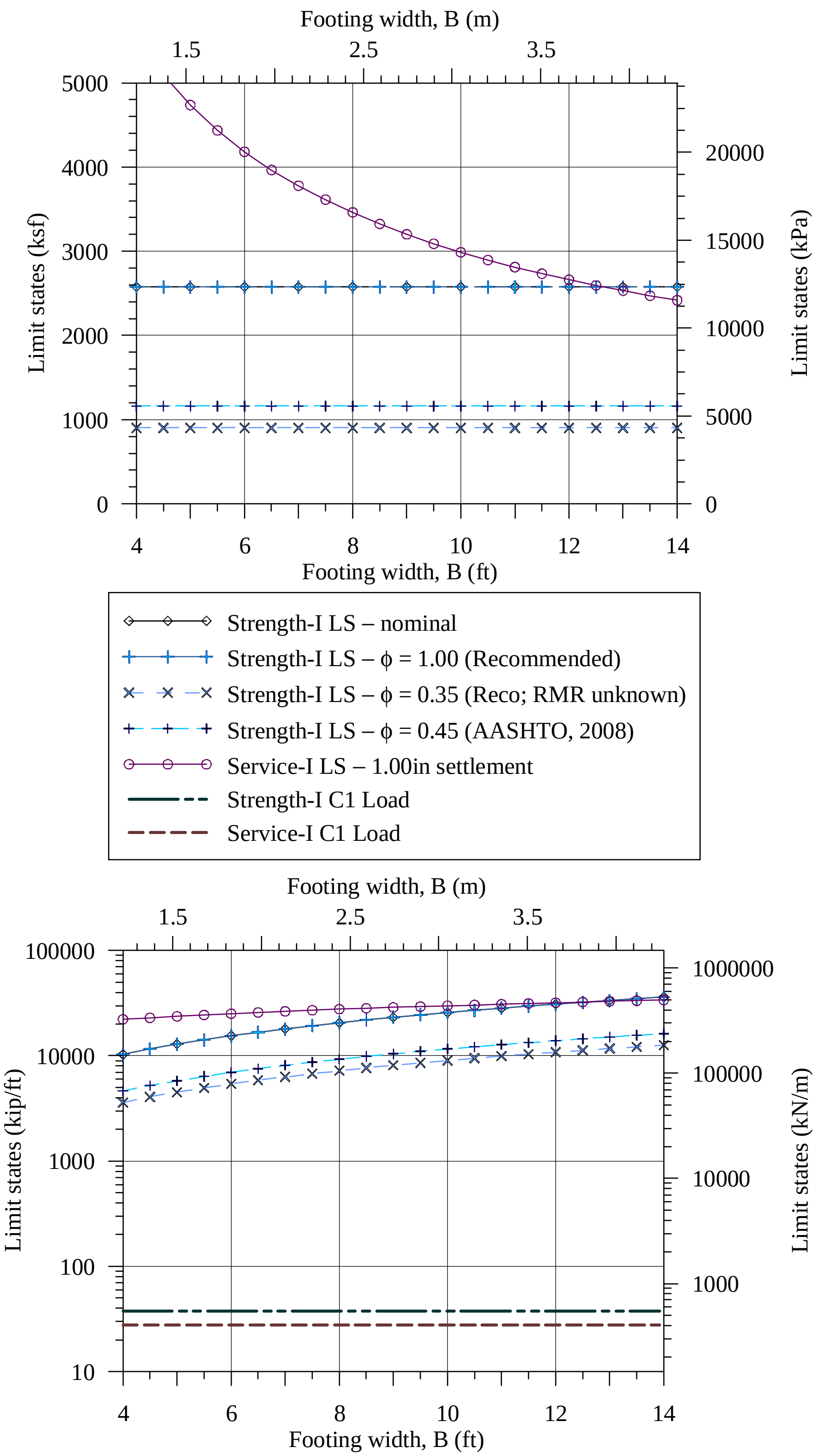

Figure H-18. Variation of factored bearing resistance for Strength-I C1 loading, estimated using Carter and Kulhawy (1988) method, with footing width for Example 7; loads are expressed per unit length of the foundation (L = 38.4ft)

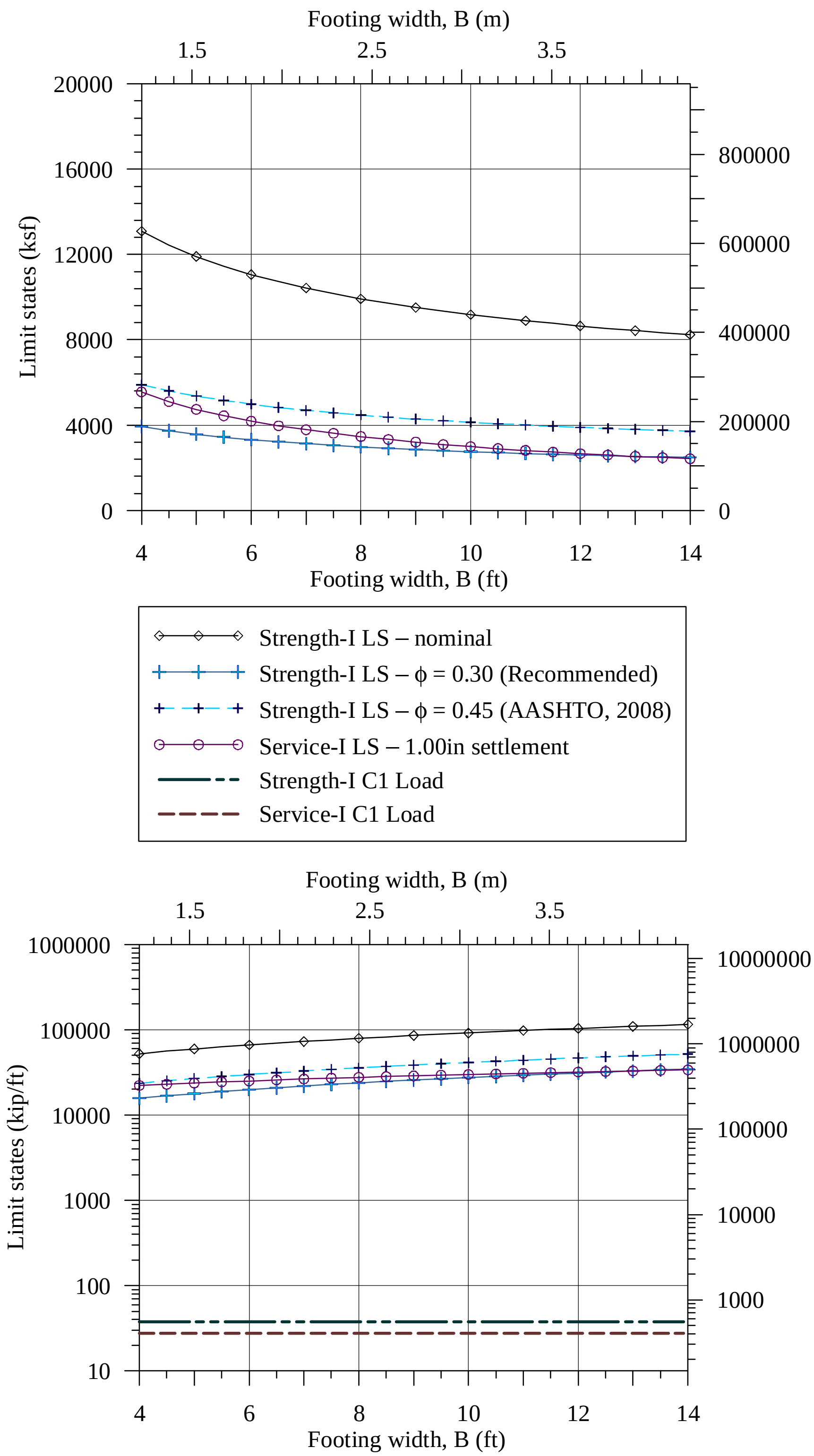

Figure H-19 Variation of factored bearing resistance for Strength-I C1 loading, estimated using Goodman (1989) method for non-fractured rock, with footing width for Example 7; loads are expressed per unit length of the foundation (L = 38.4ft)

For the constructed footing size of B = 10.5ft, the estimated factored bearing resistance using Carter and Kulhawy (1988) method is estimated to be 27.1×10^3 kip/ft when RMR is considered for the selection of ϕ and 9.5×10^3 kip/ft when RMR range is ignored; the factored bearing resistance is 28.4×10^3 kip/ft using Goodman (1989) method, while the estimated load required for 1.0in settlement is 30.4×10^3 kip/ft. All of these capacities fulfill the required Strength-I and Service-I LS loadings by very large margins.

H.7.7 Sliding Resistance

The concrete/rock adhesion should result with an interface shear strength equal to the lower of the two. Considering a reduction factor (beyond the scope of the presented research) should show sufficient width to resist all loads. For the purpose of demonstration only, the sliding resistance is evaluated assuming contact with granular material, serving also as the lowest possible resistance.

For $\phi_f = 30°$, the interfacial friction angle between the footing base and the rock ledge obtained from the recommended relation in this study, though strictly valid from the interface of concrete and granular soils, is as follows:

$$\tan(\delta_s) = 0.91 \tan(30.0) \quad \Rightarrow \delta_s = 27.7°$$

Note, in the actual design calculation, δ_s was taken as equal to 30°. The recommended resistance factor for cast in-situ footings when at-rest earth pressure is acting is $\phi_\tau = 0.40$ and that when active earth pressure is acting is $\phi_\tau = 0.45$. The current AASHTO (2007) specification recommends $\phi_\tau = 0.80$. Here, the lateral earth load considered during the design process is related to the at-rest earth pressure. For the back-fill with soil friction angle $\phi_f = 35°$, the ratio of the lateral active earth pressure coefficient to the lateral at-rest earth pressure coefficient, $K_a / K_0 = 1 / (1 + \sin \phi_f) = 1/1.574$, assuming Rankine's active earth pressure and at-rest earth pressure for normally consolidated cohessionless sand.

For the constructed footing of B = 10.5ft, the minimum factored vertical load and the corresponding lateral loads under Strength-I and Service-I loadings, thereby, the factored sliding resistance in each case are as follows.

Service-I LS:

At-rest earth pressure:

The minimum vertical load = 27.8kips/ft, and the corresponding lateral load = 6.4kips/ft from the earth pressure alone (Table H-38.2)

$\delta_s = 27.7°$: Sliding resistance $\phi_\tau F_{2\tau} = 0.40 \times 27.8\times\tan(27.7) = 5.8$kips/ft < 6.4kips/ft

$\delta_s = 30.0°$: Sliding resistance $\phi_\tau F_{2\tau} = 0.40 \times 27.8\times\tan(30.0) = 6.4$kips/ft = 6.4kips/ft

Active earth pressure:

The corresponding lateral load involving active earth pressure is (Tables H-36 and H-38.2)

$$F_{2Ea} = \frac{1}{1.574}\times 6.4 = 4.07\text{kips/ft}\,.$$

$\delta_s = 27.7°$: Sliding resistance $\phi_\tau F_{2Ea} = 0.45 \times 27.8\times\tan(27.7) = 6.5$kips/ft > 4.07kips/ft

$\delta_s = 30.0°$: Sliding resistance $\phi_\tau F_{2Ea} = 0.45 \times 27.8\times\tan(30.0) = 7.2$kips/ft > 4.07kips/ft

Current AASHTO:

δ_s = 27.7°: Sliding resistance $\phi_\tau F_{2\tau}$ = 0.80 × 27.8×tan(27.7) = 11.7kips/ft > 6.4kips/ft

δ_s = 30.0°: Sliding resistance $\phi_\tau F_{2\tau}$ = 0.80 × 27.8×tan(30.0) = 12.8kips/ft > 6.4kips/ft

Strength I LS:

At-rest earth pressure:

The minimum vertical load = 28.7kips/ft, and the corresponding lateral load = 9.6kips/ft (=1.50×6.4) from the earth pressure alone (Table H-38.2)

δ_s = 27.7°: Sliding resistance $\phi_\tau F_{2\tau}$ = 0.40 ×28.7×tan(27.7) = 6.0kips/ft < 9.6kips/ft

δ_s = 30.0°: Sliding resistance $\phi_\tau F_{2\tau}$ = 0.40 ×28.7×tan(30.0) = 6.6kips/ft < 9.6kips/ft

Active earth pressure:

The corresponding lateral load involving factored active earth pressure is (load factors given in Table H-4.2)

$$\gamma_i F_{2Ea} = 1.50 \times \left(\frac{1}{1.574} \times 6.4 \right) = 6.10\text{kips/ft}\,.$$

δ_s = 27.7°: Sliding resistance $\phi_\tau F_{2Ea}$ = 0.45 × 28.7×tan(27.7) = 6.8kips/ft > 6.1kips/ft

δ_s = 30.0°: Sliding resistance $\phi_\tau F_{2Ea}$ = 0.45 × 28.7×tan(30.0) = 7.4kips/ft > 6.1kips/ft

Current AASHTO:

δ_s = 27.7°: Sliding resistance $\phi_\tau F_{2\tau}$ = 0.80 × 28.7×tan(27.7) = 12.0kips/ft > 9.6kips/ft

δ_s = 30.0°: Sliding resistance $\phi_\tau F_{2\tau}$ = 0.80 × 28.7×tan(30.0) = 13.2kips/ft > 9.6kips/ft

This shows that the footing of width B = 10.5ft is safe in sliding except when the at-rest earth pressure for Service-I LS vertical load is acting and ϕ_τ recommended in the present study is applied to the sliding resistance. The interfacial friction angle δ_s either can be assumed to be equal to 30.0° or can be obtained from the correlation presented.

H.7.8 Discussions and Conclusions

The design footing width required for limiting the eccentricity is found to be very large; 44.3ft if limiting eccentricity of B/6 is considered or 29.5ft if B/4 is considered. The load eccentricity in this example creates negative eccentricity, which acts "in favor" in terms of bearing capacity as has been discussed in section 3.7 (Loading direction effect for inclined eccentric loading) and shown in Figure 69 in section 3.7 of the report. Without considering the limiting eccentricity, a small footing of size B = 4.0ft is found to be sufficient for bearing resistances in Strength as well as Service limit states. Strength-I limit state governs the design for B ≤ 12.0ft. The recommended footing is between 4.0ft×38.4ft and 12.0ft×38.4ft, at the discretion of the geotechnical and structural engineer, depending on the local practice.

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