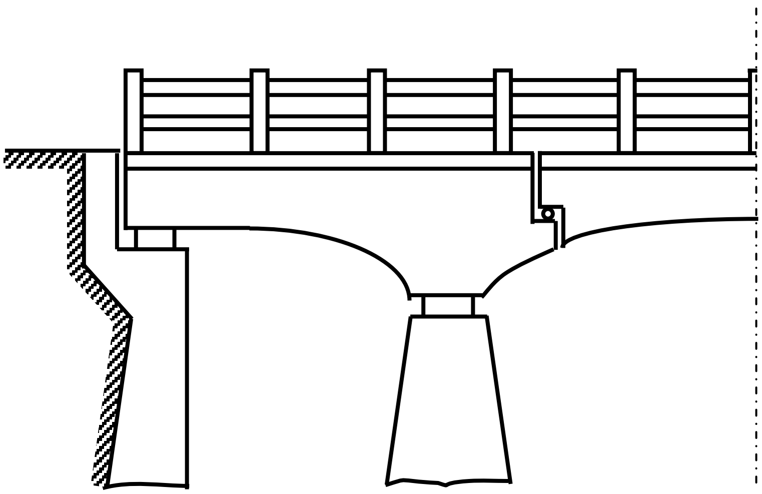
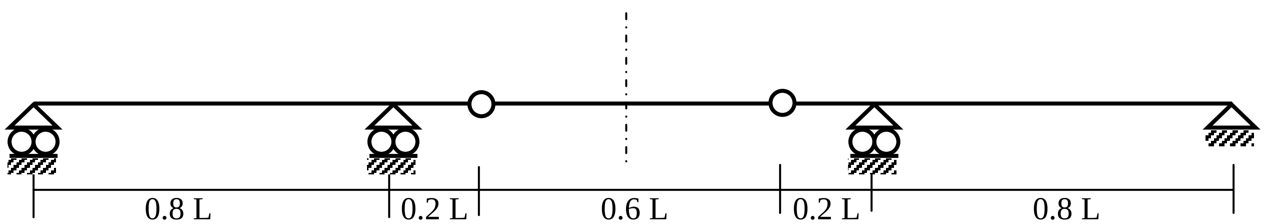
Design of a Balanced-Cantilever Bridge



CL (Bridge is symmetric about CL)



L = 80 ft $\Rightarrow$ Bridge Span = 2.6 L = 2.6 $\times$ 80′ = 208′

Bridge Width = 30′

No. of girders = 6, Width of each girder = 15″

Loads: LL = HS20, Wearing Surface = 30 psf

Material Properties: f'_c = 3 ksi, f_s = 20 ksi

1. Design of Slab

Bridge width = 30′, Number of girders = 6@15″

∴ Clear span between girders S = (30′– 6×15″)/5 = 4.5′

∴ The c/c distance between girders = 4.5′ + 15″ = 5.75′

Assuming slab thickness = 6″⇒ Slab weight = 75 psf = 0.075 ksf

∴ Wearing Surface = 30 psf ⇒ w_{DL} = 75+30 =105 psf = 0.105 ksf

∴ Dead-load moment, $M_{DL} = \pm w_{DL}S^2/10 = 0.105\times4.5^2/10 = 0.213$ k′/′

AASHTO specifies Live-load moment, $M_{LL} = \pm\ 0.8\ (S+2)/32\ P_{20}$

$= \pm\ 0.8\ [(4.5+2)/32]\times16 = \pm\ 2.6$ k′/′

Impact factor, I = 50/(S+125) = 0.386 >0.3 ⇒ I = 0.3

Impact moment, $M_{IMP} = M_{LL} \times I = \pm\ 2.6 \times 0.3 = \pm\ 0.78$ k′/′

∴ Total moment, $M_T = M_{DL} + M_{LL} + M_{IMP} = \pm\ (0.213 + 2.6 + 0.78) = \pm\ 3.593$ k′/′

For design, f'_c = 3 ksi ⇒ $f_c = 0.4f'_c$ = 1.2 ksi

∴ n = 9, k = 9/(9+20/1.2) = 0.351, j = 1– k/3 = 0.883

∴ R = ½×1.2×0.351×0.883 = 0.186 ksi

⇒ $d_{req} = \sqrt{(M_T/Rb)} = \sqrt{(3.593/0.186\times1)} = 4.40''$

t = 6″⇒ d = 6″–1.5″ = 4.5″> d_{req}

⇒ d = 4.5″, t = 6″ (OK).

∴ Required reinforcement, $A_s = M_T/(f_s jd) = 3.593\times12/(20\times0.883\times4.5) = 0.543$ in²/′

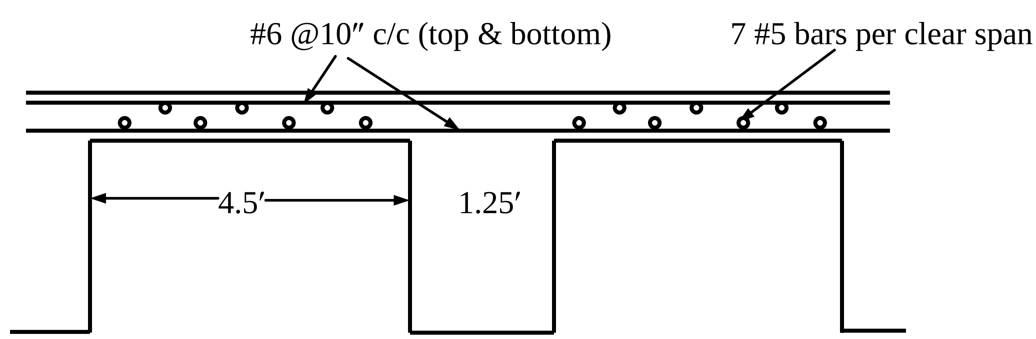
∴ Use #5 @7″ c/c [or #6 @10″ c/c]

Also, $2.2/\sqrt{S} = 2.2/\sqrt{4.5} = 1.04>0.67$

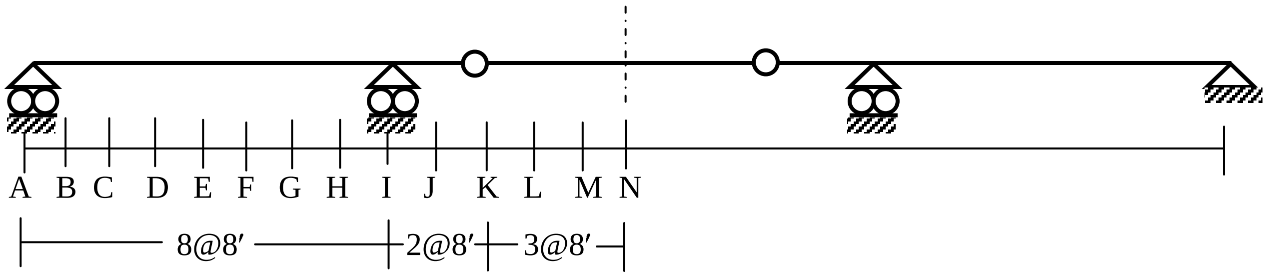
∴ Distribution steel, $A_{s(dist)} = 0.67\ A_s = 0.67\times0.543 = 0.364$ in²/′

∴ $A_{s(dist)}$ per c/c span = 5.75′× 0.364 in²/′

= 2.09 in²; i.e., 7 #5 bars (to be placed within the clear spans).



2. Dead Load Analysis of Interior Girders



Girder depths remain constant between A-D and vary parabolically between D-I and I-N. The variation is symmetric about I. If the girder depths at D and N are both 40″ (L/2 in inches) and that at I is 70″ (about 70-80% larger), the depths at the other sections can be calculated easily. The depths calculated are the following

h_A = 40″, h_B = 40″, h_C = 40″, h_D = 40″, h_E = 41.2″, h_F = 44.8″, h_G = 50.8″, h_H = 59.2″, h_I = 70″,

h_J = 59.2″, h_K = 50.8″, h_L = 44.8″, h_M = 41.2″, h_N = 40″

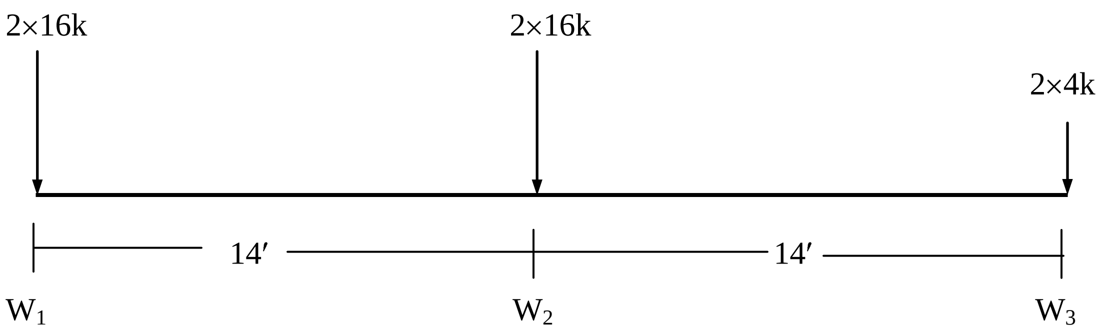
Using these dimensions (with an additional 30 psf; i.e., 2.4″ concrete layer) for the analysis of the girder for self-weight using the software GRASP, the following results are obtained.

Table 2.1 Dead Load Shear Forces and Bending Moments

Section	x from left (′)	h (″)	V (k)	M (k′)
A	0	40	27.40	0.00
B	8	40	18.32	182.84
C	16	40	9.24	293.04
D	24	40	0.16	330.59
E	32	41.2	–9.00	295.21
F	40	44.8	–18.46	185.39
G	48	50.8	–28.51	–2.48
H	56	59.2	–39.47	–274.39
I (L)	64	70	–51.62	–638.72
I (R)	64	70	51.78	–638.72
J	72	59.2	39.62	–273.14
K	80	50.8	28.67	0.00
L	88	44.8	18.61	189.10
M	96	41.2	9.16	300.16
N	104	40	0.00	336.78

3. Live Load Analysis of Interior Girders

The live load analysis of interior girders is carried out for HS20 loading with wheel loads of 4 k, 16 k, and 16 k at 14′ distances, as shown below.



For live-load analysis, each wheel load (4 k, 16 k, 16 k) needs to be multiplied by a factor

$S/5 \geq 1.0$

In this case, S = 5.75′; $\therefore$ Factor = 5.75/5 = 1.15 > 1.0, OK

$\therefore$ Wheel Loads for live load analysis are

(16 × 1.15 =) 18.4 k, (16 × 1.15 =) 18.4 k and (4 × 1.15 =) 4.6 k

Also, the impact factor I = 50/(L_0+125) ≤ 0.30, where L_0 = Loaded length

Assuming L_0 = 0.6L = 48′ (conservatively), I = 50/(48+125) = 0.289

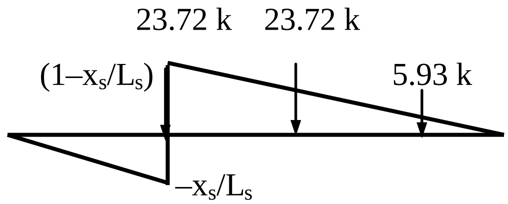
$\therefore$ The impact shear forces and bending moments can be obtained by multiplying live load shears and moments by I (= 0.289).

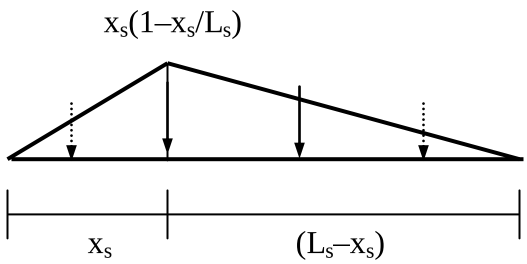
As an alternative to using separate moments for live load and impact, one can do them simultaneously by multiplying the wheel loads by (1 + I) = 1.289; i.e., taking wheel loads to be (18.4 × 1.289 =) 23.72 k, (18.4 × 1.289 =) 23.72 k and (4.6 × 1.289 =) 5.93 k.

The combined (live load + impact) shears and bending moments can be obtained by moving the wheels from A to N (keeping W_1 or W_3 in front) and recording all the shear forces (V) and bending moments (M). The software GRASP can be used for this purpose.

Instead of such random wheel movements, Influence Lines can be used to predict the critical position of wheels in order to get the maximum forces. This can considerably reduce the computational effort. The subsequent discussions follow this procedure.

The IL for V and M at the ‘simply supported span’ K-L-M-N and the critical wheel arrangements are as follows.





Using $< x > = x$, if $x \geq 0$, or $= 0$ otherwise

$V_{LL+IMP}(x_s) = (23.72/L_s)\,[(L_s–x_s) + (L_s–x_s–14) + < L_s–x_s–28>/4]$

$M_{LL+IMP}\,(x_s) = x_s\, V_{LL+IMP}(x_s)$; if $x_s \leq L_s/3$.

$= (23.72/L_s)\,[x_s\,(L_s–x_s) + x_s\,(L_s–x_s–14) + (x_s–14)\,(L_s–x_s)/4]$; otherwise.

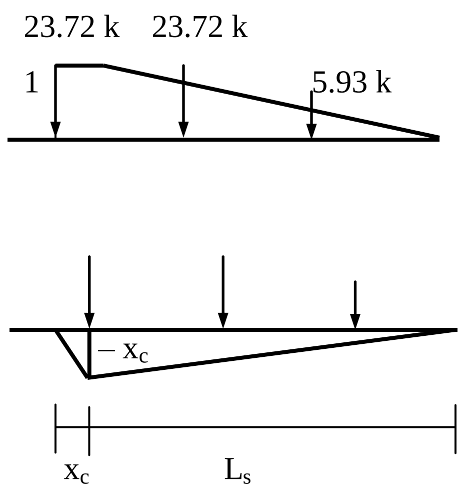
Using these equations, with $L_s = 48'$, the following values are obtained

[These calculations can be carried out conveniently in EXCEL]

Table 3.1 V_{LL+IMP} and M_{LL+IMP} for K-N

Section	x_s (′)	V (k)	M (k′)
K	0	42.99	0.00
L	8	34.10	272.78
M	16	25.20	403.24
N	24	16.81	432.89

The IL for V and M at the span 'cantilever span' I(R)-J-K and the critical wheel arrangements are as follows.



Using $< x > = x$, if $x \geq 0$, or $= 0$ otherwise

$V_{LL+IMP}(x_c) = 23.72\ [1 + \{1 – <14–x_c>/L_s\} + \{1–(28–x_c)/L_s\}/4]$

$= 23.72\ [2.25 – \{<14–x_c> + (28–x_c)/4\}/L_s]$

$M_{LL+IMP}\ (x_c) = –23.72\ (x_c\ /L_s)\ [L_s + (L_s–14) + (L_s–28)/4]$

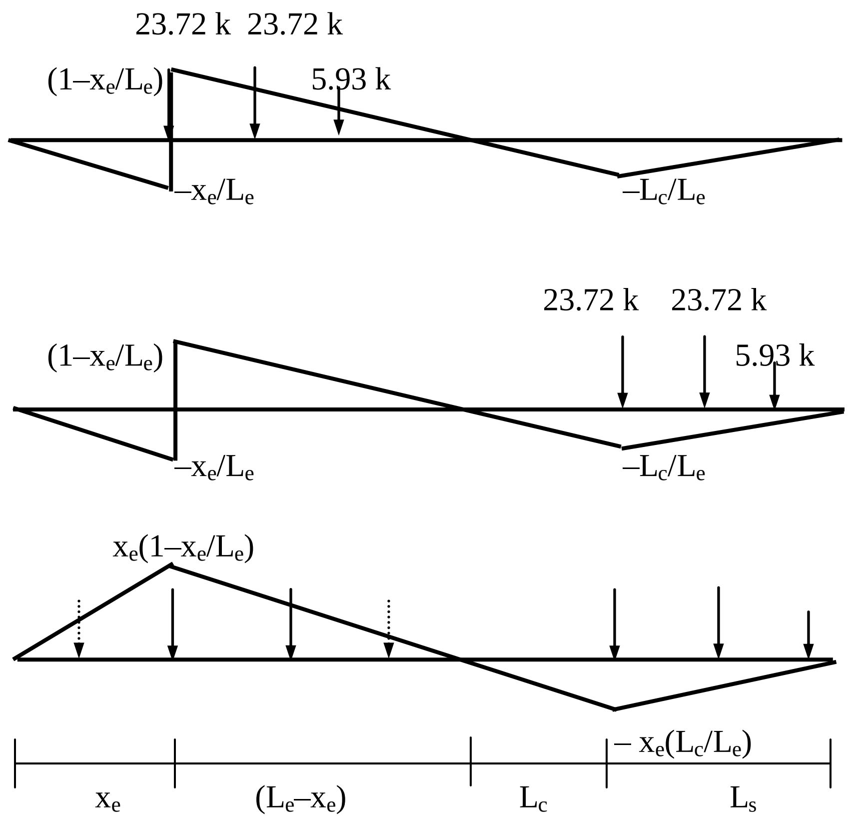
$= –\ (23.72\ x_c)\ [2.25 – 21/L_s]$

Using these equations, with $L_s = 48'$, the following values are obtained

Table 3.2 V_{LL+IMP} and M_{LL+IMP} for I(R)-J

Section	x_c (′)	V (k)	M (k′)
I (R)	16	51.89	–687.88
J	8	47.93	–343.94

The IL for V and M at the 'end span' A-B-C-D-E-G-H-I(L) and the critical wheel arrangements are as follows.



Here the results for $x_e \leq L_e/2$ will be calculated and symmetry will be used for the other half.

Using $< x > = x$, if $x \geq 0$, or $= 0$ otherwise

For positive shear and moment

$V^+_{LL+IMP}(x_e) = (23.72/L_e)\,[(L_e-x_e) + (L_e-x_e-14) + (L_e-x_e-28)/4] \geq 0$

$M^+_{LL+IMP}(x_e) = x_e\, V^+_{LL+IMP}(x_e)$; if $x_e \leq L_e/3$.

$= (23.72/L_e)\,[x_e\,(L_e-x_e) + x_e\,(L_e-x_e-14) + (x_e-14)\,(L_e-x_e)/4]$; otherwise.

For negative shear and moment

$V^-_{LL+IMP}(x_e) = -\,(23.72/L_e)\,[x_e + (x_e-14) + (x_e-28)/4] \leq 0$

or $= -\,(23.72\, L_c/L_e)\,[1 + (1-14/L_s) + (1-28/L_s)/4]$

$M^-_{LL+IMP}(x_e) = -\,(23.72\, x_e\, L_c/L_e)\,[1 + (1-14/L_s) + (1-28/L_s)/4]$

Using the derived equations, with $L_c = 16'$ & $L_e = 64'$, the following values are obtained

Table 3.3 V_{LL+IMP} and M_{LL+IMP} for A-I(L)

Section	x_e (′)	V^+(k)	M^+(k′)	V^- (k)	M^- (k′)
A	0	45.59	0.00	–10.75	0.00
B	8	38.92	311.33	–10.75	–85.99
C	16	32.24	515.91	–10.75	–171.97
D	24	25.57	624.13	–12.23	–257.96
E	32	18.90	646.37	–18.90	–343.94
F	40	*	624.13	–25.57	–429.93
G	48	*	515.91	–32.24	–515.91
H	56	*	311.33	–38.92	–601.90
I(L)	64	*	0.00	–45.59	–687.88

The ‘simply supported span’ K-L-M-N

$V_{LL+IMP}(x_s) = (23.72/L_s)\,[(L_s–x_s) + (L_s–x_s–14) + < L_s–x_s–28>/4]$

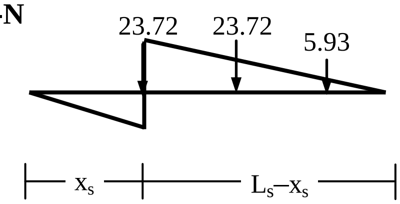
$M_{LL+IMP}\,(x_s) = x_s\,V_{LL+IMP}(x_s)$; if $x_s \le L_s/3$.

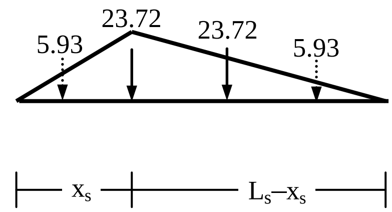
$= (23.72/L_s)\,[x_s\,(L_s–x_s) + x_s\,(L_s–x_s–14) + (x_s–14)\,(L_s–x_s)/4]$; otherwise.

Using these equations, with $L_s = 48'$, the following values are obtained

Table 3.1 V_{LL+IMP} and M_{LL+IMP} for K-N

Section	x_s (′)	V (k)	M (k′)
K	0	42.99	0.00
L	8	34.10	272.78
M	16	25.20	403.24
N	24	16.81	432.89





The ‘cantilever span’ I(R)-J-K

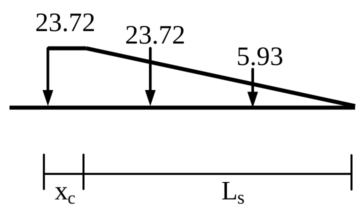
$V_{LL+IMP}(x_c) = 23.72\,[2.25 – \{<14–x_c> + (28–x_c)/4\}/L_s]$

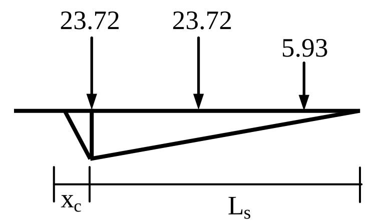
$M_{LL+IMP}\,(x_c) = –\,(23.72\,x_c)\,[2.25 – 21/L_s]$

Using these equations, with $L_s = 48'$, the following values are obtained

Table 3.2 V_{LL+IMP} and M_{LL+IMP} for I(R)-J

Section	x_c (′)	V (k)	M (k′)
I (R)	16	51.89	–687.88
J	8	47.93	–343.94





The ‘end span’ A-B-C-D-E-G-H-I(L)

Here the results for $x_e \le L_e/2$ will be calculated and symmetry will be used for the other half.

For positive shear and moment

$V^+_{LL+IMP}(x_e) = (23.72/L_e)\,[(L_e–x_e) + (L_e–x_e–14) + (L_e–x_e–28)/4] \ge 0$

$M^+_{LL+IMP}(x_e) = x_e\,V^+_{LL+IMP}(x_e)$; if $x_e \le L_e/3$.

$= (23.72/L_e)\,[x_e\,(L_e–x_e) + x_e\,(L_e–x_e–14) + (x_e–14)\,(L_e–x_e)/4]$; otherwise.

For negative shear and moment

$V^–_{LL+IMP}(x_e) = –\,(23.72/L_e)\,[x_e + (x_e–14) + (x_e–28)/4] \le 0$

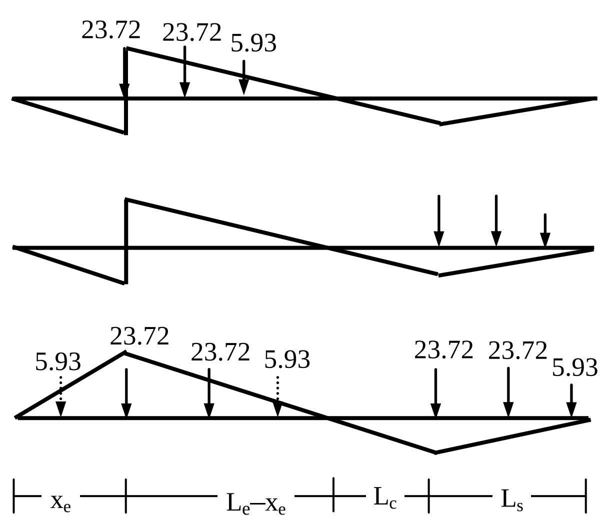
or $= –\,(23.72\,L_c/L_e)\,[1 + (1–14/L_s) + (1–28/L_s)/4]$

$M^–_{LL+IMP}(x_e) = –\,(23.72\,x_eL_c/L_e)\,[1 + (1–14/L_s) + (1–28/L_s)/4]$

Using the derived equations, with $L_c = 16'$ and $L_e = 64'$, the following values are obtained

Table 3.3 V_{LL+IMP} and M_{LL+IMP} for A-I(L)

Section	x_e (′)	V^+(k)	M^+(k′)	$V^–$ (k)	$M^–$ (k′)
A	0	45.59	0.00	–10.75	0.00
B	8	38.92	311.33	–10.75	–85.99
C	16	32.24	515.91	–10.75	–171.97
D	24	25.57	624.13	–12.23	–257.96
E	32	18.90	646.37	–18.90	–343.94
F	40	*	624.13	–25.57	–429.93
G	48	*	515.91	–32.24	–515.91
H	56	*	311.33	–38.92	–601.90
I(L)	64	*	0.00	–45.59	–687.88



4. Combination of Dead and Live Loads

The dead load and (live load + Impact) shear forces and bending moments calculated earlier at various sections of the bridge are now combined to obtain the design (maximum positive and/or negative) shear forces and bending moments.

[These calculations can be conveniently done in EXCEL, and subsequent columns should be kept for shear & flexural design]

Table 4.1 Combination of DL & LL+IMP to get V_{Design} and M_{Design}

Section	V_{DL} (k)	V_{LL+IMP} (k)	V_{Design} (k)	M_{DL} (k′)	M_{LL+IMP} (k′)	M^{+}_{Design} (k′)	M^{-}_{Design} (k′)
A	27.40	45.59 -10.75	72.99	0.00	0.00	0.00	0.00
B	18.32	38.92 -10.75	57.24	182.84	311.33 -85.99	494.17	0.00
C	9.24	32.24 -10.75	41.48	293.04	515.91 -171.97	808.95	0.00
D	0.16	25.57 -12.23	25.73	330.59	624.13 -257.96	954.72	0.00
E	-9.00	18.90 -18.90	-27.90	295.21	646.37 -343.94	941.58	-48.75
F	-18.46	-25.57	-44.03	185.39	624.13 -429.93	809.52	-244.24
G	-28.51	-32.24	-60.75	-2.48	515.91 -515.91	513.43	-518.39
H	-39.47	-38.92	-78.39	-274.39	311.33 -601.90	36.94	-876.29
I(L)	-51.62	-45.59	-97.21	-638.72	-687.88	0.00	-1326.60
I(R)	51.78	51.89	103.67	-638.72	-687.88	0.00	-1326.60
J	39.62	47.93	87.55	-273.14	-343.94	0.00	-617.08
K	28.67	42.99	71.66	0.00	0.00	0.00	0.00
L	18.61	34.10	52.71	189.10	272.78	461.88	0.00
M	9.16	25.20	34.36	300.16	403.24	703.40	0.00
N	0.00	16.81	16.81	336.78	432.89	769.67	0.00

5. Design of Interior Girders

Shear Design

The shear design of interior girders is performed by using the conventional shear design equations of RCC members. The stirrup spacing is given by the equation

$$S_{(req)} = A_s\ f_s\ d/(V–V_c)$$

where f_s = 20 ksi. If 2-legged #5 stirrups are used, $A_s = 0.62\ in^2$.

$$V_c = 0.95\sqrt{f'_c}\ bd = 0.95\sqrt{(0.003)}\ 15\ d = 0.7805\ d$$

$$S_{(req)} = 12.4\ d/(V–0.7805\ d)$$

$$d_{(req)} = V/(2.95\sqrt{f'_c}\ b) = V/2.4237$$

where d and V vary from section to section.

ACI recommends that the maximum stirrup spacing (S) shouldn't exceed

$$d/2,\ \text{or}\ 24'' \text{ or } A_s/0.0015b = 0.62/0.0225 = 27.56''$$

The calculations are carried out in tabular form and listed below.

It is convenient to perform these calculations in EXCEL.

Table 5.1 Design for Shear Force

Section	x from left (′)	h (″)	d (″)	V (kips)	$d_{(req)}$ (″)	$S_{(req)}$ from formula (″)	$S_{(provided)}$ (″)
A	0.00	40.00	33.50	72.99	30.12	8.87	8
B	8.00	40.00	33.50	57.24	23.62	13.36	13
C	16.00	40.00	33.50	41.48	17.11	16.75	16
D	24.00	40.00	33.50	25.73	10.62	16.75	16
E	32.00	41.20	34.70	-27.90	11.51	17.35	16
F	40.00	44.80	38.30	-44.03	18.17	19.15	16
G	48.00	50.80	44.30	-60.75	25.06	20.99	16
H	56.00	59.20	52.70	-78.39	32.34	17.54	16
I(L)	64.00	70.00	63.50	-97.21	40.11	16.53	16
I(R)	64.00	70.00	63.50	103.67	42.77	14.55	14
J	72.00	59.20	52.70	87.55	36.12	14.08	14
K(*)	80.00	50.80	44.30 (*)	71.66	Articulation	*	*
L	88.00	44.80	38.30	52.71	21.75	19.15	16
M	96.00	41.20	34.70	34.36	14.18	17.35	16
N	104.00	40.00	33.50	16.81	6.94	16.75	16

Flexural Design

The flexural design of interior girders is performed by using the conventional flexural design equations for singly/doubly reinforced RCC members (rectangular or T-beam section).

For positive moments, the girders are assumed singly reinforced T-beams with

$$A_s = M/[f_s\,(d–t/2)]$$

However, the compressive stresses in slab should be checked against f_c (= 1.2 ksi here).

For negative moments, the girders are rectangular beams. For singly reinforced beams, the depth $d \geq d_{(req)} = \sqrt{(M/Rb)}$, and the required steel area (A_s) at top is

$$A_s = M/(f_s\,j\,d)$$

For doubly reinforced beams, $d<d_{(req)}$; i.e., $M>M_c$ (= Rbd^2). The moment is divided into two parts; i.e., $M_1 = M_c$ and $M_2 = M–M_c$. The required steel area (A_s) at top is given by

$$A_s = A_{s1} + A_{s2} = M_1/(f_s\,j\,d) + M_2/\{f_s(d–d')\}$$

Here d' is the depth of compression steel from the compression edge of the beam. In addition, compressive steels are necessary (in the compression zone at bottom), given by

$$A_s' = M_2/\{f_s'(d–d')\}, \text{ where } f_s' = 2\,f_s(k–d'/d)/(1–k) \leq f_s$$

Development Length of #10 bars = $0.04\times1.27\times40/\sqrt{0.03}\times1.4 = 51.94''$

Table 5.2 Design for Bending Moment

Section	x from left (′)	d (″)	M^+ (k′)	A_s^+ (in^2)	M^- (k′)	M_c^- (k′)	A_{s1}^- (in^2)	A_{s2}^- (in^2)	A_s^- (in^2)	A'_s (in^2)
A	0.00	33.50	0.00	0.00	0.00	260.92	0.00	0.00	0.00	0.00
B	8.00	33.50	494.17	9.72	0.00	260.92	0.00	0.00	0.00	0.00
C	16.00	33.50	808.95	15.91	0.00	260.92	0.00	0.00	0.00	0.00
D	24.00	33.50	954.72	18.78	0.00	260.92	0.00	0.00	0.00	0.00
E	32.00	34.70	941.58	17.82	-48.75	279.95	0.95	0.00	0.95	0.00
F	40.00	38.30	809.52	13.76	-244.24	341.05	4.33	0.00	4.33	0.00
G	48.00	44.30	513.43	7.46	-518.39	456.28	7.00	0.89	7.89	0.98
H	56.00	52.70	36.94	0.45	-876.29	645.72	8.32	2.76	11.08	2.95
I(L)	64.00	63.50	0.00	0.00	-1326.60	937.50	10.03	3.83	13.86	3.99
I(R)	64.00	63.50	0.00	0.00	-1326.60	937.50	10.03	3.83	13.86	3.99
J	72.00	52.70	0.00	0.00	-617.08	645.72	7.96	0.00	7.96	0.00
K (*)	80.00	44.30 (*)	0.00	0.00	0.00	456.28	Articulation	*	*	*
L	88.00	38.30	461.88	7.85	0.00	341.05	0.00	0.00	0.00	0.00
M	96.00	34.70	703.40	13.31	0.00	279.95	0.00	0.00	0.00	0.00
N	104.00	33.50	769.67	15.14	0.00	260.92	0.00	0.00	0.00	0.00

6. Design of Articulation

The width of the girder will be doubled at the articulation; i.e., $b_a = 30''$, the gradual widening will start at a distance = 6b = 90″. The design parameters are the following,

Weight of the cross-girder = $0.15\times 2 \times(50.8''/12)\times5.75' = 7.30$ k

Design shear force = $V_K = (71.66 + 7.30)$ k = 78.96 k

Length of the articulation, $A_L = 2'$; $\therefore$ Design moment $M_{K(a)} = 78.96 \times 2'/2 = 78.96$ k′

A bearing plate or pad will be provided to transfer the load.

Assume bearing strength = 0.5 ksi $\Rightarrow$ Required bearing area = 78.96/0.5 = 157.92 in^2

$\therefore$ The bearing area is (12″×16″), with thickness = 6″ (assumed for pad)

The depth of girder at K is = 50.8″

$\therefore$ Design depth at articulation = (50.8–6)/2 = 22.4″ $\Rightarrow$ Effective depth $d_K \cong 19.4''$

The required depth from shear $d_{(req)} = V/(2.95\sqrt{f'_c}\, b_a) = 16.29''$, which is <19.4″, OK.

$\therefore$ Stirrup spacing, $S_{(req)} = A_s f_s d/(V–V_c)$

where f_s = 20 ksi. If 2-legged #5 stirrups are used, A_s = 0.62 in^2.

$V_c = 0.95\sqrt{f'_c}\, bd_K = 0.95\sqrt{(0.003)}\ 30\ d_K = 1.561\ d_K$

$\Rightarrow S_{(req)} = 12.4\ d_K /(V–1.561\ d_K) = 12.4\times19.4/(76.22–1.561\times19.4) = 4.94''$

$\therefore$ Provide 2-legged #5 stirrups @4.5″c/c

In addition, inclined bars will be provided for the diagonal cracks. These will be the same size as the main bars and their spacing will be governed by d/2 (of the main girder).

Here, d = 44.3″ $\Rightarrow$ Spacing = 22.15″

Since the length of articulation is 2′ = 24″, provide 2 #10 bars @ 12″c/c

The required depth from bending, $d_{(req)} = \sqrt{(M/Rb_a)} = 13.04''$, which is <19.4″

$\Rightarrow$ Singly reinforced section, with required steel,

$A_s = M/(f_s\, j\, d) = 78.96\times12/(20\times0.883\times19.4) = 2.77$ in^2

These will be adjusted with the main reinforcements in design.

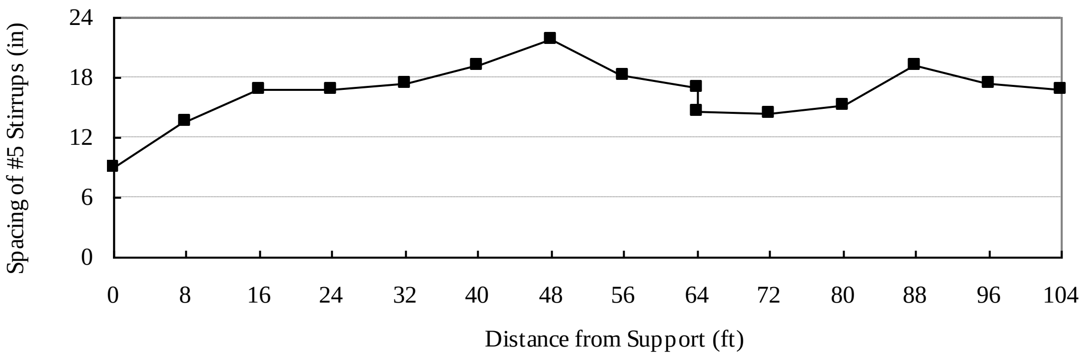


Fig. 1: Design for Shear

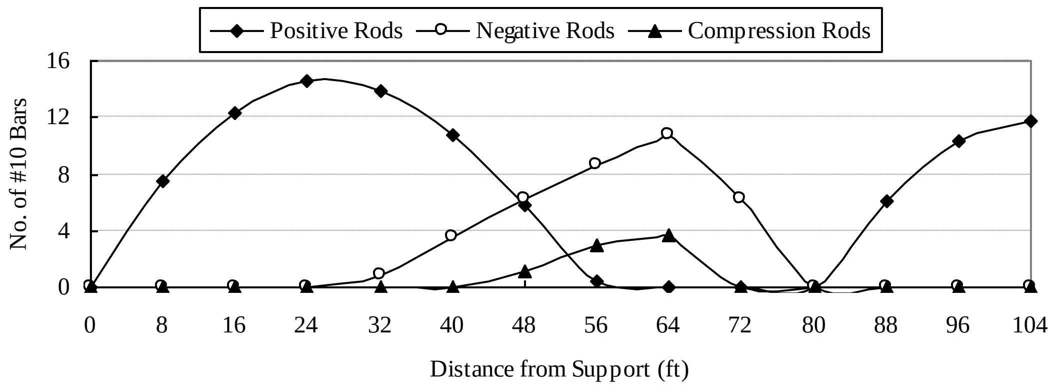
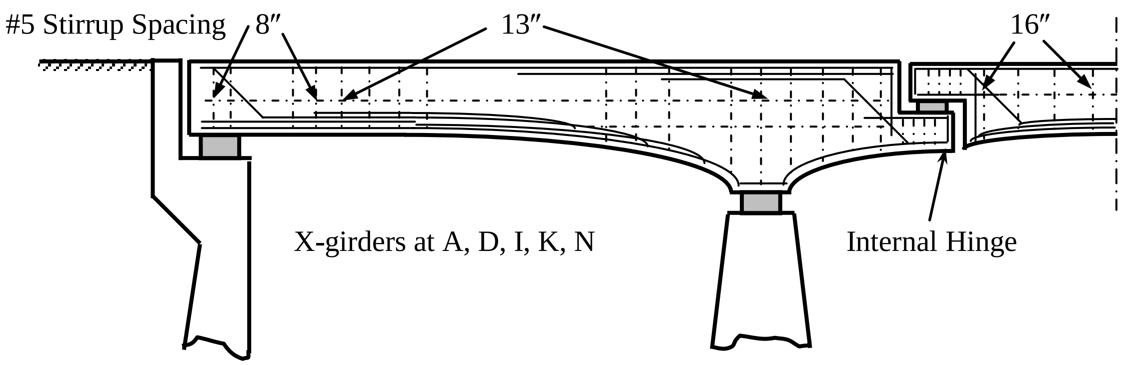
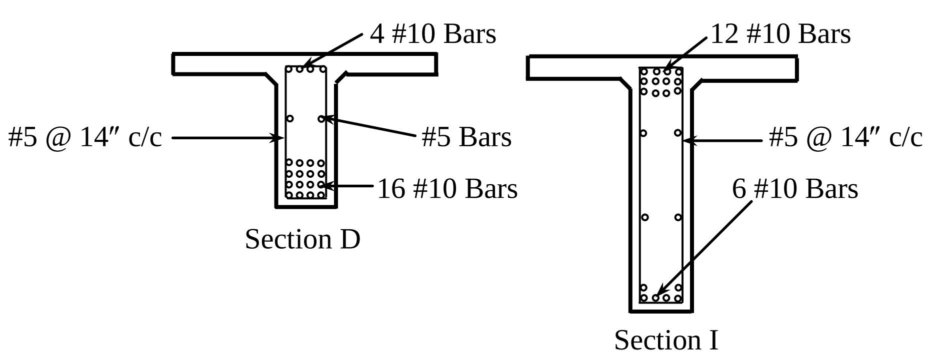
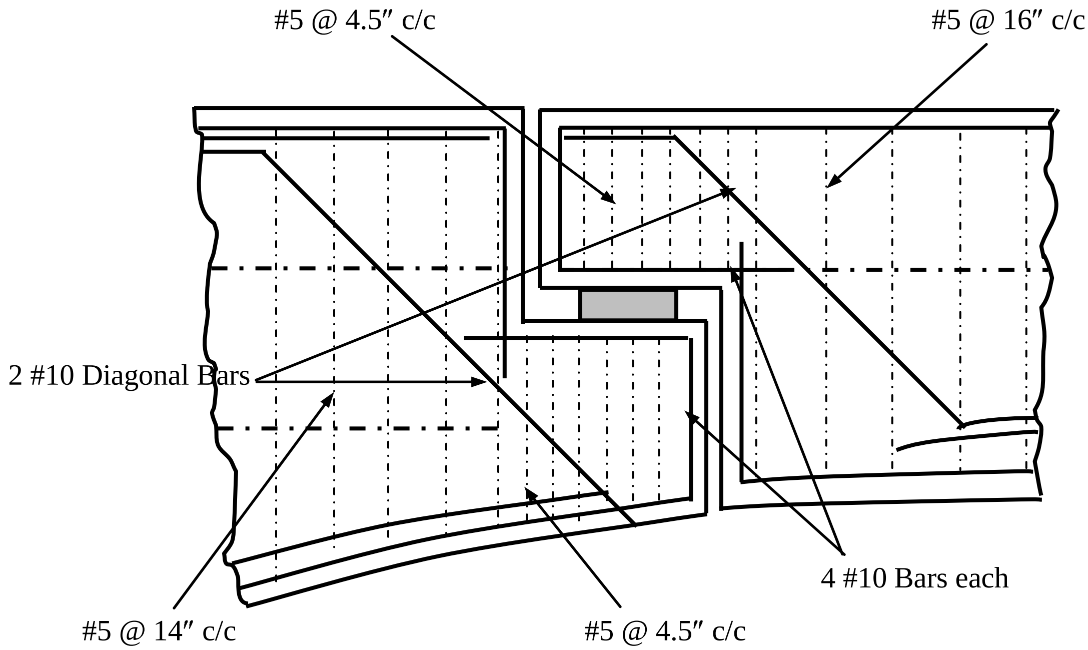


Fig. 2: Design for Moment

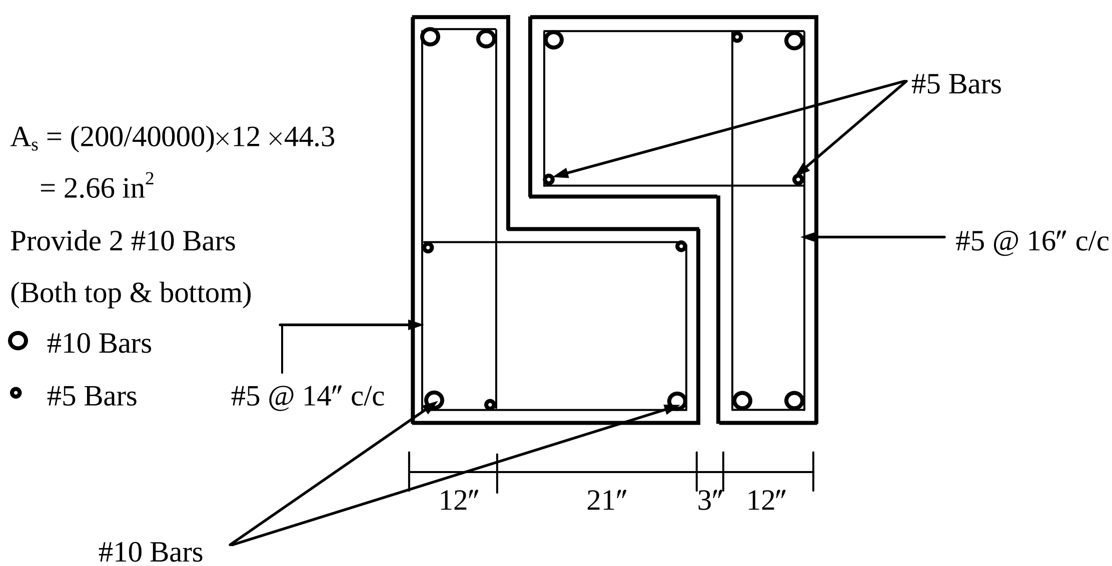
Sections	A	B	C	D	E	F	G	H	I	J	K	L	M	N
Top #10	4	4	4	4	4	6	8	10	12	8	4	4	4	4
Bottom #10	6	10	16	16	16	12	8	6	6	6	6	8	12	12







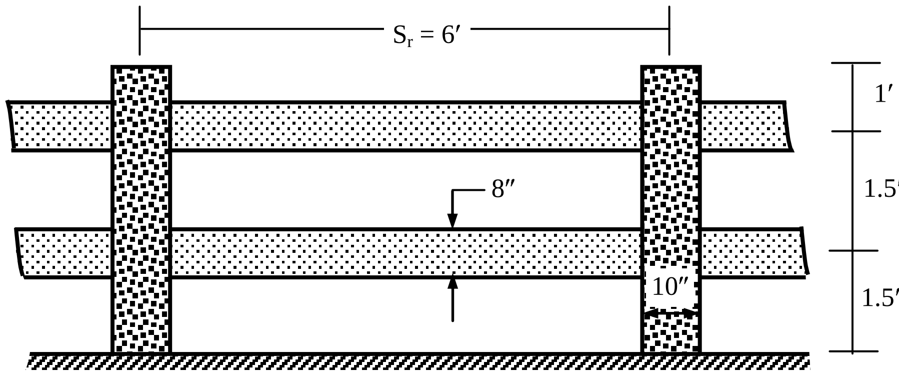
Longitudinal Section of Articulation



Cross-Girder at Articulation

7. Design of Railings and Kerb

The following arrangement is chosen for the railing



Assume (for the assignment)

- Span of Railing, $S_r = L_s/8$, Width (b) of railing section = $(S_r + 2)$ in
- Height of Railpost = $S_r/4 + S_r/4 + S_r/6$, Width (b) of railpost section = $(S_r + 4)$ in

Railing

The assumed load on each railing = 5 k

$\therefore$ Design bending moment $M_{(\pm)} = 0.8\ (PL/4) = 0.8\ (5\times6/4) = 6.0$ k′

If the width b = 8″, $d_{(req)}$ from bending = $\sqrt{(M_{(\pm)}/Rb)} = \sqrt{\{6\times12/(0.197\times8)\}} = 6.76''$

Shear force V = 5.0 k $\Rightarrow d_{(req)}$ from shear = $V/2.95\sqrt{f'_c}\ b = 5.0/(2.95\sqrt{(0.003)} \times 8) = 3.87''$

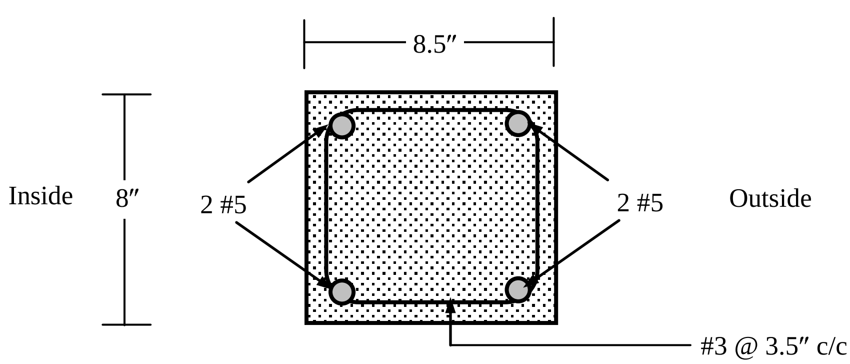
$\therefore$ Assume d = 7″, h = 8.5″

$\therefore A_s = M_{(\pm)}/(f_s jd) = 6\times12/(20\times0.883\times7) = 0.59$ in^2; i.e., use 2 #5 bars at top and bottom

$V_c = 0.95\sqrt{f'_c}\ bd = 0.95\sqrt{(0.003)} \times 8 \times 7 = 2.91$ k

$\therefore$ Spacing of 2-legged #3 stirrups, $S_{(req)} = A_s f_s\ d/(V-V_c) = 0.22\times20\times7/(5.0-2.91) = 14.76''$

$\therefore$ Provide 2-legged #3 stirrups @3.5″c/c (i.e., d/2)



Rail Post

Design bending moment $M_{(-)} = 5\times1.5 + 5\times3.0 = 22.5$ k′

If the width b = 10″, the $d_{(req)}$ from bending $= \sqrt{(M_{(-)}/Rb)} = \sqrt{\{22.5\times12/(0.197\times10)\}} = 11.71''$

Shear force V = 10.0 k

$\Rightarrow d_{(req)}$ from shear $= V/2.95\sqrt{f'_c}\, b = 10.0/(2.95\sqrt{(0.003)} \times 10) = 6.19''$

$\therefore$ Assume d = 12″, h = 13.5″

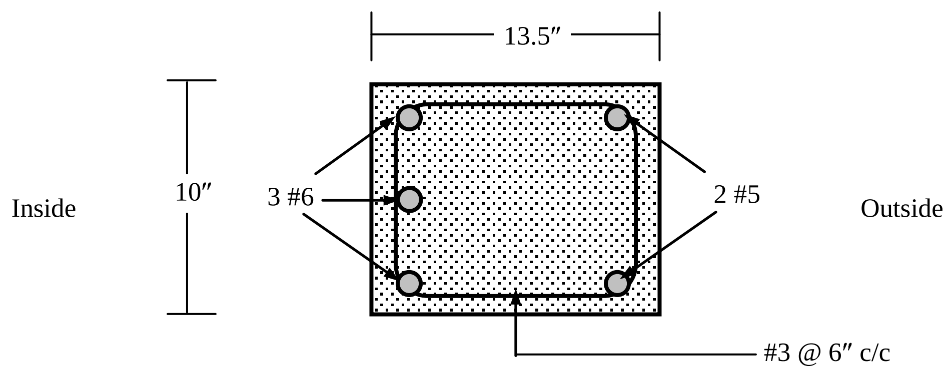
$\therefore A_s = M_{(-)}/(f_s jd) = 22.5\times12/(20\times0.883\times12) = 1.27$ in^2; i.e., use 3 #6 bars inside

$V_c = 0.95\sqrt{f'_c}\, bd = 0.95\sqrt{(0.003)} \times 10 \times 12 = 6.24$ k

If 2-legged #3 stirrups are used, $A_s = 0.22$ in^2

$\therefore$ Stirrup spacing, $S_{(req)} = A_s f_s\, d/(V-V_c) = 0.22\times20\times12/(10.0-6.24) = 14.06''$

$\therefore$ Provide 2-legged #3 stirrups @6″c/c (i.e., d/2)



Edge Slab and Kerb

Edge Slab

Design load = 16 k, assumed width = 4 ft

Design bending moment for edge slab $M_{(-)} = 16/4 \times 18/12 = 6.0$ k′/ft

$d_{(req)} = \sqrt{(M_{(-)}/Rb)} = \sqrt{\{6.0/0.197\}} = 5.52''$

Assuming d = 5.5″, t = 7.5″

$\therefore A_s = M_{(-)}/(f_s jd) = 6.0\times12/(20\times0.883\times5.5) = 0.74$ in^2/ft,

which is greater than the reinforcement (= 0.54 in^2/ft) for the main slab.

There are two alternatives, using

(a) d = 5.5″, t = 7″, with #6 @10″ c/c (like main slab) + one extra #6 after 2 main bars

(b) d = 7.5″, t = 9″, with #6 @10″ c/c (like main slab)

Use $A_{s(temp)} = 0.03t = 0.21$ in^2/ft, or 0.27 in^2/ft, i.e., #5 @14″ c/c or 12″ c/c transversely

Kerb

Design load = 10 k/4′ ⇒ Design bending moment for kerb $M_{(-)} = 10/4 \times 10/12 = 2.08$ k′/ft

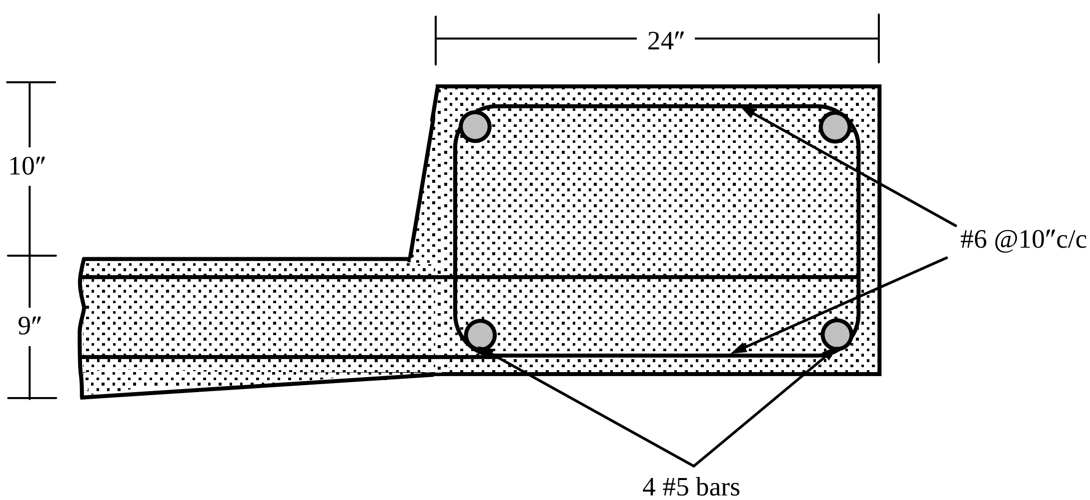
$\therefore$ The required depth, $d_{(req)} = \sqrt{(M_{(-)}/Rb)} = \sqrt{\{2.08/0.197\}} = 3.25'' \ll$ assumed d = 20″, t = 24″

$\therefore A_s = M_{(-)}/(f_s jd) = 2.08\times12/(20\times0.883\times20) = 0.071$ in^2/ft, which is not significant

$\therefore A_{s(temp)} = 0.03\,h = 0.03\times17.5 = 0.525$ in^2/ft

$\therefore$ Provide #6 bars @10″c/c over the span (i.e., consistent with #6 bars @10″c/c for the slab)

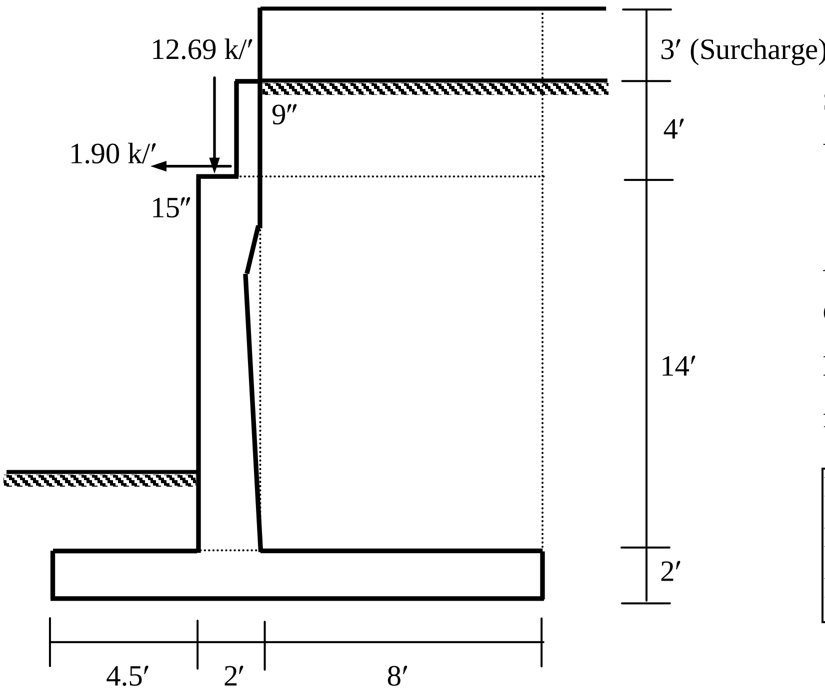
and = 0.525 × 24/12 = 1.05 in^2; i.e., 4 #5 bars within the width of kerb.



8. Design of Substructure

Design of Abutment and Wing Walls

Stability Analysis



Soil Data:

Unit Weight, $\gamma = 120$ lb/ft^3

$= 0.12$ k/ft^3

Angle of Friction, $\phi = 30°$

Coefficient of active pressure

$K_a = (1 - \sin\phi)/(1+\sin\phi) = 1/3$

f between soil and wall = 0.45

For the assignment Multiply all dimensions by Factor F = L/80

The effect of front soil is ignored and wall geometry is simplified (denoted by the dotted lines).

<u>Sliding</u>

Approximate Vertical load = 12.69 + 0.12×{(3+14)×8 + 4×8.75} + 0.15×{14.5×2 + 2×14}

= 12.69 + 16.32 + 4.2 + 4.35 + 4.2 = 41.76 k/′

∴ Horizontal resistance H_R = 0.45×41.76 = 18.79 k/′

Horizontal load H = 1.90 + 0.12×3/3×20 + 0.12×20/3×20/2 = 1.90 + 2.4 + 8.0 = 12.30 k/′

∴Factor of Safety against sliding = H_R/H = 18.79/12.30 = 1.53 > 1.5, OK

<u>Overturning</u>

Approximate resisting moment

M_R = 12.69×(4.5+0.625) + 16.32× (6.5+4) + 4.2×(5.75+4.375)+ 4.35×7.25 + 4.2×5.5

= 336.18 k′/′

Overturning moment M = 1.90×16 + 2.4×20/2 + 8.0×20/3 = 107.78 k′/′

∴Factor of Safety against overturning = M_R/M = 336.18/107.78 = 3.12 > 1.5, OK

Design of Back-wall

Design wheel load = 16 k and assumed loaded length $\cong$ 4′

$\therefore$ Load per unit width V = 16/4 = 4 k/′

Moment per unit width M = 4×2/2 = 4 k′/′

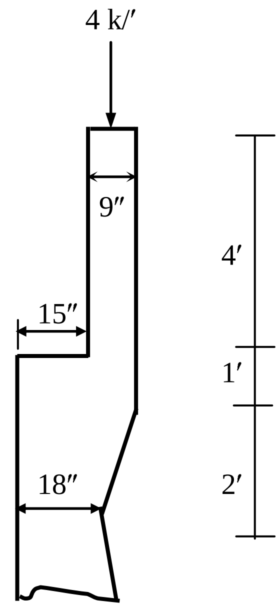
$\therefore d_{(req)}$ for moment = $\sqrt{(4/0.186)}$ = 4.64″

$\therefore$ d = 18–3 = 15″>> $d_{(req)}$

A_s = 4×12/(20×0.883×15) = 0.18 in^2/′

$A_{s(temp)}$ = 0.03×18 = 0.54 in^2/′

This should be adjusted with stem reinforcement.



Design of Stem

Design V/length = 1.9 + 0.12×18 + 0.72×18/2 = 1.9 + 2.16 + 6.48 = 10.54 k/′

Design M/length = 1.9×14 + 2.16×18/2 + 6.48×18/3 = 84.92 k′/′

$\therefore d_{(req)}$ for shear = 10.54/(0.95×$\sqrt{(0.003)}$×12) = 16.88″

and $d_{(req)}$ for moment = $\sqrt{(84.92/0.186)}$ = 21.38″

d = 24–3.5 = 20.5″ < $d_{(req)}$ $\Rightarrow$ t = 25″, d = 21.5″

A_s= 84.92×12/(20×0.883×21.5) = 2.68 in^2/′

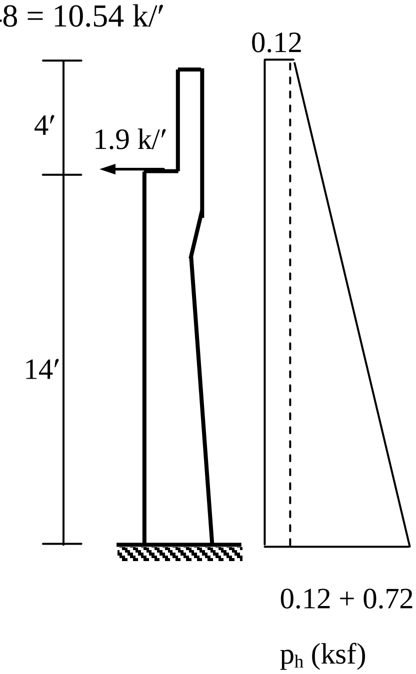
$\therefore$ Use #10 @ 5.5″ c/c (along the length near soil)

$A_{s(temp)}$ = 0.03×25 = 0.75 in^2/′

$\therefore$ Use #5 @ 5″ c/c (along the width and length farther from soil)

This should also be adequate for the back-wall

(both along length and width)



Design of Toe and Heel

Total vertical force on the soil below the wall = 41.76 k/′

The resultant moment about the far end to the toe = M_R–M =336.18–107.78 = 228.40 k′/′

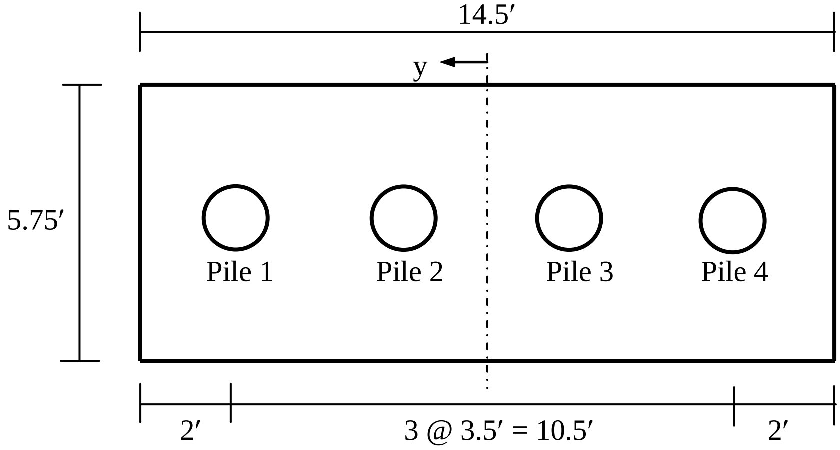
$\therefore e_o$ = 228.40/41.76= 5.47′, which is >14.5/3 and <14.5×2/3 $\Rightarrow$ uplift avoided.

The maximum soil pressure = (41.76/14.5)×[1+6(14.5/2–5.47)/14.5] = 5.00 ksf, which is > 2 ksf.

$\therefore$ Pile foundation is suggested, and the toe and heel should be designed as pile cap.

Design of Piles and Pile-cap

The following arrangement of piles is assumed for the width of the toe and heel (i.e., 14.5′) and within the c/c distance of girders (i.e., width = 5.75′).



Pile Forces

For the total width of one row of piles, $V = 41.76 \text{ k}/' \times 5.75' = 240.10 \text{ k}$

and $M = (41.76 \times 7.25 - 228.40) \text{ k}'/' \times 5.75' = 483.19 \text{ k}'$

Pile reactions are given by $F_i = V/n + My_i/\sum y_i^2$

where n = Number of piles = 4, $\sum y_i^2 = 2\times1.75^2 + 2\times5.25^2 = 61.25 \text{ ft}^2$

$\therefore$ Here, $F_i = 240.10/4 + 483.19\, y_i/61.25 = 60.03 + 7.89\, y_i$

$\therefore F_1 = 60.03 + 7.89 \times 5.25 = 101.44$ k, $F_2 = 60.03 + 7.89 \times 1.75 = 73.83$ k

$F_3 = 60.03 + 7.89 \times (-1.75) = 46.22$ k, $F_4 = 60.03 + 7.89 \times (-5.25) = 18.61$ k

Design of Piles

Using 3% steel, $P = 0.85\,(0.25 f_c' + 0.03\, f_s)\,(\pi/4)\,D^2$

$\Rightarrow 101.44 = 0.85\,(0.25 \times 3 + 0.03 \times 20)\,(\pi/4)\,D^2 \Rightarrow D = 10.61''$

$\therefore$ Use 11″ diameter piles with 6 #5 bars and #5 ties @11″ c/c

Smaller sections can be used for the piles other than Pile 1

The piles should be long enough to transfer the axial loads safely to the surrounding soil.

Assuming the entire pile load for pile 1 to be resisted by skin friction, $F_1 = \alpha_2\,(\pi DL)\,q_a/2$

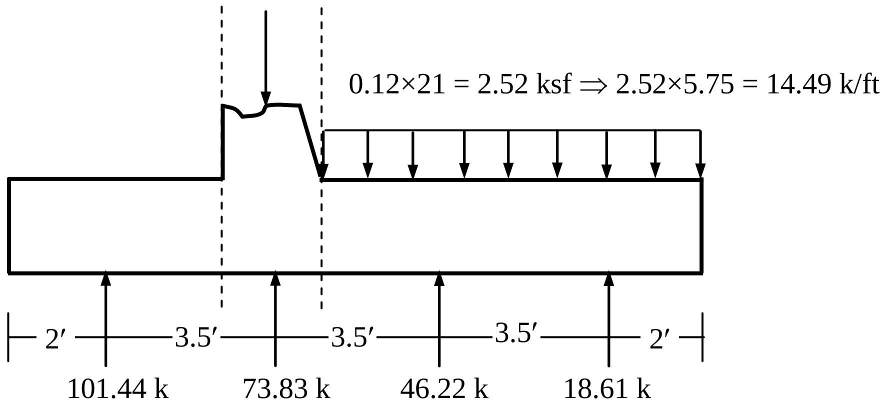
where α_2 = Reduction factor for soil disturbance $\cong 0.8$, D = Pile diameter = 11″

L = Pile length, q_a = Allowable compressive stress on soil = 2 ksf

$\therefore 101.44 = 0.8\times(\pi\times11/12\times L)\times2/2 \Rightarrow L = 44.03'$, $\therefore$ Provide 45′ long piles.

Design of Pile-cap

For the design of pile-cap, the pile loads per girder are considered along with the soil pressure converted to soil load per girder width. The pile loads on the pile-cap for c/c girder width (5.75′) are shown below.



1. Design of Toe

The assumed thickness = 2′ = 24″.

The pile-cap is assumed to have 3″ effective cover and 6″ embedment for piles.

Maximum Punching shear =101.44 k, allowable punching stress = $1.9\times\sqrt{(0.003)} = 0.104$ ksi

Punching area around a 12″ pile = π (11+d) d = 101.44/0.104 $\Rightarrow d_{(req)} \cong 13.3''$; i.e., $t_{(req)} = 22.5''$

Maximum flexural shear =101.44 k, allowable shear stress = $0.95\times\sqrt{(0.003)} = 0.052$ ksi

Shearing area = 5.75×12 d = 101.44/0.052 $\Rightarrow d_{(req)} = 28.25''$; i.e., $t_{(req)} = 37.25''$

Maximum bending moment =101.44×2.5 = 253.60 k′

$\Rightarrow d_{(req)} = \sqrt{\{253.60/(0.186\times5.75)\}} = 15.41''$; i.e., $t_{(req)} = 24.41''$

$\therefore$ t = 38″, d = 29″ $\Rightarrow A_s = 253.60\times12/(20\times0.883\times29) = 5.94$ in^2; i.e., 5.94/5.75 = 1.03 in^2/ft

$A_{s(temp)} = 0.03\times38 = 1.14$ in^2/ft

2. Design of Heel

Maximum flexural shear =14.49×7– 46.22 –18.61= 36.60 k, allowable shear stress = 0.052 ksi

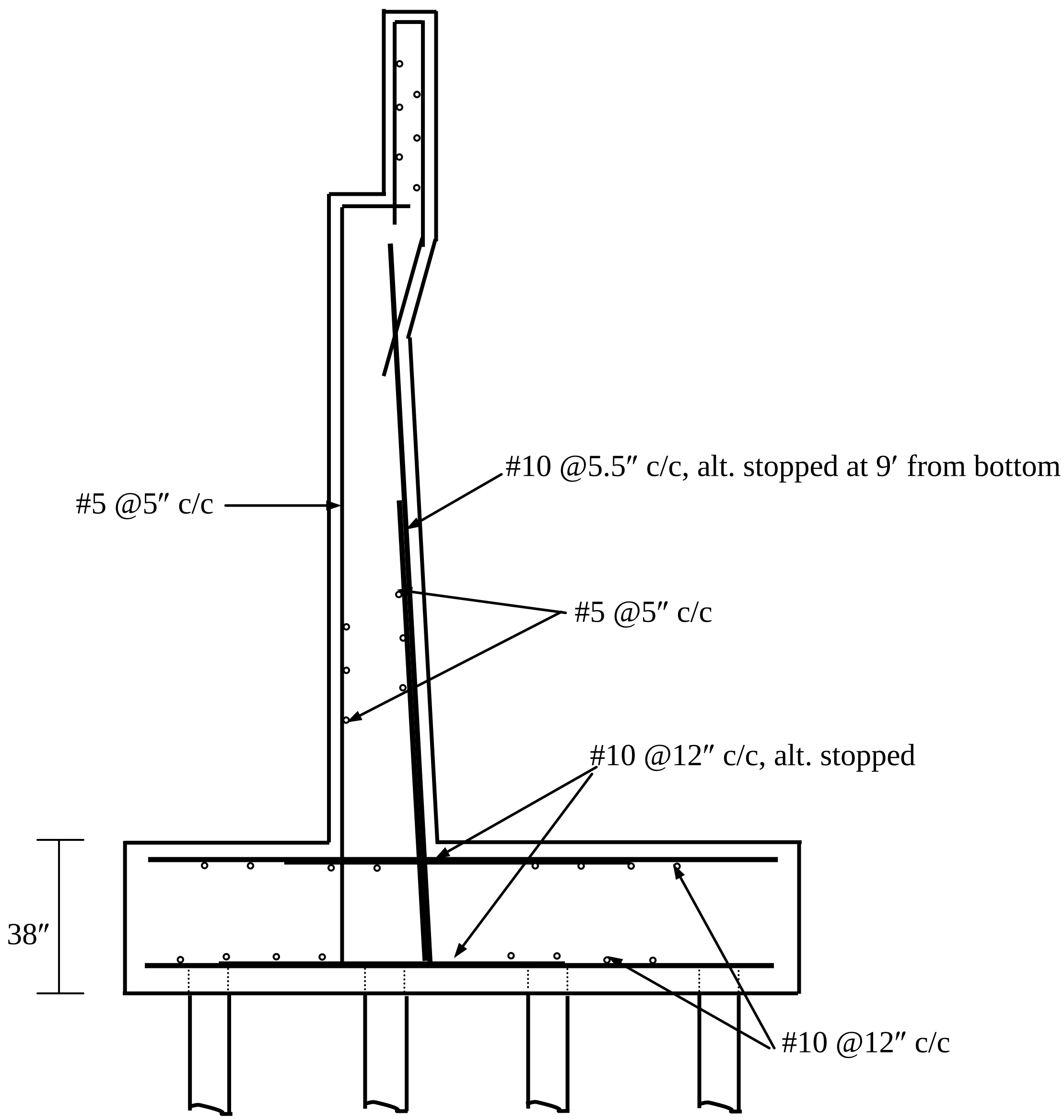
Shearing area = 5.75×12 d = 36.60/0.052 $\Rightarrow d_{(req)} = 10.19''$; i.e., $t_{(req)} = 19.19''$

Maximum bending moment = –14.49×8×8/2 + 46.22×2.5 + 18.61×6 = –236.47 k′

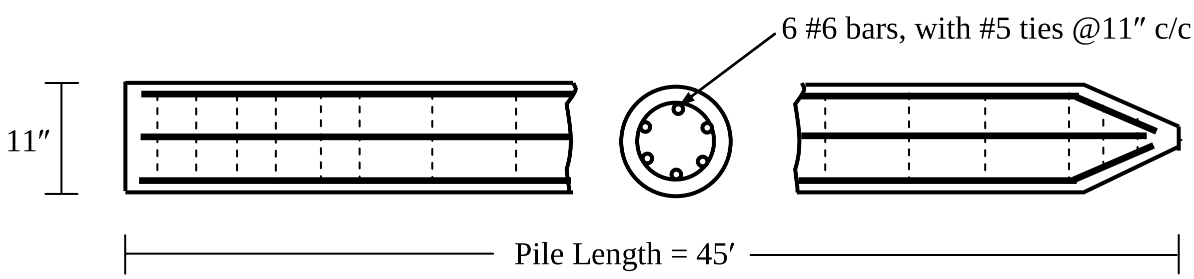
$\Rightarrow d_{(req)} = \sqrt{\{236.47/(0.186\times5.75)\}} = 14.88''$; i.e., $t_{(req)} = 23.88''$

$\therefore$ t = 38″, d = 29″ $\Rightarrow$ It will be OK for punching shear also

$\therefore A_s = 236.47\times12/(20\times0.883\times29) = 5.54$ in^2; i.e., 0.96 in^2/ft, $A_{s(temp)} = 0.03\times38 = 1.14$ in^2/ft



Abutment Reinforcements



Pile Reinforcements

5. Design of Interior Girders (USD)

Shear Design

The shear design of interior girders is performed by using the conventional shear design equations of RCC members. The stirrup spacing is given by the equation

$$S_{(req)} = A_s f_y\, d/(V_u/\phi - V_c)$$

where f_y = 40 ksi. If 2-legged #5 stirrups are used, $A_s = 0.62$ in^2.

$$V_c = 1.9\sqrt{f'_c}\, bd = 1.9\sqrt{(0.003)}\ 15\ d = 1.561\ d$$

$$S_{(req)} = 24.8\ d/(V_u/0.85 - 1.561\ d)$$

$$d_{(req)} = V/(5.9\phi\sqrt{f'_c}\, b_w) = V/4.120$$

where d and V vary from section to section.

ACI recommends that the maximum stirrup spacing (S) shouldn't exceed

$$d/2, \text{ or } 24'' \text{ or } A_s f_y/0.06 b_w = 24.8/0.90 = 27.56''$$

The calculations are carried out in tabular form and listed below.

It is convenient to perform these calculations in EXCEL.

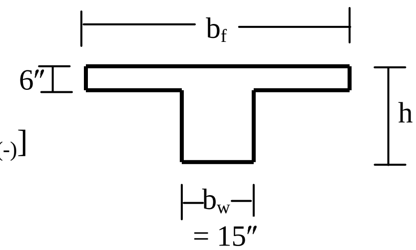
Table 5.1 Design for Shear Force

Section	x from left (′)	h (″)	d (″)	V_u (kips)	$d_{(req)}$ (″)	$S_{(req)}$ from formula (″)	$S_{(provided)}$ (″)
A	0.00	40.00	33.50	115.86	28.12	9.89	8.0
B	8.00	40.00	33.50	91.81	22.28	14.91	13.0
C	16.00	40.00	33.50	67.74	16.44	16.75	16.0
D	24.00	40.00	33.50	43.69	10.60	16.75	16.0
E	32.00	41.20	34.70	-44.73	10.86	17.35	16.0
F	40.00	44.80	38.30	-69.31	16.82	19.15	16.0
G	48.00	50.80	44.30	-94.72	22.99	22.15	20.0
H	56.00	59.20	52.70	-121.42	29.47	21.57	20.0
I(L)	64.00	70.00	63.50	-149.77	36.35	20.43	20.0
I(R)	64.00	70.00	63.50	160.70	39.00	17.51	16.0
J	72.00	59.20	52.70	136.95	33.24	16.57	16.0
K(*)	80.00	50.80	44.30 (*)	113.22	Articulation	*	*
L	88.00	44.80	38.30	84.02	20.39	19.15	16.0
M	96.00	41.20	34.70	55.66	13.51	17.35	16.0
N	104.00	40.00	33.50	28.58	6.94	16.75	16.0

Flexural Design

$p_{max} = (0.75\ \alpha f'_c/f_y)\ [87/(87+f_y)] = 0.0277$, $R_u = \phi\ p_{max} f_y\ [1 - 0.59 p_{max} f_y/f'_c] = 0.781$ ksi

The section shown is chosen for all the beams.



$\therefore$ For 3 layers of rods, $d = h - 6.5 = 13.5''$, $d' = 2.5''$

All the beams are likely to be Singly Reinforced [i.e., $M_{c(-)} > M_{(-)}$]

$A_{s(-)} = (f_c/f_y)[1 - \sqrt{\{1 - 2M_{(-)}/(\phi f_c b_w d^2)\}}]b_w d$

$= (2.55/40)\times[1 - \sqrt{\{1 - 2M_{(-)}\times 12/(0.9\times 2.55\times 15\times d^2)\}}]\times 15\times d$

$= 0.956d\ [1 - \sqrt{\{1 - 0.697M_{(-)}/d^2\}}]$

For T-beams (possible for positive moments), b_f is the minimum of

(i) $16t_f + b_w = 111''$, (ii) Simple Span/4 = $0.6L\times 12/4 = 1.8L = 144''$, (iii) c/c = $69''$

$\Rightarrow b_f = 69''$; i.e.,

$A_{s(+)} = (f_c/f_y)[1 - \sqrt{\{1 - 2M_{(+)}/(\phi f_c b_f d^2)\}}]b_f\, d$

$= (2.55/40)\times[1 - \sqrt{\{1 - 2M_{(+)}\times 12/(0.9\times 2.55\times 69\times d^2)\}}]\times 69\times d = 4.40\ d\ [1 - \sqrt{\{1 - 0.152M_{(+)}/d^2\}}]$

Development Length of #10 bars = $0.04\times 1.27\times 40/\sqrt{0.03}\times 1.4 = 51.94''$

These calculations, performed in EXCEL, are listed below.

Table 5.2 Design for Bending Moment

Section	x from left (′)	d (″)	M_+ (k′)	A_{s+} (in^2)	M_- (k′)	M_{c-} (k′)	A_{s-} (in^2)	N_{s+}	N_{s-}	N_{bot}	N_{top}
A	0.00	33.50	0.00	0.00	0.00	1095.60	0.00	0.00	0.00	6	4
B	8.00	33.50	785.24	8.03	0.00	1095.60	0.00	6.32	0.00	8	4
C	16.00	33.50	1287.30	13.42	0.00	1095.60	0.00	10.57	0.00	12	4
D	24.00	33.50	1523.85	16.04	0.00	1095.60	0.00	12.63	0.00	14	4
E	32.00	34.70	1512.12	15.29	-171.44	1175.49	1.69	12.04	1.33	14	4
F	40.00	38.30	1320.57	11.91	-470.83	1432.05	4.36	9.38	3.43	12	6
G	48.00	44.30	873.58	6.69	-880.52	1915.88	7.24	5.27	5.70	8	8
H	56.00	52.70	145.12	0.92	-1407.38	2711.33	9.87	0.72	7.77	4	10
I(L)	64.00	63.50	0.00	0.00	-2063.60	3936.48	12.02	0.00	9.47	4	10
I(R)	64.00	63.50	0.00	0.00	-2063.60	3936.48	12.02	0.00	9.47	4	10
J	72.00	52.70	0.00	0.00	-967.09	2711.33	6.54	0.00	5.15	4	10
K (*)	80.00	44.30 (*)	*	*	*	*	*	*	*	*	*
L	88.00	38.30	728.47	6.46	0.00	1432.05	0.00	5.09	0.00	8	4
M	96.00	34.70	1105.73	11.02	0.00	1175.49	0.00	8.68	0.00	10	4
N	104.00	33.50	1207.41	12.55	0.00	1095.60	0.00	9.88	0.00	10	4

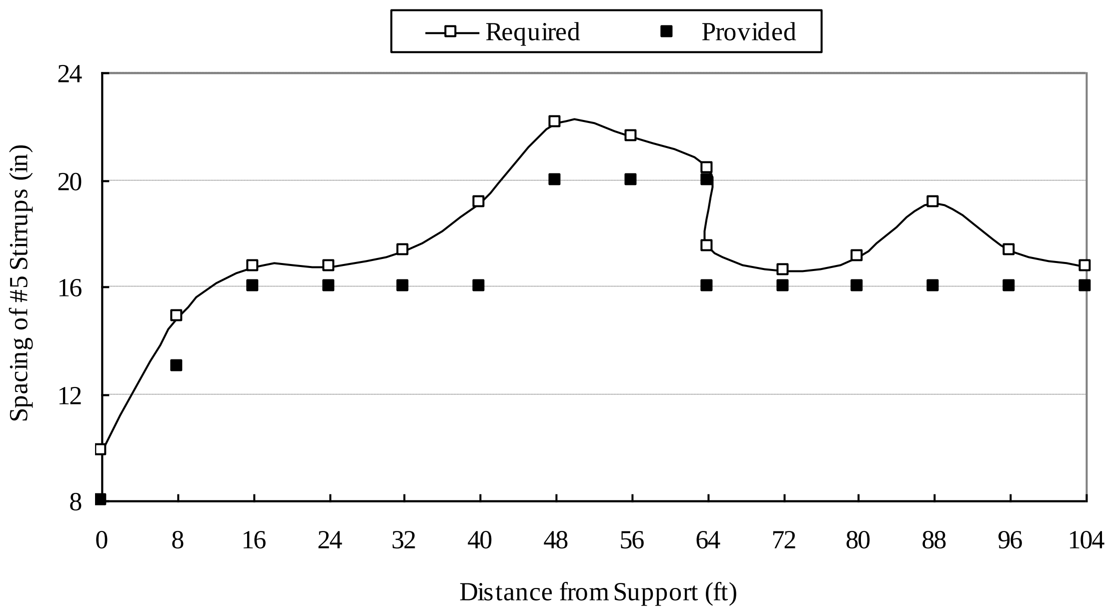


Fig. 1: Design for Shear (USD)

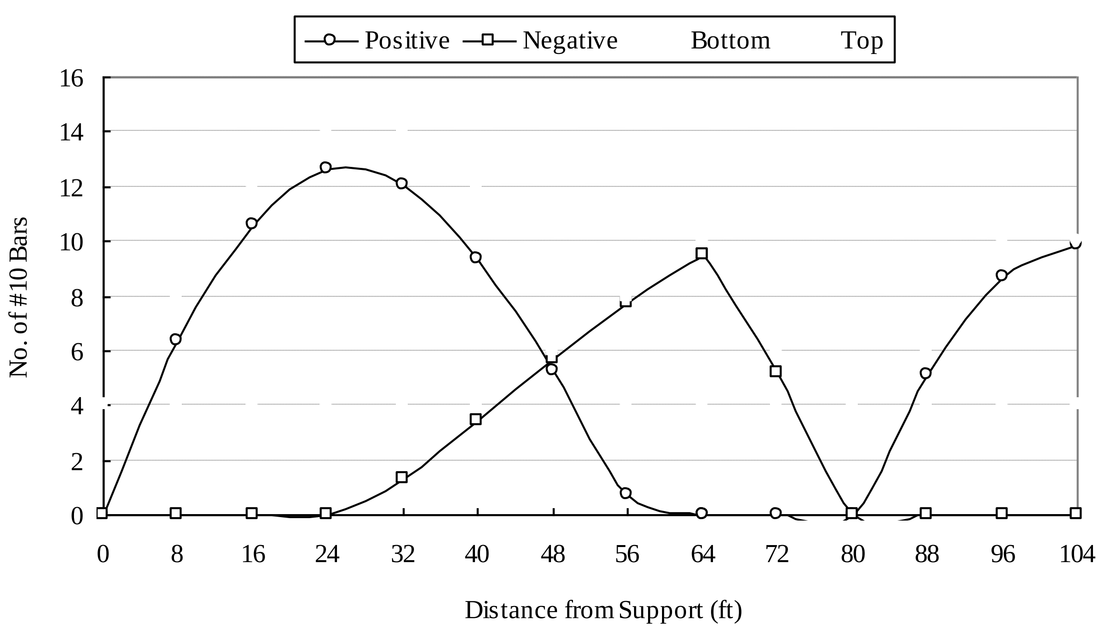
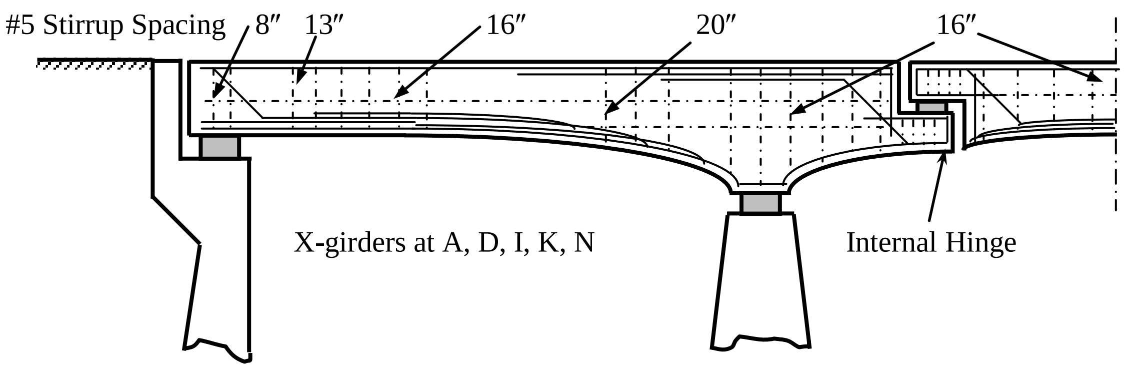
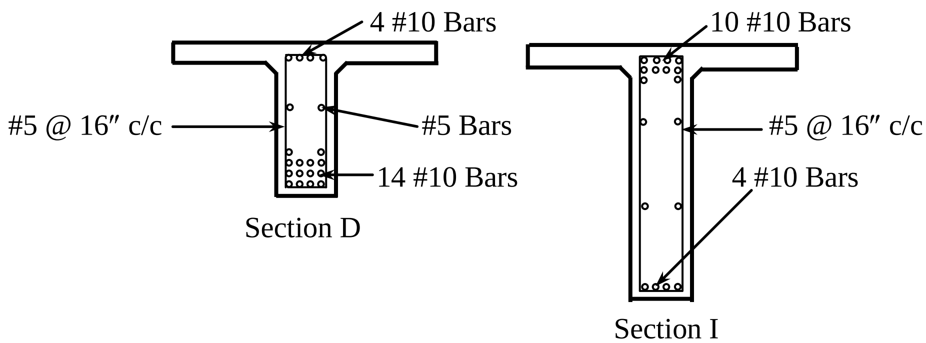


Fig. 2: Design for Moment (USD)

Sections	A	B	C	D	E	F	G	H	I	J	K	L	M	N
Top #10	4	4	4	4	4	6	8	10	10	10	*	4	4	4
Bottom #10	6	8	12	14	14	12	8	4	4	4	*	8	10	10
#5 Stirrup S	8	13	16	16	16	16	20	20	16	16	*	16	16	16





6. Design of Articulation (USD)

The width of the girder will be doubled at the articulation; i.e., $b_a = 30''$, the gradual widening will start at a distance = 6b = 90″. Length of the articulation, $A_L = 2'$.

Reaction from interior girder = 113.22 k

Weight of the cross-girder = 0.15× 2 ×(50.8″/12) ×5.75′× 1.4 = 10.23 k

Design shear force = V_K = (113.22 + 10.23) k = 123.45 k

$\therefore A_L = 2' \Rightarrow$ Design moment $M_{K(a)} = 123.45 \times 2'/2 = 123.45$ k′

A bearing plate or pad will be provided to transfer the load.

Assume bearing strength = 1.0 ksi ⇒ Required bearing area = 123.45/1.0 =123.45 in^2

∴ The bearing area is (12″×12″), with thickness = 6″ (assumed for pad)

The depth of girder at K is = 50.8″

∴ Design depth at articulation = (50.8–6)/2 = 22.4″⇒ Effective depth $d_K \cong 19.4''$

The required depth from shear $d_{(req)} = V_K/(5.9\phi\sqrt{f'_c}\, b_a) = 12.73''$, which is <19.4″, OK.

∴ Stirrup spacing, $S_{(req)} = A_s f_y\, d/(V_u/\phi - V_c)$

If 2-legged #5 stirrups are used, $A_s = 0.62$ in^2.

$V_c = 1.9\sqrt{f'_c}\, bd_K = 1.9\sqrt{(0.003)}\, 30\, d_K = 3.122\, d_K$

$\Rightarrow S_{(req)} = 24.8\, d_K/(V_K/0.85 - 3.122\, d_K) = 24.8 \times 19.4/(145.23 - 3.122 \times 19.4) = 5.68''$

∴ Provide 2-legged #5 stirrups @5.5″c/c

In addition, inclined bars will be provided for the diagonal cracks. These will be the same size as the main bars and their spacing will be governed by d/2 (of the main girder).

Here, d = 44.3″⇒ Spacing = 22.15″

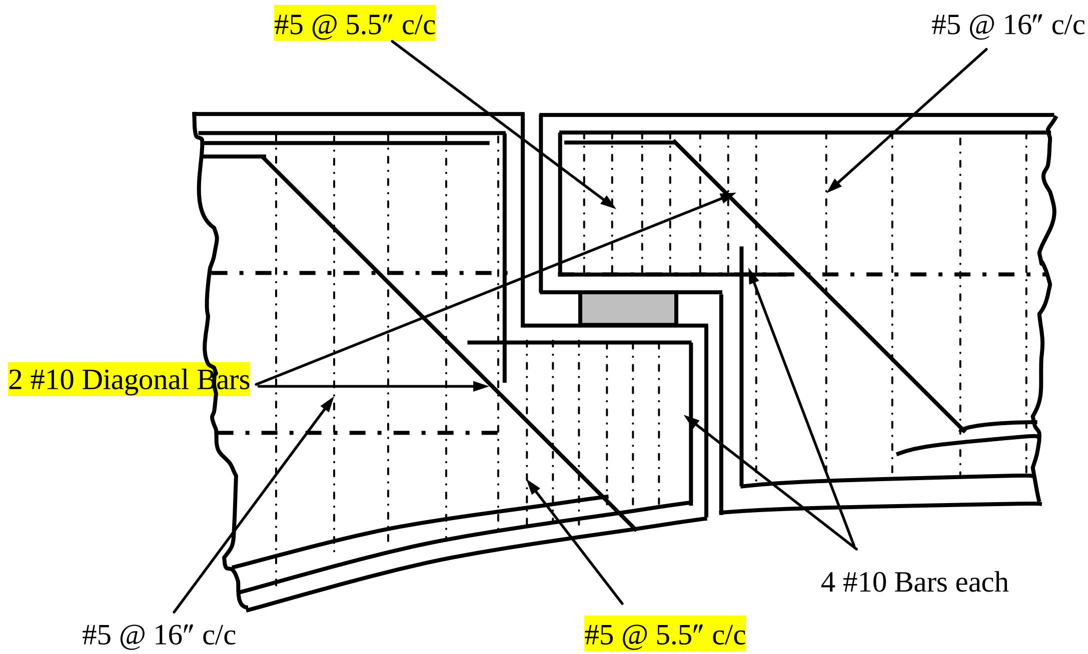
Since the length of articulation is 2′ = 24″, provide 2 #10 bars @ 12″c/c

The required depth from bending, $d_{(req)} = \sqrt{(M_{K(a)}/R_u b_a)} = 7.95''$, which is <19.4″

⇒ Singly reinforced section, with required steel,

$A_s = (f_c/f_y)[1-\sqrt{\{1-2M_{K(a)}/(\phi f_c b_a d^2)\}}]\, b_a d = 2.19$ in^2

These will be adjusted with the main reinforcements in design.



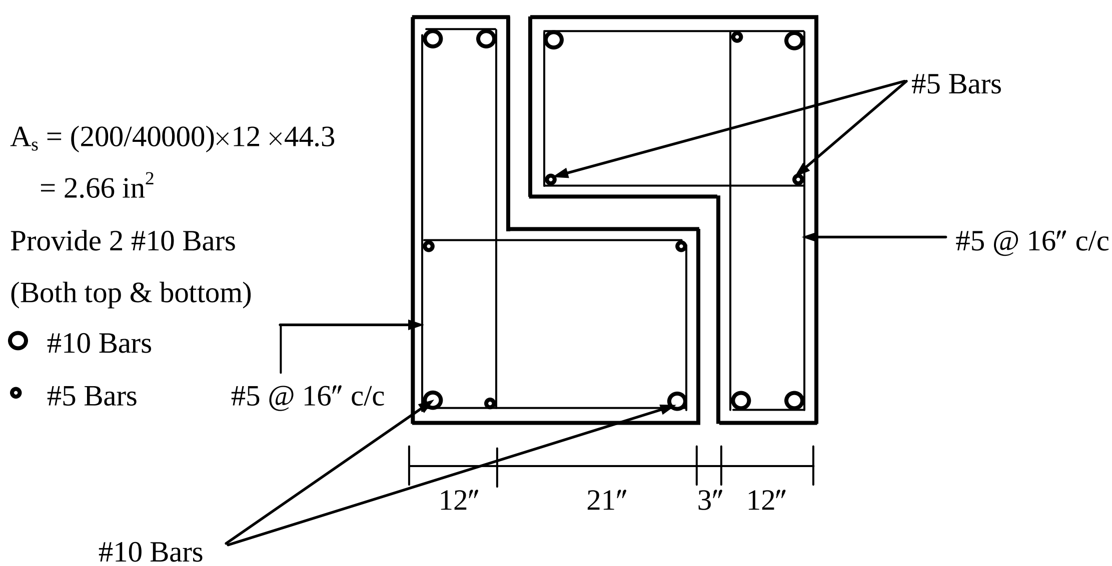
Longitudinal Section of Articulation

A_s = (200/40000)×12 ×44.3

= 2.66 in^2

Provide 2 #10 Bars

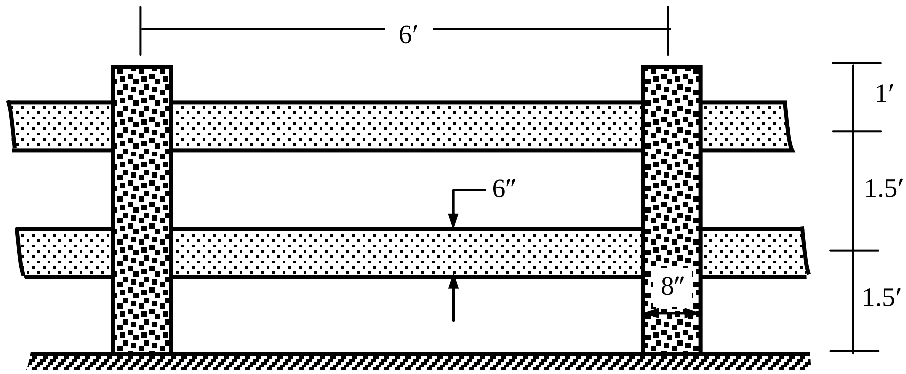
(Both top & bottom)



Cross-Girder at Articulation

7. Design of Railing and Rail Post (USD)

The following arrangement is chosen for the railing



Railing

The assumed load on each railing = 5 k

$\therefore$ Design bending moment $M_{(\pm)}$ = 0.8 (PL/4) = 0.8 (1.7×5×6/4) = 10.2 k′

If the width b = 6″, the required depth from bending is

$d_{(req)} = \sqrt{(M_{(\pm)}/R_u b)} = \sqrt{\{10.2\times12/(0.781\times6)\}}$ = 5.11″; i.e., assume d = 6″, t = 7.5″

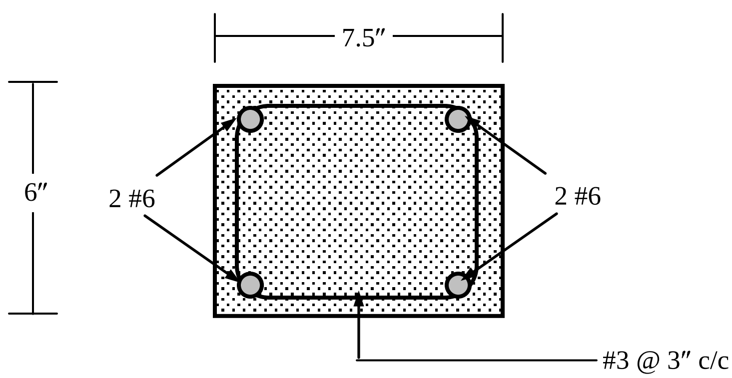
$\therefore A_s = (f_c/f_y)[1-\sqrt{\{1-2M_{(\pm)}/(\phi f_c bd^2)\}}]\ bd = 0.662\ in^2$; i.e., use 2 #6 bars at top and bottom

Shear force $V_u = 1.7 \times 5 = 8.5$ k, $V_c = 1.9\sqrt{f'_c}\ bd = 1.9\sqrt{(0.003)} \times 6 \times 6 = 3.75$ k

If 2-legged #3 stirrups are used, $A_s = 0.22\ in^2$

$\therefore$ Stirrup spacing, $S_{(req)} = A_s f_y\, d/(V_u/\phi - V_c) = 0.22\times40\times6/(8.5/0.85-3.75) = 8.44″$

$\therefore$ Provide 2-legged #3 stirrups @3″c/c (i.e., d/2)



Rail Post

Design bending moment $M_{(-)} = 1.7\times5\times1.5 + 1.7\times5\times3.0 = 38.25$ k′

If the width b = 6″, the required depth from bending is

$d_{(req)} = \sqrt{(M_{(-)}/R_u b)} = \sqrt{\{38.25\times12/(0.781\times8)\}} = 8.57$″; i.e., assume d = 10″, t = 11.5″

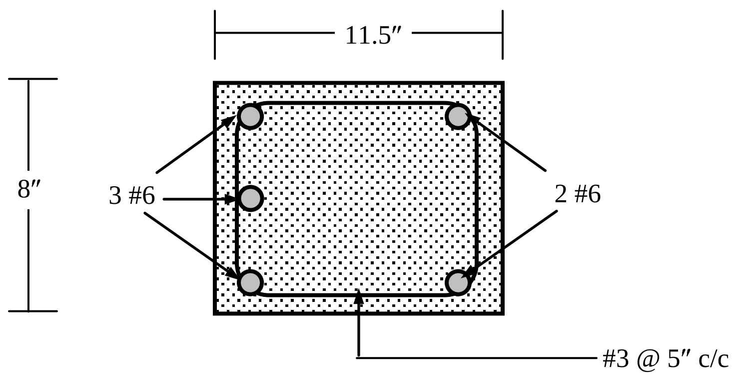
$\therefore A_s = (f_c/f_y)[1-\sqrt{\{1-2M_{(-)}/(\phi f_c bd^2)\}}]\ bd = 1.494\ \text{in}^2$; i.e., use 3 #6 bars inside

Shear force $V_u = 1.7 \times 10 = 17$ k, $V_c = 1.9\sqrt{f'_c}\ bd = 1.9\sqrt{(0.003)} \times 8 \times 10 = 8.33$ k

If 2-legged #3 stirrups are used, $A_s = 0.22\ \text{in}^2$

$\therefore$ Stirrup spacing, $S_{(req)} = A_s f_y\, d/(V_u/\phi - V_c) = 0.22\times40\times10/(17/0.85-8.33) = 7.54$″

$\therefore$ Provide 2-legged #3 stirrups @5″c/c (i.e., d/2)



Kerb

Design bending moment $M_{(-)} = 1.7\times5/4'\times(12/12) = 2.13$ k′/′

If the width b = 12″, the required depth from bending is

$d_{(req)} = \sqrt{}(M_{(-)}/R_u b) = \sqrt{}\{2.13\times12/(0.781\times12)\}<$ assumed d = 7.5″, t = 9″

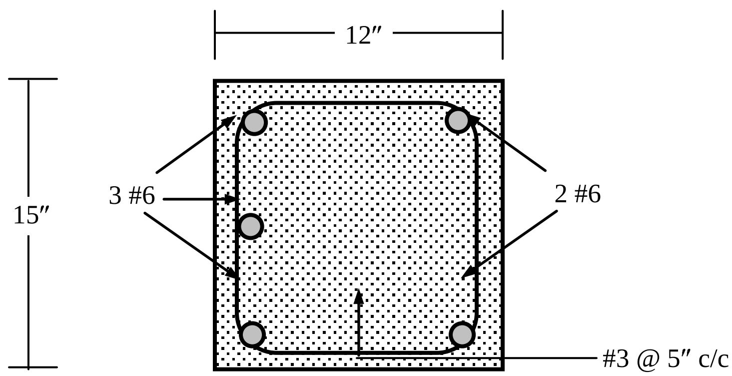
$\therefore A_s = (f_c/f_y)[1-\sqrt{}\{1-2M_{(-)}/(\phi f_c bd^2)\}]$ bd = in^2; i.e., use 3 #6 bars inside

Shear force $V_u = 1.7 \times 10 = 17$ k, $V_c = 1.9\sqrt{}f'_c$ bd = $1.9\sqrt{}(0.003) \times 8 \times 10 = 8.33$ k

If 2-legged #3 stirrups are used, $A_s = 0.22$ in^2

$\therefore$ Stirrup spacing, $S_{(req)} = A_s f_y\, d/(V_u/\phi - V_c) = 0.22\times40\times10/(17/0.85-8.33) = 7.54''$

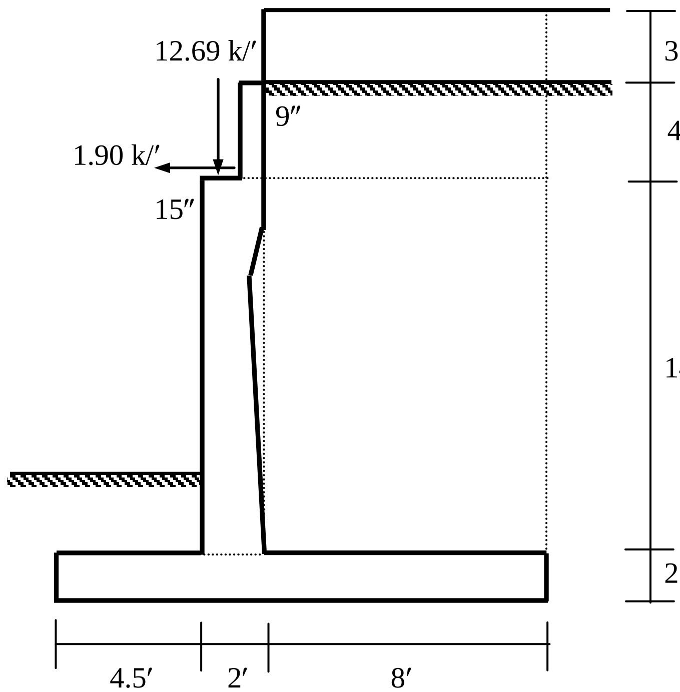
$\therefore$ Provide 2-legged #3 stirrups @5″c/c (i.e., d/2)



8. Design of Substructure

Design of Abutment and Wing Walls

Stability Analysis (Using Working Load and Allowable Pressure)



Soil Data:

Unit Weight, γ = 120 lb/ft^3

= 0.12 k/ft^3

Angle of Friction, ϕ = 30°

Coefficient of active pressure

K_a= (1– sin ϕ)/(1+sin ϕ) = 1/3

f between soil and wall = 0.45

The effect of front soil is ignored and wall geometry is simplified (denoted by the dotted lines).

<u>Sliding</u>

Approximate Vertical load = 12.69 + 0.12×{(3+14)×8 + 4×8.75} + 0.15×{14.5×2 + 2×14}

= 12.69 + 16.32 + 4.2 + 4.35 + 4.2 = 41.76 k/′

∴Horizontal resistance H_R = 0.45×41.76 = 18.79 k/′

Horizontal load H = 1.90 + 0.12×(3×20 + 20×20/2)/3 = 1.90 + 2.4 + 8.0 = 12.30 k/′

∴Factor of Safety against sliding = H_R/H = 18.79/12.30 = 1.53>1.5, OK

<u>Overturning</u>

Approximate resisting moment

M_R = 12.69×(4.5+0.625) + 16.32× (6.5+4) + 4.2×(5.75+4.375) + 4.35×7.25 + 4.2×5.5

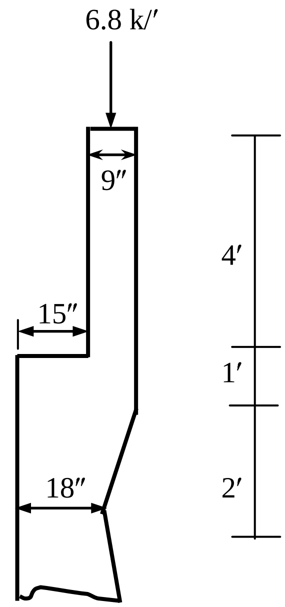
= 329.36 k′/′

Overturning moment M = 1.90×16 + 2.4×20/2 + 8.0×20/3 = 107.73 k′/′

∴Factor of Safety against overturning = M_R/M = 329.36/107.73 = 3.06>2.5, OK

Design of Back-wall

Design wheel load = $16\times1.7 = 27.2$ k, assumed loaded length $\cong 4'$



$\therefore$ Load per unit width V = $27.2/4 = 6.8$ k/′

Moment per unit width M = $6.8\times(15/2 + 9/2)/12 = 6.8$ k′/′

$\therefore d_{(req)}$ for shear = $6.8/(1.9\times0.85\times\sqrt{(0.003)}\times12) = 6.8/1.062 = 6.41''$

and $d_{(req)}$ for moment = $\sqrt{(6.8/0.781)} = 2.95''$

$\therefore d = 18–3 = 15'' >> d_{(req)}$

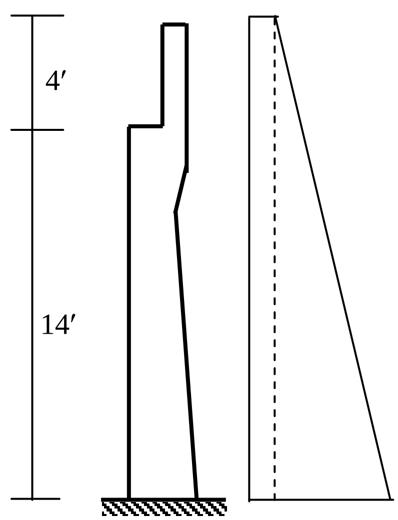
$\therefore A_s = (f_c/f_y)[1–\sqrt{\{1–2M/(\phi f_c bd^2)\}}]\ bd = 0.15$ in^2/′

$A_{s(temp)} = 0.03\times18 = 0.54$ in^2/′

This should be adjusted with stem reinforcement

Design of Stem

Design shear/length = $(0.12\times18 + 0.72\times18/2)\times1.4 = 3.02 + 9.07 = 12.09$ k/′



Design moment/length = $3.02\times18/2 + 9.07\times18/3 = 81.60$ k′/′

$\therefore d_{(req)}$ for shear = $12.09/1.062 = 11.39''$

and $d_{(req)}$ for moment = $\sqrt{(81.60/0.781)} = 10.22''$

$d = 24–3.5 = 20.5'' > d_{(req)}$

$\therefore A_s = (f_c/f_y)[1–\sqrt{\{1–2M/(\phi f_c bd^2)\}}]\ bd = 1.39$ in^2/′

$\therefore$ Use #10 @ 10.5″ c/c (along the length near soil)

$A_{s(temp)} = 0.03\times24 = 0.72$ in^2/′

$\therefore$ Use #5 @ 5″ c/c (along the width and length farther from soil)

This should also be adequate for the back-wall

(both along length and width)

Design of Toe and Heel

Total vertical force on the soil below the wall = 41.76 k/′

The resultant moment about the far end to the toe = $M_R – M = 329.36 – 107.73 = 221.63$ k′/′

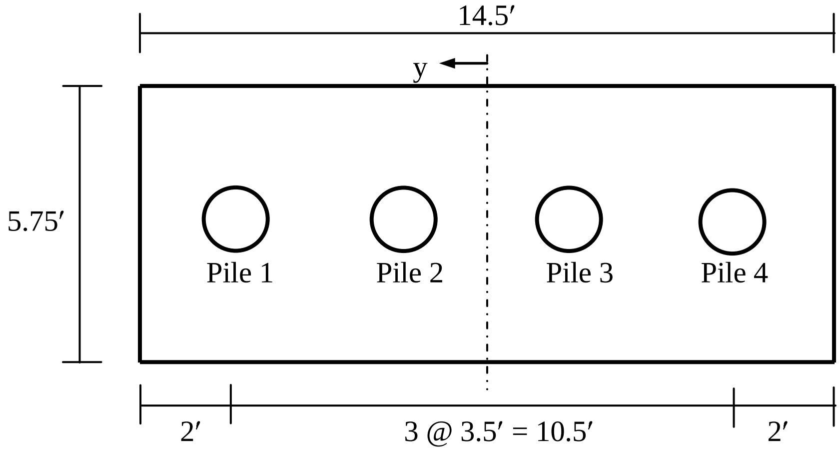
$\therefore e_0 = 221.63/41.76 = 5.31'$, which is $>14.5/3$ and $<14.5\times2/3 \Rightarrow$ uplift avoided.

The maximum soil pressure = $(41.76/14.5)\times[1+6(14.5/2–5.31)/14.5] = 5.20$ ksf, which is >2 ksf.

$\therefore$ Pile foundation is suggested, and the toe and heel should be designed as pile cap.

Design of Piles and Pile-cap

The following arrangement of piles is assumed for the width of the toe and heel (i.e., 14.5′) and within the c/c distance of girders (i.e., width = 5.75′).



Pile Forces

Vertical force and resultant moment in USD are = 60.84 k/′ and = 473.27 – 156.61 = 316.67 k′/′

For the total width of one row of piles, V = 60.84 k/′×5.75′ = 349.82 k

and M = (60.84 × 7.25 – 316.67) k′/′×5.75′ = 715.38 k′

Pile reactions are given by $F_i = V/n + My_i/\sum y_i^2$

where n = Number of piles = 4, $\sum y_i^2 = 2\times1.75^2 + 2\times5.25^2 = 61.25$ ft^2

$\therefore$ Here, $F_i = 349.82/4 + 715.38\, y_i/61.25 = 87.46 + 11.68\, y_i$

$\therefore$ $F_1 = 87.46 + 11.68 \times 5.25 = 148.77$ k, $F_2 = 87.46 + 11.68 \times 1.75 = 107.90$ k

$F_3 = 87.46 + 11.68 \times (-1.75) = 67.02$ k, $F_4 = 87.46 + 11.68 \times (-5.25) = 26.14$ k

Design of Piles

For the load F_1, $148.77 = 0.80\phi\, A_g\{f_c + p\,(f_y - f_c)\} = 0.80 \times 0.70\, A_g\,\{2.55 + 0.03\times(40-2.55)\}$

$\therefore A_g = 72.32$ in^2 $\Rightarrow$ D = 9.60″; i.e., Provide 10″-dia piles with 4 #6 bars and #3 ties @10″ c/c

The same pile will be used for all piles though smaller sections can be used for the others

The piles should be long enough to transfer the axial loads safely to the surrounding soil by skin friction and end bearing.

Assuming the entire pile load for Pile1 to be resisted by skin friction, $F_{1(WSD)} = \alpha_2\, (\pi DL)\, q_a/2$

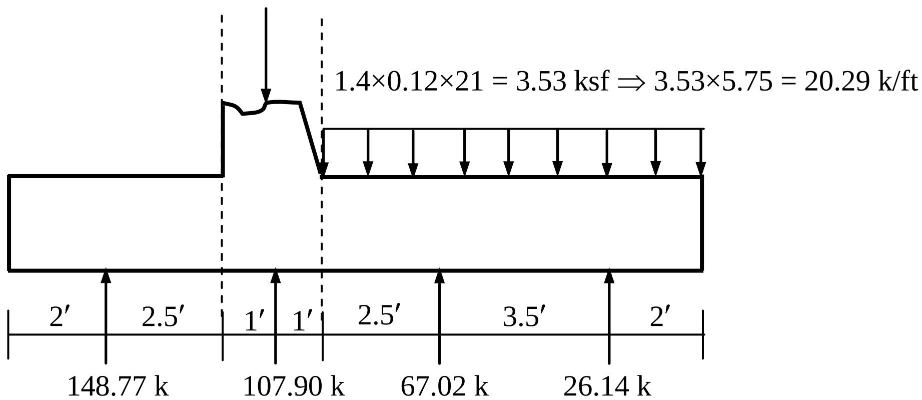
where α_2 = Reduction factor for soil disturbance $\cong$ 0.8, D = Pile diameter = 10″ = 0.833′,

L = Pile length, q_a = Allowable compressive stress on soil = 2 ksf

$\therefore 100.02 = 0.8\times(\pi\times0.83\times L)\times2/2 \Rightarrow L = 47.76′$, $\therefore$ Provide 48′ long piles

Design of Pile-cap

For the design of pile-cap, the pile loads per girder are considered along with the soil pressure converted to soil load per girder width. The pile loads on the pile-cap for c/c girder width (5.75′) are shown below.



1. Design of Toe

The pile-cap is assumed to have 3″ effective cover and 6″ embedment for piles.

Maximum punching shear = 148.77 k, shear strength = $3.8\times0.85\times\sqrt{(0.003)} = 0.177$ ksi

Punching area around a 10″ pile = π (10+d) d = 148.77/0.177 $\Rightarrow d_{(req)} \cong 12.1''$; i.e., $t_{(req)} = 21.1''$

Maximum flexural shear = 148.77 k, shear strength = $1.9\times0.85\times\sqrt{(0.003)} = 0.088$ ksi

Shearing area = 5.75×12 d = 148.77/0.088 $\Rightarrow d_{(req)} = 24.4''$; i.e., $t_{(req)} = 33.4''$

Maximum bending moment = $148.77\times2.5 = 371.93$ k′

$\Rightarrow d_{(req)} = \sqrt{\{371.93/(0.781\times5.75)\}} = 9.1''$; i.e., $t_{(req)} = 18.1''$

$\therefore$ t = 34″, d = 25″$\Rightarrow A_s = (f_c/f_y)[1-\sqrt{\{1-2M/(\phi f_c bd^2)\}}]$ bd = 7.69 in^2; i.e., 7.69/5.75 = 1.34 in^2/ft

$A_{s(temp)} = 0.03\times34 = 1.02$ in^2/ft

2. Design of Heel

Maximum flexural shear =$20.29\times7 - 67.02 - 26.14 = 48.85$ k, allowable shear stress = 0.088 ksi

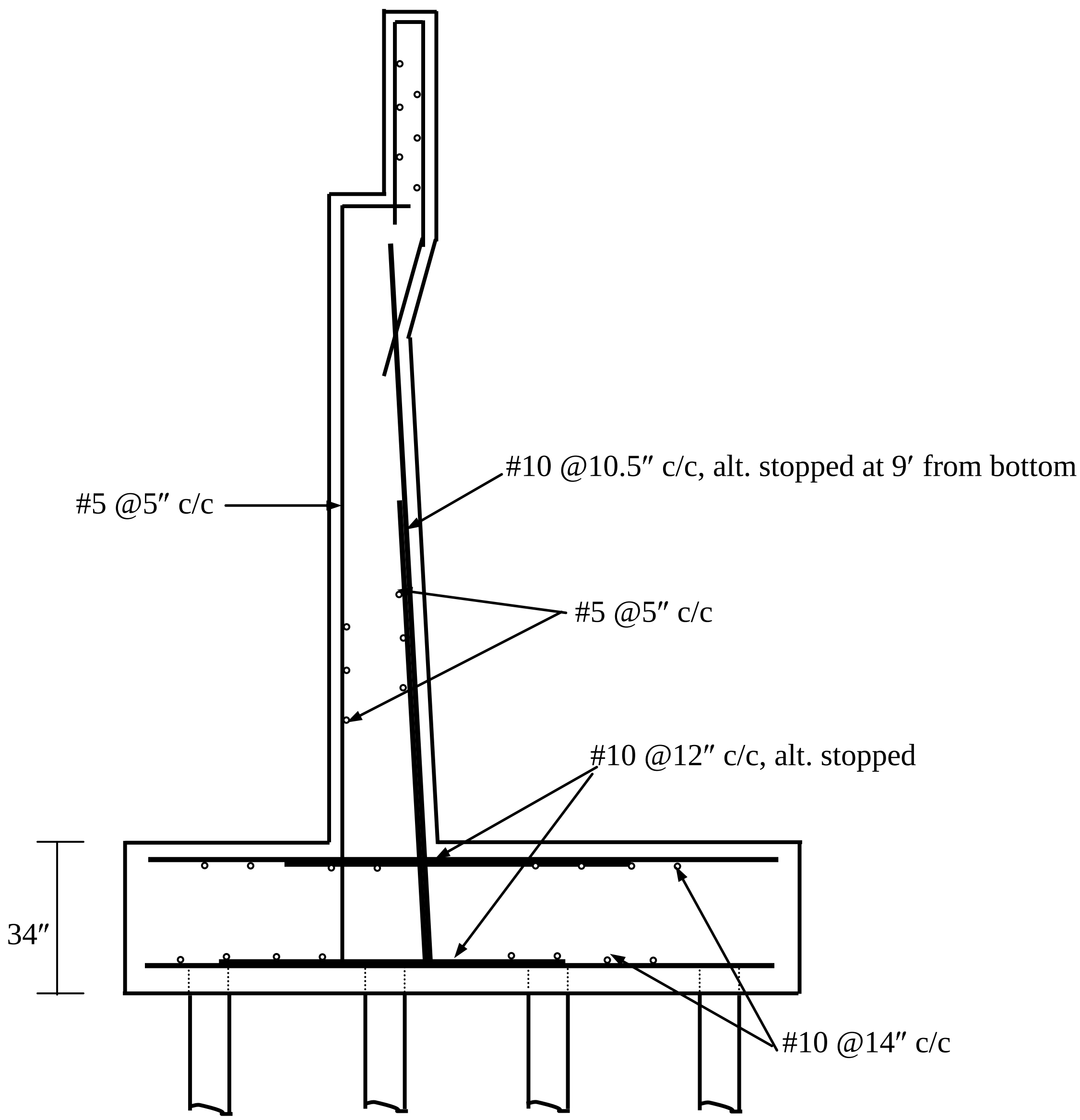
Shearing area = 5.75×12 d = 48.85/0.088 $\Rightarrow d_{(req)} = 8''$; i.e., $t_{(req)} = 17''$

Maximum bending moment = $-20.29\times8\times8/2 + 67.02\times2.5 + 26.14\times6 = -324.78$ k′

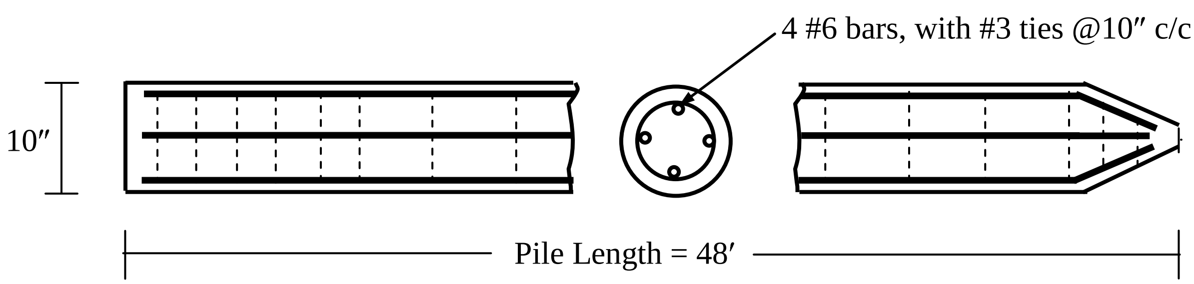
$\Rightarrow d_{(req)} = \sqrt{\{324.78/(0.781\times5.75)\}} = 8.5''$; i.e., $t_{(req)} = 17.5''$

$\therefore$ t = 34″, d = 25″$\Rightarrow$ It will be OK for punching shear also

$\therefore A_s = (f_c/f_y)[1-\sqrt{\{1-2M/(\phi f_c bd^2)\}}]$bd =5.62 in^2; i.e., 0.98 in^2/ft, $A_{s(temp)} = 0.03\times34 = 1.02$ in^2/ft



Abutment Reinforcements



Pile Reinforcements