Thornton Tomasetti

PROJECT Peapack & ODB

PROJECT NO.

DATE

BY

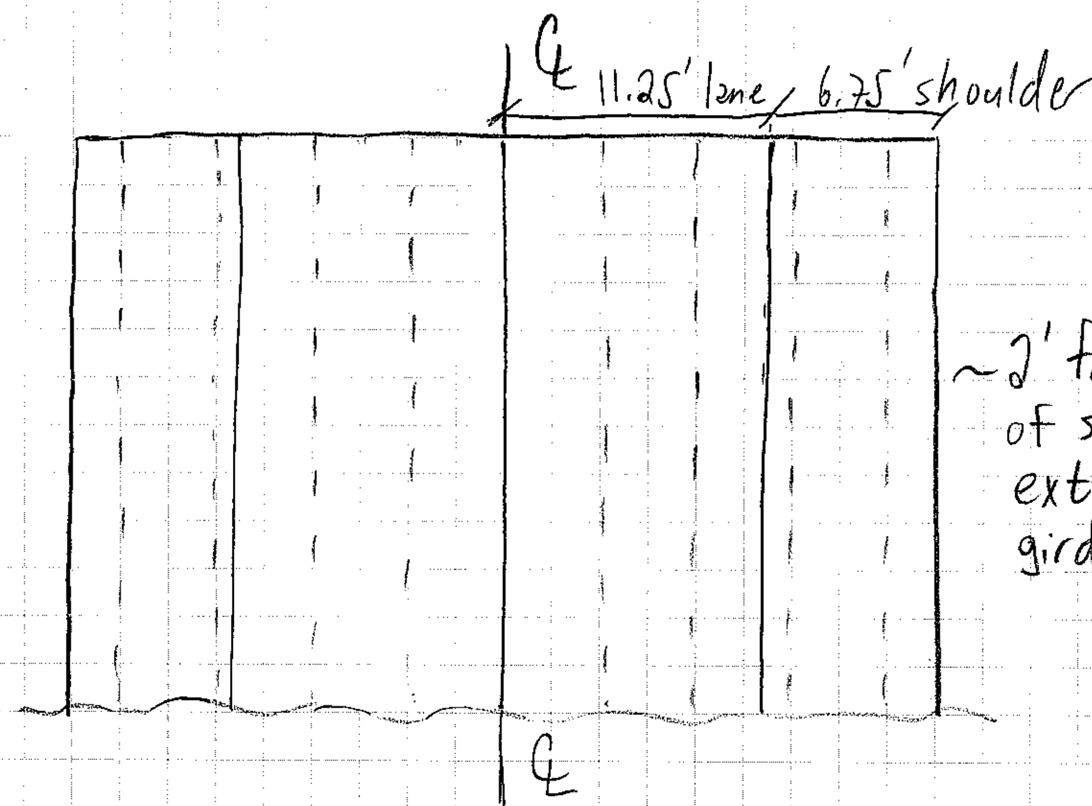
SHEET of

SUBJECT Plan View

CHECKED BY

DRAWING NO.

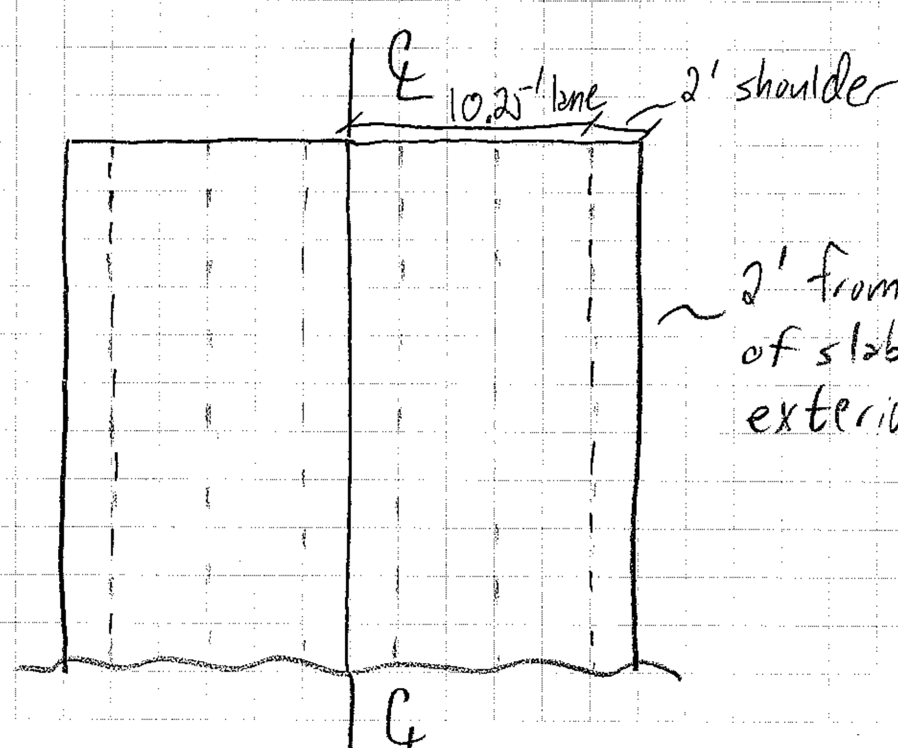
Peapack =>



~2' from edge of slab to exterior girder

- guardrails are placed @ edge of slab
- girder spacing is 4' c/c

ODB =>



~2' from edge of slab to exterior girder

scale ↓ 2'

Thornton Tomasetti

PROJECT Peapack

SUBJECT Longitudinal Section

PROJECT NO.

DATE

BY

SHEET of

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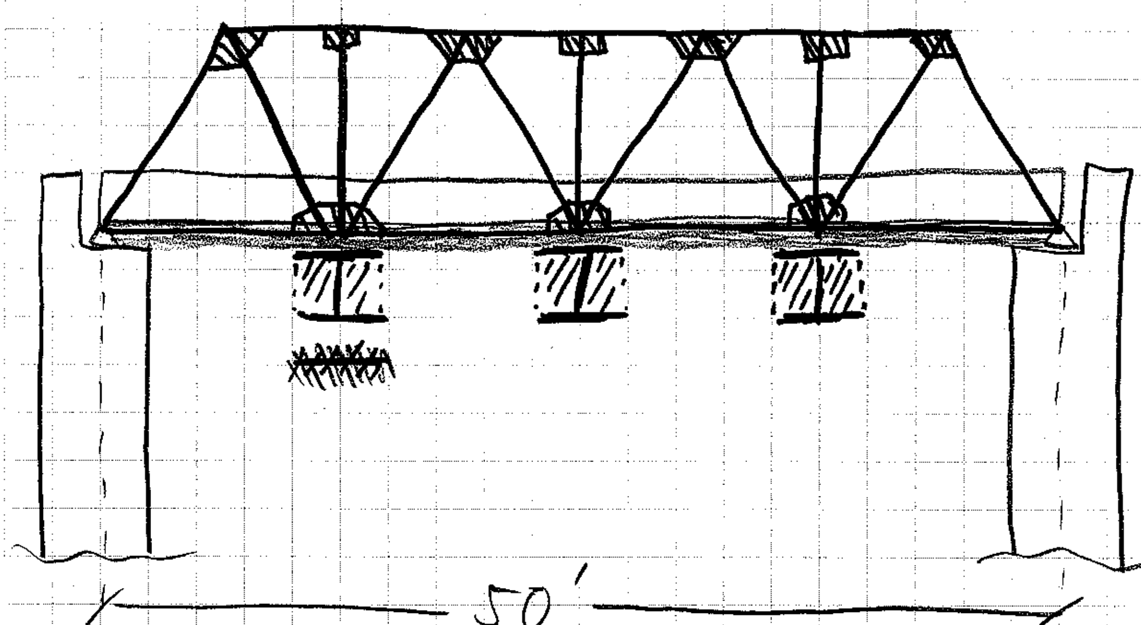
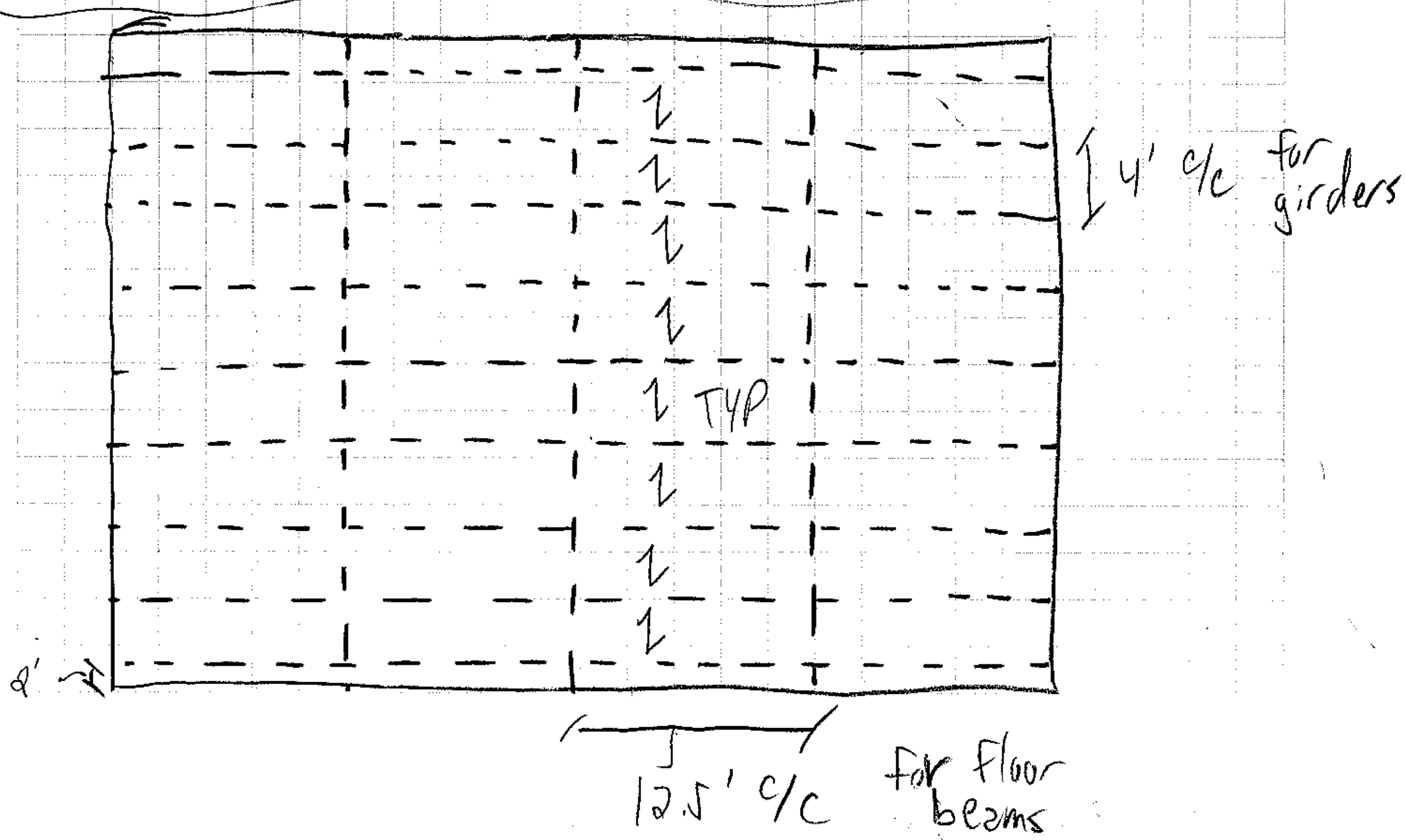
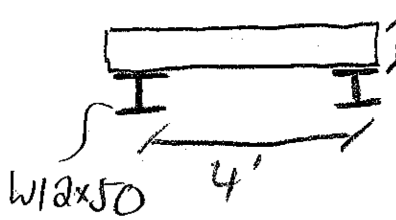
DRAWING NO.





Deck Design (Both Bridges)	Designed by → VJH	Checked by → LHJ



$d = 9'' - cc - \frac{1}{2}(\text{bar diameter})$
$= 9'' - 2.5'' - \frac{1}{2}(5/8'') = 6.19''$

$S = \text{slab span} = \text{dist. between flanges} + \frac{1}{2}(\text{flange width})$
$= [4' - (8\tfrac{1}{8}'' \times \frac{1\,ft}{12''})] + \frac{1}{2}(8\tfrac{1}{8}'' \times \frac{1\,ft}{12''})$
$= 2.98' \approx 3.65'$

w_{DL} = slab weight; future wearing surface; haunch

unfactored loads →

$(w_{DL})_1 = (9'' \times \frac{1'}{12''})(150\ lb/ft^3)(1'\ strip) = .1125\ k/ft$
$(w_{DL})_2 = (25\ lb/ft^2)(1'\ strip) = .025\ k/ft$
$(w_{DL})_3 = (3'' \times 1'/12'')(150\ lb/ft^3)(1'\ strip) = .0375\ k/ft$

Factors ⇒ $DL = 1.25$; $DL_{FWS} = 1.5$; $LL = 1.75$; $I = 1.33$

Factored loads →

$(w_{DL})_{1+3} = 1.25[.1125\ k/ft + .0375\ k/ft] = .1875\ k/ft$
$(w_{DL})_2 = 1.5[.025\ k/ft] = .0375\ k/ft$

$M_{DL} = \frac{wS^2}{10}$; cont over 2 supports ⇒ $M_{DL} = \frac{(.1875 + .0375)(3.65')}{10}$
$= .30\ k\cdot ft$

$M_{LL} = .8\left(\frac{S+2}{32}\right)P$; cont. over 2 supports ⇒ $M_{LL+I} = .8\left(\frac{3.65+2}{32}\right)(20^k \times 1.33)1.75$
$= 6.58\ ft\cdot k$

$M_u \le \phi M_n \le \phi A_s F_y(d - a/2)$; $a = \frac{A_s f_y}{.85 f'_c b}$; using #5 bars @ 12" spacing
↳ $A_s = .31\ in^2$

• $a = \frac{(.31\ in^2)(60\ ksi)}{.85(5\ ksi)(12\ in)} = .37\ in$

$\phi M_n = .9(60\ ksi)(.31\ in^2)[(9'' - 2.5'' - \frac{1}{2}(5/8'')) - .37/2] = 100\ in\cdot k / 12$
$= 8.4\ ft\cdot k$

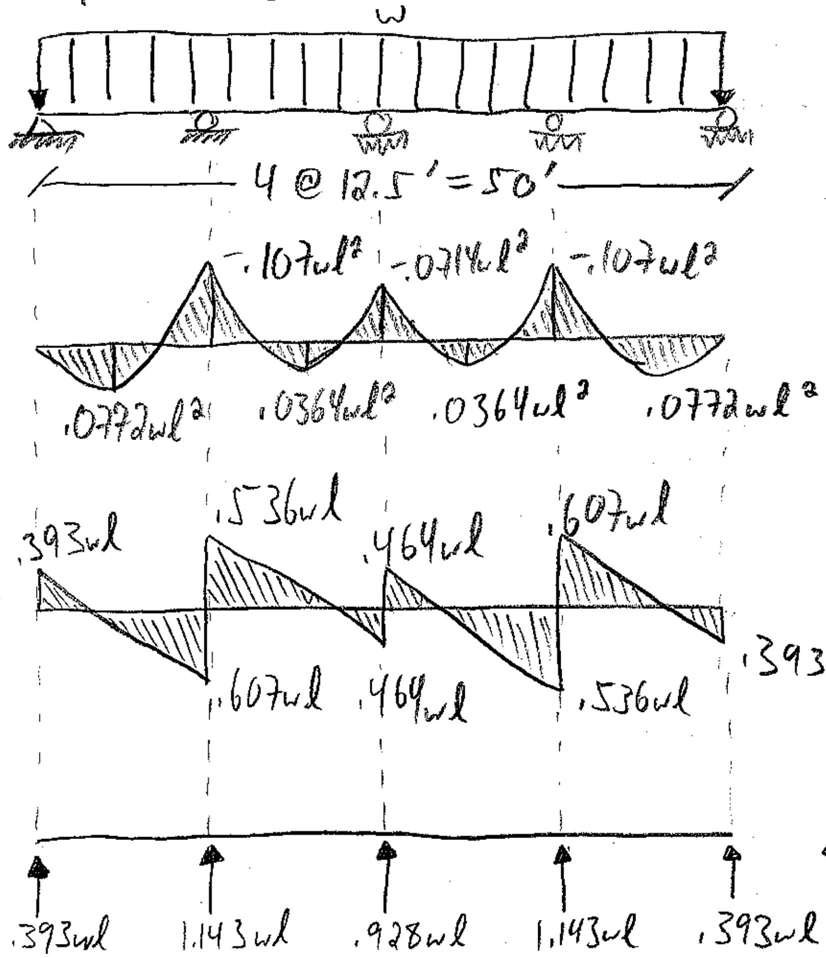
☆ $M_u \le \phi M_n \rightarrow [6.58\ ft\cdot k + .30\ k\cdot ft] = \boxed{6.88\ ft\cdot k \le 8.4\ ft\cdot k\ ✓ OK}$

Distribution steel → $\frac{220}{\sqrt{S}} = \frac{220}{\sqrt{3.65}} = 116\% \nless 67\%$; use 67%
↳ use #5@12"

ACI 318-05 7.12.2.1b — shrinkage & temperature steel → $\rho = \frac{A_s}{bd}$ → $A_s = .0018[12'' \times 6.19''] = .134\ in^2$ ⇒ use #4@12"

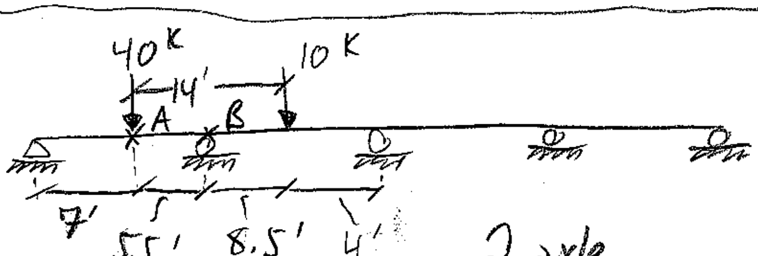
Girder Design Peapack Bridge	Designed by → LHJ	Checked by → SRV

loading, moment, and shear diagrams for girders



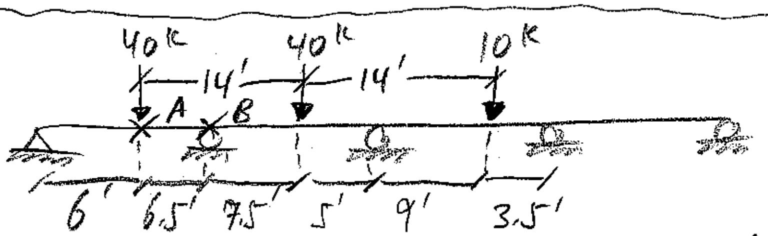
from AISC Manual Table 3-23 pg. 3-225

girder analysis from LL+I



2 axle truck

→ Produces 90 ft·k @ pt. A
Produces 58 ft·k @ pt. B
- $V_{max} = R_{max} = 40^K$ when truck axle sits on spt.



3 axle truck

→ Produces 87 ft·k @ pt. A
Produces 80 ft·k @ pt. B
- $R_{max} = 45^K$

☆ tandem load produces
$M_{max} = 86.5$ ft·k
$V_{max} = 38.4^k$
↓
@ 5' from ext. support

$\therefore M_{max} = 90^{ft\cdot K}$; $M_{LL+I} = 90 \times 1.33 \times 1.75 = 210^{ft\cdot K}$
(unfactored)

$V_{max} = 45^K$; $V_{LL+I} = 45^K \times 1.33 \times 1.75 = 105^K$

Distribution Factors → Peapack	Designed by → LHJ	Checked by → SRV

· calculate distribution factors:

$K_g = n(I + Ae_g^2)$; $n = \frac{E_{st}}{E_c} = 7.25$

~ assume a W12x50 → $A = 14.6\ in^2$; $d = 12.2\ in$; $I_x = 391\ in^4$

$e_g = 9''/2 + 1.5'' + 12.2''/2 = 12.1''$

→ $K_g = 7.25[391\ in^4 + (14.6\ in^2)(12.1'')^2] = 18{,}333\ in^4$

For interior stringers w/ 2 or more design lanes loaded

$DF_M = .075 + (S/9.5)^{.6}(S/L)^{.2}\left(\frac{K_g}{12 \cdot L \cdot t_s^3}\right)^{.1}$

$= .075 + \left(3.65/9.5\right)^{.6}\left(3.65/12.5\right)^{.2}\left(\frac{18333}{12 \cdot 12.5 \cdot 9^3}\right)^{.1}$

$DF_M = .443$

$DF_V = .2 + S/12 - (S/35)^{2.0} = .2 + \frac{3.65}{12} - (3.65/35)^2 = .493$

for ext girders →

$DF_M = .443 \times [DF]_{EXT\,M}$ → $[DF]_{EXT\,M} = .77 + \frac{d_e}{9.1} = .99 \approx 1.0$

$DF_V = .493 \times [DF]_{EXT\,V}$ → $[DF]_{EXT\,V} = .6 + \frac{d_e}{10} = .8$

where d_e = dist from web of ext girder to face of barrier $= 2^{ft}$

No excessive SDL present <u>AND</u> low exterior distribution factors;

∴ Design Assumption ⇒ leave ext. girders the same as int. girders

☆ similar assumption for ODB due to similar use, and $d_e = 2'$

Girder Design Peapack Bridge	Designed by → LHJ	Checked by → VJH

w_{DL} = slab + steel + haunch + guard rail ;

$= (9'' \times 1/12'')(150\ lb/ft^3)(1') = .1125\ k/ft$

$W_{gr} = (\frac{10''}{12})(\frac{10''}{12})(150\ lb/ft^3)$
$= .035\ k/ft$
$W_{gr} = .035 \times \frac{2\ rails}{9\ girders} = .008\ k/ft$

$w_{DL} = (50\ lb/ft)$; $w_{DL} = (3'' \times 1/12'')(150\ lb/ft^3)(1') = .0375\ k/ft$

Factored loads

$w_{DL} = 1.25[.1125 + .0375 + .008 + .05] = .26\ k/ft$

$w_{FWS} = \frac{(36')(25\ lb/ft^2)}{9\ girders} = .1\ k/ft \times 1.5 = .15\ k/ft$

w_{LL} = design lane load $= .64\ k/ft \times 1.75 = 1.12\ k/ft$

Moments & Shears ⇒ $M_{max} = .107 w l^2 = .107[.26\ k/ft + .15\ k/ft](12.5\ ft)^2$
$+ .107(1.12\ k/ft)(12.5\ ft)^2(DF_M)$
$+ 210\ ft{\cdot}k(DF_M) \Rightarrow M_u = 108.2\ ft{\cdot}k$

$V_{max} = .607 w l = .607[.26\ k/ft + .15\ k/ft](12.5\ ft)$
$+ .607[1.12\ k/ft](12.5\ ft) DF_V + 105\ k \times DF_V$
$V_u = 59.1\ k$

Girder Design

$M_u \leq \phi M_n \times C_b$; assume $C_b = 1.0$; conservative

$M_u \approx 110\ ft{\cdot}k$; $L_b = 12.5\ ft \Rightarrow \therefore \phi M_n \geq 110\ ft{\cdot}k$

~ From AISC Manual, Table 3-10, pg 3-123

↳ $\boxed{\text{use a W10x39 w/ } \phi M_n = 154\ ft{\cdot}k}$

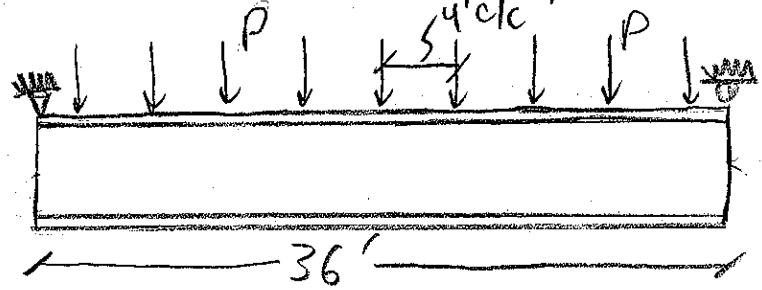
Shear Check ⇒ $V_u \approx 59.1^k$; $V_u \leq \phi V_n \leq \phi[.6 F_y A_w C_v]$ ~ chapter 6 sect. G AISC manual

$\cdot\ h/t_w = 25 \leq 2.24\sqrt{E/F_y} = 49.3$ ✓

↳ $\therefore C_v = 1$, $\phi = 1 \Rightarrow 1.0[.6 \times 50\ ksi \times .315'' \times 9\ 7/8'']$

↳ $\phi V_n = 93.3^k > V_u$ ✓ OK

Floor beam design Peapack Bridge	Designed by $\Rightarrow$ LHJ	Checked by $\Rightarrow$ SRV



+ DL moment + shear

$$M_{DL} = wx\left(\frac{x}{2}\right) = \frac{wx^2}{2}$$

reactions from girders $\Big\}$ $P = \Sigma 1.4 w l + R_{LL+I}$

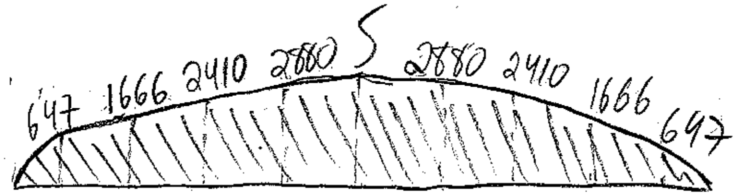
$= 1.4 \times 12.5^{ft}[.26^{k/ft} + .15^{k/ft}] + 1.4 \times 12.5^{ft}[1.12^{k/ft}] \times DF_V$

$+ 105^k \times DF_V \Rightarrow P = 68.6^k$

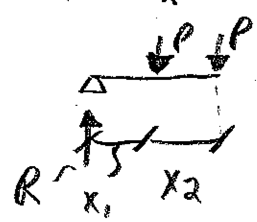
$$\Sigma M_A = 0 = -P(2' + 6' + 10' + 14' + 18' + 22' + 26' + 30' + 34') + R_B(36')$$

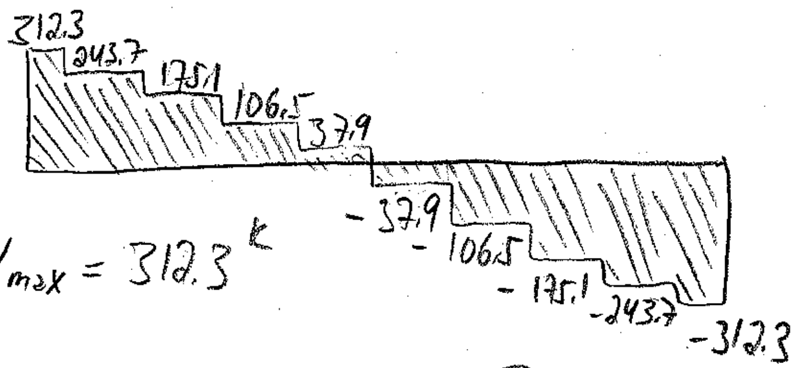
$\hookrightarrow R_B = \underset{LL}{308.7^{ft \cdot k}} + \underset{DL}{\frac{wl}{2}} = 312.3^{ft \cdot k}$

$\sim w = .200^{k/ft}$
$l = 36'$



$M = R_x + P[\Sigma x]$





$V_{max} = 312.3^k$

M_{max}

$$C_b = \frac{12.5 M_{max}}{2.5 M_{max} + 3M_A + 4M_B + 3M_C}$$

$C_b \times \phi M_n \geq M_u$;

	girder reactions	dead load	Σ
M_u	3075	32.4	3108
M_A	2124.7	8.1	2133
M_B	3075	32.4	3108
M_C	2124.7	8.1	2133

$$\hookrightarrow C_b = \frac{12.5(3108)}{2.5(3108) + 3(2133) + 4(3108) + 3(2133)}$$

$= 1.17$; $M_u = 3108^{ft \cdot k}$

$M_u \leq \phi M_n \times C_b \Rightarrow \frac{M_u}{C_b} = 2640^{ft \cdot k} \leq \phi M_n$

$l_b = 4'$

consult table 3-10 in AISC Manual $\Rightarrow$ $\boxed{\text{use a W30x211 w/ } \phi M_n = 2820^{ft \cdot k}}$

Thornton Tomasetti

PROJECT Peapack		PROJECT NO.	DATE
		BY LHJ	SHEET of
SUBJECT Floor Beam Shear Check		CHECKED BY VJH	DRAWING NO.

Shear Check for Floor Beams:

$V_u = 312.3^k \leq \phi V_n \leq \phi[.6 F_y A_w C_v]$

$h/t_w = 34.5 \leq 2.24\sqrt{E/F_y} = 49.3$ ✓

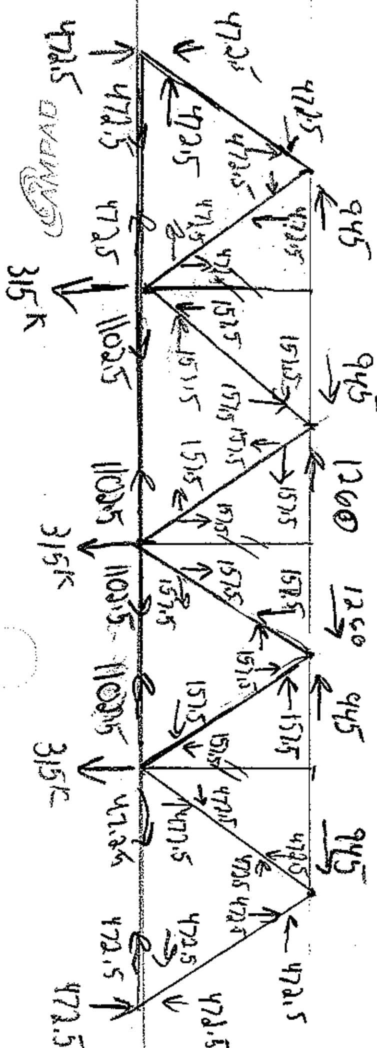
$\therefore C_v = 1, \phi_v = 1$

$\phi V_n = 1.0[.6 \times 50^{ksi} \times 3/4'' \times 31'' \times 1.0]$

$\phi V_n = 698^K$

$V_u \leq \phi V_n$ ✓ <u>OK</u>

Truss Design
Peapack Road Bridge | BY: SRV | CHECKED BY: L.S.



$P_U = 1260$ kips

$K = 1$
$L = 12.5'$

$W12 \times 106 \Rightarrow r_x = 5.47\,in \quad A_g = 31.2\,in^2$

$$\frac{KL}{R_x} = \frac{(1)(12.5)(12)}{5.47\,in} = 27.42$$

Table 4-22 AISC w/ $\frac{KL}{R} = 27.42$:

$\phi F_{cr} = 42.62$

$\phi P_n = \phi F_{cr} A_g = 42.62(31.2) = 1329$ kips

$P_U \le \phi P_n$ <u>Good for x-axis</u>

Table 4-1 AISC w/ $KL = 6.25$:

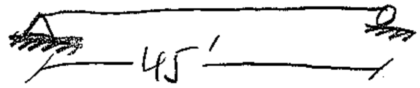
$\phi P_n = 1345$ kips

$P_U < \phi P_n$ <u>Good for y-axis</u>

$\boxed{\text{Use } W12 \times 106}$

Composite Stringer ~ ODB	Designed by → VJH	Checked by → LHJ

$L = 45'$; unshored construction ; 25 psf wearing surface



girders @ 4' c/c

• effective flange width $= b_e$ → $1/4 \times 45' = 11.25'$
c/c stringers $= 4'$ → use $b_e = 4'$
$12 \times t_{slab} + 1/2\, b_{fl} = 11.25'$

• compute loads; moments, & shears

$DL_{slab} = (9'' \times 1/12'')(6')(150\ lb/ft^3) = .675\ k/ft$

$DL_{haunch} = (1')(3'' \times 1/12'')(150\ lb/ft^3) = .0375\ k/ft$

(assume 150 lb/ft); $DL_{steel} = (100\ lb/ft)(1.05) = .105\ k/ft$

$DL_{parapet} = (10'' \times 1/12'')(10'' \times 1/12'')(50\ lb/ft^3) \times \frac{2\ parapets}{6\ stringers}$

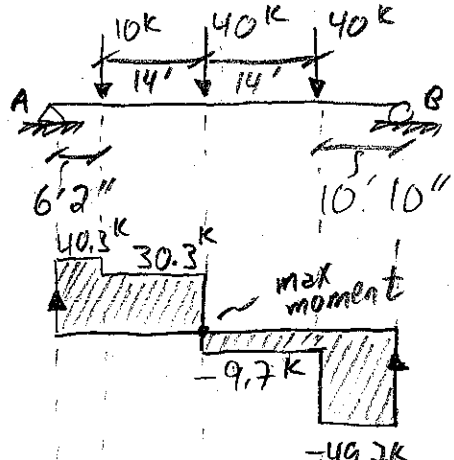
↳ $DL_p = .012\ k/ft$

$DL_{FWS} = \frac{(25\ lb/ft^2)(24.4\ ft)}{6\ stringers} = .100\ k/ft$

$LL_{lane\ load} = .64\ k/ft$; $LL+I$ → design truck loading, see below ↓

LL+I

3 axle truck ⇒



→ $\Sigma M_A = 0 = -10^k(6.17') - 40^k(20.17') - 40^k(34.17') + R_B(45')$

↳ $R_B = 49.7^k$

$\therefore R_A = 40.3^k$

$M_{max} = R_A[6'2'' + 14'] - 10^k[14']$
$= 672.7\ ft \cdot k$

$V_{max} = 49.7^k$

* tandem truck produces smaller forces for Moment & Shear; 3 axle truck controls

• $M_{DL} = \left[\begin{matrix} slab + haunch \\ + steel \end{matrix}\right] = \frac{wl^2}{8} = \frac{(.675 + .0375 + .105)(45)^2}{8}$

↳ $M_{DL} = 207\ ft \cdot k$

$V_{DL} = \frac{wl}{2} = \frac{(.675 + .0375 + .105)(45')}{2} = 18.4\ k$

• $M_{FWS} = \frac{wl^2}{8} = \frac{(.100\ k/ft)(45')^2}{8} = 25.3\ ft \cdot k$; $V_{FWS} = \frac{wl}{2} = 2.3^k$

• $M_{LL} = wl^2/8 = (.64\ k/ft)(45')^2/8 = 162\ ft \cdot k$; $V_{LL} = \frac{wl}{2} = 14.4^k$

• $M_{LL+I} = 672.7\ ft \cdot k$; $V_{LL+I} = 49.7^k$

composite stringer ~ODB	Designed by VJH/LHJ	Checked by ~ SRV

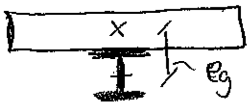
· calculate distribution factors: LL_M = live load distribution factor for moment
LL_V = live load distribution factor for shear

* assume a stringer size of W24x76

↳ $A_g = 22.4\ in^2$
$I_x = 2100\ in^4$
$d = 23.92\ in^2$

; $LL_M = .075 + \left(\frac{S}{9.5}\right)^{.6}\left(\frac{S}{L}\right)^{.2}\left(\frac{K_g}{12Lt_s^3}\right)^{.1}$ ↳ cont. over 2 supports

where $K_g = n(1 + Ae_g^2)$



e_g = distance between C.G.'s
$= 9"/2 + 3" + 23.92"/2 = 19.5"$

$n = E_{st}/E_c$; $E_c = 57000\sqrt{f'_c} = 57000\sqrt{5000} = 4000$ ksi

↳ $n = 29 \cdot 10^6 / 4 \cdot 10^6 = 7.25$

$\therefore K = 7.25[1 + 22.4\,in^2 \times (19.5")^2] = 61760\ in^4$

$LL_M = .075 + \left(\frac{3.65'}{9.5}\right)^{.6}\left(\frac{3.65'}{45'}\right)^{.2}\left(\frac{61760\,in^4}{12 \times 45' \times (9/12)^3}\right)^{.1} = .672$

$LL_V = .2 + S/12 - (S/35)^2 = .2 + 3.65/12 - (3.65/35)^2 = .493$

· Factor moments and shears: $\phi_{DL} = 1.25$, $\phi_{FWS} = 1.5$, $\phi_{LL} = 1.75$, $\phi_{LL+I} = 1.33$

~ $M_{DL} = 1.25[207\ ft{\cdot}k] = 259\ ft{\cdot}k$

$V_{DL} = 1.25[18.4^k] = 23^k$

~ $M_{FWS} = 1.5[25.3\ ft{\cdot}k] = 38\ ft{\cdot}k$

$V_{FWS} = 2.3^k[1.5] = 3.5^k$

~ $M_{LL} = 1.75[162\ ft{\cdot}k](.672) + 1.75[672.7\ ft{\cdot}k](1.33)(.672)$
$= 1243\ ft{\cdot}k$

$V_{LL} = 1.75[14.4^k](.493)$
$+ 1.75[49.7^k](1.33)(.493) = 70^k$

$\Rightarrow \Sigma M = 259 + 38 + 1243 = 1540\ ft{\cdot}k$

$\Sigma V = 23 + 3.5 + 70 = 96.5^k \approx 100^k$

* strength I factors control; compared to strength II
(1.25DL + 1.5DLFWS + 1.75LL+I) (1.5DL + 1.5DLFWS)

composite stringer ~ODB	Designed by → LHJ	Checked by → VJH

• check plastic strength ∴ $a = \frac{A_s F_y}{.85 f'_c b_e} = \frac{(22.4\ in^2)(50\ ksi)}{(.85)(5\ ksi)(4' \times 12'')} = 5.5''$

$M_p = A_s F_y (d - a/2) = (22.4\ in^2)(50\ ksi)\left(\frac{23.92''}{2} + 3'' + 8.5'' - \frac{5.5''}{2}\right)(1/12'')$

$M_p = 1933\ ft{\cdot}k$

• $d_{slab} = 6.2''$, $a = 5.5'' \Rightarrow$ ∴ PNA lies in concrete

• W24x76 is a compact section; $M_n = M_p$

↳ $M_u + (1/3)\cancel{f_\ell}\,0\,S_{xt} \le \phi_b M_n \Rightarrow 1540\ ft{\cdot}k \le 1933\ ft{\cdot}k$

• shear check: $V_u = 100^k$; $h/t_w = 49$; $2.24\sqrt{E/F_y} = 49.3 \ge t_w$

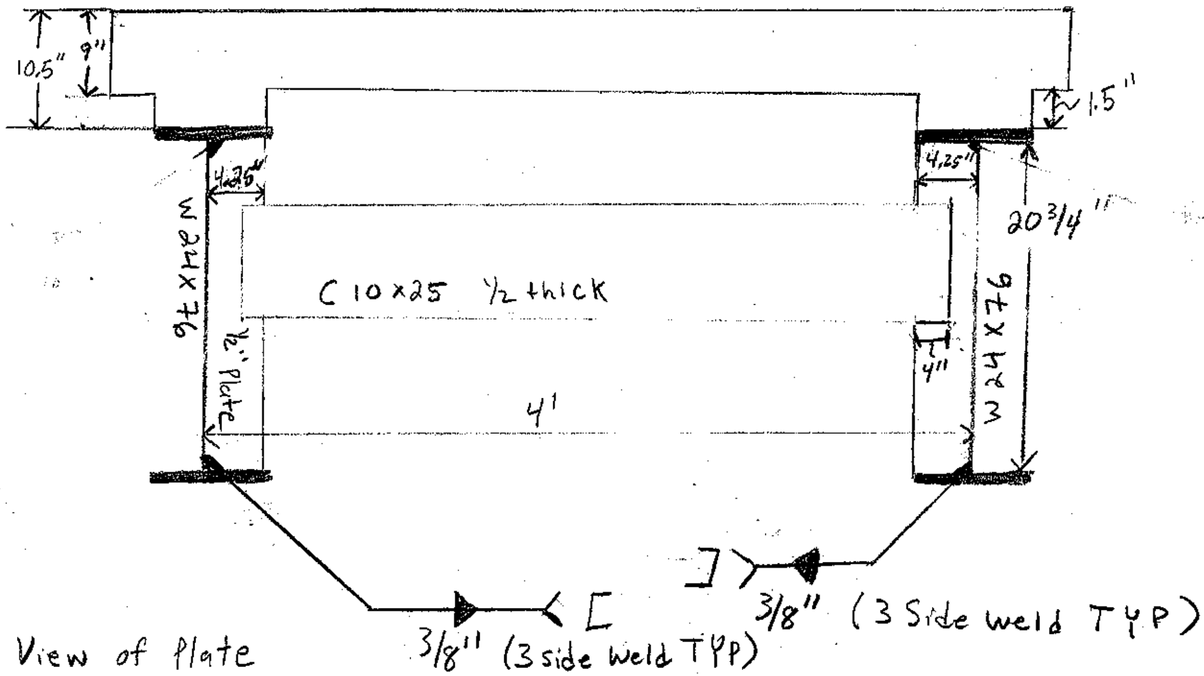
↳ ∴ $\phi_v = 1.0$, $C_v = 1.0$

$\phi V_n = .6 F_y A_w C_v \phi_v = .6(50\ ksi)(23.9'')(.44'')(1.0)(1.0)$

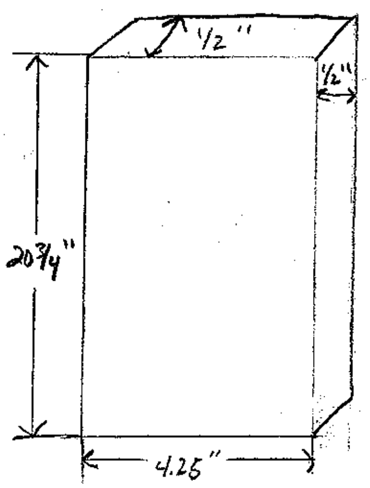
↳ $\phi V_n = 315.5^k$ ✓ OK

ODB - Diaphragms + Plate + Bolts | Checked by → SRV

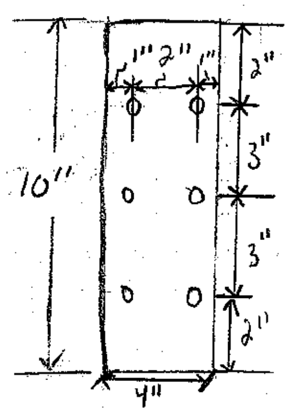
Designed by → VJH



View of Plate



Bolt Specs



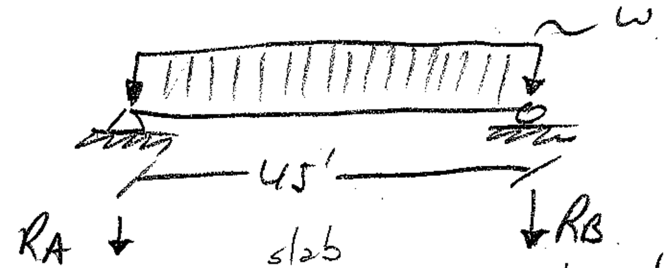
ASTM A325

$\emptyset_b = 7/8''$

Slip Critical Bolt
Standard Hole
Single Shear
$\emptyset_v r_n = 15.4\ ^{k}/_{Bolt}$

$\emptyset v r_n = 92.4\ k$ total

ODB	Load estimation for geotech & abutments	By → LHJ



$$DL \rightarrow w = \underbrace{(.675\ ^{k}/ft)}_{\text{slab conc}} + \underbrace{(.0375\ ^{k}/ft)}_{\text{conc haunch}} + \underbrace{(.076\ ^{k}/ft)1.05}_{\text{steel wght}} + \underbrace{(.012\ ^{k}/ft)}_{\text{wood parapet}} + \underbrace{(.1\ ^{k}/ft)}_{\text{FWS}}$$

$$w_{DL} = .9\ ^{k}/ft$$

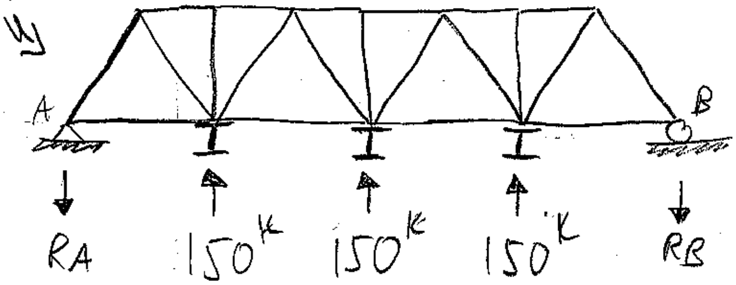
$LL \rightarrow w = .64\ ^{k}/ft$; $LL + I \rightarrow R = 50^{k}$ From truck

$$\Sigma w = .9 + .64 \approx 1.55\ ^{k}/ft$$

$$R = \frac{wl}{2} = \frac{(1.55\ ^{k}/ft)(45')}{2} \approx 35^{k} + 50^{k} = 85^{k}/\text{girder}$$

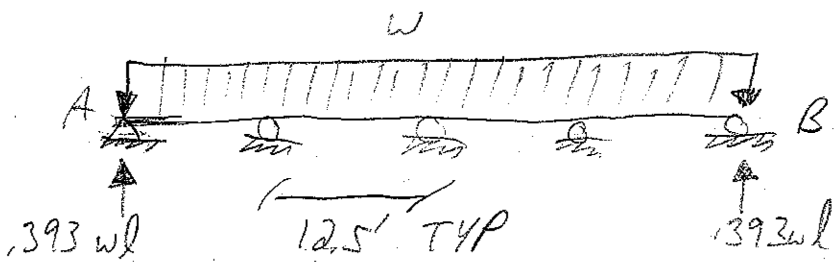
Peapack	Load Estimation for geotech & abutm	By → LHJ

truss forces / floor beam forces (already Factored)



$\Sigma M_A = 0 =$
$150(12.5')$
$150(25')$
$150(37.5')$
$- R_B(50')$
$\to R_B = 225^k$
$R_A = 225^k$

girder reactions → LL+I → 40^k when truck crosses support



$\to w_{DL} = (.1125 + .0375 + .008 + .05 + .1) = .308\ ^k/_{ft}$
$w_{LL} = (.64\ ^k/_{ft})$

DL + LL

$\therefore R_A = R_B = [.393 \times 12.5'][.308\ ^k/_{ft}]$
$+ [.393 \times 12.5'][.64\ ^k/_{ft}] \times DF_v$
$+ 40^k \times DF_v$

$\therefore$ Reaction/girder $= 22.8\ ^k/$girder

AASHTO LRFD Bridge Design Specifications	Peapack Road Bridge	FHWA.Gov

Top of abutment elevation - 46.918 m
Bottom of River Channel - 42.314 m
Footing 3'2" Below Lowest point in the River

Total abutment Height = $(46.918^m - 42.314^m) + 3.167 = 18'-3"$

18' - 3" = 94"

Material Properties: Concrete Density W_c = 150 pcf
f'_c = 4000 psi or 4 ksi
f_y = 60,000 psi or 60 ksi

Reinforcement Steel Requirements:

Backwall back cover -	$Cover_b$ =	2.5 in	(S Table 5.12.3-1)
Stem Back Cover -	$Cover_s$ =	2.5 in	(")
Footing Top Cover -	$Cover_{ft}$ =	2.0 in	(")
Footing Bottom Cover -	$Cover_{fb}$ =	3.0 in	(")

Girder Spacing - S = 4'
Number of Girders - N = 9
Span Length - L_{span} = 48'
Out to Out width - W_{deck} = 36'

Abutment + Wingwall Detail S 2.3.3.2

Abutment Stem Height = 18' - 3"
Wingwall Stem Design Height = 18' - 3" (3/4 Point)

Abutment + Wingwall Length S.11.6.1.4

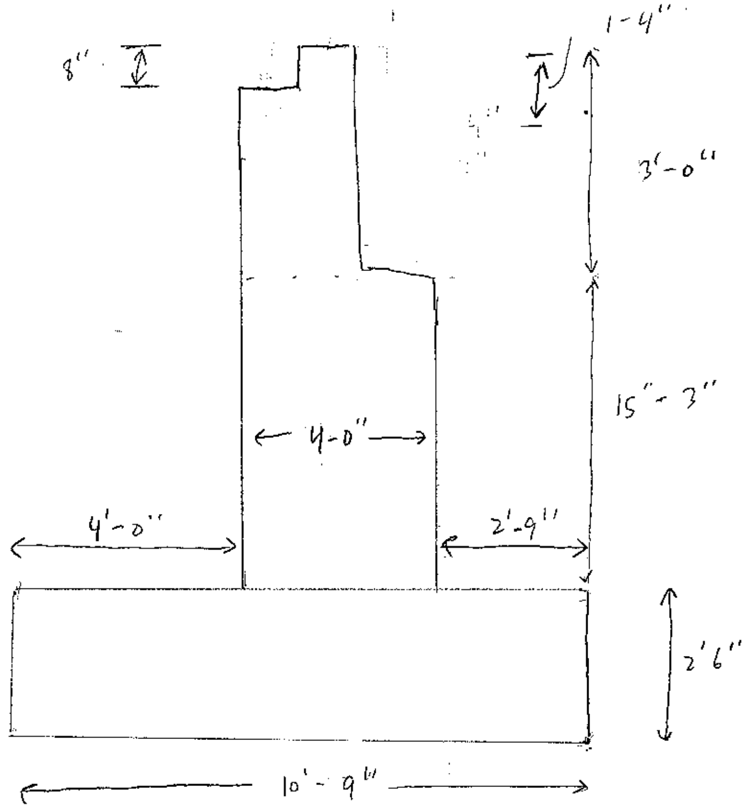
Abutment Length = 32'
Wingwall Length = 17'

Full - Depth Reinforced Concrete Cantilever Abutment

No Sliding analysis necessary due to piles.

Overturning Analysis Provided.

Preliminary Sketch



Compute Dead Load effects

1) Back Wall Dead Load: $(4.0'' \times 3' \times 0.150\ kcf) \times 70\% = 1.18\ ^{k}/_{ft}$ (Estimated)

2) Stem Dead Load $DL_{ST} = (15.25' \cdot 4' \cdot 0.120) = 7.32\ ^{k}/_{ft}$

3) Footing Dead Load $DL_{Ft} = (10.75')(2.5')(0.120\ pcf) = 3.22\ ^{k}/_{ft}$

4) Earth Dead Load $DL_{EA} = (18.25')(4')(0.120\ kcf) = 10.56\ ^{k}/_{ft}$

$\gamma_s = 0.120\ ^{k}/_{ft^3}$

Loads Due to Lateral Earth Pressure. – Analysis at 3- Key points
1) Backwall only
2) BW + stem
3) BW + Stem + Footing

$P = K_a \cdot \gamma_s \cdot z$

Load occurs at 2/3 From Top.

$K_a = 0.3$ (Estimated)
$\gamma_s = 0.120$ KCF (Estimated)
$z =$ Depth – varies.

Backwall effect: $z = 3'$

$P = (0.3)(0.120)(3) = \underline{0.11 \text{ KSF}}$ @ $\underline{2' \text{ From Top}}$

Reaction Due to Lateral Earth Pressure $= R_{EH_{BW}}$

$R_{EH_{BW}} = \left(\frac{1}{2}\right)(P)(h_{BKWL}) = \left(\frac{1}{2}\right)(0.11)(3) = 0.165$ k/ft

Stem Effect
$K_a = 0.3$
$\gamma_s = 0.120$ KCF
$z = 18'3''$

$P = (0.3)(0.120 \text{ KCF})(18.25') = 0.657$ KSF

$R_{EH_{St}} = \left(\frac{1}{2}\right)(0.657)(18.25) = \underline{6.00 \text{ k/ft}}$

Reaction @ $\underline{12.16' \text{ From Top.}}$

Bottom of Footing
$K_a = 0.3$
$\gamma_s = 0.120$
$z = 20'-9''$

$P = (0.3)(0.120 \text{ KCF})(20.75') = 0.747$

$R_{EH_{BF}} = (0.5)(.747)(20.75') = \underline{7.75 \text{ k/ft} \text{ @ } 13.83' \text{ From Top}}$

Live Load Surcharge $Q = 250$ PSF (Referenced Foundations Book)
Acts at 1/2 Height of Stem (Referenced Foundation Book)

Superstructure

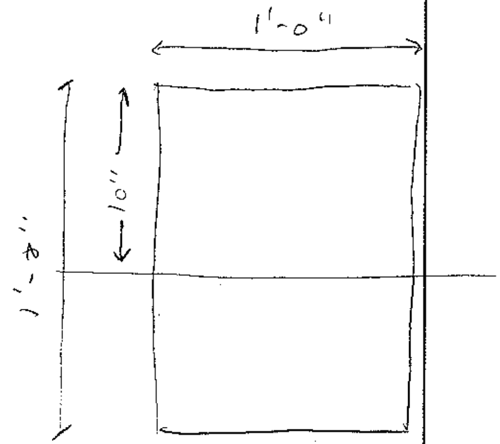
450 k per Abutment

: Reinforcement:

Design Abutment Backwall

Per Foot Analysis

Assume #5 Bars
$\phi = 0.625\ in^2$
Area $= 0.31\ in^2$



cracking strength is governed by $M_{cr} = \frac{f_r \cdot I_g}{y_t}$

$f_{cr} = 0.24\sqrt{f'c} = 0.24\sqrt{4\ ksi}$

$F_{cr} = 0.48\ ksi$

$I_g = \left(\frac{1}{12}\right)(12\,in)(20'')^3 = 8000\ in^4$

$y_t = 10\ in$

$$M_{cr} = \frac{\frac{f_r \cdot I_g}{y_t}}{ft} = \frac{\frac{(0.48\ ksi)(8000\ in^4)}{10\ in}}{ft} = 32\ \frac{kip}{ft}$$

Factored moment $= 1.2(32) = 38.40\ ft \cdot kip/ft$

Flexural resistance equal to the lesser of 1.2 cracking strength or 1.33 times max Factored moment.

$M_{ubwmax} = 14.38\ k\text{-}ft/ft$ $\quad 1.33\ M_u = 19.13\ k\text{-}ft/ft$ - controls

Effective Depth $\quad de =$ total backwall thickness - cover - ½ Ø bar

$t_{bw} = 20''$ $\quad de = 20'' - 2.5'' - \frac{0.625\,in}{2} = 17.19\ in$

Solve for amount of reinforcing steel

$\phi f = 0.90$ $\quad b = 12\,in$ $\quad R_n = \frac{M_{ubwdes} \cdot 12\,in}{(\phi_f \cdot b \cdot de^2)} = 0.072\ k/in^2$

$$\rho = 0.85\left(\frac{f'c}{f_y}\right)\left[1.0 - \sqrt{1.0 - \frac{2R_n}{0.85 \cdot f'c}}\right] \qquad \rho = 0.00121$$

$A_s = \rho\,\frac{b}{ft} \cdot de$ $\quad A_s = 0.25\ in^2/ft$ $\quad \frac{bar\_Area}{A_s} = 14.9\ in$

#5 Bars $\quad$ bar space $= 9\,in$

$A_s =$ bar area $\left[\frac{12\,in}{bar\ space}\right] = A_s = 0.41\ in^2$ per Foot

Check Maximum Reinforcement

$T = A_s f_y \qquad T = 24.80\ k$ S. 5. 7. 3. 2. 1

$a = \dfrac{T}{0.85\, f'_c \cdot b} \qquad a = 0.61\ in$

$\beta_1 = 0.85 \qquad c = \dfrac{a}{\beta_1} = 0.72\ in$

S. 7. 2. 2

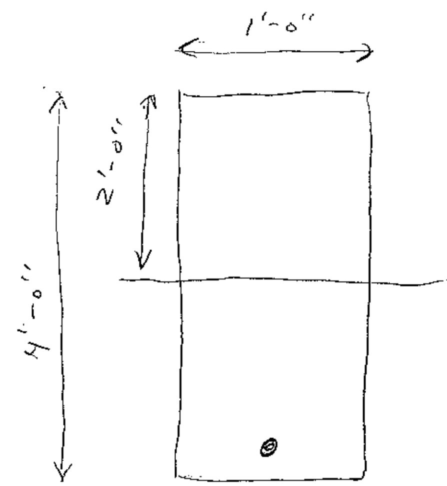
$\dfrac{c}{d_e} = 0.04 \qquad \dfrac{c}{d_e} < 0.42$ ‖

$0.4 \leq 0.42 \qquad$ o.k! S. 7. 3. 3. 1

Design Abutment stem:

Assume # 9 Bars

$\phi = 1.128\ in$
$A = 1.00\ in^2$
$f_y = 60\ ksi$



$M_{cr} = \dfrac{f_r \cdot I_g}{y_t}$

$f_{cr} = 0.48\ ksi$
$I_g = \left(\frac{1}{12}\right)(12)(48\ in)^3 = 110{,}592\ in^4$

$y_t = 24'' \qquad M_{cr} = 184.3\ \dfrac{kip\,ft}{ft}$

$1.2(M_{cr}) = 221.18$ Controls $\qquad M_{u\,stem} = 221.18$

$d_e = 48'' - 2.5'' - \frac{1}{2} = 45''$

$\phi_f = 0.90$
$b = 12''$
$f'_c = 4\ ksi$

$R_n = \dfrac{M_{u\,stem} \cdot 12}{(\phi_f \cdot b \cdot d_e^2)} \qquad R_n = 0.124\ k/in^2$

$R_n = .121\ k/in^2$

$\rho = 0.85 \left(\dfrac{f'_c}{f_y}\right)\left[1.0 - \sqrt{1.0 - \dfrac{(2)(R_n)}{0.85 \cdot f'_c}}\right] \qquad \rho =$

$.204\,[0.07] = 0.0039$

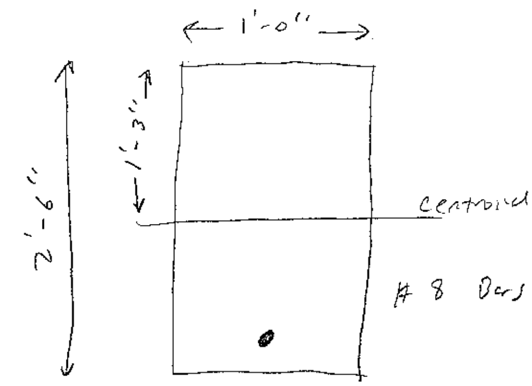
$A_s = \rho \cdot \frac{b}{ft} \cdot d_e = (0.0039)\left(\frac{12}{ft}\right)(45'') = 2.1\ in^2/ft$

Required Bar Spacing $= \frac{\text{bar Area}}{2.1\ ft/ksi} = \frac{1.0}{2.1} = (0.47)(12) = 5.75$ in

$A_s = (1.00)\left(\frac{12\ in}{5\ in}\right)$ $A_s = 2.4\ in^2$ per foot

#9 at 5''

Design Abutment Footing



Assume #8 Bars

$\phi = 1.00$ in

bar Area $= 0.79\ in^2$

$f_y = 60$ ksi

$f_r = 0.48$ ksi

$I_g = (1/12)(12\ in)(30\ in)^3 = 27{,}000\ in^4$

$y_t = 15$ in

$M_{cr} = 72.00$ k/ft/ft

$1.2\ M_{cr} = 86.40$ k-ft/ft

1.33 x

$M_{u\ toe} = 73.71$ k-ft/ft $1.33\ M_u = 98.04$ k-ft/ft

$d_e = 30 - 3 - 1/2$ $d_e = 26.50$ in

$\phi f = 0.90$ $b = 12$ in $f'_c = 4.0$ ksi

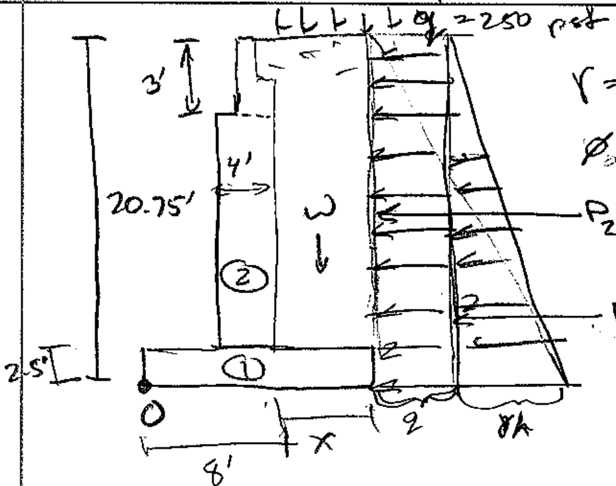
$R_n = 0.137\ k/in^2$

$\rho = 0.85\left(\frac{f'_c}{f_y}\right)\left[1.0 - \sqrt{1.0 - \frac{2R_n}{0.85\,f'_c}}\right]$ $\rho = 0.00233$

$S = \frac{1.00}{0.74} = 12.8$ in   # 8 Bars 12 in spacing

$A_s = 0.79\ in^2$ per foot

Overturning



Overturning

Overall base and height calculations of abutment

Superstructer wt not incl. in calculation

$K_a = \tan^2\left(45 - \frac{\phi}{2}\right) = .33$

$\sigma = \gamma h = (120)(20.75) = 2490 \text{ psf}$

$P_1 = \frac{1}{2} K_a \sigma h = \frac{1}{2}(.33)(2490)(20.75) = 8525 \text{ lb}$

$P_2 = (20.75)(250) = 5188 \text{ lb}$

Moment about Point O from Soil and surcharge load

$M_1 = P_1\left(\frac{2}{3}h\right) = 8525\left(\frac{2}{3}(20.75)\right) = 59 \text{ ft} \cdot \text{kip}$

$M_2 = P_2\left(\frac{1}{2}h\right) = (5188)\left(\frac{1}{2}\right)(20.75) = 54 \text{ ft} \cdot \text{kip}$

Moment from weight of soil

$W = xh\gamma = (18.25x)(120) = 2190x$

$M_w = W\left(8 + \frac{1}{2}x\right) = (2190x)\left(8 + \frac{1}{2}x\right) = (17520x + 1095x^2) \text{ ft} \cdot \text{lb}$

Weight of Concrete

$WC1 = (8 + x)(2.5)(150) = (3000 + 375x) \text{ lb/ft}$

$WC2 = (15.25)(4)(150) = 9150 \text{ lb/ft}$

Moment from wt of Concrete

$M_{wc1} = (3000 + 375x)\left(4 + \frac{x}{2}\right) = 12000 + 3000x + 188x^2$

$M_{wc2} = (9150)(6) = 54{,}900 \text{ ft} \cdot \text{lb/ft}$

Sum of Moments

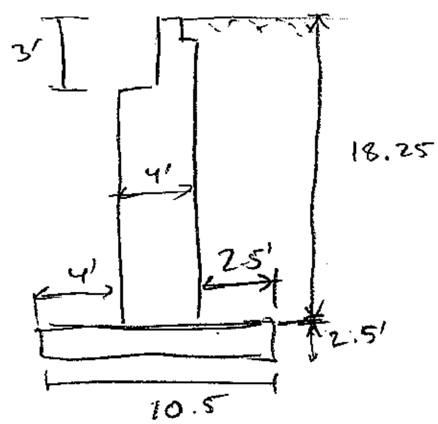
$$M_1 + M_2 = M_w + M_{wc1} + M_{wc2}$$

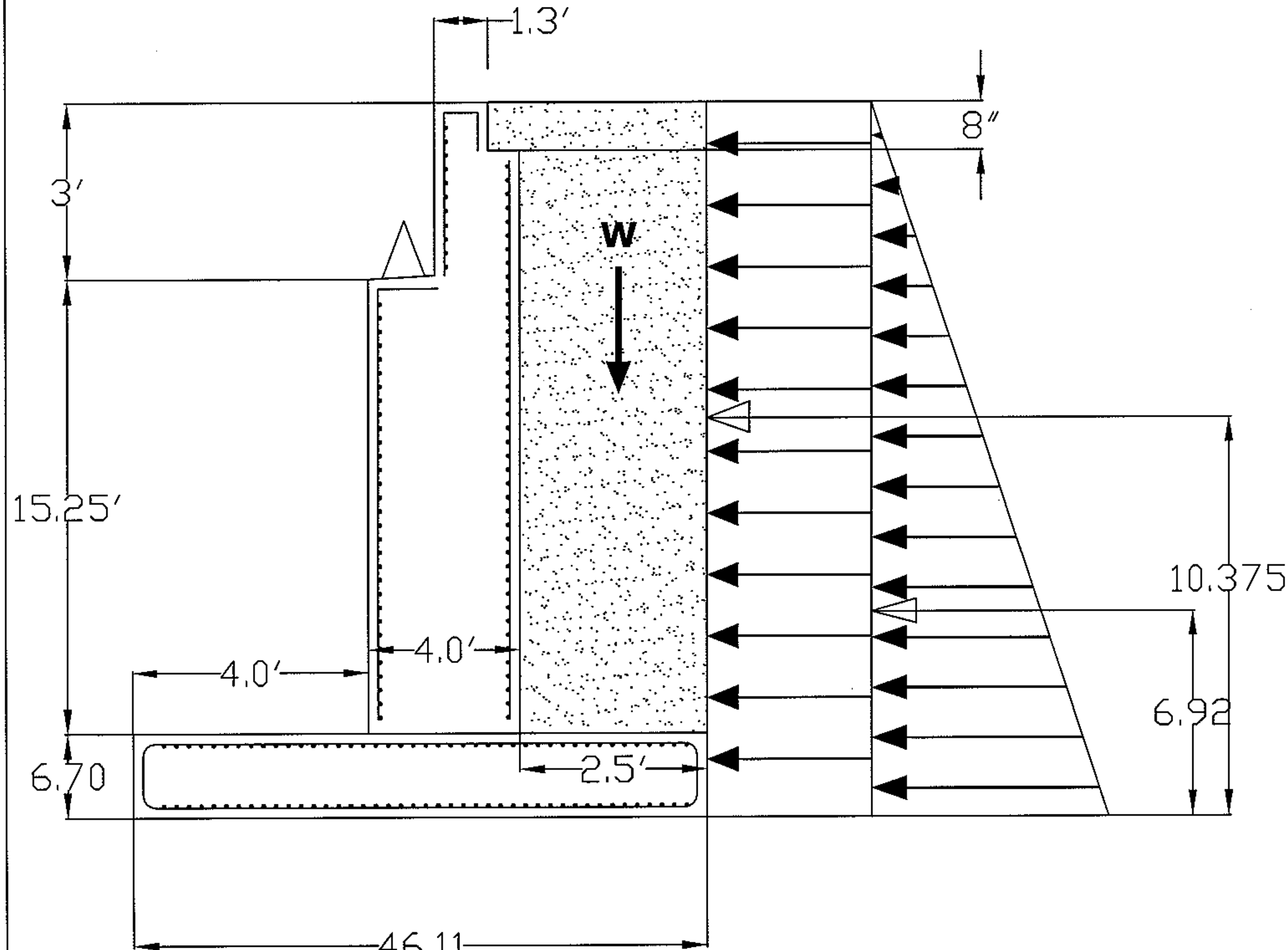
$$54 + 59 = 17.5x + 1.095x^2 + 12 + 3x + .188x^2 + 54.9$$

$$113 = 1.283x^2 + 20.5x + 54.9$$

$$1.283x^2 + 20.5x - 58.1 = 0$$

$$x = 2.45'$$




1.3′
8″
3′
W
15.25′
10.375
4.0′
4.0′
6.92
6.70
2.5′
46.11