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Sciences
Inc.

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Phoenix, Arizona 85020

35TH AVENUE OVER THE
ARIZONA CANAL DIVERSION CHANNEL
FLOOD CONTROL DISTRICT OF
MARICOPA COUNTY
CONTRACT NO. FCD 84-10
BOOKS 1 & 2 OF 6

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35TH AVENUE BRIDGE OVER the ARIZONA CANEL DIVERSION CHANNEL

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Section 1

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35TH AVENUE BRIDGE OVER

THE ARIZONA CANAL DIVERSION CHANEL

PROPOSED (PRELIMINARY) CONSTRUCTION SEQUENCE

ALT. 1

2 SPAN

1. CONSTRUCT DETOUR BRIDGE

CONSTRUCT WEST SIDE OF BRIDGE (SOUTH BOUND)

2. DETOUR TRAFFIC – SOUTH BOUND TO NORTH BOUND "TWO" LANE ROAD WAY
   – NORTHBOUND TO DETOUR ROADWAY

3. REMOVE SOUTHBOUND EXISTING ROADWAY, CURB & GUTTER, TRAFFIC ISLAND FROM ≈℄ INSIDE NB LANE WEST
   –NOTE– 12" WATER CUT OFF DURING THE CONSTRUCTION OF NEW SB ROADWAY

4. EXCAVATE ACDC TO INVERT AND BRIDGE NEAT LINE – ADVANCE AS FAR EAST AS POSSIBLE, MAY REQUIRE SHEETING ALONG ℄

5. CONSTRUCT ABUTMENTS & CENTER PIER TO ℄ BRIDGE

6. CONSTRUCT ACDC SIDE WALLS

7. BACK FILL ABUTMENTS & RETAINING WALLS

8. SET BRIDGE GIRDERS - MAY REQUIRE REMOVAL OF OVERHEADS (POWER & TELEPHONE)

9) FORM & POUR DECK TO ℄ BRIDGE

10. SET (C.I.P.) TRAFFIC & PEDESTRIAN BARRIERS - WEST SIDE OF BRIDGE

11. PLACE P.C. APPROACH PAVEMENT

12. SET TEMP. BARRIERS - 2 LANE SB

CONSTRUCT NB ROADWAY

13. DETOUR SB TRAFFIC TO NEW BRIDGE - 2 LANE

14. REMOVE NB EXISTING R/W

15. EXCAVATE TO AC DC INVERT (BEYOND BRIDGE) - MAY REQUIRE RELOCATING 42" WATER

16. CONSTRUCT PIER & ABUTMENTS

17. CONSTRUCT AC DC RETAINING WALLS

18. BACKFILL

19. SET BRIDGE GIRDERS

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20. Pour Deck

21. — Barriers

22. Approaches

23. ~~Opt~~ Strip —

NEW 35TH AVENUE BRIDGE

OVER THE ARIZONA CANAL DIVERSION CHANEL

PRELIMINARY CONCEPTS

REQUIRED:

86'-0 CURB TO CURB CLEAR ROADWAY WITH A 5'-6 SIDEWALK ON EACH SIDE OF THE ROADWAY SEPERATED BY A JERSEY TYPE TRAFFIC BARRIER. MAINTAIN THE DOUBLE-RAIL HANDRAIL ON EXTERIOR TO MATCH EXISTING BRIDGE.
(SEE SHT. 2 FOR SECTION)

℄ BRIDGE IS AT AN APPROXIMATE 22° SKEW WITH ℄ CHANEL.
CHANEL IS 110'-0 WIDE @ BOTTOM WITH AN APPROXIMATE WATER DEPTH OF 18'

PROVIDE A MINIMUM. 21'-6 CLEARENCE FROM CHANEL INVERT TO BOTTOM OF BRIDGE

MAINTAIN EXISTING PROFILE GRADE AND CROSS SLOPES OF ROADWAY

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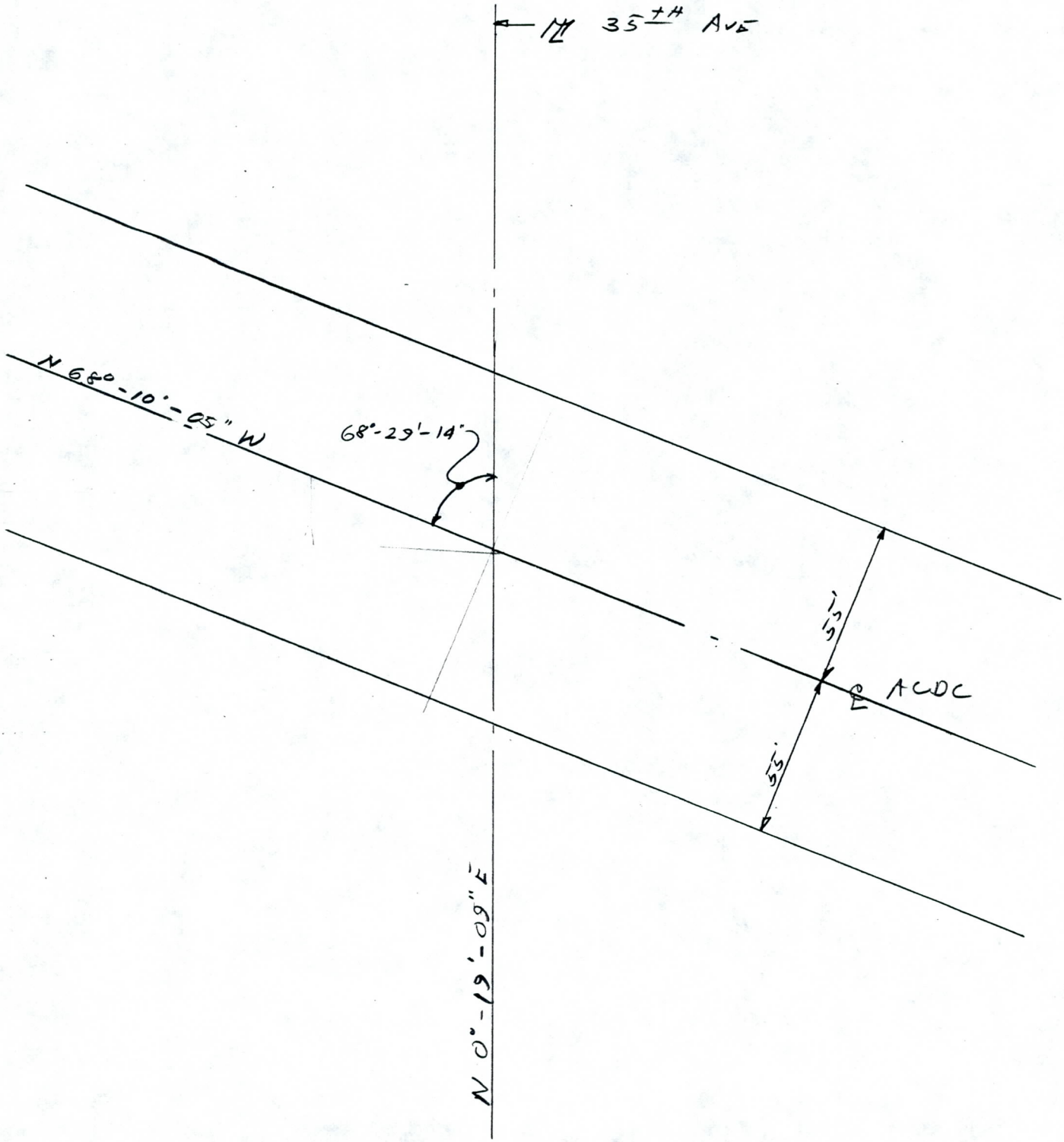
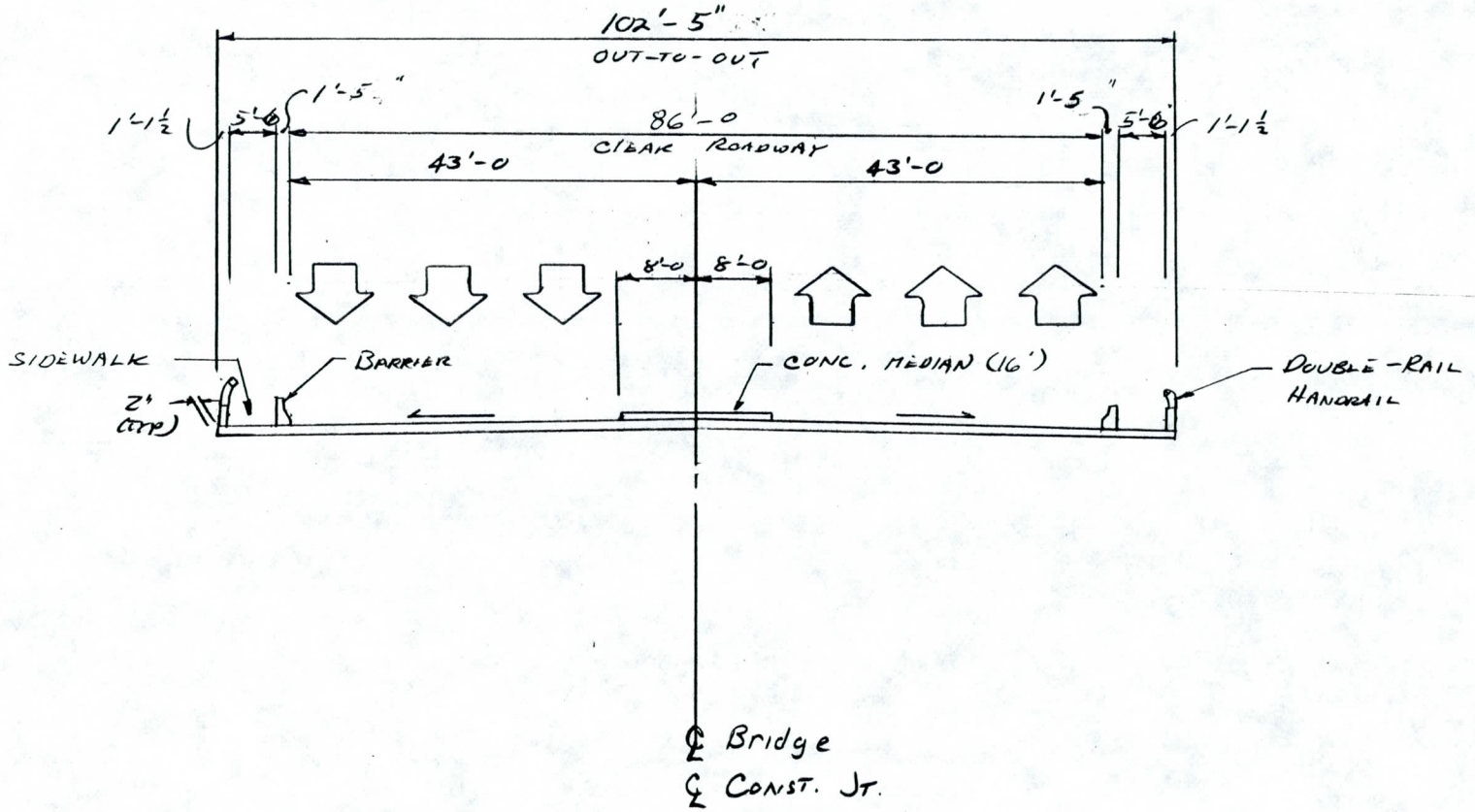


ARIZONA CANAL DIVERSION CHANNEL
35TH AVE BRIDGE



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Bridge Span Length

℄ ACDC to Edge of Channel

55'/SIN 68.50° = 59.11'

Assume 2' wall 2.00'

Assume 4'-0" cap 2.00'

Σ = 63.11' (2) = 126.24 FT ℄ Brg - ℄ Brg

USE ℄ Brg-to-℄ Brg = 126.25 FT
ABUT. 1 to ABUT. 2

Minimum Height from Bridge Deck (Top of) to Channel Bottom

Channel Bot. Elev.	1202.98	COE Datum @ Edge of Wall
	- 0.12	Correction to C.O.P. Datum
	1202.86	GES Survey Datum (COP
Approx. Top of Deck	1229.60	(Gutter Line)

ΔEL = 26.74 FT
- 21.50 Min. Clr.
5.24 FT Superst. Depth (Max)

ALLOWABLE STRESSESS

PRESTRESSED CONCRETE DESIGN AASHTO 1.6.6

PRESTRESSED GIRDER

CLASS AA CONCRETE $f'_c = 5000$ psi

TEMPORARY STRESS - BEFORE CREEP & SHRINKAGE LOSSES

$f'_{ci} = 4000$ psi

$f_c = 0.60 f'_{ci} = 2400$ psi

$f_t = 0$ psi (NO TENSION)

@ SERVICE LOAD - AFTER ALL LOSSES

$f_c = 0.40 f'_c = 2000$ psi

$f_t = 3\sqrt{f'_c} = -212$ psi

PRESTRESSING STEEL

7 WIRE 270K STRAND

$f_{pu} = 270$ ksi $f_{py} = 0.85 f_{pu} = 229.5$ ksi

TEMPORARY - BEFORE LOSSES $f_{ps} = 0.70 f_{pu} = 189$ ksi

@ SERVICE LOAD - AFTER "ALL" LOSSES $f_{ps} = 189 - 45 = 144$ ksi

ASSUMED LOSSES

BRIDGE DECK
PIER CAP
DRILLED SHAFTS

CLASS A CONCRETE $f'_c = 3500$ psi

$f_c = 0.40 f'_c = 1400$ psi

$v_c = 0.95\sqrt{f'_c} = 56$ psi

GRADE 40 REINFORCEMENT*

$f_s = 20$ ksi

(GRADE 60 MAY BE SUBSTITUTED AT CONTRACTORS OPTION)

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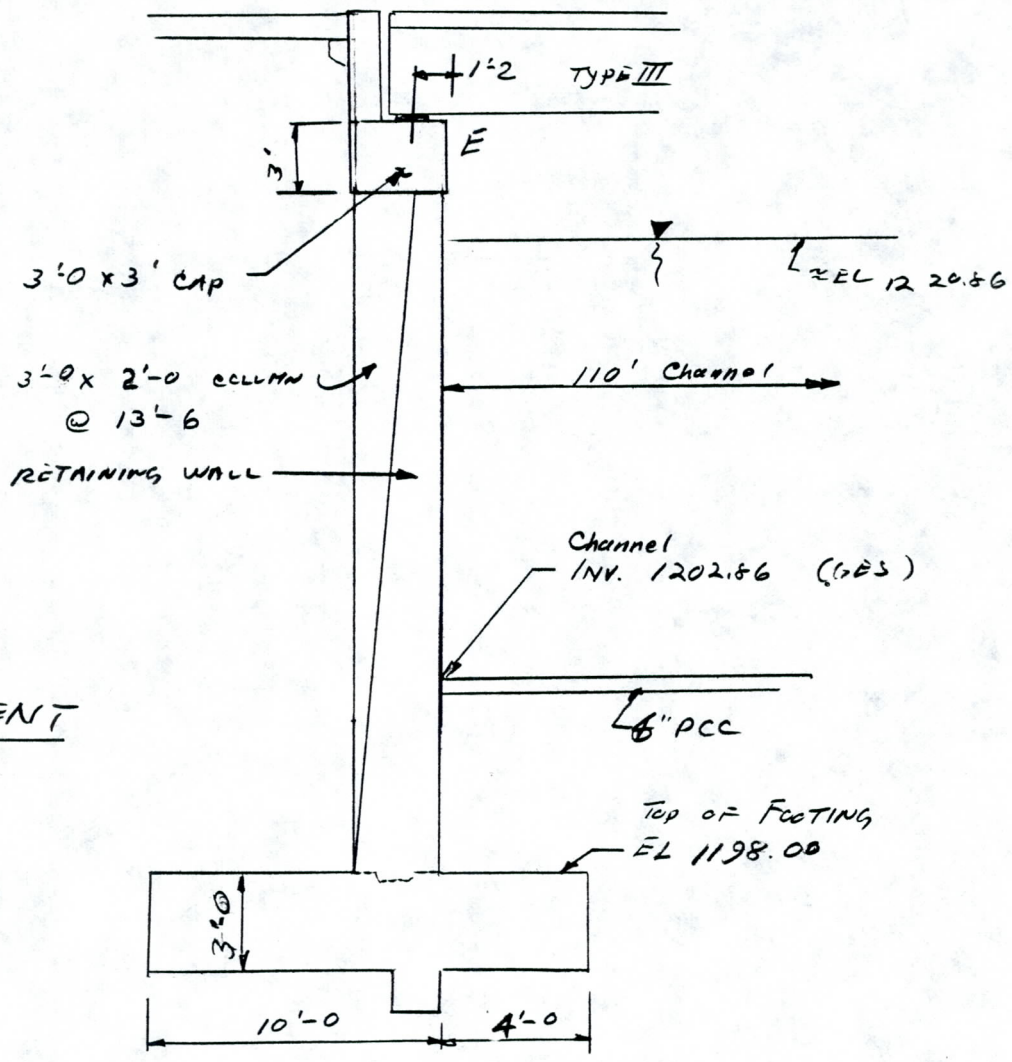
REVISE SPAN LENGTH TO REFLECT THE SETING OF THE BEAMS ON THE RETAINING WALL (ABUTMENT - RETAINING WALL)



ABUTMENT

SPAN = [55' + 1.50 - 0.75] / SIN 64.50° = 59.92'

SAY 60'

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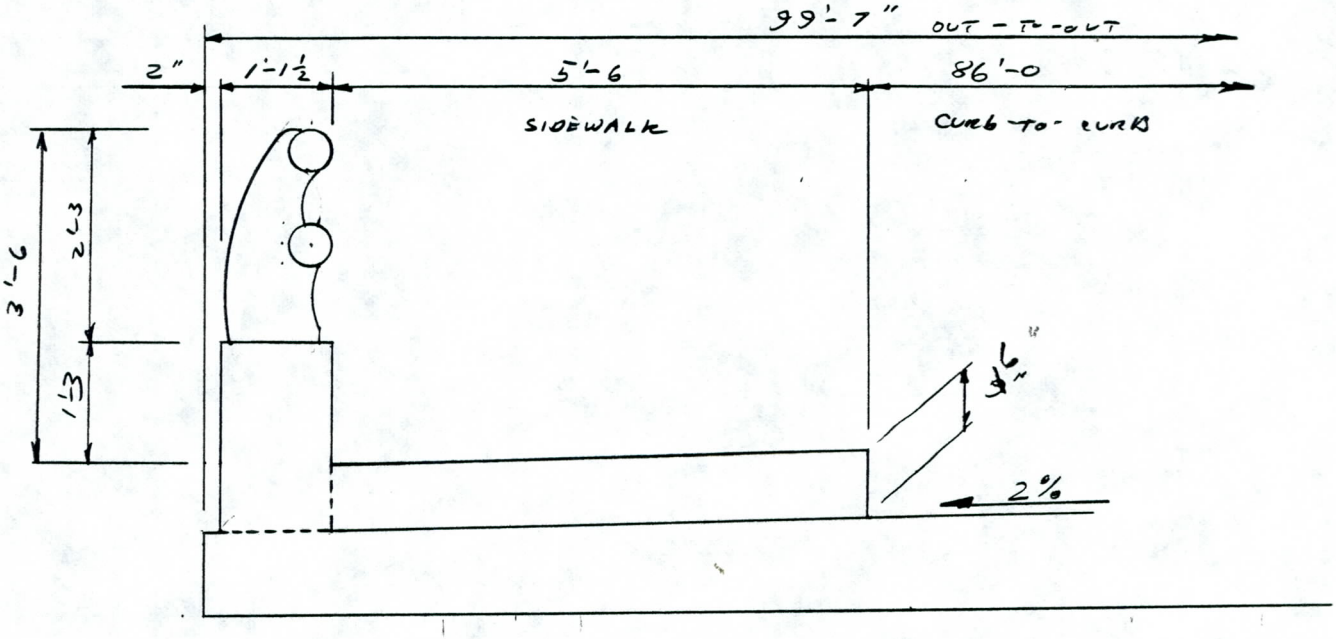

TRY 14 TYPE III GIRDERS @ 7'-2 = 93'-2" x 60'

DEAD LOADS

NON-COMPOSITE SIMPLE SPAN

TYPE III	560/144 (0.150)	=	0.583 kef
HAUNCH	1.33 (2/12) 0.150	=	0.033
DIAPHRAGM	8/12 (21.44 SF) / 60 (0.150	=	0.036
SLAB	7/12 (7.167) 0.150	=	0.627
			1.279 kef
			SAY 1.30 kef

COMPOSITE SIMPLE SPAN

BARRIER	2 (1.125) 2 (0.150) / 14	=	0.048 kef
RAILING	2 (0.250) / 14	=	0.036
SIDEWALK	9/12 (5.50) 0.150) / 14	=	0.044
FWS	[illegible] (0.025) 7.167	=	0.179
MEDIAN	16 (9/12) 0.150 / 14	=	0.129
			0.436 kef
			SAY 0.45 kef

LIVE LOAD

COMPOSITE 2-SPAN CONTINUOUS

HS 20-44

MIDSPAN BENDING MOMENTS

NON-COMPOSITE DL

$$M_{DL}^{NC} = 1.30(60^2)/8 = 585 \text{ FT-k}$$

COMPOSITE DL

$$M_{DL}^{C} = 0.45(60^2)/8 = 203 \text{ FT-k}$$

LIVE LOAD

$$I = 50/125+60 = 1.27$$

TRUCK $[32(0.2031 + 0.1440) + 8(0.1365)]60 = 732$ FT-k

$$M_{LL} = 732(1.27)\,7.167/11 = 606 \text{ FT-k}$$

GIRDER PROPERTIES

NON-COMPOSITE

$A = 560 \text{ IN}^2$
$S_B = 6186 \text{ IN}^3$
$S_T = 5070 \text{ IN}^3$

COMPOSITE

$A = 560 \text{ IN}^2$
$S_B = 8062 \text{ IN}^2$
$S_T = 3410 \text{ IN}^3$
$S_T^C = 15560 \text{ IN}^3$

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CONCRETE STRESSES @ MIDSPAN

$$P/A \pm Pe/S \mp M/S = f_c$$

@ SERVICE LOAD

BOT. $P/560 + Pe/6186 - \left[585/6186 + \frac{203+606}{8062} \right] 12000 = -212$ psi

$P/560 + Pe/6186 = 2127$ psi $\Big]\ 6186/5070$ (1)

TOP $P/560 - Pe/5070 + \left[585/5070 + \frac{203+606}{30410} \right] 12000 = +2000$ psi

$P/560 - Pe/5070 = 296$ psi (2)

(1) $1.22\ P/560 + Pe/5070 = 2595$
+ (2) $P/560 - Pe/5070 = 296$

$2.22\ P/560 = 2892$

$\therefore P = 729{,}514$ lb
$e = 6.99$ in

TRY AN 18-6 GIRDER

$P = 24(22{,}000) = 528{,}000$ lb
$e = 15.770$ in

BOT. $528000/560 + 528000(15.77)/6186 - 2339 = f_c$

$943 + 1346 - 2339 = -50$ psi > -212 psi
$\therefore$ ok

TOP $943 - 1642 + 1743 = +1044$ psi $< +2000$ psi
$\therefore$ ok

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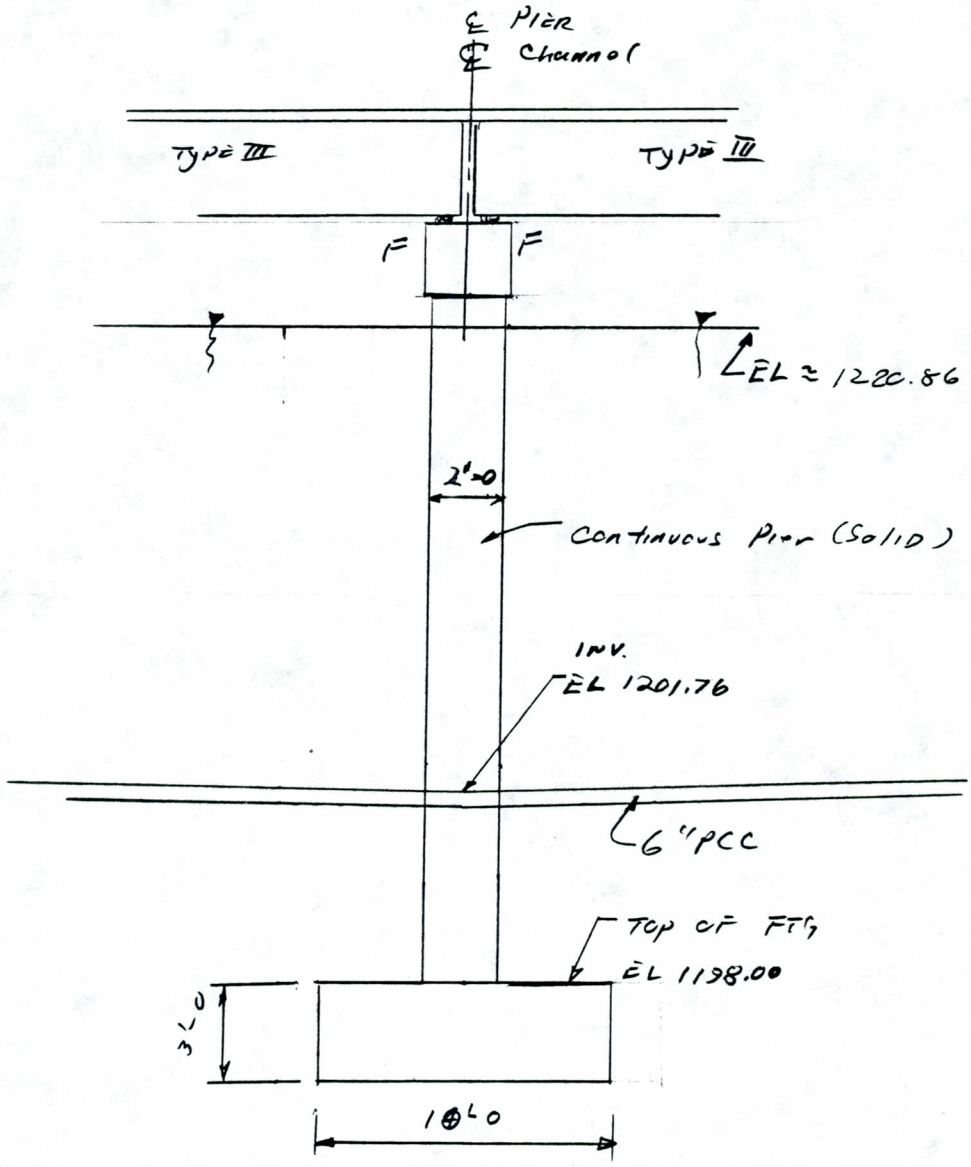

Pier @ ℄

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℄ PIER

LOADS ON FOOTER

STRUCTURE (SUPER)	(1.30 + 0.45) 14 GIRDER (60')	=	1470^k
LL.	6 LANES (32 + (32+8) 0.9247)	=	414^k
PIER CAP	3'-6 W x 3'-0 D x 103'-9 L	=	164^k
PIER (SOLID)	2'-0 D x 103'-9 L x 23'-6 H	=	731^k
FTG	10'-0 W x 3'-0 D x 106'-0 L	=	477^k
SOIL	3.80' (10 - 2) 0.120 (106.0)	=	387^k
WATER	18 (10 - 2) 106.0 (0.0625)	=	954^k
			4597^k
			/ 106 ft
		=	43.37 kef
			/ 10
		=	4.34 ksf

ASSUME FTG. FOUNDED ON SG-C LAYER

$f_{brg} \cong 10$ ksf

∴ OK

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2 - SPAN BRIDGE

ESTIMATED QUANITIES

PRECAST, PRESTRESSED TYPE III AASHTO GIRDERS

28 EA. x 61' = 1708 LF

CONCRETE DECK

$f'_c = 4000$ psi CLASS AA

$\frac{7}{12}(99'-7'')/27 = 2.151$ CY/LF

DECK LENGTH $\doteq 122.'$ (2.151) = 262.48 CY

DIAPHRAMS

ABUTMENTS $\frac{8}{12}\left[(7.1667)97.50' - 13\left(\frac{560}{144}\right)\right] 2$ ABUT = 864.26 CF

PIER $1.50\left[7.1667(13)4' - 13\left(\frac{560}{144}\right)\right]$ = 483.17 CF

MIDSPAN $\frac{8}{12}\left[7.1667(13)4 - 13\left(\frac{560}{144}\right) - \frac{7}{12}(7.1667 - 1.833)(13)\right] 2 =$ 375.56 CF

Σ = 1722.99 CF
= 63.81 CY

HAUNCH

$\frac{2''}{12}(1.333)$ 1708 LF / 27 = 14.06 CY

Σ CLASS AA = 340.4 CY SUPER-STRUCT.

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PIER 1

ALL CONCRETE CLASS A $f'_c = 3500$ psi

PIER CAP

3.50(3.00)103.75/27 = 40.35 CY

PIER 2.00(23.50)103.75/27 = 180.60 CY

FOOTER 10.00(3.00)(106.00)/27 = 117.78 CY

Σ 338.73 CY
CLASS A SUB-STRUCTURE

STRUC. EXEC. - BELOW W.L.
EL 1220.86 TO 1195.00

13(25.86)109/27 = 1357.17 CY

STRUC. BACKFILL
EL 1201.81 TO 1195.00

[16.81(13)109 - 10(3)106]/27 = 239.62 CY

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ABUTMENTS - 2 REQUIRED

ALL CONCRETE CLASS A $f'_c = 3500$ psi

PIER CAP

3.00(3.00) 107.00/27 = 35.67 CY

BACK WALL

4.75(1.00) 107.00/27 = 18.82 CY
4.75(2.00)(1.00)2/27 = 0.70 CY

COLUMNS

8 COLUMN (2.00)3.00(24.00)/27 = 42.67 CY

RETAINING WALL

[7(11.50) + (107 - 7(13.5) - 2)] 24.00 (1/2)(1.00 + 3.00)/27
= 161.78 CY

FOOTER

3.00 (14.00) 110.00/27 = 171.11 CY
1.00(1.50)110.00/27 = 6.11 CY

Σ 436.86 CY
2 ABUT => 873.72 CY

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STRUC. EXC TO W.L.
EL.

17.00 (113.00) 35.00/27 = 2490.19 CY
- 7.67 (113.00)(8.74) /27 = - 280.41
2209.78 CY
X2 = 4419.56 CY

STRU. BACKFILL

31.25 (113) 8.5 /27 = 1111.69 CY
3 (1.5) 113 /27 = 18.83
14 (3) 3.0 /27 = 4.67
4.36 (5.50) 113/27 = 100.36
1.50 (3.0) 113/27 = 18.83
24 (½) 2 [(107 - 8(2)] /27 = 80.89
1335.27 CY
X 2

= 2670.54 CY

GENERAL

REMOVAL EXISTING R/W

97.00 (155.00) /9 = 1670.56 SY

2 - RAIL RAILING = 218.00 LF

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PARAPET

CLASS A CONCRETE 3000psi

2.00 (121.00) 1.125 (2)/27 = 20.17 CY
12.00 (4.25) 1.125 (2) /27 = 4.25 CY
24.42 CY

SIDEWALK

CLASS B, f'c = 2500 psi

5.50 (0.75) 121.00 (2)/27 = 36.97 CY

MEDIAN

CLASS B

AVG WIDTH ≅ 10'-0

10.00 (0.75) 121 /27 = 33.61 CY

Σ CLASS B = 70.58 CY

STRUCTURAL EXC

TO W.L

121.00 (106.00) 8.00 /27 = 3800.30 CY

CONCRETE APPROACH SLABS CLASS A, f'c = 3000 psi

99.5833 (13.00) 1.00 (2) /27 = 110.65 CY

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COST ESTIMATE

2-SPAN PRECAST PRESTRESSED GIRDER BRIDGE

CONCRETE

CLASS AA f'_c = 4000 psi — 340.4 CY @ $250 = $85,100

CLASS A f'_c = 3500 psi — 1212.5 CY @ $200 = $242,500

CLASS A f'_c = 3000 psi — 135.1 CY @ $125 = $16,888

CLASS B f'_c = 2500 psi — 70.6 CY @ $170 = $12,002

Σ CONCRETE $356,490

PRECAST, PRESTRESSED CONCRETE BRIDGE MEMBERS

1708 LF @ $65
$ 111,020

STRUCTURAL EXCAVATION

9577.0 CY @ $5
= $47,885

STRUCTURAL BACKFILL

2910.2 CY @ $10
= $29,102

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REINFORCING STEEL
5#/CF = 135#/CY

1795 CY (135) = 242,325 Lb @ $0.40
= $ 96,930

REMOVE EXISTING ROADWAY
1670.6 SY @
= $15,000

Σ 1983 DOLLARS $ 656,427

ADJ. TO 1987 (20%) 131,285
787,712

10% CONTIGINCIES 78,771
866,483

12% PROFIT 103,978

$ 970,461 / 121.5 (86)

= $92.87/SF

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PRELIMINARY COST ESTIMATE

TWO-SPAN PRECAST, PRESTRESSED AASHTO TYPE III GIRDER BRIDGE

PAY ITEM	DESCRIPTION	ESTIMATED QUANITY	UNITS	UNIT COST	TOTAL COST
206. -1	STRUCTURAL EXCAVATION	9577.0	CY	5.00	47,885
206. -2	STRUCTURAL BACKFILL	2910.2	CY	15.00	43,653
301 -1	SUBGRADE PREPARATION	335	SY	48.00	16,080
505 -1	CONCRETE (CLASS AA) f'_c = 4000psi	340.4	CY	250.00	85,100
505 -6	CONCRETE (CLASS A) f'_c = 3500psi	1212.5	CY	200.00	242,500
505 -5	CONCRETE (CLASS A) f'_c = 3000psi	135.1	CY	125.00	16,888
505 -9	CONCRETE (CLASS B) f'_c = 2500psi	70.6	CY	170.00	12,002
506 -1	PRECAST, PRESTRESSED CONCRETE BRIDGE MEMBERS (TYPE III)	1708	LF	65.00	111,020
520 -2	STEEL HANDRAIL (DOUBLE)	225	LF	32.00	7,200
727-1	STEEL REINFORCEMENT	242,325	Lb	0.40	96,930
	TOTAL				$679,260

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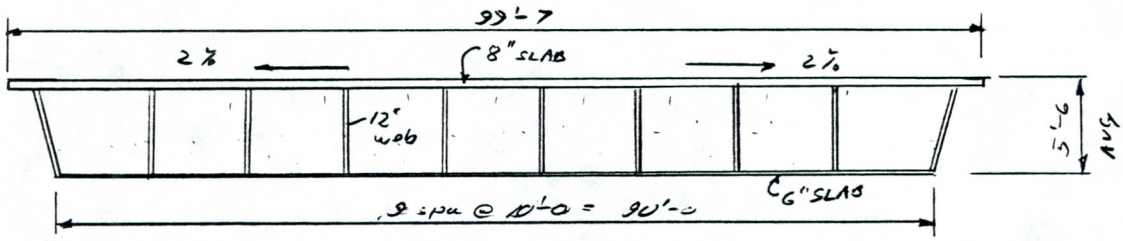
SINGLE SPAN BOX GIRDER 121.50'



	A	$\bar{y}_b$	$A\bar{y}_b$	I_o	d	Ad^2
TOP SLAB 1188(8)	9504	62	589,248	50,688	25.73	6,293,917
WEBS 10(12)52	6240	32	199,680	1,406,080	4.27	113,560
BOT. SLAB 1092(6)	6552	3	19,656	19,656	33.266	7,250,619
Σ	22,296		808,584	1,476,424		13,658,096

$$\bar{y}_b = \Sigma A\bar{y} / \Sigma A = 808584 / 22296 = 36.266 \text{ IN}$$

$$\bar{I} = 1,476,424 + 13,658,096 = 15,134,520 \text{ IN}^4$$

$$S_b = \bar{I} / 36.266 = 417,320 \text{ IN}^3$$

$$S_b / S_T = 0.8199$$

$$S_T = \bar{I} / 29.734 = 508,997 \text{ IN}^3$$

$$r = \sqrt{\bar{I}/A} = 26.05 \text{ IN}$$

ADDITIONAL AREA DUE TO FILLETS (4x4)

$$\tfrac{1}{2}(4)(4)(9)(4) = 288 \text{ IN}^2$$

$$\Sigma A = 22,584 \text{ IN}^2 = 156.833 \text{ SF} = 5.809 \text{ CY/LF}$$

DESIGN LOADS

DEAD LOAD

BOX GIRDER	22584/144 (0.150)	=	23.525 kef
RAILING	2(0.250)	=	0.500
MEDIAN	16(9/12) 0.150	=	1.800
SIDEWALK	9/12(5.50)0.150(2)	=	1.238
PARAPET	1.125(2)0.150(2)	=	0.675
DIAPHRAGMS	8/12 (90) 3/121.50 (0.150)	=	0.222
FWS	0.025 (86)	=	2.150
POST-TENSIONING		≅	0.300
			30.410 kef
			SAY 31 kef

$$M_{DL} = 31(121.50^2)/8 = 57,205 \text{ ft-k}$$

LIVE LOAD

$$I = 50/125 + 121.50 = 0.20$$

6 LANES $M_{LL} = 1910$ ft-k (6 LANE) 1.20 = 13,750 ft-k

AASHTO LANE

$$M_{DL} + M_{LL} = 70,955 \text{ ft-k}$$

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Description PRELIM

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CONCRETE STRESSES

$$P/A \pm Pe/S \mp M/S = f_c$$

BOT.

$$P/22584 + Pe/417320 - 70935(12000)/417320 = -212 \text{ psi}$$

$$P/22584 + Pe/417320 = 1828 \text{ psi} \quad (1)$$

TOP

$$P/22584 - Pe/508997 + 70935(12000)/508997 = +2000 \text{ psi}$$

$$P/22584 - Pe/508997 = 327 \text{ psi} \quad (2)$$

$$417320/508997\,(1) + (2) \qquad 1.8199\,P/22584 = 1826$$

$P = 22,659,700$ lb

$e = 15.18''$ $\quad e_b = 21.09''$

IF $e_b = 6''$ $\quad e = 30.27''$

$P = 15,648,900$ lb

$\Rightarrow$ 711 1/2" ϕ 270k STRANDS

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Description

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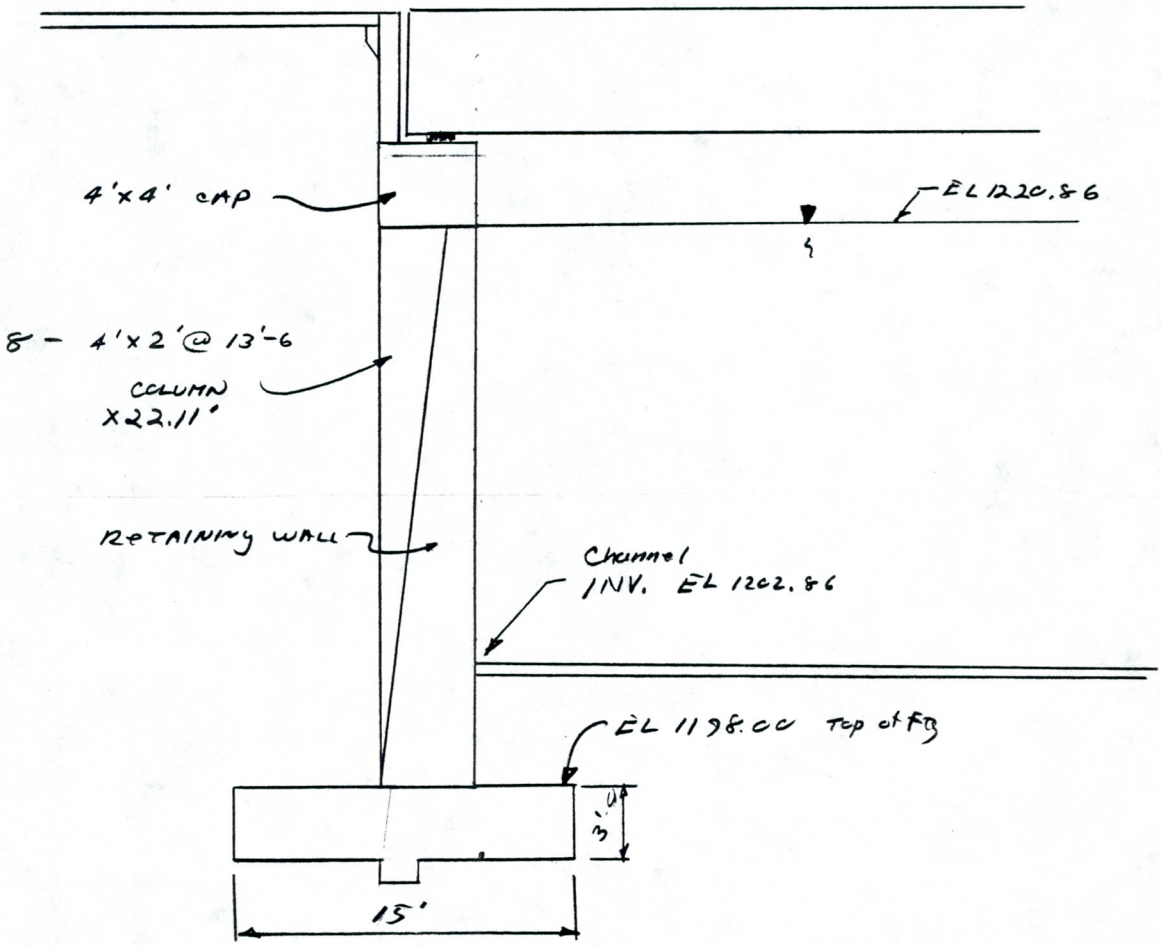
ABUTMENT



$R_{DL} = 31\,(121.50/2) = 1883.25^k$

$R_{LL} = 66.5\,(6) = 399.0$

$\Sigma = 2282^k$

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Description

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ESTIMATED QUANITIES

BOX GIRDER CONCRETE

CLASS AA f'c = 5000 psi

BOX 5.809 CY/LF (124.5) = 723.22 CY

END DIAP. 4.333 (90 − 8) (5 (1.0)) / 27 = 63.80 CY
& MIDSPANS

Σ 789.02 CY

ABUTMENT − 2 REQUIRED

CLASS A f'c = 3500 psi

PIER CAP

4 (4) 107/27 = 63.41 CY

BACK WALL

5.50 (1.00)(107)/27 = 21.80 CY
5.50 (3.00) 1.00/27 (2) = 1.22 CY

COLUMNS

8 COL (4)(2) 22.11/27 = 52.41 CY

RETAINING WALL

(107 − 8(2)) ½ (1+4) 22.11/27 = 186.30 CY

FOOTER

3 (15) 110/27 = 183.33 CY
1 (1.50) 110/27 = 6.11 CY

Σ 514.58 CY (2) = 1029.16 CY

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2-RAIL RAILING 218 LF

REMOVE EXISTING ROADWAY 1670.56 SY.

PARAPET

CLASS A f'c = 3000 psi 24.42 CY

SIDEWALK

CLASS B f'c = 2500 psi 36.97 CY

MEDIAN

CLASS B f'c = 2500 33.61 CY

APPROACH SLAB

CLASS A f'c = 3000 psi 110.65 CY

STRUCTURAL EXCA TO WATER LINE

$[121.50 - 2(4+1.5)]\ 8.00 / 27\ (106) = 3470.52$ CY

ABUT. STRUC. EXC

$18(110)\ 35/27 = 2566.67$ CY

2 ABUT X => 5133.33 CY

BACKFILL

$[8.50\ (31) + 3(1.50)]\ 110/27 = 1091.85$

$[107 - 8(2)]\ \frac{1}{2}(3)(22.11)/27 = 111.78$

$2(1.50)\ 31.(4)/27 = 13.78$

$1217.41 \times 2 = 2434.82$ CY

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POST - TENSIONING

711 STRAND (124.5') 0.425 = 37,621 lb

REINFORCING

5 #/CF

(789.02 + 1029.16 + 24.42 + 36.97 + 33.61 + 110.65) 27 (5) = 273,217 lb

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COST ESTIMATE

SINGLE SPAN BOX GIRDER BRIDGE

CONCRETE

CLASS AA $f'_c = 5000$ psi — 789.0 CY @ $300 = $236,700

CLASS A $f'_c = 3500$ psi — 1029.2 CY @ $200 = $205,840

CLASS A $f'_c = 3000$ psi — 135.1 CY @ $125 = $16,888

CLASS B $f'_c = 2500$ psi — 70.6 CY @ $170 = $12,002

STRUCTURAL EXCAVATION

8603.9 CY @ $5 = $43,020

STRUCTURAL BACKFILL

2434.8 CY @ $10 = $24,348

REINFORCING STEEL

273,217 Lb @ $0.40 = $109,287

REMOVAL OF EXISTING ROADWAY

1670.6 SY @ = $15,000

POST-TENSIONING STEEL

37,621 Lb @ $0.70 = $26,335

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Σ 1983 DOLLARS	$ 689, 420
ADJ TO 1987 (20%)	137, 884
	827,304
10% CONTINGENCIES	82, 730
	910 034
12% PROFIT	109, 204
	$ 1, 019, 238

/ 121.5 / 86

= $ 97.54/SF

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PRELIMINARY COST ESTIMATE

SINGLE SPAN CAST-IN-PLACE BOX GIRDER BRIDGE

PAY ITEM	DESCRIPTION	ESTIMATED QUANTITY	UNITS	UNIT COST	TOTAL COST
206-1	STRUCTURAL EXCAVATION	8603.9	CY	5.00	43,020
206-2	STRUCTURAL BACKFILL	2434.8	CY	15.00	36,522
301-1	SUBGRADE PREPERATION	335	SY	48.00	16,080
505-3	CONCRETE (CLASS AA) f'_c = 5000psi	789.0	CY	300.00	236,700
505-6	CONCRETE (CLASS A) f'_c = 3500psi	1029.2	CY	200.00	205,840
505-5	CONCRETE (CLASS A) f'_c = 3000psi	135.1	CY	125.00	16,888
505-9	CONCRETE (CLASS B) f'_c = 2500psi	70.6	CY	170.00	12,002
506-6	POST-TENSIONING STEEL ½"ϕ 270k 7-WIRE	37,621	LB	0.80	30,097
520-1	STEEL (DOUBLE) HANDRAIL	225	LF	32.00	7,200
727-1	STEEL REINFORCEMENT	273,217	LB	0.40	109,287
	TOTAL				$713,640

PRELIMINARY COST ESTIMATE FOR THE NEW 35th AVENUE BRIDGE OVER THE ARIZONA CANAL DIVERSION CHANNEL

IN THE DESIGN OF THE NEW 35th AVENUE BRIDGE OVER THE ARIZONA CANAL DIVERSION CHANNEL (ACDC) TWO BRIDGE TYPES WERE EXAMINED. A SINGLE-SPAN CAST-IN-PLACE CONCRETE MULTI-CELL POST-TENSIONED BOX GIRDER AND A TWO-SPAN PRECAST, PRESTRESSED CONCRETE AASHTO TYPE III GIRDER BRIDGE WERE PRELIMINARY DESIGNED AND A COST ESTIMATE WAS CALCULATED. BOTH DESIGN WERE ESTIMATED TO COST APPROXIMATLY $700,000 FOR THE BRIDGE STRUCTURE WITH THE BOX GIRDER BRIDGE BEING SLIGHTLY HIGHER (SEE COST ESTIMATES). BASED ON THE FOLLOWING DESCRIPTIONS OF EACH DESIGN IT CAN BE SHOWN THAT THE TWO-SPAN BRIDGE IS THE MORE FESIABLE FOR THIS SITE.

THE SINGLE-SPAN BOX GIRDER CONSISTS OF A MULTI-CELL CAST-IN-PLACE POST-TENSIONED CONCRETE STRUCTURE. THE SINGLE SPAN CAST-IN-PLACE CONFIGURATION IS OPTIMUM FROM THE CHANNEL HYDRAULICS AND OVERHEAD CONSTRUCTION CLEARENCES STANDPOINT BUT THERE OTHER CONDITIONS THAT MUST BE EXAMINED. A POST-TENSIONED BOX GIRDER IS A COMPLICATED FORMING AND PRESTRESSING STRUCTURE WHICH MAY LIMIT THE CONTRACT TO A FEW CONTRACTORS. THE BOX GIRDER ALSO DOES NOT LEND ITSELF BEING CONSTRUCTED AND PLACED IN SERVICE ONE HALF AT A TIME. THIS TYPE STRUCTURE IS DESIGN TO ACT AS AS A CONTINUOUS UNIT AND IS THE MOST EFFICIENT ONLY WHEN SUCH IS THE CASE. ANOTHER CONSIDERATION TO BE AWARE OF IS THAT SINCE THIS IS A CAST-IN-PLACE CONCRETE STRUCTURE, CONSTRUCTION TIME WILL BE INCREASED, POSSIBLY

AS MUCH AS 30 TO 60 DAYS. ON SUCH A HIGH VOLUMNE ROADWAY, ANY ADDITIONAL OR DELAYS IN CONSTRUCTION TIME IS UNDESIRABLE.

THE TWO-SPAN AASHTO GIRDER BRIDGE CONSIST OF 14 TYPE III PRECAST, PRESTRESSED AASHTO GIRDERS WITH A SOLID PIER LOCATE ON THE CHANNEL CENTER LINE. THE PRIMARY ADVANTAGE OF THIS CONFIGURATION IS THE EASE AND SPEED OF CONSTRUCTION AND THE LARGE NUMBER OF QUALIFIED CONTRACTORS. THE ONLY DISADVANTAGE THAT CAN BE CONTRIBUTED TO THE SITE IS DURING THE ERECTION OF THE SOUTH SPAN THE 235 KV OVERHEAD POWER LINE WILL BE REQUIRED TO BE SHUT-OFF AND POSSIBLY DISCONNECTED, BUT S.R.P. AUTHORITIES CONFIRM THAT THIS WOULD ONLY BE A MATTER OF CORDINATION.

IN CONCLUSION IT HAS BEEN SHOWN THAT THE TWO-SPAN CONCRETE GIRDER DESIGN IS THE PROPER CHOICE FOR THIS SITE. THE BOX GIRDER ALTERNATE WOULD CAUSE NUMEROUS PROBLEMS AND DELAYS IN CONSTRUCTION. IF IT IS FURTHER DEEMED THAT A SINGLE SPAN STRUCTURE IS REQUIRED IT WOULD BE NECESSARY TO DESIGN THE BRIDGE AS TWO PARRALEL STRUCTURES WITH A CORESPONDING INCREASE IN COST. IT IS THIS ENGINEERS RECOMMENDATION TO USE THE TWO-SPAN PRECAST, PRESTRESSED CONCRETE AASHTO TYPE III GIRDER ALTERNATE AS PRESENTED ON THE ENCLOSED DRAWINGS

RAYMOND L. COX, P.E.

PRESTRESSED GIRDER ALT.

TWO 62'-4½ (SAY 62.50') SPAN

SIMPLE UNDER DEAD LOADS
2-SPAN CONTINUOUS DUE TO LIVE LOAD

MIN GIRDER DEPTH = $\ell/30$ = 2.08 FT
MIN STRUCTURE DEPTH = $\ell/25$ = 2.50 FT

TRY 15 - TYPE III AASHTO GIRDERS
@ 6'-10 SPACING = 95'-8"

DEAD LOADS

NON-COMPOSITE
SIMPLE SPAN

TYPE III GIRDER	560/144 (0.150)	=	0.583 kef
DIAPHRAGM	6/12 (6.83) 0.15/62.5	=	0.008
HAUNCH	2/12 (1.63) 0.150	=	0.042
SLAB	7/12 (6.83) 0.150	=	0.598
			1.231
			SAY 1.25 kef

COMPOSITE
SIMPLE-SPAN

BARRIERS	2(0.305)/15	=	0.041
RAILING	2(0.250)/15	=	0.033
FWS	0.025(6.83)	=	0.171
			0.245
			SAY 0.25 kef

LIVE LOAD

COMPOSITE
2-SPAN CONTINUOUS

HS 20 - 44

BENDING MOMENTS

@ MIDSPAN

NON-COMPOSITE DL $M_{NC}^{DL} = w\ell^2/8 = 1.25(62.50^2)/8 = 610^{FT-k}$

COMPOSITE DL $M_{COM}^{DL} = 0.25(62.50^2)/8 = 122^{FT-k}$

COMPOSITE LL $I \cong 1.30$

TRUCK $[32^k(0.2031 + 0.1440) + 8^k(0.1365)]62.50 = 763^{FT}$

$M_T^{LL} = 763(1.30)\,6.83/11 = 616^{FT-k}$ ← CONTROLS

LANE

$0.680(0.0938)62.5^2 + 18(0.2031)62.5 = 478^{FT-k}$

$M^{LL} = 478(1.30)\,6.83/11 = 386^{FT-k}$

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GIRDER PROPERTIES

TYPE III

NON-COMPOSITE

$A = 560 \text{ IN}^2$

$\bar{y}_b = 20.27 \text{ IN}$

$I = 125,390 \text{ IN}^4$

$S_B = 6186 \text{ IN}^3$

$S_T = 5070 \text{ IN}^3$

COMPOSITE

	A	$\bar{y}_b$	$A\bar{y}_b$	I_o	d	Ad^2
TYPE III	560	20.27	11,351.2	125,390	15.30	131,090
7" * 82" SLAB	574	50.5	28,987.0	2344	14.93	127,947
	1134 IN²		40,338.2 IN³	127,734		159,037
						127,734

$\bar{y}_b = 40338.2 / 1134 = 35.57 \text{ IN}$

$\bar{I}_o = 286,771 \text{ IN}^3$

$S_B = 8062 \text{ IN}^3$

$S_T = 30,410 \text{ IN}^3$

$S_T^c = 15,560 \text{ IN}^3$

CONCRETE STRESSES

$$f = P/A \mp Pe/S \pm M/S$$

$$P_j/P = 189\,ksi/144\,ksi = 1.3125$$

$$\therefore P_j = 1.3125\,P$$

@ SERVICE LOAD
MIDSPAN

BOT.

$$P/A + Pe/S_B - M_{nc}^{DL}/S_B - M_c^{DL}/S_B^{comp} - M^{LL}/S_B^{comp} = f_c^t$$

$$P/560 + Pe/6186 - \left[610/6186 + \frac{122+616}{8062}\right]12000 = -212$$

$$\frac{6186}{5070}\left[\ \therefore\ P/560 + Pe/6186 = 2070\right. \qquad (A)$$

TOP

$$P/A - Pe/S_T + M_{nc}^{DL}/S_T + \frac{M_c^{DL} + M^{LL}}{S_T^{comp}} = f_c^c$$

$$P/560 - Pe/5070 + \left[610/5070 + \frac{122+616}{30410}\right]12000 = +2000$$

$$\therefore P/560 - Pe/5070 = 265 \qquad (B)$$

$$1.220\,P/560 + Pe/5070 = 2526$$
$$P/560 - Pe/5070 = 265$$

$$2.220\,P/560 = 2791$$

$$\therefore P = 704.04^{k}$$
$$e = 7.14^{in}$$

TRY A- TYPE III GIRDER
18-6 PATTERN

$P = 24(22) = 528^k$
$e = 15.770$ IN

@ SERVICE LOAD

BOT $943 + 1346 - 2282 = +7$ psi ≥ -212 psi ∴ ok

TOP $943 - 1642 + 1735 = +1036$ psi $\leq +2000$ psi ∴ ok

USE 15 - TYPE III AASHTO GIRDERS
w/ 7" SLAB

NOTE: LESS GIRDERS MAY HAVE BEEN USED BY SPECIFING AN INCREASED SPACING, BUT THE INCREASED STRUCTURE FLEXIBILITY WOULD BE DETRIMENTAL. THE SUBSTRUTURE WOULD NOT BE APPRECIABLY CHANGED DUE TO LESS GIRDERS

EST. SUPERSTRUTURE QUANITIES

TYPE III GIRDERS x 62'-9"	30	EA.
CONCRETE CLASS S $f'_c = 4500$ psi	288	CY
102.4167 (130) 7/12/27		
BARRIER, HALF	260	LF
RAILING, TWO	260	LF

2 SPAN UNIT

Center PIER

REACTION

DEAD LOAD

15 GIRDERS (1.50 kips) 62.50 = 1410 k

LIVE LOAD

6 LANE [61.5(2) - 32] = 546 k

PIER CAP

4'(4') 100' (0.150) = 240 k

Σ 2200 k

ASSUME 36" DRILLED SHAFTS
0.60 ksf SIDE FRICTION = 5.65 klf
12 ksf end Bearing = 84.8 k

Top 19' of shaft unsupported

$w = \frac{\pi}{4} 3^2 (0.150) = 1.10$ klf

$2200^k + n(12)1.10 = n(84.8 + 5.65\ell)$

$63.9n + 5.65\ell n = 2200$

TRY 11 SHAFTS @ 9'-0 = 90'-0

$\ell = 24'$ below channel invert
$(L = 45')$

ABUTMENTS

DEAD LOAD = 705^k

LIVE LOAD 6(61.5) = 370^k

Pier cap = 240^k

$\bar{\Sigma} = 1315^k$

$63.9n + 5.65\ell n = 1315$

TRY 7 shafts @ 14'-6 = 87'-0

$\ell = 22'$ below channel invert
$(L = 45')$

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ESTIMATED QUANTITIES & COSTS

DRILLED SHAFTS 36"

[11 shafts + 2(7 shafts)] 45' = 1125 LF @ $70 / $78,750

PIER CAP

4(4)100/27 = 60 CY @ $160 / $9600

ABUTMENT CAP(S)

[4(4)102.5/27]2 = 122 CY @ $160 / $19,520

GIRDERS

30 GIRDERS (63.50') = 1905 LF @ $45 / $85,725

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2-Span

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BRIDGE DECK

102.5'(130')7/12/27 = 290 CY
@ $160
$46,400

REINFORCING
5#/CF

(290 + 60 + 122) 27 (5) = 63,720 Lb
@ 35¢
$22,302

PARAPET & RAILING 2602 LF
@ $45
$11,700

MEDIAN
8/12 (16)/27 (130) = 51.50 CY
@ 150
$7725

STRUCTURAL EXCAVATION

115(10')130'/27 + 100(18)7/27 = 6005
@ $5
$30,020

Σ 310,000 ⇒ $35.08/SF ROWY

SINGLE SPAN AASHTO PRESTRESSED GIRDERS

126.25 FT SPAN

$M^{LL} = 2000$ FT-K / LANE NO IMPACT AASHTO App. A

$I = 50/100 + 126.25 = 0.22$

$M^{LL} = 2440$ FT-K / LANE

MIN BEAM DEPTH $= \ell/25 = 5.05$ FT

TRY A TYPE V GIRDER

FIND TOTAL CAPACITY OF A TYPE V 26/6 GIRDER

$P = 32(22) = 704^k$

$e = 30.210$ IN @ MIDSPAN

$I^{NC} = 521,180$ IN^4 $A = 1013$ IN^2

$S_b^{NC} = 16,307$ IN^3

ASSUME COMPOSITE GIRDER $S_b^c = 1.3\, S_b^{NC}$

$S_b^c = 21,200$ IN^3

STRESSES $P/A + Pe/S_b^{NC} - M^{LL}/S_b^c = -212$ psi

$704000/1013 + 704000(30.210)/16307 - M/21200 = -212$

$M = 3906$ FT-K

ASSUME $M = 2.5\ M^{LL}$

$M^{LL} = 1562^{FT-k}/GIRDER$

$1562^{FT-k}/GIRDER = 2240^{FT-k}/LANE\ [S/11\ LANES/GIRDER]$

$\therefore S = 7.67^{FT}$

TRY 10 TYPE V @ 10'-0 = 90'-0

LOADING

DEAD LOADS

NON-COMPOSITE

TYPE V GIRDER		=	1.055 kef
DIAPHRAGM	$3[6/12(46.5\ SF)\ 0.150/126.25]$	=	0.083
HAUNCH	$2/12\ (3.50)\ 0.150$	=	0.088
SLAB	$7/12\ (10)\ 0.150$	=	0.875
			2.101 kef

$M_{\mathcal{L}}^{+} = 2.10(126.25^2)/8 = 4186^{FT-k}$

COMPOSITE

BARRIERS	$2(0.305)/10$	=	0.061
RAILING	$2(0.250)/10$	=	0.050
FWS	$0.025(10)$	=	0.250
			0.361 kef

$M_{\mathcal{L}}^{+} = 0.361(126.25^2)/8 = 719^{FT-k}$

LIVE LOAD

HS 20 - 44

$$M^{LL} = \frac{0.640(126.25)^2}{8} + \frac{18(126.25)}{4} = 1845 \text{ FT-K/LANE}$$

$$I = 1.22$$

$$M^{LL} = 1845(1.22)\,10/11 = 2045 \text{ FT-K/GIRDER}$$

GIRDER PROPERTIES

NON-COMPOSITE

$I = 521,180 \text{ IN}^4$
$\bar{y}_b = 31.96 \text{ IN}$
$A = 1013 \text{ IN}^2$

$S_b = 16,307 \text{ IN}^3$
$S_t = 16,791 \text{ IN}^3$

COMPOSITE

	A	$\bar{y}_b$	$A\bar{y}$	$\bar{I}$	d	Ad^2
TYPE V	1013	31.96	32,375.5	521,180	13.42	182,438
7" SLAB x 84"	588	68.50	40,278.0	2401	23.12	314306
	1601 IN²		72653.5	523,581		496,744

$$\bar{y}_b = 72653.45/1601 = 45.38 \text{ IN}$$

$$\bar{I}_c = 523,581 + 496,744 = 1,020,325 \text{ IN}^4$$

$$S_b = 22,484 \text{ IN}^3$$

$$S_T = 57,907 \text{ IN}^3$$

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CONCRETE STRESSES

SERVICE LOAD @ MIDSPAN

BOT. $P/A + Pe/S - M_{DL}/S - M_{LL}/S \geq -212$ psi

$$P/1013 + Pe/16307 - \left[4186/16307 + \frac{719+2045}{22484}\right]12000 = -212$$

$$\frac{16307}{16791}\left[\ P/1013 + Pe/16307 = 4344 \qquad (A)\right.$$

TOP $P/A - Pe/S + M_{DL}/S + M_{LL}/S \leq 2000$ psi

$$P/1013 - Pe/16791 + \left[4186/16791 + \frac{719+2045}{37907}\right]12000 = 2000$$

$$P/1013 - Pe/16791 = -1564 \qquad (B)$$

$$\begin{array}{ll} (A) & 0.971\,P/1013 + Pe/16791 = 4219 \\ +\ (B) & P/1013 - Pe/16791 = -1564 \\ \hline & 1.971\,P/1013 = 2655 \text{ psi} \end{array}$$

$\therefore$ $P = 1{,}364{,}395$ lb

$e = 35.82$ in

$Pe = 48\,873$ in-k

LARGEST TYPE V IS 38

$P = 836^k$

$e = 27.38$ in

$Pe = 22{,}890$ in-k

$\therefore$ NG

Type VI

TRY 11 TYPE VI @ 9'-0 = 90'-0

DEAD LOADS

NON-COMPOSITE

TYPE VI	GIRDER	=	1.130 kcf
DIAPHAGM	3[47.97 sf (6/12) 0.150]/126.25	=	0.086
HAUNCH	3/12 (3.50) 0.150	=	0.088
6½" SLAB	6.5/12 (9) 0.150	=	0.731
			2.035 kcf
			SAY 2.10 kcf

$$M_{\mathcal{E}}^{DL-NC} = 2.10(126.25^2)/8 = 4184^{FT-k}$$

COMPOSITE

BARRIERS	2 (0.390)/11	=	0.071 klf
RAILING	2 (0.250)/11	=	0.046
FWS	86 (0.025)/11	=	0.195
			0.312 kcf
			SAY 0.32 kcf

$$M_{\mathcal{E}}^{DL-C} = 0.32(126.25^2)/8 = 638^{FT-k}$$

LIVE LOAD

HS 20-44

$$M_{\mathcal{E}}^{LL} = 1845^{FT-k}/LANE\ (1.22)\ 9/11 \underset{I\ \ 9/11}{} = 1842^{FT-k}/GIRDER$$

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GIRDER SECTION PROPERTIES

TYPE VI 6'-5 or greater spacing

NON-COMPOSITE

$A = 1085 \text{ IN}^2$

$I = 733,320 \text{ IN}^4$

$\bar{y}_b = 36.38 \text{ IN}$

$S_b = 20,157 \text{ IN}^3$

$S_T = 20,587 \text{ IN}^3$

COMPOSITE

	A	$\bar{y}_b$	$A\bar{y}_b$	I_o	d	Ad^2
TYPE VI	1085	36.38	39472.3	733,320	13.02	183,817
6½" x 78" SLAB	507	77.25	39165.8	1785	27.85	393,354
	1592 IN²		78,638 IN³	735,105		577,170

$\bar{y}_b = 78638/1592 = 49.396 \text{ IN}$

$\bar{I} = 735105 + 577170 = 1,312,275 \text{ IN}^4$

$S_b^c = 1312275/49.396 = 26,566 \text{ IN}^3$

$S_t^c = 1312275/22.604 = 58,055 \text{ IN}^3$

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STRESSES @ MIDSPAN

BOT $\quad P/1085 + Pe/20157 - \left[4184/20157 + \frac{638+1842}{26566}\right]12000 \geq -212$

$20157/20587 \left[P/1085 + Pe/20157 = 3399 \right. \qquad (A)$

TOP $\quad P/1085 - Pe/20587 + \left[4184/20587 + \frac{638+1842}{58055}\right]12000 \leq 2000$

$P/1085 - Pe/20587 = -951 \qquad (B)$

(A) + (B) $\quad 1.977\, P/1085 = 2377$

$\therefore \; P = 1{,}303{,}210^{LB}$

$e = 33.995^{IN}$

$Pe/22000 = 2013.8^{IN-STRAINS}$

40/18 GIRDER $\quad Pe/22000 = 1668 < 2014$

$\therefore$ REDUCE GIRDER SPACING

TRY 12 TYPE VI @ 8'-3 = 90'-9
W/ 6½" SLAB

DEAD LOADS

NON-COMPOSITE

TYPE VI GIRDER		=	1.130 kef
DIAP.	3[43.35 SF (6/12) 0.150]/126.25	=	0.077
HAUNCH	3/12 (3.5) 0.150	=	0.088
6½" SLAB	6.5/12 (8.25) 0.150	=	0.670
			1.965 kef
			say 2 kef

$$M_{\mathcal{E}} = 2(126.25^2)/8 = 3985^{FT-k}$$

COMPOSITE

BARRIERS & RAILING	2(0.390+0.250)/12	=	0.107
FWS	86(0.025)/12	=	0.179
			0.286
			say 0.30 kef

$$M_{\mathcal{E}} = 0.30(126.25^2)/8 = 598^{FT-k}$$

LIVE LOAD

$$M_{\mathcal{E}} = 1845(1.22)^{8.25}/11 = 1688^{FT-k}$$

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STRESSES

SERVICE @ MIDSPAN

BOT

$$P/1085 + Pe/20157 - \left[3985/20157 + \frac{598+1688}{26566} \right] 12000 = -212$$

$$20157/20587 \left[P/1085 + Pe/20157 \right. = 3193 \qquad (A)$$

TOP

$$P/1085 - Pe/20587 + \left[3985/20587 + \frac{598+1688}{38035} \right] 12000 = 2000$$

$$P/1085 - Pe/20587 = -795 \qquad (B)$$

(A) + (B)

$$1.979\, P/1085 = 2331$$

$$\therefore P = 1,278,100 \text{ lb}$$
$$e = 31.78 \text{ in}$$

$$Pe/22000 = 1846$$

40/18 $\quad Pe/22000 = 1668 < 1846$

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TRY 14 TYPE VI @ 7'-4 = 95'-4
w/ 6½" SLAB

DEAD LOADS

NON-COMPOSITE

TYPE VI GIRDER		=	1.130 kef
HAUNCH	2/12 (3.3) 0.150	=	0.088
DIAP.	3 [37.69] 6/12 (0.150) / 126.25	=	0.067
SLAB	6.5/12 (7.333) 0.150	=	0.596
			1.881
			SAY 1.90 kef

$$M_{\mathcal{L}} = 1.90(126.25^2)/8 = 3786^{FT-k}$$

COMPOSITE

BARRIERS & RAILING	2(0.390 + 0.250)/14	=	0.091
FWS	86(0.025)/14	=	0.154
			0.245
			SAY 0.25 kef

$$M_{\mathcal{L}} = 0.25(126.25^2)/8 = 498^{FT-k}$$

LIVE LOAD

$$M_{\mathcal{L}} = 1845(1.22)\,7.33/11 = 1500^{FT-k}$$

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STRESSES

BOT $\quad P/1085 + Pe/20157 - \left[3786/20157 + \frac{498+1500}{26566}\right]12000 = -212$

$20157/20587 \left[P/1085 + Pe/20157 = 2944 \right. \qquad (A)$

TOP $\quad P/1085 - Pe/20587 + \left[3786/20587 + \frac{498+1500}{58055}\right]12000 = 2000$

$P/1085 - Pe/20587 = -620 \qquad (B)$

(A) + (B) $\quad 1.978\, P/1085 = 2262.5$

$\therefore P = 1{,}240{,}365$ lb

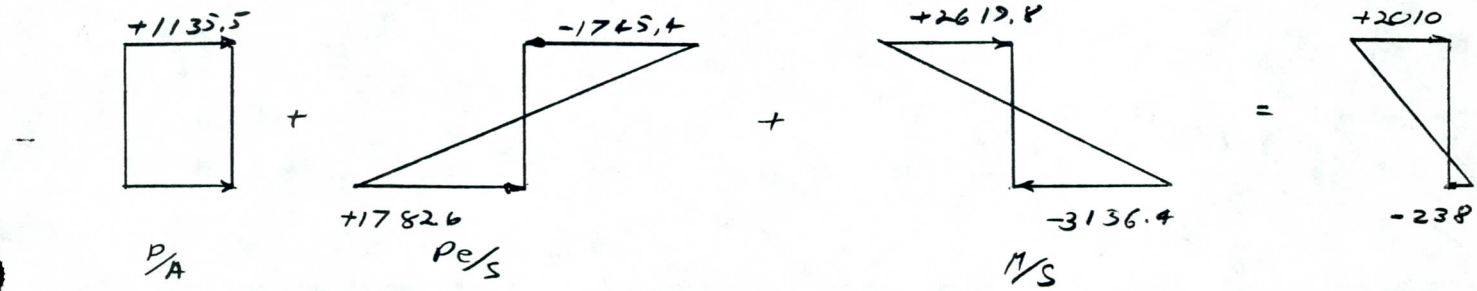
$e = 29.265$

$Pe/22000 = 1650$

TRY 40/16 GIRDER

$P = 56(22000) = 1{,}232{,}000$ lb

$e = 29.166$ in



$\therefore$ OK

CHECK Defl.

CHECK LIVE LOAD DEFLECTION

AASHTO 1.7.6 MAX $\Delta = \ell/800$ DUE TO LL+I

$$\Delta_{\mathcal{L}} = 5w\ell^4/384EI$$

$$E = 57000\sqrt{f'_c} = 57000\sqrt{5000}$$
$$= 4{,}030{,}510 \text{ psi}$$

$$I^c = 1{,}312{,}275 \text{ IN}^4$$

$$\ell = 126.25^{FT} = 1515 \text{ IN}$$

$$w = 640 \text{ plf}(1.22)\,7.33/11 = 520.5 \text{ plf}$$
$$= 43.4 \text{ pli}$$

$$\Delta_{\mathcal{L}} = 5(43.4)1515^4 / 384(4030510)1312275$$

$$= 0.56^{IN} \quad \text{DUE TO UNIFORM } w$$

$$\Delta_{\mathcal{L}} = P\ell^3/48EI$$

$$P = 26000 \text{ LB}$$

$$\Delta_{\mathcal{L}} = 26000(1515^3)/48(4030510)1312275 = 0.36^{IN}$$

$$\Sigma\Delta_{\mathcal{L}} = 0.92^{IN} \leq \ell/800 = 1.89^{IN}$$

$\therefore$ OK

USE 14 TYPE VI AASHTO GIRDERS

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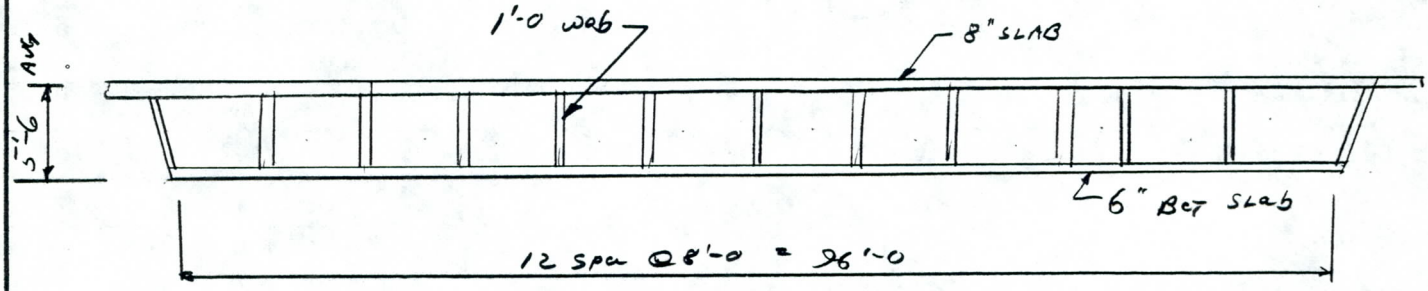
TRY a SINGLE SPAN BOX GIRDER BRIDGE



	A	$\bar{y}$	$A\bar{y}$	I_o	d^2	Ad^2
Top SLAB 102.33(12) 8	9824	62.00	609,088	52 394.7	26.3	6 795 1623
Webs 12(52) 13	8112	32.00	259,584	1,527,904	3.7	111,5323
BOT SLAB 6(96(12))	6984	3.00	20 952	20 952	32.7	7,467,921.6
	24,920 in²		889,624 in³	1,901,250.7		14,374,137

$$\bar{y}_b = 889624 / 24920 = 35.70 \text{ in}$$

$$\bar{I} = 1,901,250.67 + 14,374,137.2 = 16,275,390 \text{ in}^4$$

$$S_b = \bar{I} / 35.70 = 455,893 \text{ in}^3$$

$$S_T = \bar{I} / 30.30 = 537,142 \text{ in}^3$$

$$r = \sqrt{\bar{I}/A} = 25.56 \text{ in}$$

DESIGN LOADS

DEAD LOAD

BOX GIRDER	24920/144 (0.150)	=	25.958 kcf
RAILING	2(0.390 + 0.250)	=	1.280
MEDIAN	6/12 (6) 0.150	=	1.200
DIAPHRAGMS	3[8/12 (85) 4.33] 0.150/126.25	=	0.875
FILLETS	12[1/2 (4)(4)]/144 (4) 0.150	=	0.400
FWS	(56-16)(0.025)	=	1.750
POST-TENSIONING		=	0.370
			31.833 kcf

SAY 32 kcf

$$M_{DL} = 32(126.25^2)/8 = 63,756 \text{ FT-K}$$

LIVE LOAD

$$\text{IMPACT} = 50/125 + 126.25 = 0.20$$

6 LANE

$$M^{LL} = 2000^{\text{FT-K}} (6) 1.20 = 14,400 \text{ FT-K}$$

$$M_{DL} + M_{LL} = 78,160 \text{ FT-K}$$

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CHECK CONCRETE STRESSES

$$P/A \pm M/S \mp Pe/S = f$$

TOP SLAB

$$P/24920 - Pe/537142 + 78160(12000)/537142 = 2000 \text{ psi}$$

BOT SLAB

$$P/24920 + Pe/455,893 - 78160(12000)/455893 = -212 \text{ psi}$$

$$P/24920 - Pe/537142 = 253.9 \quad] \; 537142/455893$$

$$P/24920 + Pe/455893 = 1845.3 \quad] +$$

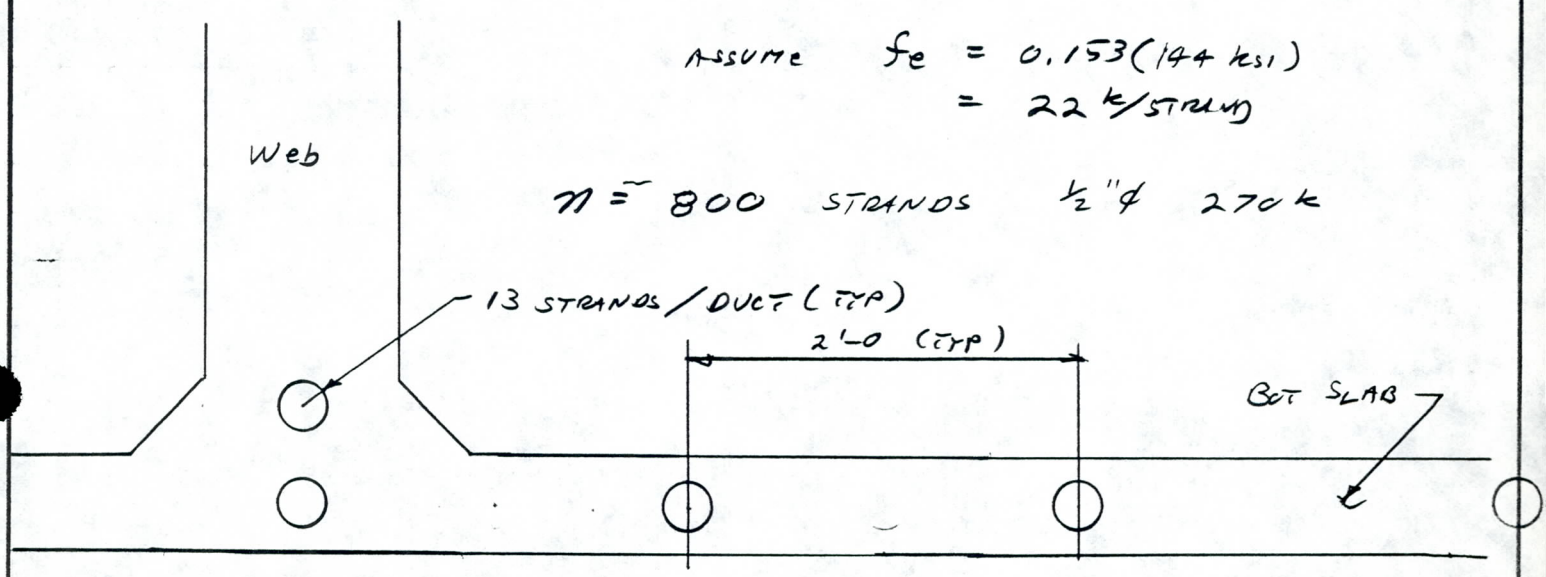
$$2.178 P/24920 = 2144.5$$

$$\therefore P = 24,536,703 \text{ lb}$$
$$e = 16.00 \text{ in}$$

IF $e = 29.70$ in
$P = 17,528,332$ lb

ASSUME $f_e = 0.153(144 \text{ ksi})$
$= 22$ k/STRAND

$n = 800$ STRANDS ½"φ 270k



SUBSTRUCTURE DESIGN

CONC PIER CAP ON DRILLED SHAFTS (36")

ASSUME: SIDE FRICTION ON SHAFT = 0.60 ksf

$\pi(3)0.60 = 5.65$ kef FRICTION

: END BEARING = 12 ksf

$\frac{\pi}{4} 3^2 (12) = 84.8^k$

PIER REACTION

DEAD LOAD

$\frac{1}{2}(32 \text{ kef})\, 126.25 = 2020^k$

LIVE LOAD $6 \text{ LANE}(67^k) = 402^k$

2422^k/PIER

PIER CAP $4'(4')\,102'(0.150) = 245^k$

TOTAL 2670^k/PIER

SHAFT $\frac{\pi}{4}(3^2)0.150 = 1.1$ kef

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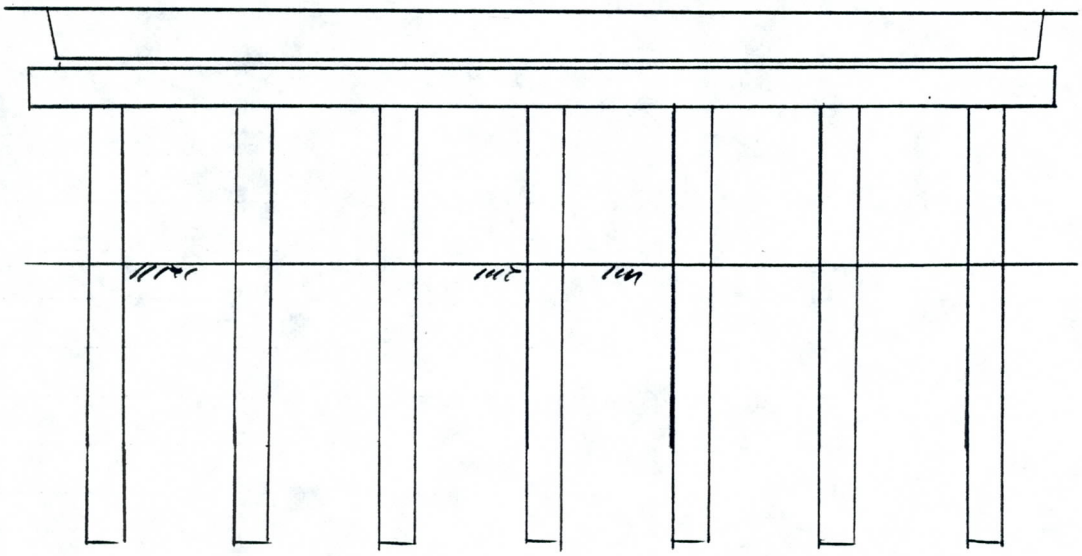
17.50' UNSUPPORTED SHAFT

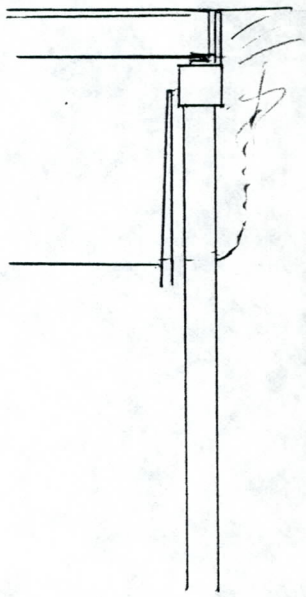
$$2670^k + n(17.50)1.10 = n(84.8 + 5.65\ell)$$

$$65.55n + 5.65\ell n = 2670$$

TRY 12 shafts @ 8'-6 = 93'-6

$\ell \cong 28$ FT below CHANNEL INVERT
($L \cong 45'$)





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ESTIMATED QUANITIES & COST

DRILLED SHAFTS 36"

12' SHAFTS (45 FT) 2 PIERS = 1080 L.F.
$70 / $75,600

PIER CAP

[4'(4') 102'] 2 PIERS / 27 = 121 CY.
$160/CY / $19,360

MAIN SPAN

BOX [24 920/144] 130 FT / 27 = 833.23 CY
DIAP. π(4.33)12 (2' + 2' + 3(1')) / 27 = 94.4

Σ 930 CY.
$200/CY
/ $186,000

STRUCTURAL EXCAVATION

115' WIDE (10' DEEP) 130' LONG / 27 = 5540 CY

$5 / $27,700

REINFORCING STEEL

5 # / CF

(121 + 930) 27 (5) = 142,000 LB

35¢ / LB

$49,700

POST TENSIONING

800 STRAND (0.52 DIA) 130' = 54,080 LB

28¢ / $15,140

Parapet & RAILING

260 LF EA

× $45 / LF

$11,700

MEDIAN

5/12 (16) / 27 (130) = 51.50 CY

$150

$7725

Σ ≅ $395,000 => $44.70 / SF ROWAY

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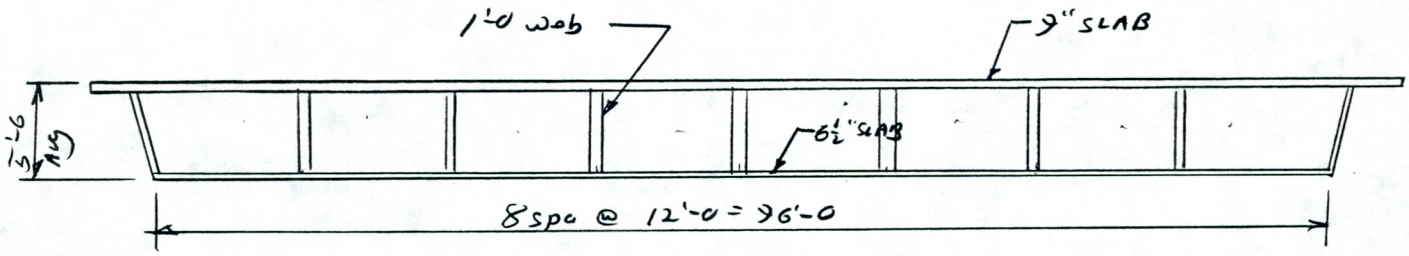
MODIFY SECTION TO REDUCE COST



	A	$\bar{y}$	$A\bar{y}$	I_o	d	Ad^2
Top SLAB	11052	61.50	679,698	74,601	25.05	6934604
Webs	5454	31.75	173,164.5	1,159,085.6	4.70	120,530
Bot SLAB	7566	3.25	24 589.5	26,638.6	33.20	8 340 050
	24072 IN^2		877,452	1,260,328		15 395 185

$= 167.17\ FT^2$

$\bar{y}_b = 877452 / 24072 = 36.451^{IN}$

$\bar{I} = 16,655,513\ IN^4$

$S_b = \bar{I} / 36.451 = 456,929\ IN^3$

$S_t = \bar{I} / 29.549 = 563,657\ IN^3$

DEAD LOAD

BOX GIRDER	167.17 SF (0.150)	=	25.075 kcf
FILLETS	½(4)(4)/144 (8)4 (0.150)	=	0.267
DIAPHRAGMS	8/12 (11) 4.21 (3) 0.150 (8)/126.25	=	0.880
RAILING	(0.390+0.250) 2	=	1.280
MEDIAN	8/12 (16) 0.150	=	1.200
FDS	70(0.025)	=	1.750
POST-TENSIONING		=	0.400
			30.85 kcf

$$M^{DL} = 30.85\,(126.25^2)/8 = 61{,}470^{FT-k}$$

$$M^{LL} = \underline{14{,}400^{FT-k}}$$

$$\Sigma\ 75{,}870^{FT-k}$$

CONCRETE STRESSES

TOP $\quad P/24072 - Pe/563657 + 75870\,(12000)/563657 = 2000$

BOT $\quad P/24072 + Pe/456929 - 75870\,(12000)/456929 = -212 \quad \left]\ \frac{456929}{563657}\right.$

$\Sigma \quad 1.811\,P/24072 = 1828 \quad \Rightarrow \quad P = 24{,}304{,}540^{LB}$

$e = 14.49''$

IF $e = 32.50''$

$P = 15.803^{k}/22 = 718 - \frac{1}{2}''$ STRANDS

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END ABUTMENTS SIMILAR TO PREVIOUS

EST COST.

THE CHANGE IN THE MAIN BOX GIRDER SECTION WILL NET AN APPROXIMATE COST DECREASE OF ONLY $8200 (.94¢/SF)

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TRY 24" SHAFTS

$P_{side} = \pi(2)0.60 = 3.77$ kef

$P_{END} = \frac{\pi}{4}(2^2)12 = 37.67$ k

$w = \frac{\pi}{4} 2^2 (0.150) \cong 0.50$ kef

$\therefore \quad 2670^k + n(17.50)0.50 = n(37.67 + (3.77 - 0.50)\ell)$

$2670 = 28.92\, n + 3.27\, \ell\, n$

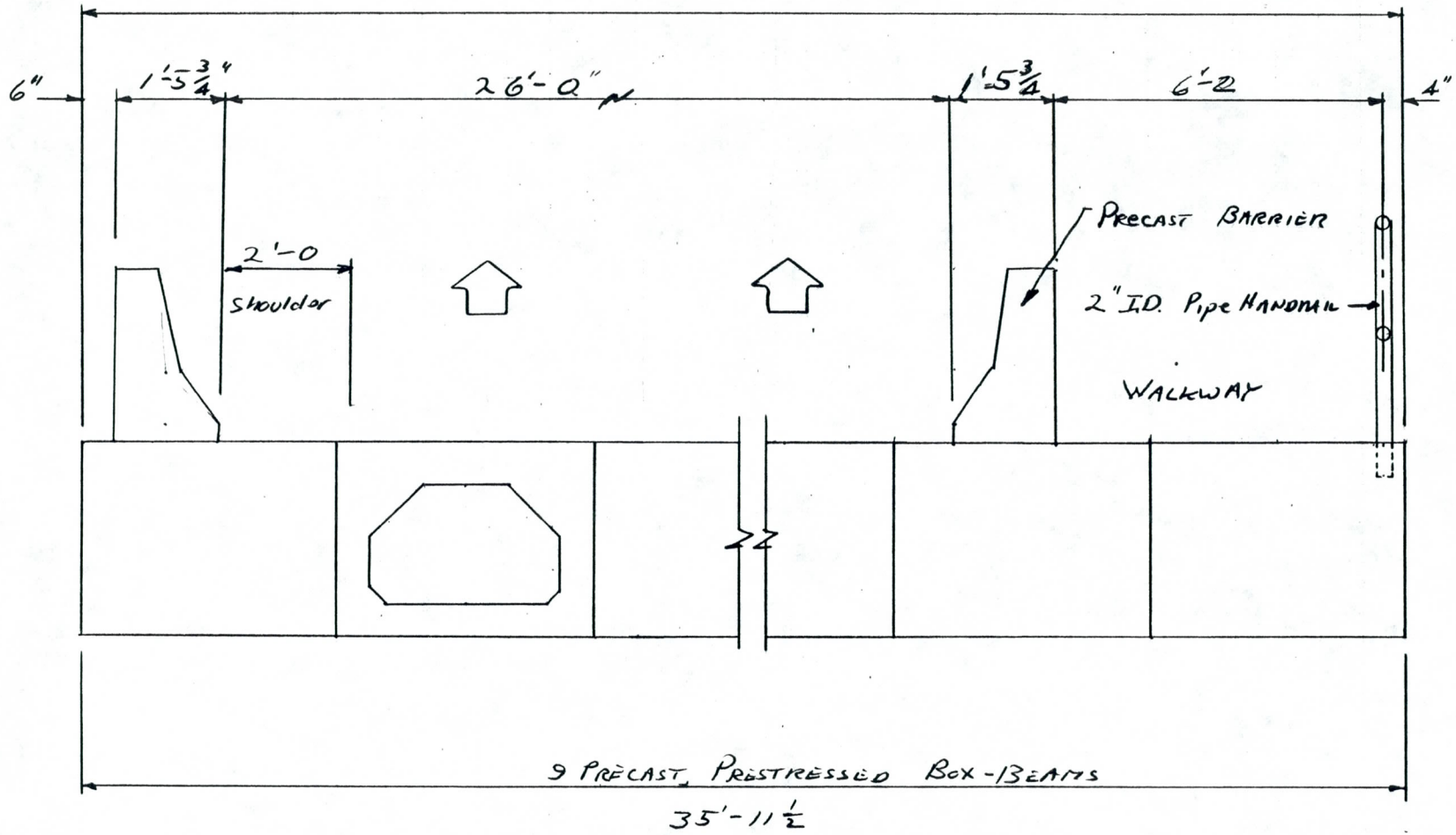
TRY 20 SHAFTS @ 5'-0 = 95'-0

$\ell \cong 32'$ below GL
(L = 50') below cap

QUANITY

20 (50) 2 piers = 2000 LF
@ \$21
\$42,000

Less than 36" shafts by ≈ \$34,000



DECK SECTION

LOOKING STA. Ahead

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Job 27-23045-1603 Page

Description Detour Bridge

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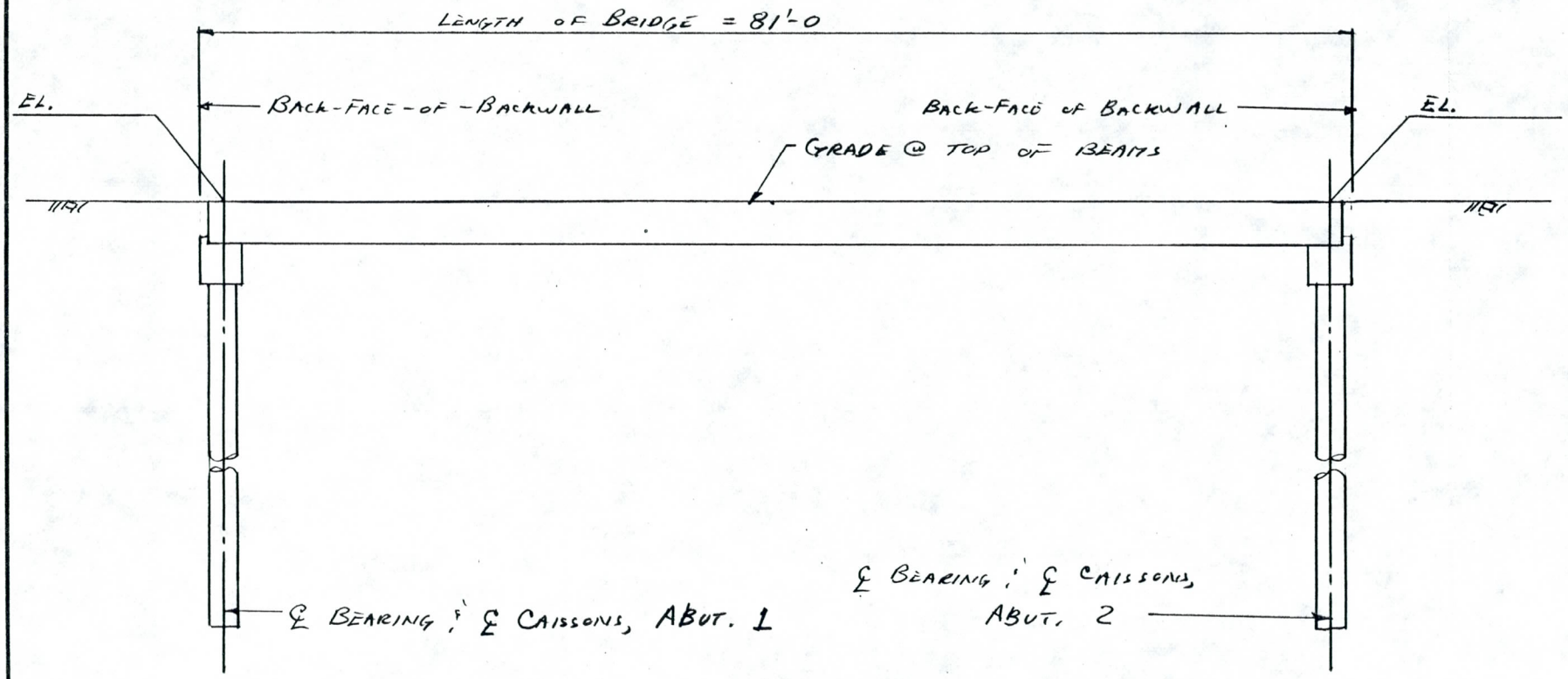

SECTION ALONG CONSTRUCTION ℄

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Job 31-23843-7605 RLC

Description DETOUR BRIDGE

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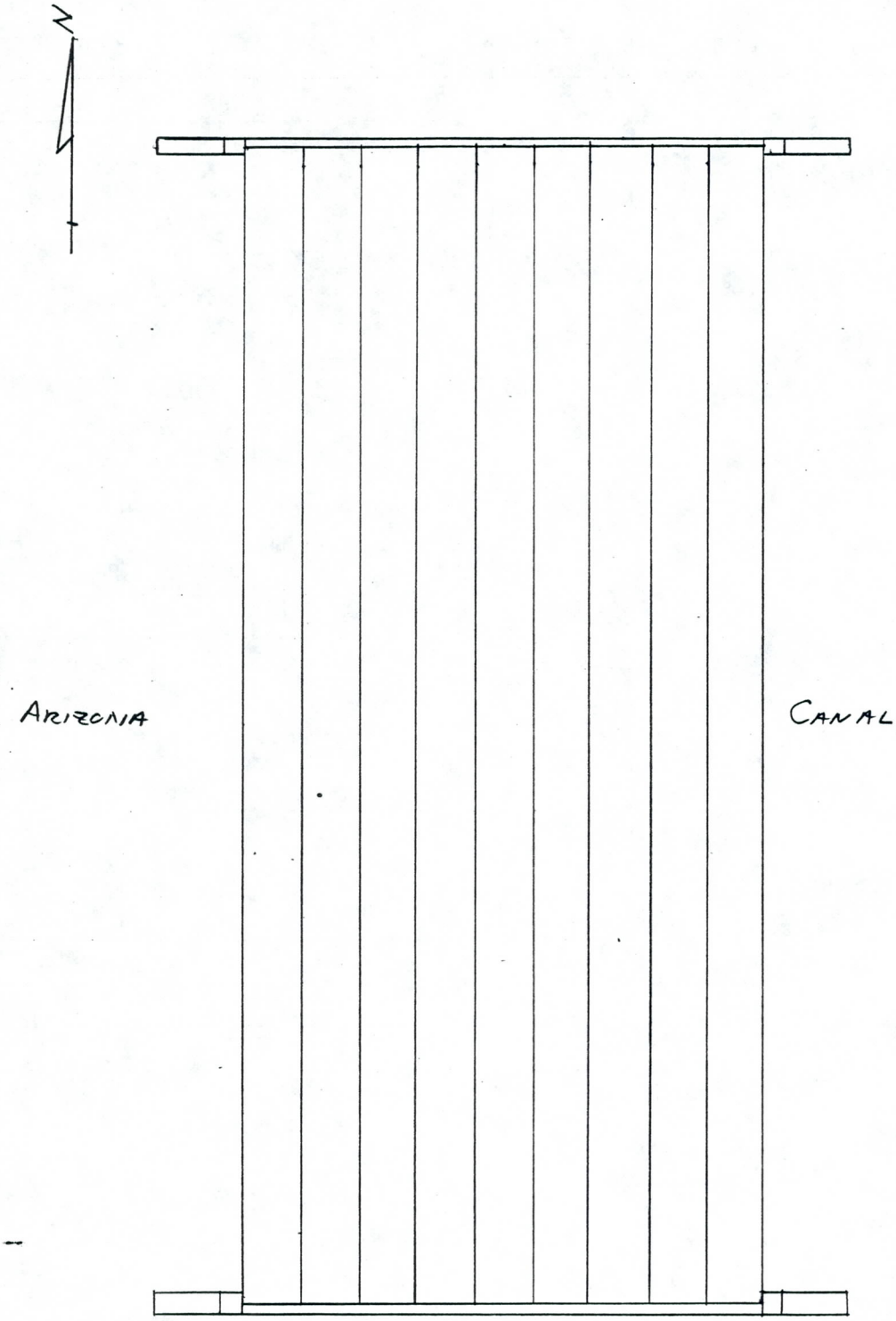

LOCATION PLAN

TEMPORARY SINGLE SPAN PRESTRESSED CONCRETE BOX-BEAM DETOUR BRIDGE

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REF. AC DC Bridge
35TH AVE

ED RALEIGH FCDMC
262-1501

MON 2 JULY 1984
2 PM

Temporary Detour Bridge

1. ARE THE 80FT (78FT SPAN) BOX BEAMS AS SHOWN ON THE PLANS FOR THE TEMP. SERVICE BRIDGE OVER THE ARIZ. CANAL EAST OF 51st AVE (JUNE 83) THE BEAMS INTENDED BY THE CITY FOR USE ON THE 35TH AVE DETOUR BRIDGE?

yes

2. ARE THEY, OR WILL THEY, BE AVALIBLE DURING THE CONSTRUCTION PERIOD? APPROXIMATLY 9 WILL BE REQUIRED.

yes, have on hand or will cast as required

3. WHAT IS THE DESIGN LOADING FOR THE BEAMS?

A 65.4k ↓ ← 26'-2" → ↓ 55.7k ↓ ← 7" → ↓ AXLE LOADING

AXLE wheels

IS SHOWN ON THE PLANS (NOT AN AASHTO TRUCK LOAD).

— SCRAPPER LOAD, ONE ON BRIDGE AT A TIME

4. WERE THE BOX BEAMS DESIGN TO INCLUDE A JERSEY TYPE BARRIER (2) ON THEM?

5. A 2" ϕ hole IS SHOWN AT EACH END OF THE BEAMS. ARE THESE for HOLD DOWN BOLTS? (NOT SHOWN ON ABUTMENTS.)

yes, hold downs. Ed could not remember why no hold down bolts were shown on the dwgs. posibly be cause it is a temp. Bridge. holes were in case they were used on a permanent structure.

6. ARE THE BEAMS ASSUMED TO ACT AS A UNIT DUE TO THE LATERAL TIE RODS? (The shear key DOES NOT SEEM TO BE ABLE TO TRANSMIT THE SHEAR LOADS AND WITH THE ½" GAP, THE TIERODS ARE INSIGNIFICANT)

NO, designed to carry a wheel Load on EACH BEAM. Ed WILL SEND ME A COPY OF THE CALCS for MY info.

7. Normal PRACTICE IS TO USE Drilled-in-place CASSIONS for ABUTMENT

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BRIDGE (DETOUR) CONSTRUCTION SEQUENCE

ABUTMENT CONSTRUCTION

1) CONSTRUCT DRILLED SHAFTS
   a) AUGER HOLE TO DEPTH
   b) FORM AS REQUIRED
   c) PLACE REINFORCING
   d) PLACE CONCRETE

2) CONSTRUCT PIER CAP EXCLUSIVE OF WING WALLS AND BACKWALL
   a) SET FORMS
   b) SET REINFORCING
   c) PLACE CONCRETE
   d) REMOVE FORMS AFTER CONCRETE SET

3) REPEAT STEPS 1 & 2 FOR OPPOSITE ABUTMENT

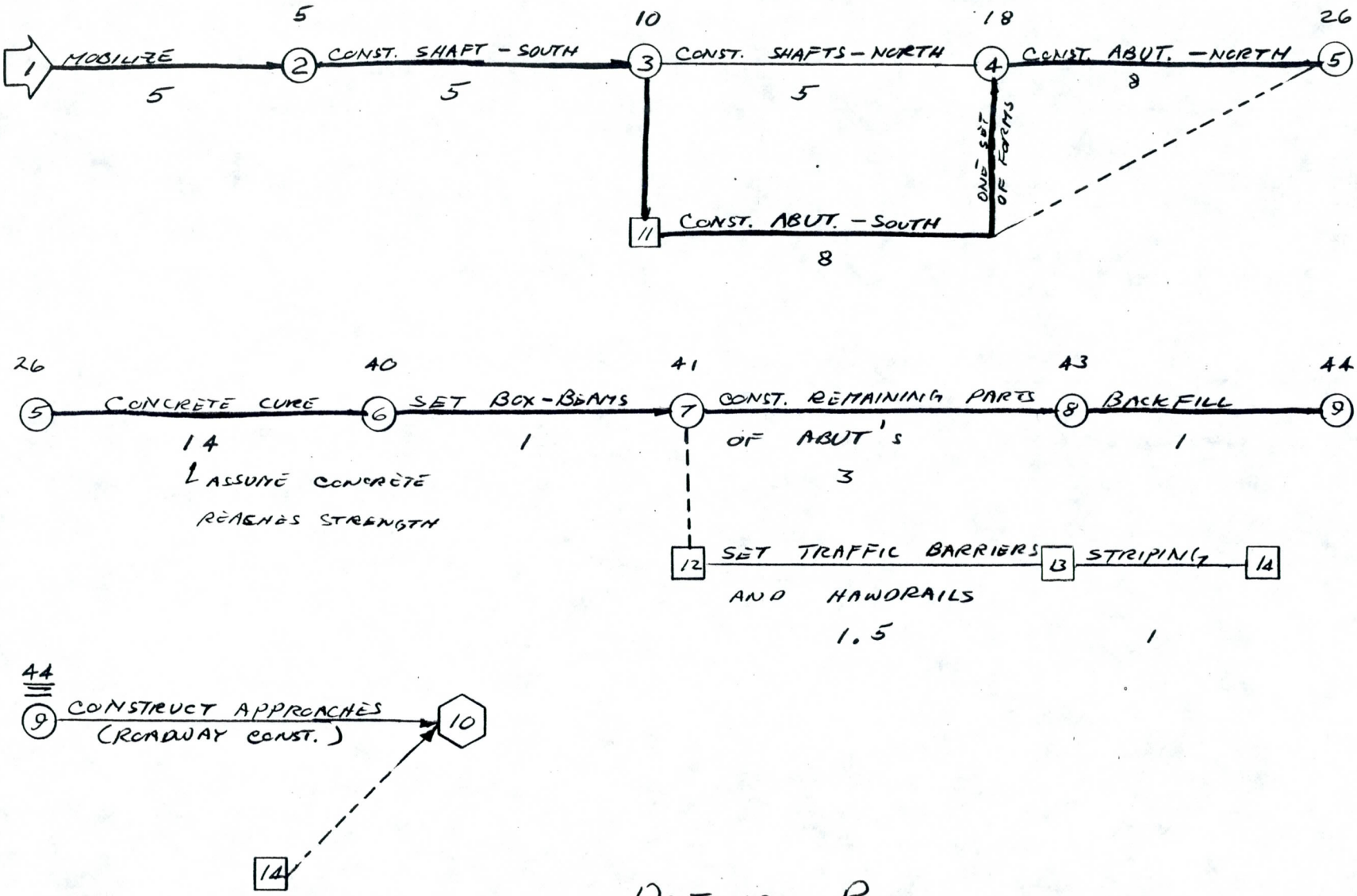
4) SET PRECAST 80' BOX-BEAMS (SUPPLIED BY FCDMC) REQUIRES ACCESS OF BEAM TRANSPORT AND CRANE TO CANAL ROAD (ASSUME CRANE BOOM WILL NOT INTERFER WITH OVERHEAD POWER LINE; OTHERWISE BEAMS MUST BE SET FROM SOUTH END; ? ACCESS)

5) FORM AND POUR ABUTMENT WINGWALLS AND BACKWALLS

6) PLACE TRAFFIC BARRIERS AND HANDRAILS

7) PLACE BACKFILL AND APPROACH ROADWAY

(EST CONST TIME = 44 DAYS)



DETOUR ROAD

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Description DETOUR

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TEMP BRIDGE EST. CONSTRUCTION TIME

DAYS		
5	MOBILIZE - EQUIP. TO SITE - MAT'L. TO SITE (NOT INCLUDING ORDERING) - STORAGE	
5	DRILL SHAFTS - DRILL, FORM, SET STEEL, PLACE CONCRETE	
8	CONSTRUCT ABUTMENTS - FORM	2
	- SET REINF	1
	- PLACE CONC.	1/2
	- CONC. SET	4
	- REMOVE FORMS	1/2
14	CONCRETE CURE	
1	SET BOX - BEAM (9)	
3	CONSTRUCT REMAINS OF ABUTMENTS - FORM	1/2
	- SET REINF	1/4
	- PLACE CONC.	1/4
	- CONC. SET	2 3/4
	- REMOVE FORMS	1/4
1	BACK-FILL	
1.5	SET TRAFFIC BARRIERS AND HANDRAILS	
1	STRIPPING	

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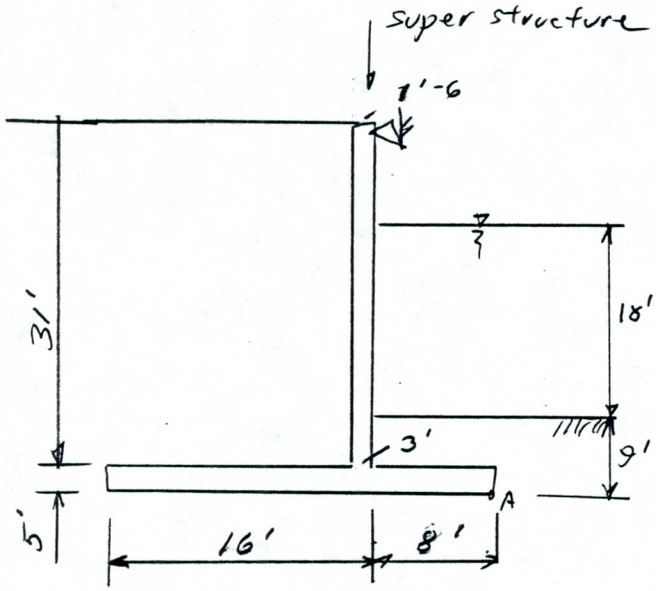
Abutment Wall

Preliminary



$\gamma = 132$ kcf $\qquad \phi = 36.5°$

$K_o = 1 - \sin\phi = 0.41$

2' surcharge

superstructure $\cong$ 10 klf (D+L)

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Σ Moment about A

Vertical - Resisting

			V (kips)	ARM (FT)	M_A (FT-k)
SOIL	31(13) 0.132	=	53.2	17.50	931.0
surcharge	2(13) 0.132	=	3.4	17.50	59.5
WATER/SOIL	18(8) 0.063	=	9.1	4	36.4
SOIL/WATER	4(0.132)8	=	4.2	4	16.8
STEM	½(1.5+3) 31 (0.150)	=	10.5	9.5	99.8
BASE	5(24) 0.150	=	18.0	12	216.0
			98.4 k		
superstructure			10	9.5	95.0
			108.4 k		1454.5 ft-k

Overturning - propel cantilever

surcharge	2(0.132)5(31)/8	=	5.11	2(0.132)31²/8	=	31.7
soil	2(31(0.132)31)/5	=	50.7	31(0.132)31²/15	=	262.2
Assume no water (worst case)						293.9 ft-k

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{1454.5 - 293.9}{108.4} = 10.71'$$

$$e = \frac{B}{2} - \bar{x} = \frac{24}{2} - 10.71 = -1.29 \text{ FT} < \frac{24}{6} = 4.0 \quad \therefore \text{ok}$$

$$q = \frac{\Sigma V}{L}\left(1 \pm \frac{6e}{L}\right) = \frac{108.4}{24}\left(1 \pm \frac{6(1.29)}{24}\right) = 4.52(1 \pm 0.32)$$

q_{max} (toe) = 6 ksf

q_{min} (heel) = 3.1 ksf

per soils report

toe settlement $\approx 0.5''$
heel settlement $\approx 0.25''$

diff = 0.25" @ toe

To top

$$R = \sqrt{16^2 + 36^2} = 39.4'$$

$$\alpha = \tan^{-1}\left[\frac{0.25}{24(12)}\right] = 0.0497^\circ$$

$$\sin\alpha \cong \tan\alpha = 0.00087$$

$$\Delta_{top} = 39.4\,(0.00087)\,12\,\text{in} = 0.41\text{ in}$$

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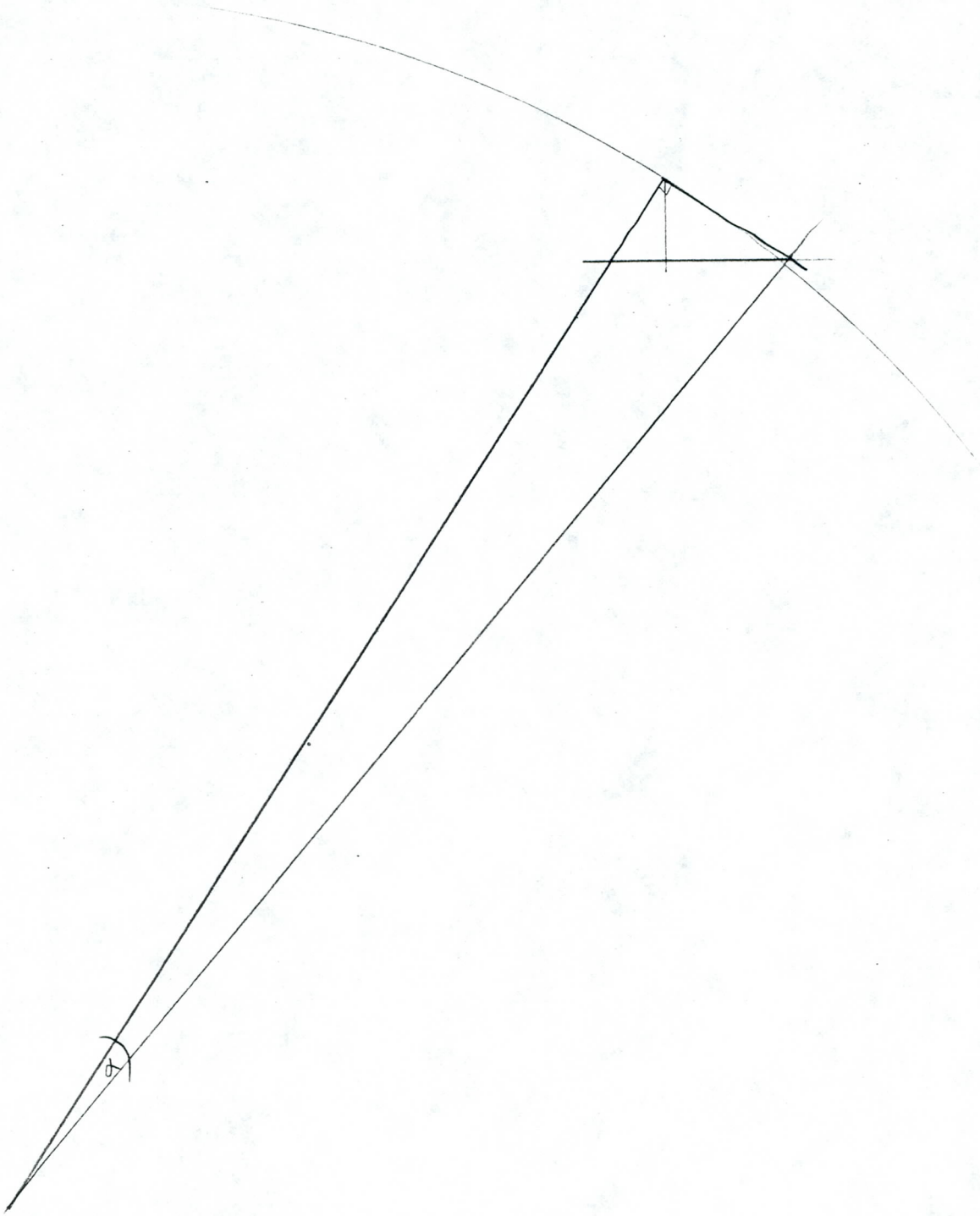

Overturning - Free cantilever (consv.)

				MRM	
surcharge	$2(0.132)36$	=	9.5^k	18	= 171.1
soil	$\frac{1}{2}(36(0.132))36$	=	85.5^k	12	= 1026.0
			96^k		$1197.1'^k$

$$\bar{x} = 1254.5 - 1197.1 / 108.4 = 2.37'$$

$$e = 24/2 - 2.37 = 9.63'$$

$$q = \frac{108.4}{24}\left(1 \pm \frac{6(9.63)}{24}\right) = 4.52(1 \pm 2.41) = 15.4 \text{ ksf}$$
$$= -6.4 \text{ ksf}$$

Section 2

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Job 5123042 35TH AVE ACDC

Description INDEX

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CALCS CHECKED:

Registered Professional Engineer (Structural) CERTIFICATE NO. 15328 GEORGE G. HAGEDON Date Signed 2/5/85 ARIZONA, U.S.A.

FINAL DESIGN OF
NEW BRIDGE OVER the
ARIZONA CANAL DIVERSION CHANNEL
ON 35th AVENUE

BOOK 2 - SUPERSTRUCTURE

SHT(S)	DESCRIPTION
1	DESIGN DATA
4	LOADS & MOMENTS
12	SUPERSTRUCTURE SECTION PROPERTIES
15	Prestress Girder DESIGN
22	stress DIAGRAM
23	FLEXURAL strength
26	Deflection / Camber
31	Continuity
51	Shear
80	Deck SLAB Design
100	Post-TENSIONED Girder (ALTERNATE)

35th AVE BRIDGE OVER the ARIZONA CANAL DIVERSION CHANNEL

2-SPAN TYPE III AASHTO Precast, Prestressed Girder Bridge

Solid Center Pier
RETAINING WALL TYPE Abutment
} ON SPREAD FOOTERS

Deck SLAB/Girders shall act as a strut to resist Earth PRESSURE LOADS

Concrete

Prestressed Girders	5000 psi
Deck Slab	4000 psi
Substructure	3000 psi

Reinforcing

#6 and smaller	fy = 40,000 psi
#7 AND LARGER	60,000 psi

DESIGN SPECIFICATIONS:

AMERICAN ASSOCIATION OF STATE TRANSPORTATION OFFICIALS (AASHTO) "STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES", THIRTEENTH EDITION, 1983

SEISMIC RISK ZONE: II A = 0.05

LIVE LOAD:	HS20 - 44	
DEAD LOAD:	GIRDER	= 583 PLF
	SLAB	= 94 PSF
	FUTURE WEARING SURFACE	= 25 PSF
UTILITY LOAD:	WATERLINE	= 120 PLF
	CATV	= 32 PLF

CONSTRUCTION SPECIFICATIONS:

MARICOPA ASSOCIATION OF GOVERNMENTS (M.A.G.), "UNIFORM STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION", 1979 EDITION AS SUPPLEMENTED BY THE "1984 CITY OF PHOENIX SUPPLEMENT" AND THE "SUPPLEMENTAL SPECIAL PROVISIONS".

CONCRETE:

CONCRETE SHALL CONFORM TO THE DESIGN SPECIFICATIONS AND TO THE APPLICABLE SECTIONS OF THE CONSTRUCTION SPECIFICATIONS, INCLUDING BUT NOT NECESSARILY LIMITED TO SECTIONS 505 AND 725.

USE	CLASS	F_c (PSI)	
PRECAST, PRESTRESSED TYPE III GIRDER	AA	5000	F_{ci} = 4000 PSI
CAST-IN-PLACE DECK	AA	4000	F_c = 1600 PSI
SUBSTRUCTURE	AA	4000	F_c = 1600 PSI
BARRIERS	AA	4000	F_c = 1600 PSI
APPROACH SLABS	A	3000	F_c = 1200 PSI

CONSTRUCTION JOINTS SHALL BE PERMITTED ONLY AT INDICATED LOCATIONS. ADDITIONAL CONSTRUCTION JOINTS OR CHANGES TO THOSE SHOWN SHALL BE APPROVED BY THE ENGINEER.

ALL EXPOSED CONCRETE EDGES SHALL HAVE A 3/4 X 3/4 INCH CHAMFER UNLESS NOTED OTHERWISE.

ALL REINFORCING STEEL SHALL HAVE A MINIMUM 2 INCH CONCRETE COVER UNLESS INDICATED OTHERWISE.

CONCRETE SURFACES SHALL BE FINISHED IN ACCORDANCE WITH THE CONSTRUCTION SPECIFICATIONS.

REINFORCING STEEL:

REINFORCING STEEL SHALL CONFORM TO THE APPLICABLE SECTIONS OF THE CONSTRUCTION SPECIFICATIONS INCLUDING BUT NOT NECESSARILY LIMITED TO SECTION 727. THE REINFORCING STEEL SHALL HAVE THE FOLLOWING CHARACTERISTICS:

BAR SIZE	GRADE	MIN. YIELD STRENGTH	MAX. WORKING STRESS
#6 AND SMALLER	40 OR 60	40,000 PSI	20,000 PSI
#7 AND LARGER	60	60,000 PSI	24,000 PSI

ALL REINFORCING SPACINGS ARE CENTER TO CENTER OF BARS UNLESS INDICATED OTHERWISE.

ALL BAR BENDING DIMENSIONS ARE OUT TO OUT OF BARS.

REINFORCING STEEL SHALL HAVE BENDS, DEVELOPMENT LENGTHS, AND SPLICE LENGTHS IN ACCORDANCE WITH THE DESIGN SPECIFICATIONS.

PRESTRESSING WORK:

PRESTRESSED WORK SHALL CONFORM TO THE DESIGN SPECIFICATIONS AND TO THE APPLICABLE SECTIONS OF THE CONSTRUCTION SPECIFICATIONS INCLUDING BUT NOT NECESSARILY LIMITED TO SECTION 506.

CONTRACTOR SHALL BE RESPONSIBLE FOR PICK-UP POINT INSERTS AND LOCATIONS, SPECIAL PICK-UP REINFORCING AND STRONG BACKS, AND ALL PICK-UP AND PLACING OPERATIONS.

WELDING:

ALL WELDING SHALL CONFORM TO THE LATEST EDITION OF THE AMERICAN WELDING SOCIETY STRUCTURAL WELDING CODE D1.1 AS MODIFIED BY THE AASHTO STANDARD SPECIFICATIONS FOR WELDING OF STRUCTURAL STEEL HIGHWAY BRIDGES.

ALL WELDING SHALL BE BY WELDERS HOLDING CURRENT VALID CERTIFICATES AND HAVING CURRENT EXPERIENCE IN THE WELD CALLED FOR.

WELDING RODS ARE TO BE LOW HYDROGEN TYPE.

REINFORCING STEEL SHALL NOT BE WELDED.

SUPPLEMENTARY NOTES:

PROVIDE ALL TEMPORARY BRACING, SHORING, GUYING OR OTHER MEANS TO AVOID EXCESSIVE STRESSES AND TO HOLD STRUCTURAL ELEMENTS IN PLACE DURING CONSTRUCTION. ESTABLISH AND VERIFY ALL, OPENINGS AND INSERTS PRIOR TO CONSTRUCTION.

COST OF ADDITIONAL FIELD AND OFFICE WORK NECESSITATED BY REQUEST BY CONTRACTOR FOR AN OPTION OR DUE TO ERRORS OR OMISSIONS IN CONSTRUCTION SHALL BE BORNE BY THE CONTRACTOR. OPTIONS ARE FOR CONTRACTOR'S CONVENIENCE. HE SHALL BE RESPONSIBLE FOR ALL CHANGES NECESSARY IF HE CHOOSES AN OPTION AND HE SHALL COORDINATE ALL DETAILS.

ANY ENGINEERING DESIGN PROVIDED BY OTHERS AND SUBMITTED FOR REVIEW SHALL BEAR THE SEAL OF AN ENGINEER REGISTERED IN ARIZONA.

CONTRACTOR SHALL VERIFY IN FIELD ALL EXISTING CONDITIONS SHOWN ON DRAWINGS.

COST FOR INCIDENTAL ITEMS, NOT SPECIFICALLY COVERED OTHERWISE, SHALL BE INCLUDED IN THE UNIT PRICE FOR THE SUBJECT BID ITEM.

CONSTRUCTION PHASING:

CONSTRUCTION OF THE NEW BRIDGE SHALL BE PHASED IN SUCH A MANNER AS TO MAINTAIN TWO-WAY PUBLIC TRAFFIC AT ALL TIME WITHOUT CAUSING UNDUE OR UNREASONABLE DELAYS OR INCONVENIENCE. THE CONTRACTOR SHALL SUBMIT TO THE ENGINEER FOR APPROVAL AT LEAST 10 DAYS PRIOR TO BEGINNING OF CONSTRUCTION HIS SEQUENCE OF CONSTRUCTION AND MAINTENANCE OF TRAFFIC PLAN.

FOUNDATIONS:

GEOTECHNICAL INVESTIGATION WAS PERFORMED BY SPEEDIE AND ASSOCIATES, PHOENIX, ARIZONA.

ALL FOUNDATIONS ARE SPREAD FOOTINGS BEARING ON EXISTING SANDY GRAVEL.

LATERAL EARTH PRESSURES ARE BASED ON AN EQUIVALENT FLUID PRESSURE OF 50 PCF FOR THE ACTIVE CASE AND 400 PCF FOR THE PASSIVE CASE.

EXCAVATIONS FOR FOUNDATIONS SHALL BE INSPECTED BY A GEOTECHNICAL ENGINEER TO VERIFY ASSUMED ALLOWABLE SOIL BEARING AND LOW SETTLEMENT AND SWELL POTENTIAL, AND TO MAKE ANY ADDITIONAL RECOMMENDATIONS.

STRUCTURAL EXCAVATION AND STRUCTURAL BACKFILL:

STRUCTURAL EXCAVATION AND BACKFILL SHALL CONFORM TO THE APPLICABLE SECTIONS OF THE CONSTRUCTION SPECIFICATIONS INCLUDING BUT NOT NECESSARILY LIMITED TO SECTION 206.

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Job 5123042 35th Ave ACDC

Description New Bridge

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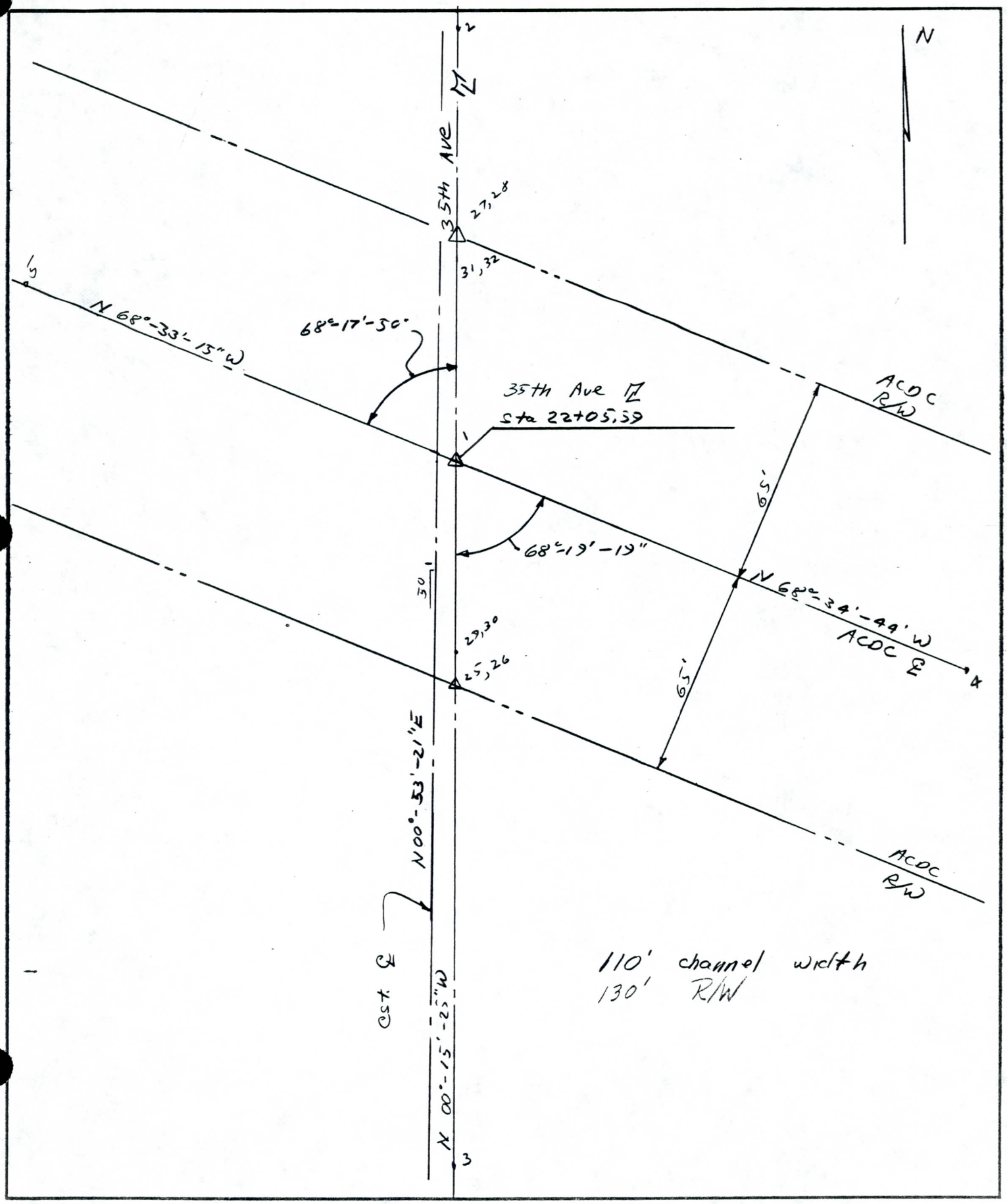
Checked by G.G.H Date 4 DEC 84 of



ACDC ℄ AVG BEARING

N 68-34-44 W
N 68-33-15 W
137-07-59
/2
AVG. N 68° - 33' - 59.5" W ✓

ANGLE w/ cst. ℄

N 68 -33- 59.5 W
+ N 00 -53 - 21 E
69° -27' -20.5"
= 69.456° ✓

Channel width = 110' perpendicular to ACDC ℄

110/SIN 69.456° = 117.47' along cst. ℄

single span length ≅ 117.5/2 = 58.74'

say 59'-0" ℄ to ℄ Brg ✓

Assuming EQUAL distance from ℄ pier to ℄ Brg and edge of channel to ℄ Brg Abut.

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Job 312042 35th Ave A UC
Description New Bridge
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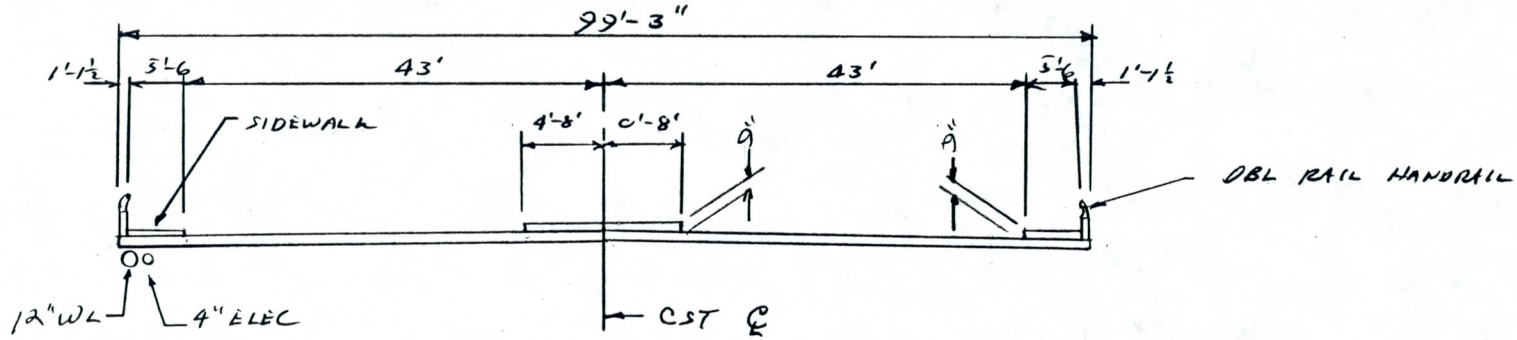
Checked by G.G.H. Date 17 Dec 94 of



UTILITIES

12" WATERLINE	x 120 plf	pipe + water	
4" ELECTRICAL	x 32 plf	duct + cable	

TRY 13 - TYPE III GIRDER @ 7'-10" oc = 94'-0
with 2'-7½" overhangs

min slab thickness $t_s = \frac{S+10}{30} \geq 0.542'$ AASHTO 8.9.2

$s = 7.83'$ $t_s = \frac{7.833+10}{30} = 0.594'(12) = 7.13^{IN}$

say $7\frac{1}{2}^{IN}$ slab

TYPE III GIRDER

DESIGN LOADS

DEAD LOAD
NON-COMPOSITE

TYPE III GIRDER	$560 \text{ in}^2/144\ (0.150)$	=	0.583 klf
$7\frac{1}{2}$" slab	$7\frac{1}{2}/12\ (7.83)\ 0.150$	=	0.734
2^{IN} haunch	$2/12\ (16/12)\ 0.150$	=	0.033
			1.350 klf/GIRDER

MIDSPAN DIAPHRAGM $8/12\ (\approx 2540 \text{ sq in}/144\ (0.150) = 1.76$ k/DIAP.

DEAD LOAD
-composite

SIDEWALK	9/12 (5.50) 2 (0.150)	=	1.238 klf
PARAPET	13½"/12 (2'-4") 0.150 (2)	=	0.675
RAILING	50 plf/1000 (2)	=	0.100
MEDIAN	9/12 (16) 0.150)	=	1.800
WATER LINE	120 plf/1000	=	0.120
ELEC. LINE	32 plf/1000	=	0.032
F.W.S	25 psf/1000 (86)	=	2.150
			6.115 klf / 13 Girders

AASHTO 3.23.2.3.1.1

= 0.47 klf / Girder ✓

LIVE LOAD

HS20-44

$I = 50/(L+125) = 50/(59+125) = 0.272$ ✓

LANE/Girder = 5/11 = 7833/11 = 0.712 LANES/Girder ✓ [3.23.2.2]

LANE 0.640 klf & 18 k / LANE ✓

TRUCK 32k ↓ 14' 32k ↓ 14' 8k ↓ / LANE ✓

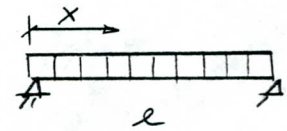
∴ DESIGN GIRDER AS A SIMPLE SPAN BEAM WITH A MAXIMUM ALLOWABLE TENSION OF $6\sqrt{f'_c}$

INVESTIGATE GIRDER AS A 2-SPAN CONTINUOUS BEAM WITH A MAXIMUM TENSION OF $3\sqrt{f'_c}$

Simple Beam Moments

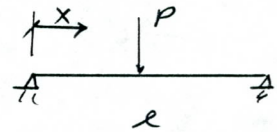
UNIFORM LOAD

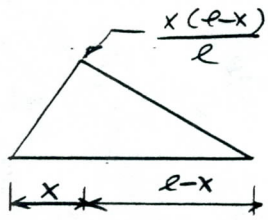
$$M_x = \frac{wx}{2}(\ell - x)$$



Point Load @ midspan

$$M_x = Px/2 \qquad x \le \ell/2$$





M influence

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Description

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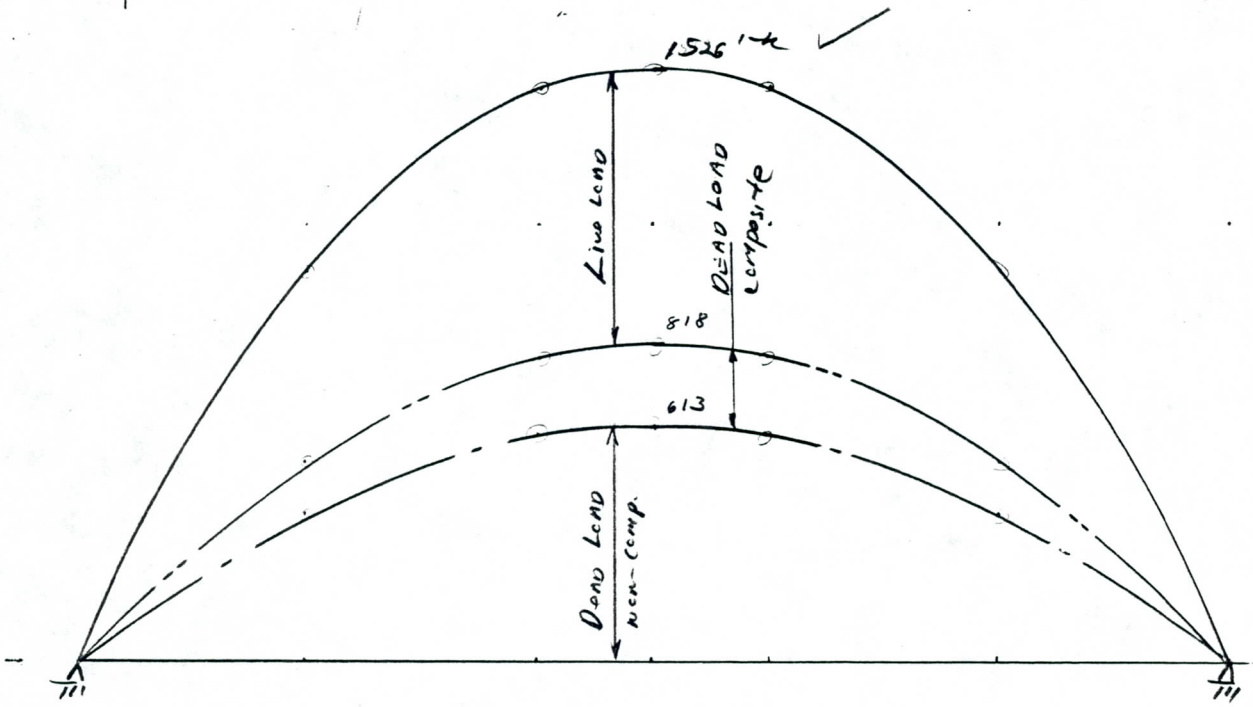
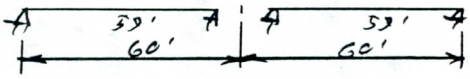
x/ℓ	X (FT)	M_{DL}^{NC} (FT-k)	M_{DL}^{C} (FT-k)	M_{LL}^{Truck} FT-k/LANE	M_{LL}^{LANE} FT-k/LANE	$M_{LL}(I)\frac{S}{11}$ (FT-k)	
0.0	0	0	0	0	0	0	support
0.2	11.8'	386	131	545.3*	348.1	494	
0.4	23.6	585	196	773.1*	522.2	700	
0.5	29.5	613.	205	782.0 *	544.0	708	symm. about midspan
0.6	35.4	585	196	773.1 *	522.2	700	
0.8	47.2	386	131	545.3 *	348.1	494	
1.0	59.0	0	0	0	0	0	support



Simple Beam Moments

2-Span Continuous Moments

REF: "Moments, Shears, and Reactions for Continuous Highway Bridges" AISC publ. No T106 TABLE A20



L = 60'

DEAD LOAD - Noncomposite

simple span L = 59'

see pgs.

DEAD LOAD - composite

$w_{DL}^{c} = 0.47$ klf

Live Load

HS 20 - 44

I = 0.272 ✓

DF = 0.712 ✓

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Description
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X/L	X (FT)	TOTAL AREA coef	M_{DL}^{c} $C*w*L^2$	+ AREA coeff.	P^+ coef.	M_{LL}^+ $C*w_{LL}*L^2 + P^T*P*L$	− AREA coeff	P^- coeff	M_{LL}^- $C*w_{LL}*L^2 + P^-*P*LL$
				LANE LOAD MOMENTS/LANE					
0	0	0	0	0	0	0	0	0	0
0.20	12	0.0550	93	0.0675	0.1504	318	-0.0125	-0.0192	-50
0.40	24	0.0700	118	0.0950	0.2064	442	-0.0250	-0.0384	-99
0.50	30'	0.0625	106	0.0938	0.2031	435	-0.0313	-0.0480	-124
0.60	36'	0.0450	76	0.0825	0.1824	387	-0.0375	-0.0576	-149
0.80	48'	-0.0200	-34	0.0300	0.1024	180	-0.0500	-0.0768	-198
1.0	60'	-0.1250	-212	0	0	0	-0.1250	-0.0960(2)	-495 *

X/L	X (FT)	$+P^{32}$	$+P^{32}$	$+P^{8}$	M_{LL}^+	$-P^{32}$	$-P^{32}$	$-P^{8}$	M_{LL}^-	M_{LL}^+	M_{LL}^-
		TRUCK LOAD MOMENTS/LANE								MAX/Girder	
0	0	0	0	0	0	0	0	0	0 =	0	
0.20	12	0.1504	0.0939	0.0484	496 *	-0.0192				449	
0.40	24	0.2064	0.1092	0.0839	646 *	-0.0384				585	
0.50	30	0.2031	0.1026	0.0916	631 *	-0.0480				572	
0.60	36'	0.1824	0.0994	0.0627	571 *	-0.0576	-0.0473	-0.0374	-219 *	517	-198
0.80	48	0.1024	0.0371	0.0079	272 *	-0.0768	-0.0630	-0.0498	-292 *	246	-264
1.0	60	0	0	0	0	-0.0960	-0.0788	-0.0623	-365	0	-448

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Description

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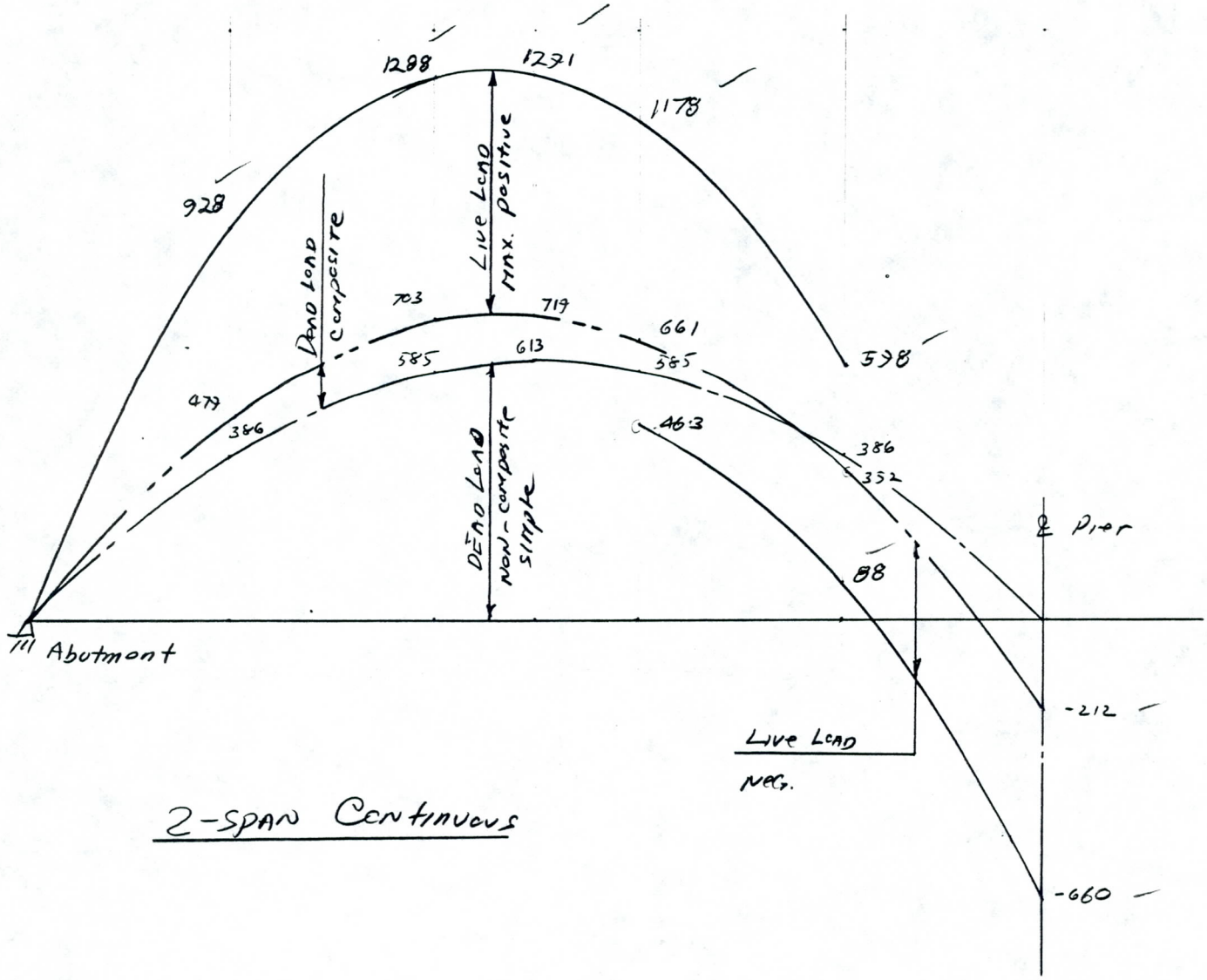


2-span Continuous

SECTION PROPERTIES

NON-COMPOSITE SECTION

PCI Design Handbook, pg. 2-53

TYPE III GIRDER

$A_x = 560 \text{ in}^2$

$\bar{I} = 125,390 \text{ in}^4$

$\bar{y}_b = 20.27 \text{ in}$

$\bar{y}_t = 45 - 20.27 = 24.73 \text{ in}$

$S_b = \bar{I}/\bar{y}_b = 125390/20.27 = 6186 \text{ in}^3$

$S_t = \bar{I}/\bar{y}_t = 125390/24.73 = 5070 \text{ in}^3$

Composite Section Properties

EFFECTIVE TOP FLANGE WIDTH [9.8.1, 8.10.1]

$$b = \text{Least of} \begin{cases} \ell/4 = 59/4\,(12) = 177'' \\ 12t_s = 12(7.5) = 90'' \leftarrow \text{controls} \\ S = 7.83\,(12) = 94'' \end{cases}$$

modify EFF. FLANGE width to account for concrete strengths

slab $f'_c = 4000$ psi $\quad E_c^s = 57\sqrt{4000} = 3605$ ksi

Girder $f'_c = 5000$ psi $\quad E_c^g = 57\sqrt{5000} = 4030$ ksi

$$\alpha = E_c^s / E_c^g = 3605/4030 = 0.895$$

$$b' = \alpha b = 0.895(90) = 80.55 \text{ in}$$

Assume $0''$ haunch [conservative] for section properties

	A	y_b	Ay_b	I_o	d	Ad^2
SLAB $7\frac{1}{2}$ (80.55)	604.1	48.75	29,451	2832	13.70	113,388
GIRDER	560.0	20.27	11,351	125,390	14.78	122,331
Σ	1164.1		40,802	128,222		235,719

$$\bar{I} = 128,222 + 235,719 = 363,941 \text{ in}^4$$

$$\bar{y}_b = \Sigma Ay_b / \Sigma A = 40802 / 1164.1 = 35.05 \text{ in}$$

$$\bar{y}_t = 45 - 35.05 = 9.95 \text{ in}$$

$$S_B = \bar{I} / \bar{y}_b = 10,383 \text{ in}^3$$

$$S_T = \bar{I} / \bar{y}_T = 36,577 \text{ in}^3$$

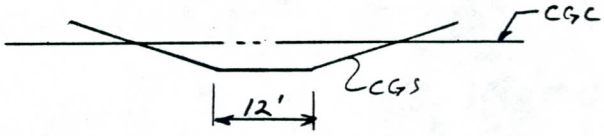
GIRDER

SLAB

$$\bar{y}_c = 9.95 + 7\tfrac{1}{2} = 17.45 \text{ in}$$

$$S_c = \bar{I} / \bar{y}_c = 20,856 \text{ in}^3$$

TRIAL STRAND PATTERN



@ MIDSPAN

TRY 10 @ $2\frac{1}{2}$" = 25
10 @ $4\frac{1}{2}$" = 45
2 @ $6\frac{1}{2}$" = 13
22 83

$M_{DL}^{NC} = 613'^{k}$ ✓
$M_{DL}^{C} + M_{LL} = 205 + 708 = 913'^{k}$ ✓

Type 16/6

$e_{ps} = 20.27 - 83/22 = 16.50$ in

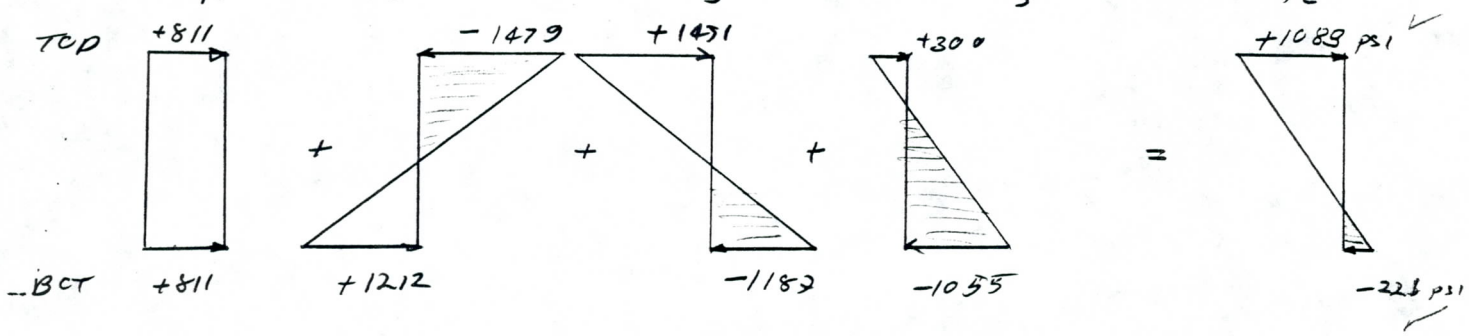
Assume $F_f = 0.5 f_{pu} = 135$ ksi
Losses = 55 ksi ✓

$\frac{1}{2}$"ϕ strand ✓

$P_f = 135\text{ ksi}(0.153 in^2/str)\,22 = 454.4^k$ ✓

CONCRETE STRESSES

$P/A \mp Pe/S \pm M_{DL}^{NC}/S^{NC} \pm M_{DL}^{C}+M_{LL}/S^{C} = f_c$



✓

-221 psi $\cong -3.01\sqrt{f'_c} < -6\sqrt{f'_c}$ ✓

∴ Ok

1083 psi $< 0.4 f'_c = 2000$ psi

CALCULATE PRESTRESS LOSSES

STRESS RELIEVED STRAND
½"ϕ 270 ksi 7 wire $A_{ps} = 0.153\ in^2$

INITIALLY TENSIONED TO 0.70 fpu = 189 ksi after anchor set.

NON-composite section properties

$A_x = 560\ in^2$
$I = 125,390\ in^4$
$y_b = 20.27\ in$
$y_T = 24.73\ in$
$w = 583$ plf

composite section properties

$A_x = 1164.1\ in^2$
$I = 363,941\ in^4$
$\bar{y}_b = 35.05\ in$
$\bar{y}_t = 7.95\ in$

STRAND PATTERN

22 strands

@ ENDS $e_{ps} = 16.50\ in$
@ ℄ Brg $e_{ps} = 7.22\ in$

Deflection point @ 6'-0 from midspan

$f'_{ci} = 4000$ psi
$E_{ci} = 3640$ ksi

$f'_c = 5000$ psi
$E_c = 4030$ ksi

Vol/surface RATIO = 4.06

REF: AASHTO sec 9.16.2

TOTAL LOSS $\Delta f_s = SH + ES + CR_c + CR_s$
@ service LIFE

SHRINKAGE

$SH = 17{,}000 - 150 * RH$

RH = RELATIVE HUMIDITY FIG 9.16.21.1 pg 95
= 40%

$SH = 17000 - 150(40) = 11{,}000$ psi

ELASTIC SHORTENING

$ES = \frac{E_s}{E_{ci}} f_{cir}$

$E_s = 28 \times 10^6$ psi

$E_{ci} = 33 w^{3/2} \sqrt{f'_c} = 33(145^{3/2})\sqrt{4000} = 3.64 \times 10^6$ psi

M_{DL} DUE to GIRDER self wt. @ MIDSPAN

$w_{DL}^B = 583$ plf $M_{DL}^B = 0.583(59^2)/8 = 254$ ft-k

ASSUME $f_0 = 0.63 f_{pu} = 170.1$ ksi

$P_0 = 22(0.153)170.1 = 572.6^k$

$f_{cir} = P_0/A + P_0 e_{ps}^2/I - M_{DL}^B e_{ps}/I$

$= 572.6(10^3)/560 + 572.6(10^3)16.50^2/125390 - 254(12000)16.50/125390$

$f_{cir} = 1022 + 1243 - 401 = 1864$ psi

$ES = 28(10^6)/3.64(10^6)\,1864 = 14{,}340$ psi

Creep of Concrete

$CR_c = 12 f_{cir} - 7 f_{cds}$

$f_{cds} = M_{DL}^{nc} e_{ps}/I + M_{DL}^{c} \bar{e}_{ps}/\bar{I}$

$M_{DL}^{nc} = (1.35 - 0.383)\,59^2/8 + 1.76(59)/4 = 360$ '-k

$e_{ps} = 16.50$ in $\quad I = 125{,}390$ in^4

$M_{DL}^{c} = 205$ '-k

$\bar{e}_{ps} = 35.05 - 83/22 = 31.28$ in

$\bar{I} = 363{,}941$

$f_{cds} = 360(12000)\,16.50/125390 + 205(12000)\,31.28/363941$

$= 568 + 212 = 780$ psi

$CR_c = 12(1864) - 7(780) = 16{,}910$ psi

RELAXATION of PRESTRESSING Steel

$$CR_s = 20{,}000 - 0.4ES - 0.2(SH + CR_c)$$

$$= 20000 - 0.4(14340) - 0.2(11000 + 16910)$$

$$= 8680 \text{ psi}$$

TOTAL LOSSES @ SERVICE LIFE

$$\Delta f_s = 11{,}000 + 14340 + 16910 + 8680 = 50{,}930 \text{ psi}$$

26.95%

Estimated Total Loss @ RELEASE

$$\Delta f_{si} = 0.10\,SH + ES + 0.50\,CR_s$$

$$= 0.10(11000) + 14340 + 0.5(8680) = 19{,}780 \text{ psi}$$

10.5%

∴ OK

$$f_{se} = 189 - 50.93 = 138.07 \text{ ksi}$$

$$F_f = 0.153 \text{ in}^2 (138.07) = 21.12 \text{ k/STR.}$$

$$f_{sei} = 189 - 19.78 = 169.22 \text{ ksi}$$

$$F_o = 0.153(169.22) = 25.89 \text{ k/STR.}$$

CHECK CONCRETE STRESSES

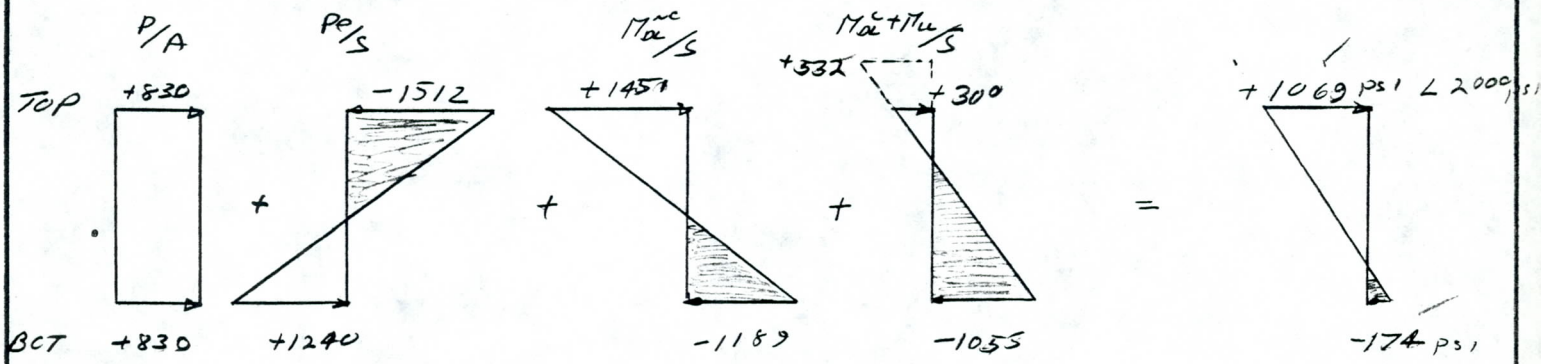
@ SERVICE LOAD

@ MIDSPAN

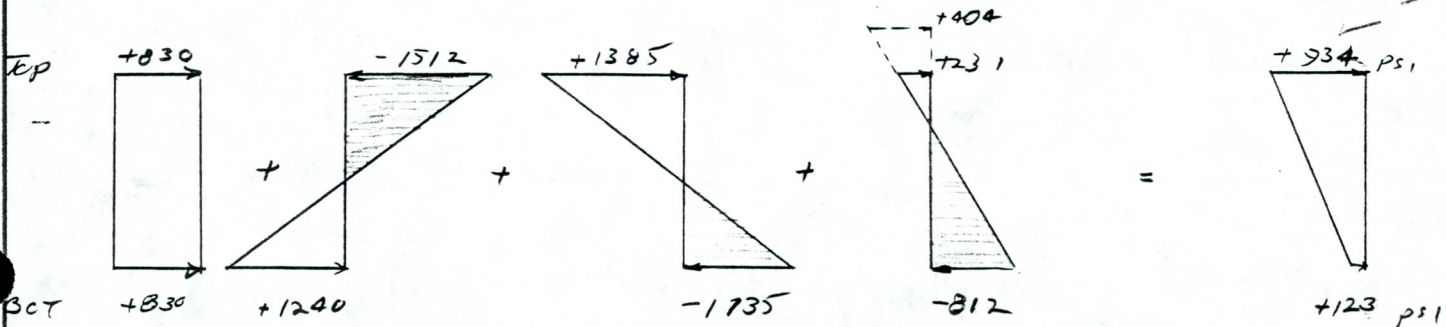
$P_f = 22(0.153)138.07 = 464.7^k$

$e_{ps} = 16.50$ in

	simple	2-SPAN
M_{DL}^{NC}	613 '-k	585
M_{DL}^{C}	205	118
M_{LL}	708	585



SIMPLE SPAN @ ℄ SPAN



2-SPAN CONTINUOUS @ 0.4L

∴ OK

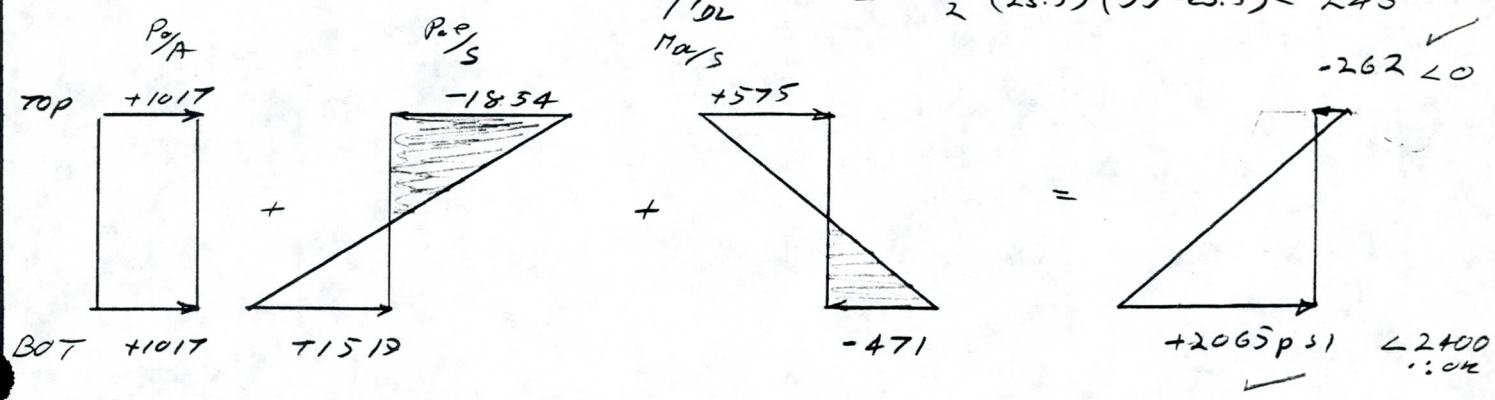
@ RELEASE

@ HOLD-DOWNS

$P_o = 22(0.153)169.22 = 569.6^k$

$e = 16.50$ in

$M_{DL}^{NC-BEAM} = \frac{0.583}{2}(23.5)(59-23.5) = 243$ ft-k

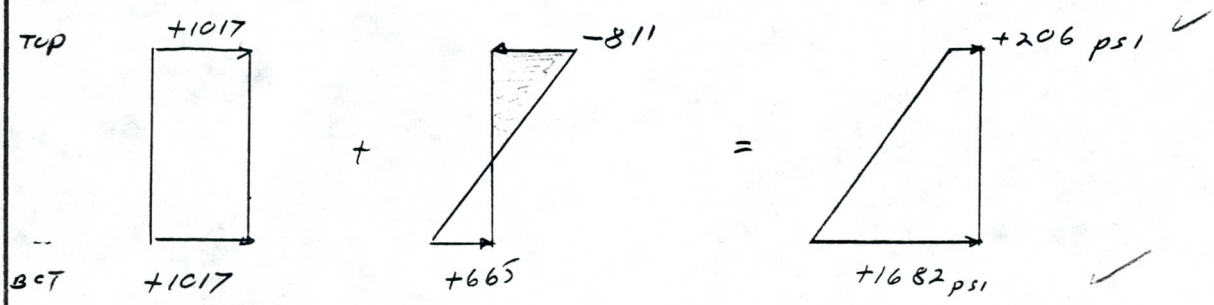


∴ DESIGN using mild steel OR REDUCE strands

@ ℄ Bearing

$P_o = 569.6^k$

$e = 7.22$ in



By inspection AND ADDITIONAL GESI program "PCGD3" RUNS
∴ VARYING STRAND PATTERNS will still result in TENSION AT the top of the GIRDER AT the HOLD-DOWNS AT RELEASE

RESIST EXCESS TENSILE STRESSES BY USING MILD steel [AASHTO 9.15.2.1]

$$P/A - Pe/S_T + M_{DL}^{BEAM}/S_T + M_{cr}/S_T = 0 \text{ psi}$$

$$569.6(1000)/560 + [-569.6(1000)16.50 + 243(12000) + M_{cr}(12000)]/5070 = 0$$

+ 262 p.l. (P24)

$$\therefore M_{cr} = [0 - 1017 + 1854 - 575]\,5070/12000 = 111 \text{ Ft-k}$$

GRADE 40 reinf $\Rightarrow$ $f_s = 20000$ psi
$f'_{ci} = 4000$ psi $\Rightarrow$ $f_{ci} = 1600$ psi

$$n = E_s/E_c = 29(10^6)/33(145^{3/2})\sqrt{4000} = 7.96 \quad \text{say } 8$$

$$k = 1/1 + f_s/nf_c = 1/1 + \frac{20}{8(1.6)} = 0.390$$

$$j = 1 - k/3 = 0.870$$

$$A_s = M(\text{in-lb})/f_s\, j\, d$$

$$d = 45 - 2\tfrac{1}{2} = 42.50 \text{ in}$$

$$A_s = 111(12000)/20000(0.870)42.50 = 1.80 \text{ in}^2$$

USE 6 #5 $A_s = 1.86 \text{ in}^2$

Alternative

Try 4- 7-wire strands

$e_{ps} = 42.5 - 20.27 = 22.23$ in

$$262 = P/A + Pe/S_T$$

$$= P/560 + P(22.23)/5070$$

$$= P[1/560 + 1/228.07]$$

$\therefore P = 42,461$ lb

Assuming a 10% initial loss

stress to $42461/0.90 = 47,179$ lb

or 11794 say 12000 k/strand

use 4 - 3/8"ϕ or larger 7-wire strand stressed to 12,000 k each

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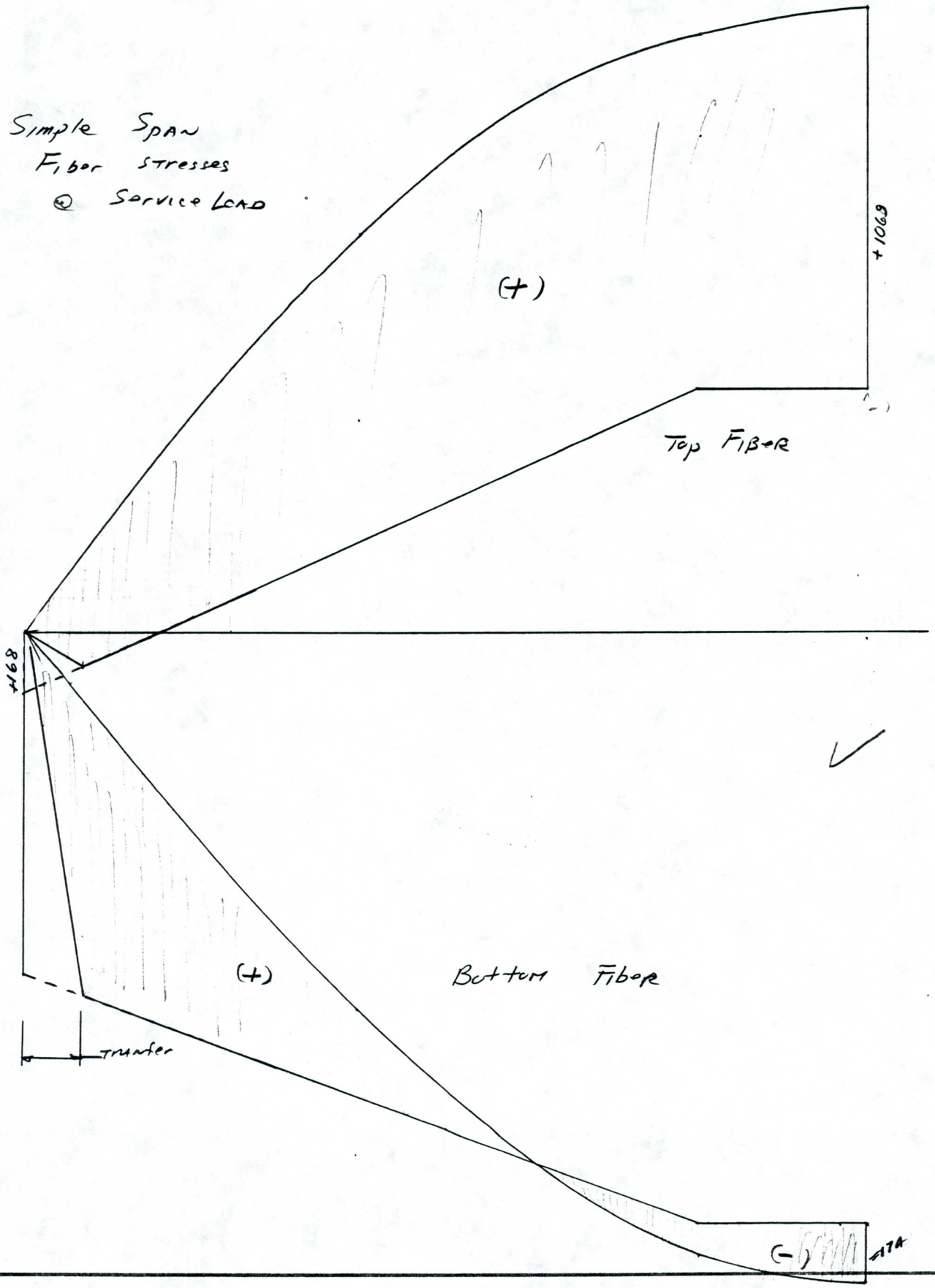


CHECK FLEXURAL STRENGTH [9.17]

$\rho_{ps} = A_{ps}/bd$

$b = 80.55^{in}$

$d = 45 + 7\frac{1}{2} - 20.27 + 16.50 = 48.73^{in}$

$A_{ps} = 22(0.153) = 3.366\ in^2$

$\rho_{ps} = 3.366/80.55(48.73) = 0.0008575$

$f_{su}^* = f_{pu}(1 - 0.5\omega')$

$\omega' = \rho_{ps} f_{pu}/f_c' = 0.0008575(270)/5 = 0.0463$

$f_{su}^* = 270(1 - 0.5(0.0463)) = 263.75$ ksi

$\omega = \rho_{ps} f_{su}^*/f_c' = 0.0008575(263.75)/5 = 0.0452 < 0.30$

$\therefore$ ok

$1.4 d\omega = 1.4(48.73)0.0452 = 3.09^{in} < 7\frac{1}{2}''$

$\therefore$ RECTANGULAR SECTION

-- provided $M_u = A_{ps} f_{su}^* d(1 - 0.6\omega)$

$= 3.366(263.75)48.73(1 - 0.6(0.0452))/12$

$= 3507^{ft\text{-}k}$ PROVIDED

REQUIRED M_u @ MIDSPAN

$$M_u = 1.3[DL + \tfrac{5}{3}(LL+I)]$$

$$= 1.3[613+205 + \tfrac{5}{3}(708)] = 2597^{\text{ft-k}} < 3507$$

$\therefore$ ok ✓

MINIMUM STEEL

$$M_u \geq 1.2 M_{cr}$$

$$f_{cr} = 7.5\sqrt{f'_c} = 7.5\sqrt{5000} = 530 \text{ psi} \checkmark$$

$$f_{cr} = P/A + Pe/S_b - M_{cr}/S_b$$

$$-530 = 464.7(1000)/560 + 464.7(1000)16.50/6186 - M_{cr}\,12000/10383$$

$$\therefore M_{cr} = [829.8 + 1239.5 + 530]\,10383/12000$$

$$= 2249^{\text{ft-k}} \checkmark$$

$$1.2 M_{cr} = 2699^{\text{ft-k}} < 3507^{\text{ft-k}} \quad \therefore \text{ok} \checkmark$$

ANCHORAGE ZONE

[AASHTO 9.21.3]

$F_{EB} = 4\% * P_o$

$= 4/100 (569.6) = 22.78^k$

$A_s^{EB} = F_{EB}/f_s = 22.78/20 = 1.14 \text{ in}^2$ in $d/4$

$d = 45 - 20.27 + 7.22 = 31.95^{in}$

$d/4 = 7.99$ in

REF: "Design of Prestressed Concrete Structures" 3rd Ed
T.Y. Lin, N.H. Burns pp 284

$A_t = 0.021 \frac{T}{f_s} \cdot \frac{h}{l_t}$ = required total area of stirrups at the end of the girder, uniformly distributed over a length equal to $h/5 = 9$ in

T = effective prestress force at release
$= 569.6^k$

l_t = transfer length of strand $\cong 50 d_s = 25$ in

$A_t = 0.021 \frac{569.6}{20} \cdot \frac{45}{25} = 1.08 \text{ in}^2$

USE 4 legs #5 stirrup $A = 1.24 \text{ in}^2$

place 6-#4 @ 9" enclosing bottom prestressing (each end)

DEFLECTION / CAMBER

↓ (+)

@ RELEASE

DUE TO BEAM SELF-WEIGHT

$$\Delta_{DL}^{BEAM} = 5wl^4 / 384 E_c I \quad @ \text{ MID-SPAN}$$

$w = 583 \text{ plf} = 48.58 \text{ pli}$

$E_c = E_{ci} = 33 (145^{3/2}) \sqrt{4000} = 3.64 \times 10^6 \text{ psi}$

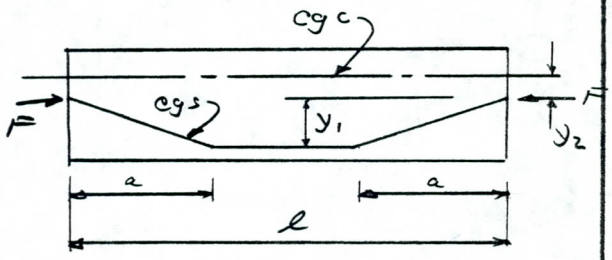
$I = 125{,}390 \text{ in}^4$

$l = 59^{FT} = 708 \text{ in}$

$$\Delta_{DL}^{BEAM} = 5(48.58)\, 708^4 / 384 (3.64) 10^6 (125390) = 0.348^{in} \downarrow$$

DUE TO PRESTRESSING

$$\Delta_{PS} = -\frac{l^2}{8EI}\left[M_2 + M_1 - \frac{M_1}{3}\left(\frac{2a}{l}\right)^2 \right]$$



$M_1 = F y_1 \qquad M_2 = F y_2$

$l = 708 \text{ in}$

$a = (59\tfrac{1}{2} - 6)\, 12 = 282^{in}$

$E = 3.64 \times 10^6$ psi

$F = 569.6^k$ (pg 21)

$I = 125{,}390 \text{ in}^4$

$y_1 = 16.50 - 7.22 = 9.28^{in}$

$y_2 = 7.22$ in

$M_1 = 569.6\,(9.28) = 5{,}285{,}888$ in-lb

$M_2 = 569.6\,(7.22) = 4{,}112{,}512$ in-lb

$$\Delta_{\frac{1}{2}} = -\frac{708^2}{8(3.64)10^6(125390)}\left[4112512 + 5285888 - \frac{5285888}{3}\left(\frac{2(282)}{708}\right)^2\right]$$

$= -1.135^{in}$ (↑)

TOTAL CAMBER @ RELEASE

$\Delta_r = -1.135 + 0.348 = -0.787^{in}$ (↑)

$\approx \frac{3}{4}^{in}$

DUE to Composite topping

$w = 734 + 33 = 767$ plf $= 63.92$ pli (pg 5)

$P = 1{,}760^{lb}$

$$\Delta_{DL}^{vc} = \frac{5wl^4}{384EI} + \frac{Pl^3}{48EI}$$

$E = 33\,(145^{3/2})\sqrt{5000} = 4.07 \times 10^6$ psi

$I = 125\,390 \text{ in}^4$ $l = 708^{in}$

$$\Delta_{DL}^{NC} = 5(63.92)\,708^4 / 384(4.07)\,10^6(1253.3c) + 1760(708^3)/48(4.07)\,10^6(1253.3c)$$

$$= 0.410 + 0.025 = 0.435^{in} \checkmark \quad (\downarrow)$$

$$\approx 7/16^{in}$$

TOTAL CAMBER AT ERECTION

REF: PCI DESIGN HANDBOOK, PG 3-27, TABLE 3.4.1

$$\Delta^{SL} = 1.85(0.348) - 1.80(1.135) + 0.435$$

$$= -1.399 + 0.435 = -0.964^{in} \checkmark \quad (\uparrow)$$

camber @ erection

$$\approx 1^{in}$$

USE A 2^{in} HAUNCH ✓

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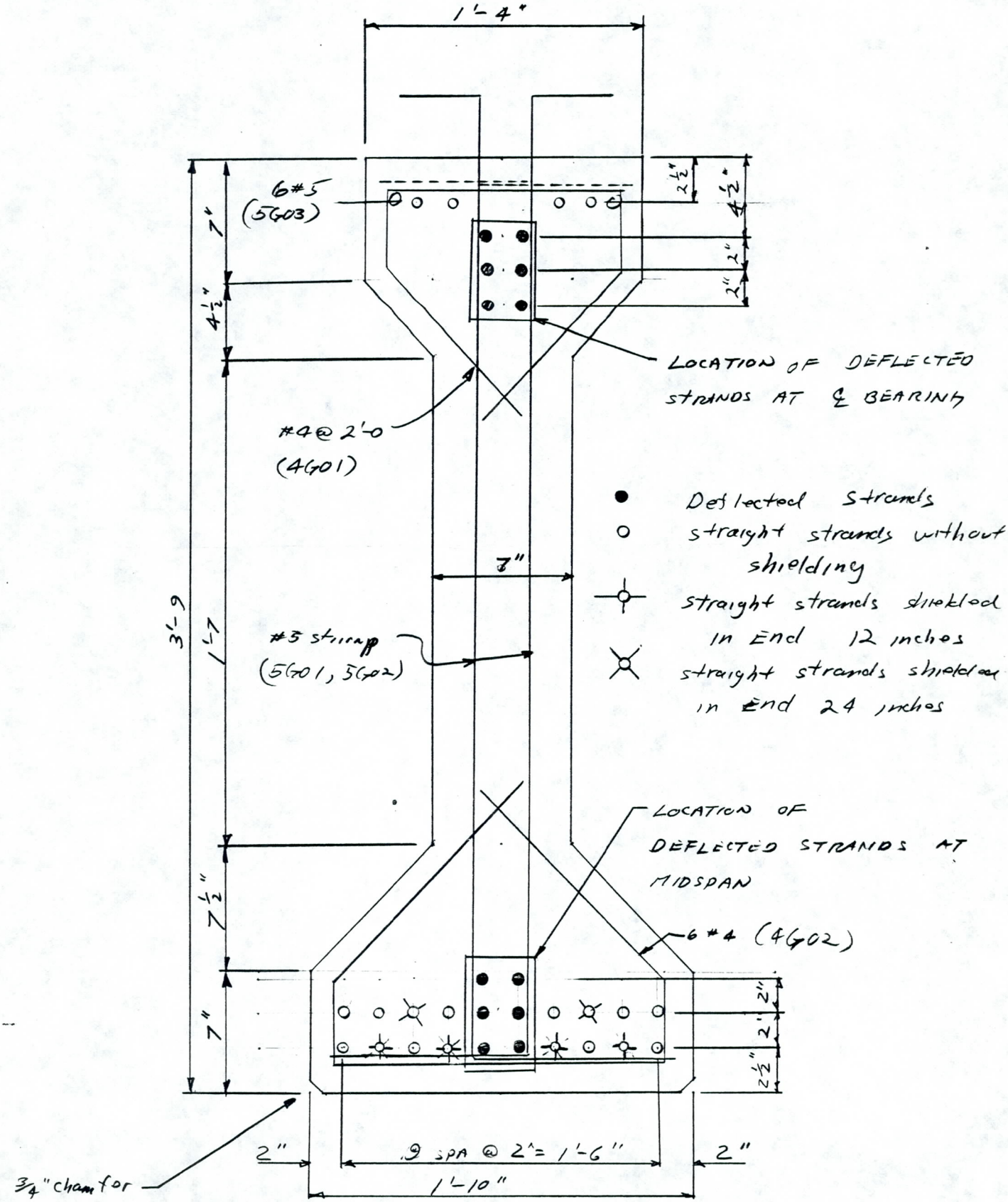

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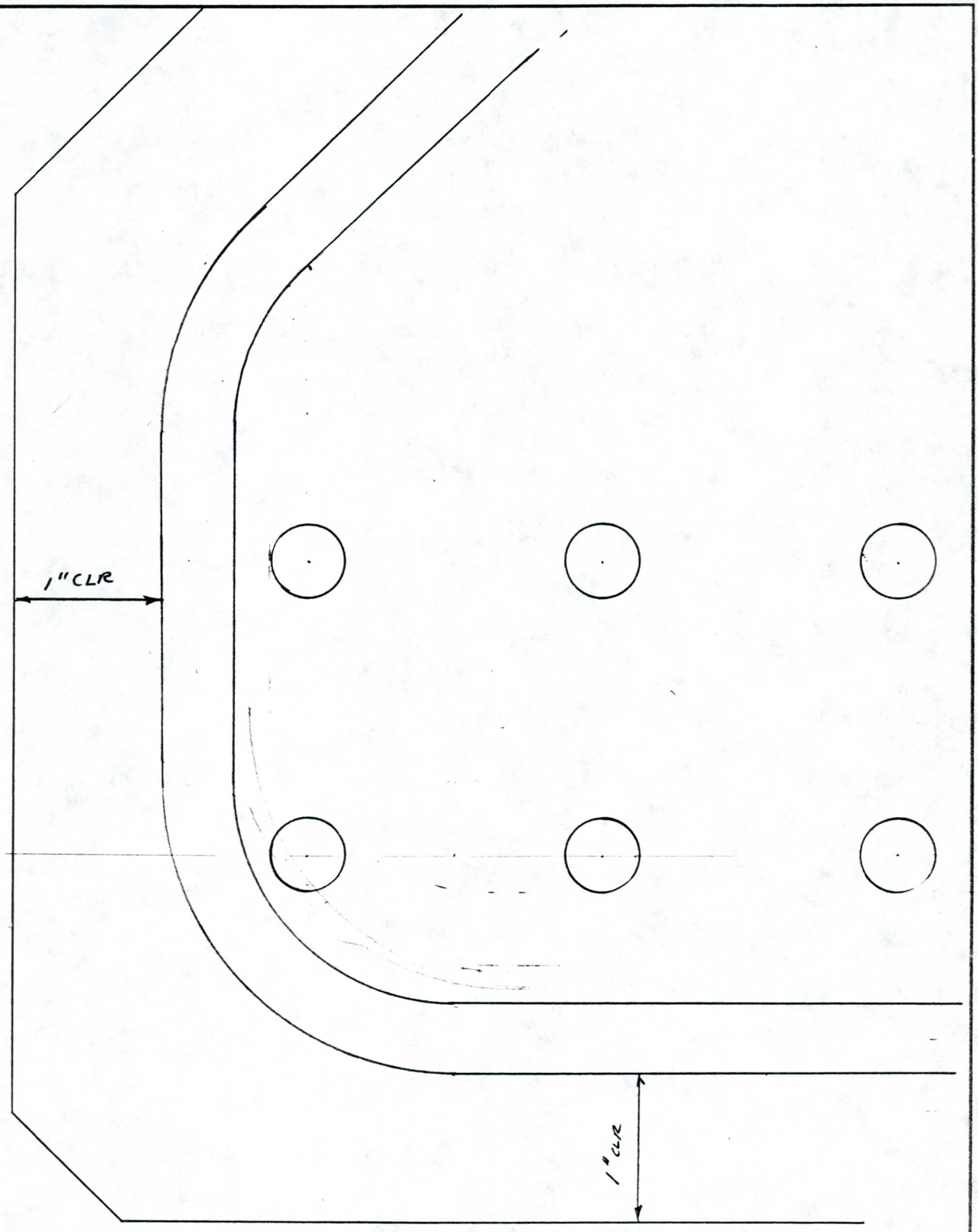
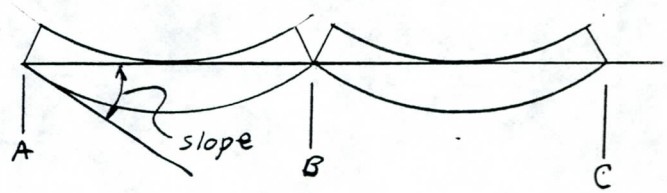


Continuity

REF: "DESIGN OF CONTINUOUS HIGHWAY BRIDGES WITH PRECAST, PRESTRESSED CONCRETE GIRDERS



SHRINKAGE RESTRAINT MOMENT

$$M_s = \varepsilon_s E_b A_b \left(e_2' + t/2 \right)$$

E_b = DECK SLAB ELASTIC MODULUS
$= 33 (145^{3/2}) \sqrt{4000} = 3.64 \times 10^6$ psi

A_b = SLAB TOTAL CROSS-SECTIONAL AREA
$= 90 (7.5) = 675 \text{ in}^2$
└ max b_{eff}

$(e_2' + \frac{t}{2})$ = distance between the mid-depth of the slab and the centroid of the "composite" section

$e_2' = \bar{y}_t = 9.95^{in}$
t = slab thickness $= 7\frac{1}{2}''$

$$e_2' + t/2 = 9.95 + \frac{7.5}{2} = 13.70^{in}$$

ε_s = DIFFERENTAL SHRINKAGE STRAIN

ASSUME continuity connection made AT 45 days AFTER PRESTRESS RELEASES
40% HUMIDITY

proportion of FINAL shrinkage

$= 0.50$ (FIG. 8)

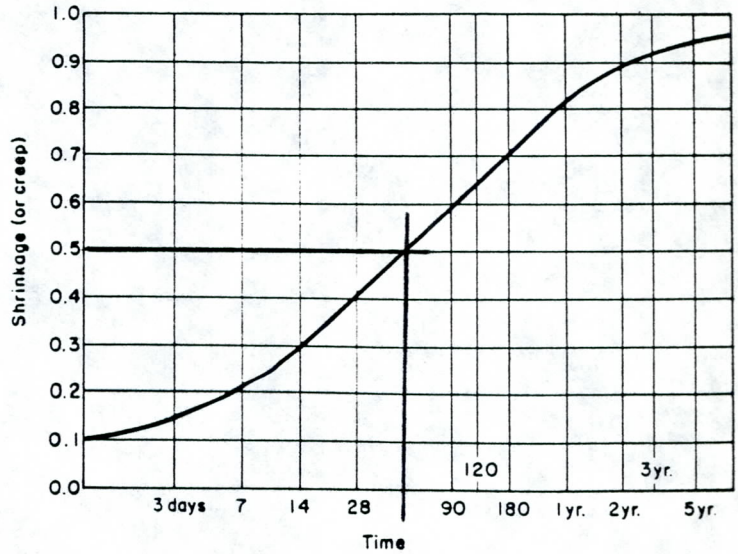


Fig. 8. Proportion of final shrinkage or creep vs. time

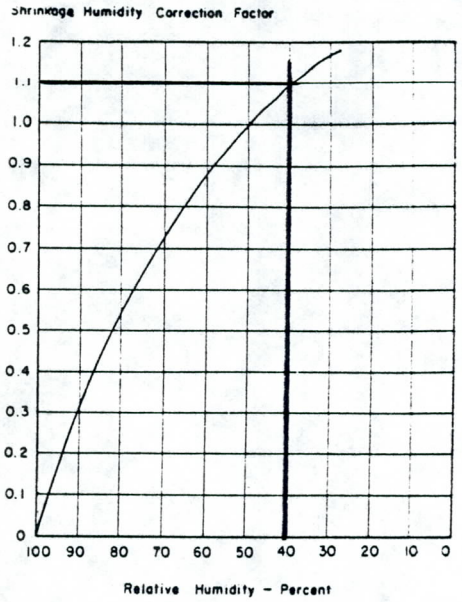


Fig. 10. Shrinkage humidity correction factor

HUMIDITY CORRECTION = 1.10 ✓

$$\varepsilon_s = (0.600 \times 10^{-3})\, 0.50\, (1.10)$$

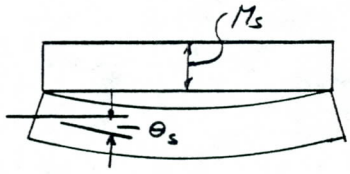
$$= 3.300 \times 10^{-4} \checkmark$$

$$\therefore M_s = 3.300(10^{-4})\, 3.64(10^6)\, 675\, (13.70)$$

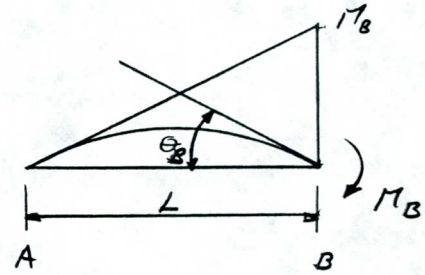
$$= 11.109 \times 10^6 \text{ in-lb} = 925.7 \text{ Ft-k} \checkmark$$

SLOPE AT END OF the simple-span BEAMS

$$\theta_s = M_s L / 2EI \checkmark$$



Apply MOMENT AT B to RETURN BEAM END TO A HORIZONTAL position



SLOPE AT B $= -\frac{M_B L}{2EI} * \frac{2}{3}$

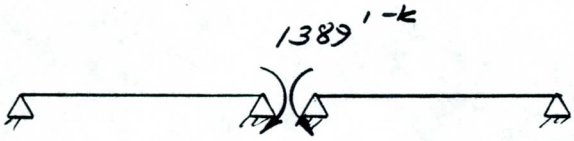
$= -M_B L / 3EI$

set slope EQUAL to slope due to shrinkage

$-M_B L / 3EI = M_S L / 2EI$

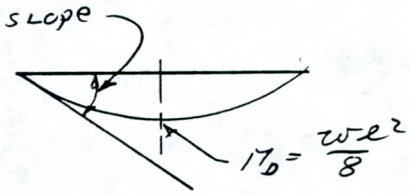
$\therefore M_B = -\frac{3}{2} M_S$

$= -\frac{3}{2}(925.7) = \underline{\underline{-1389}}$ FT-k ✓



SHRINKAGE
NOT ADJUSTED for CREEP EFFECTS

DEAD LOAD CREEP RESTRAINT MOMENT



$$\text{SLOPE} = M_D L / 3EI$$

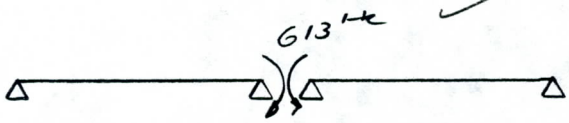
Apply moment at B to return beam end to horizontal

$$-M_B L / 3EI = M_D L / 3EI$$

$$\therefore M_B = -M_D$$

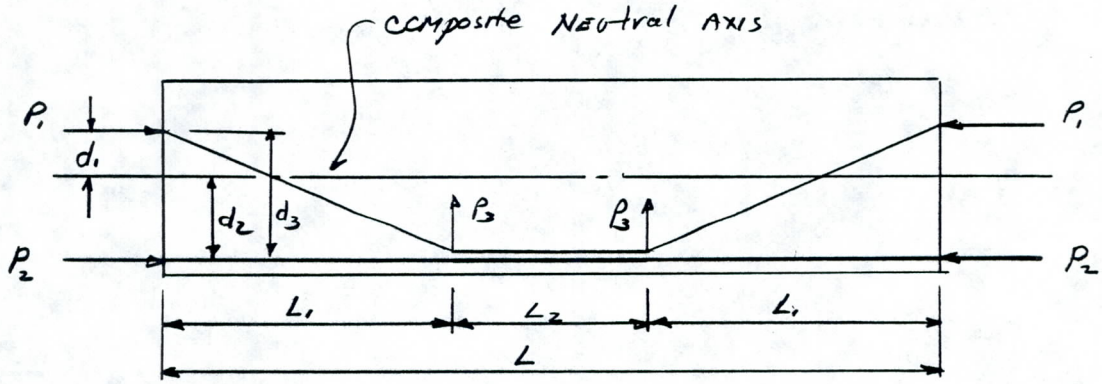
$$M_D = 613^{1-k} \Rightarrow M_B = -613^{1-k}$$

simple span non-comp. DL



DEAD LOAD CREEP NOT reduced by creep effects

PRESTRESS CREEP RESTRAINT MOMENT



15% of total Loss ✓ OK

$L_2 = 12'$ $\qquad L_1 = (59-12)\frac{1}{2} = 23.50'$

Assume stress losses @ time of connection ≅ 20% fsi

$f_e = 189 - 0.20(189) = 151.2$ ksi

P_1

6 strands $(0.153 \text{ in}^2/\text{str})\ 151.2$ ksi $= 138.8^k$ ✓

2	@	40.5	=	81
2	@	38.5	=	77
2	@	36.5	=	73
6				231

$e_1 = 35.05 - 231/6 = -3.45$

$d_1 = -e_1 = 3.45$ in ✓

P_2

16 strands $(0.153 \text{ in}^2/\text{str})\ 151.2$ ksi $= 370.1^k$ ✓

8	@	$2\frac{1}{2}$	=	20
8	@	$4\frac{1}{2}$	=	36
16				56

$e_2 = 35.05 - 56/16 = 31.55$ in $= d_2$ ✓

$d_3 = d_1 + d_2 = 35$ in ✓

$$P_3 = P_1 d_3 / L_1 = 138.8(35) / 23.5(12) = 17.23^k$$

slope DUE to PRESTRESS

$$[P/S \text{ slope}] = [P_1 d_1 L - P_2 d_2 L - P_3 L_1^2 - P_3 L_1 L_2] / 2EI$$

$\frac{1}{2} P_1 d_1 L = 138.8(3.45)59.0(12)/2 = 169,516$

$-\frac{1}{2} P_2 d_2 L = 370.1(31.55)59.0(12)/2 = -4,133,536$

$-\frac{1}{2} P_3 L_1^2 = 17.23(23.50(12))^2/2 = -685,099$

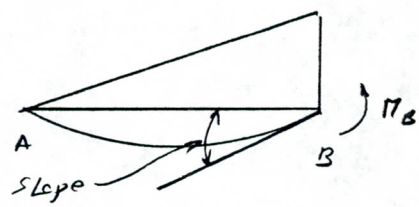
$-\frac{1}{2} P_3 L_1 L_2 = 17.23(23.50)(12)144/2 = -349,838$

$\Sigma \; -4,998,957$

$$[P/S \text{ slope}] = -4,998,957 / EI$$

APPLY moment to B to bring beam end to horizontal

$$-[P/S \text{ slope}] = M_B L / 3EI$$



$$M_B = -3EI/L \, [P/S \text{ Slope}]$$

$$= -3EI/59 \, [-4998957/EI] / 144 \, IN^2/FT^2$$

$$= 1765 \, ^{\prime}\text{-}k$$

PRESTRESSING NOT including creep effects

CREEP EFFECT FACTOR

for an concrete elastic Modulus at Release

$E_{ci} = 3.64 \times 10^6$ psi

specific creep AT 20yrs $= 0.41 \times 10^{-6}$ in/in/psi ✓

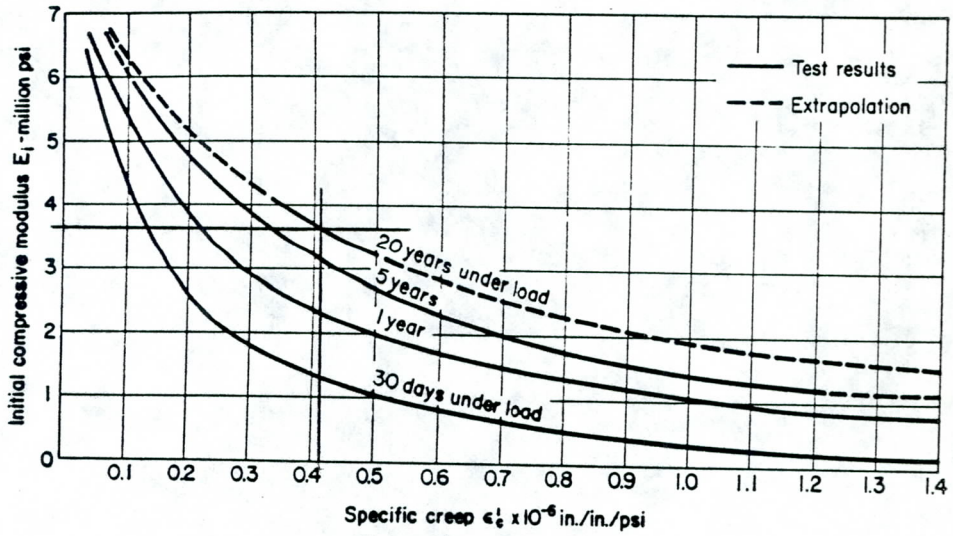


Fig. 5. Prediction of basic creep from elastic modulus

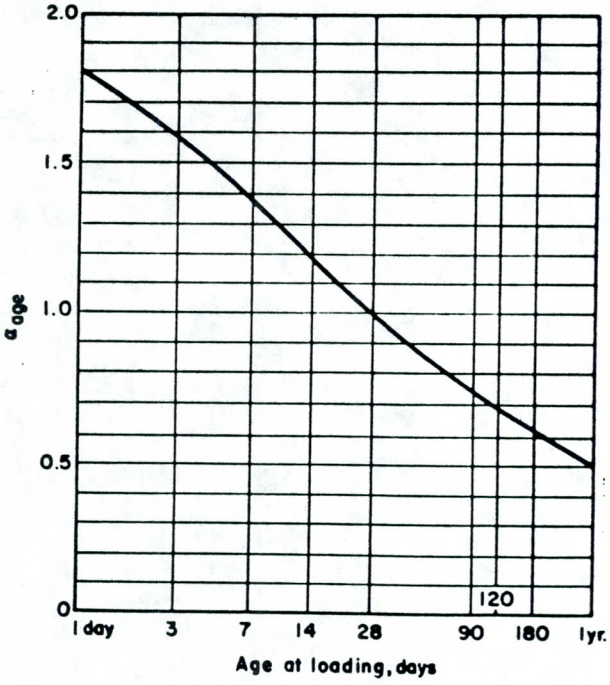


Fig. 6. Creep vs. age at loading

Assume release of prestress AT 1 day

$\alpha_{age} = 1.8$

Adjusted $\epsilon_c' = 0.41 (10^{-6})\ 1.8$

$= 0.74 \times 10^{-6}$ in/in/psi ✓

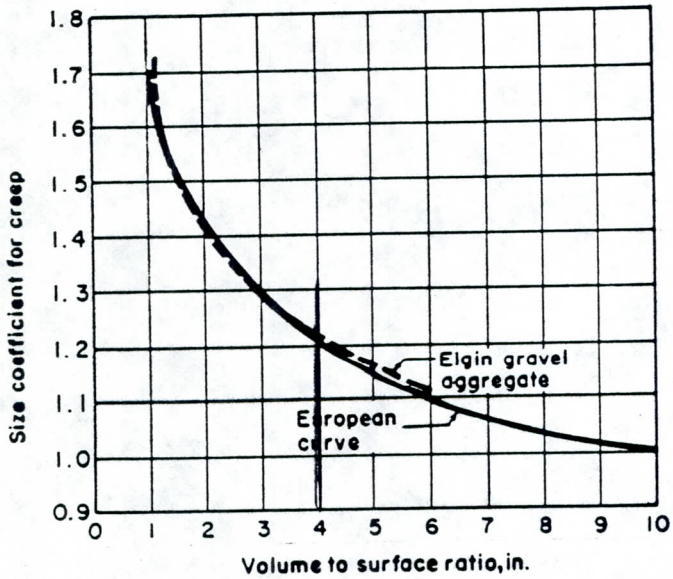


Fig. 7. Creep vs. volume-to-surface ratio

Table 1. Creep data for AASHO-PCI I-beams

AASHO-PCI girder type	Volume/surface ratio in.	Creep volume/surface ratio correction factor (from Fig. 7)
I	3.0	1.28
II	3.4	1.25
III	4.0	1.20
IV	4.7	1.16
V	4.4	1.18
VI	4.4	1.18

SIZE coefficient for a V/S RATIO = 4.0 , 1.20

$\varepsilon_c' = 1.20\,(0.74 \times 10^{-6}) = 0.886 \times 10^{-6}$ in/in/psi ✓

@ Erection 50% of creep has occured (Fig 8)
leaving 50% to occur

or $\varepsilon_c' = 0.50\,(0.886 \times 10^{-6}) = 0.44 \times 10^{-6}$ in/in/psi ✓

$\phi = \varepsilon_c / \varepsilon_e = \varepsilon_c E_c = 0.44(10^{-6})\,4.03(10^{6}) = 1.784$ ✓

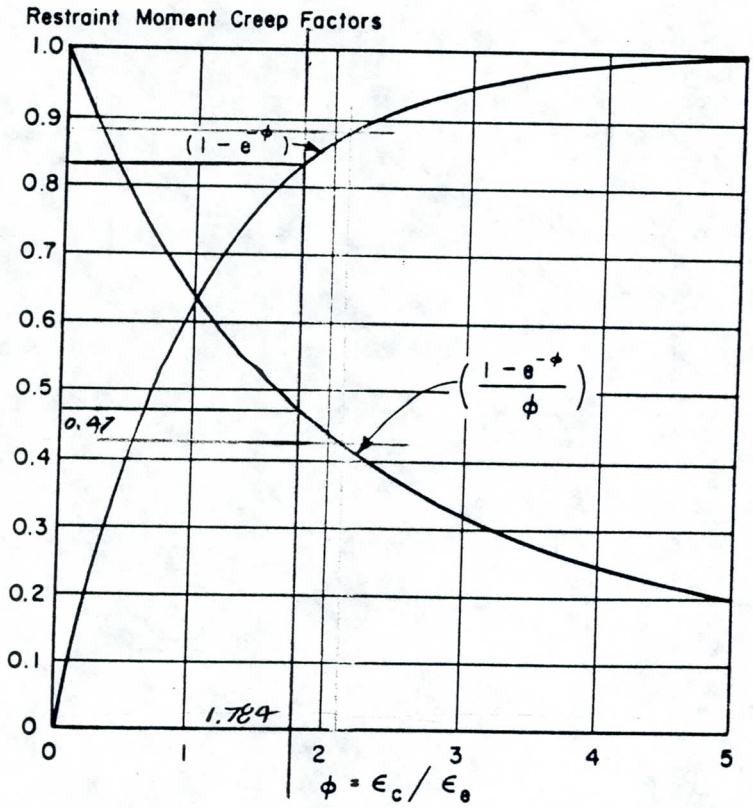


Fig. 9. Variations in restraint moment creep effect factors for both creep and shrinkage

$(1-e^{-\phi}) = 0.83$ ✓

$\left(\frac{1-e^{-\phi}}{\phi}\right) = 0.47$ ✓

Summation of Restraint Moments

$$M_r = (M_{ps} - M_{DL})(1-e^{-\phi}) - M_s\left(\frac{1-e^{-\phi}}{\phi}\right)$$

$$= (1765-613)(0.83) - 1389(0.47)$$ ✓

$$= \underline{303^{\text{ F-k}}}$$ ✓

Design of Positive Moment Connection

$M_R = 303^{\text{Ft-k}}/\text{Girder}$

$M^+_{LL+I} = 0^{\text{ft-k}}$ $M^-_{LL+I} = -448^{\text{ft-k}}$

Assume Grade 60 Rebar $f_s = 24$ ksi

Diaphragm & Deck Conc.

$f'_c = 4$ ksi $f_c = 1.4$ ksi

$n = E_s/E_c = 28(10^3)/57\sqrt{4000} = 7.77$ say 8

$k = \dfrac{1}{1+\frac{f_s}{nf_c}} = \dfrac{1}{1+\frac{24}{8(1.4)}} = 0.318$

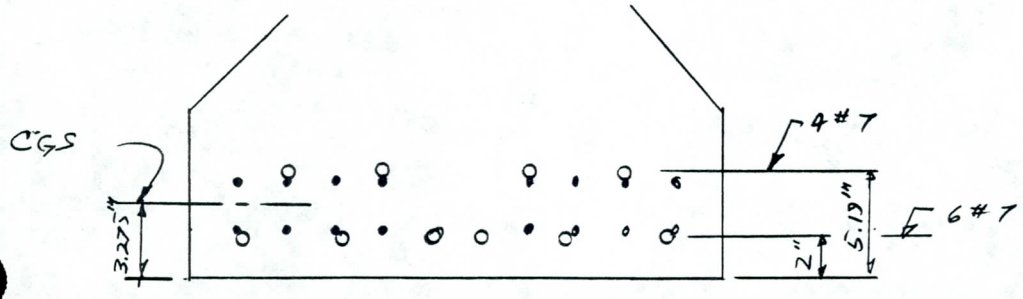
$j = 1 - k/3 = 0.894$

$a = f_s j/12 = 24(0.894)/12 = 1.79$

$d \cong 45 + 7.5 - 3\frac{1}{2} = 49^{\text{in}}$

$A_s = M/ad = 303/1.79(49) = 3.46 \text{ in}^2$

TRY 10 # 7 $A_s = 6 \text{ in}^2$



$d = 45 + 7.5 - 3.275 = 49.23^{\text{in}}$

Greiner Engineering

Greiner Engineering Sciences, Inc.

Tucson 602 327-3413
Phoenix 602 275-5400

Job 51230 A2 35th Ave ACDC

Description

Calculated by RLC Date 21 Nov 84 Sheet No. A1

Checked by G.G.H. Date 6 Dec 84 of

IF $A_s = M/dd$

$a = f_s j / 12$

$\therefore \quad f_s = 12M / A_s j d$

$= 12(303) / 6(0.894)49.23 = 13.77$ ksi

FOR $M = 303 - 448 = -145$ 1-k

$f_s = -145(12) / 6(0.894)49.23 = -6.59$ ksi

FATIGUE stress limits [8.16.8.3]

$f_f = 21 - 0.33 f_{min} + 8(r/h)$

$= 21 - 0.33(-6.59) + 8(0.3) = 25.6$ ksi

$\pm M_{LL+I} = 448$ 1-k

$f_s^r = 448(12) / 6(0.894)49.23 = 20.4$ ksi < 25.6 ∴ OK

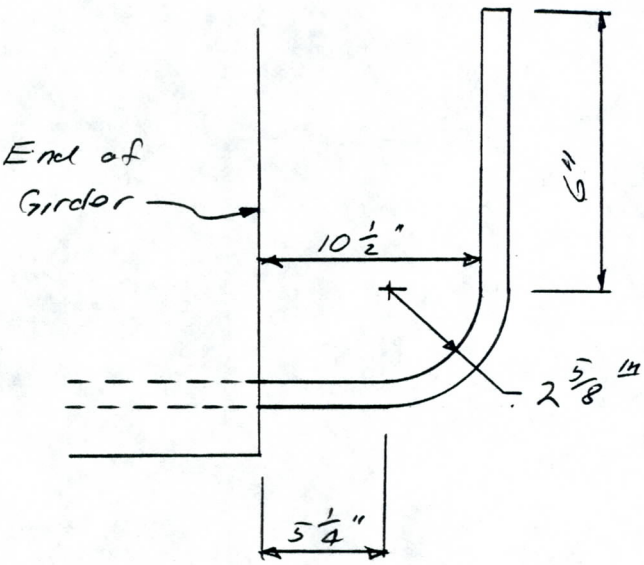
ALLOWABLE BOND stress $= 0.1 f'_c = 0.1(4000)$
$= 400$ psi, use 350 psi max

perimeter #7 $= 2.749$ in ✓

Req'd embedment length $= \left(f_s / 350\text{psi}\right) \times A_s$ ← A_s/bar

$= \left(13770 / 350(2.749)\right) \times 0.6$

$= 8.6$ in ✓



5.25
4.81 1/4 bend
10.06 ✓
6.00 ✓ Extension
16.06" > 8.6 in ✓
∴ ok

embed bar 3'-10" into Girder ✓

ALTERNATE POSITIVE MOMENT Connection

TRY EXTEND Bottom 8 strands

FIND f_s

$A_s = 8(0.153) = 1.224 \text{ in}^2$

$d = 45 + 7\frac{1}{2} - 2\frac{1}{2} = 50 \text{ in}$

TRY $kd = 4 \text{ in}$

$n = 8$

		A	y	Ay
SLAB	80.55(4)	322.2	2	644.4
steel	1.224(8)	9.8	50	489.6
	Σ	332.0		1134.0

$y = 1134/332 = 3.42 \text{ in}$

TRY $y = 3.36 \text{ in}$

		A	y	Ay	Ay^2	I_o
SLAB	80.55(3.36)	270.6	1.68	454.7	763.9	257
steel	1.224(8)	9.8	50	489.6	24,480	—
	Σ	280.4		944.3	25,243.9	257

$y = 944.3/280.4 = 3.37 \text{ in} \approx 3.36$ OK

$I = 25244 + 257 - 280.4(3.37^2) = 22,321 \text{ in}^4$

Distance from N.A. to steel $= 50 - 3.37 = 46.63 \text{ in}$

steel stress

$$f_s = Mc/I \; n = 303^{\cdot}(12000)\,46.63 / 22321\,(8)$$

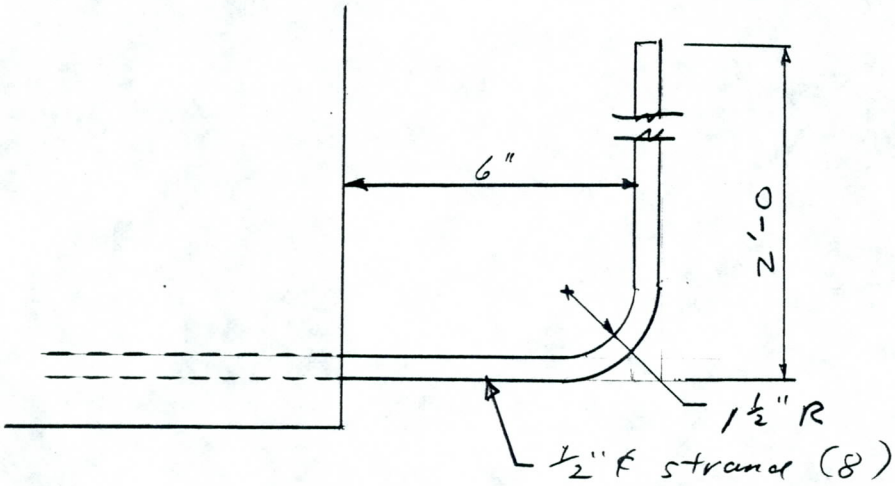
$$= 60,767 \text{ psi}$$

allowable bond stress = 350 psi

perimeter ½" strand ≅ 1.571 in (#4 bar)

$$\text{REQ'D Embedment Length} = 60767\,(0.153) / 350\,(1.571)$$

$$= 16.9 \text{ in}$$



4.50"
2.75" ¼ bond
7.25
22.00 extension
29.25 IN > 16.9 in
∴ OK

EFFECT OF LATERAL FORCE AT TOP OF CAP ON GIRDERS

PG 21A

$V = 10.16$ kip (112 LF) / 13 Girders D+L+E

$= 87.53$ k/Girder

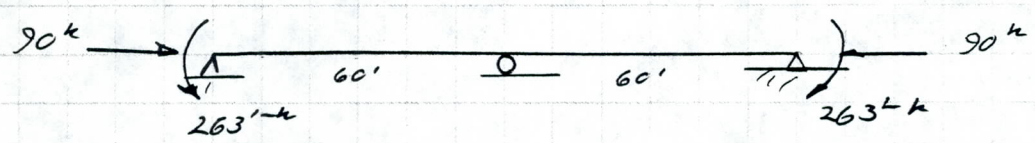
say 90 k/Girder

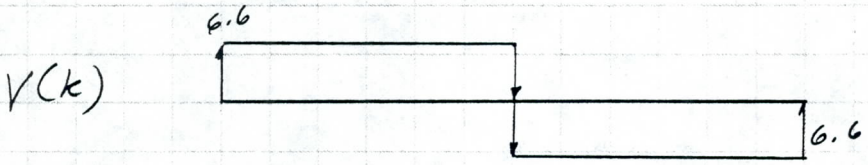
End Moment due to eccentricity

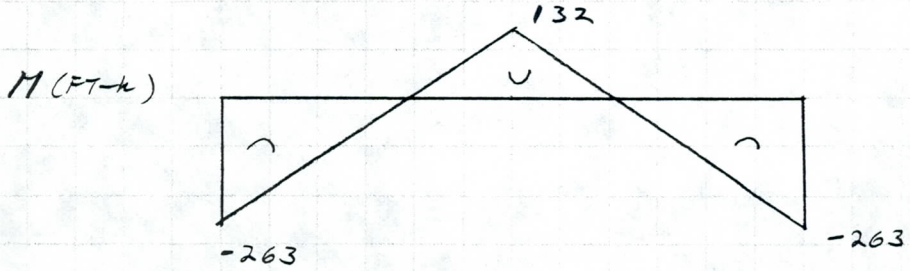
PG 14 superstructure

$M = 90 (35.05'' / 12) = 263$ FT-k/FT

on composite section







Composite Section Properties

$A = 1164 \text{ in}^2$

$S_B = 10,383 \text{ in}^2$

$S_T = 36,577 \text{ in}^2$

} Girder

Stress due to End Forces

End

$90/1164 - 263(12)/36577 = 9 \text{ psi}$ T Top

$+ 263(12)/10383 = 381 \text{ psi}$ C Bot

Mid-span

$90/1164 - 66(12)/36,577 = 56 \text{ psi}$ C Top

$+ 66(12)/10383 = 154 \text{ psi}$ C Bot

@ ℄ Brg

due to prestress — pg 21

Top 168 psi C − 9 = 159 psi C ∴ OK

Bot 1372 psi C + 381 = 1753 psi C < 2000 ∴ OK

@ Midspan

∴ OK by inspection

check Positive Moment Connection

$M = 303 + 132 = 435^{1-k}$ (PS 40)

$A_s = M/ad = 435/1.79(49) = 4.96 \text{ in}^2$ (ℓ_{PS40})

6 in² supplied (10-#7) ∴ ok

$f_s = 435(12000)/6(0.8)4)49.23 = 19,768$ psi

$\ell_d = (19,763/350(2.749))0.60 = 12.33$ in ($\ell_{\#7}$ ℓ_{A_s})

16.06 in supplied ∴ ok

ALT. 8 strands

$f_s = 12000(435)46.63/22321(8) = 87,239$ psi

$\ell_d = 87239(0.153)/350(1.371) = 24.28$ in (Aps)

29.25 supplied ∴ ok

∴ No change in girder design

NEGATIVE Reinforcement over Pier

USE LOAD FACTOR DESIGN

$M_{DL}^{nc} = 0^{1-k}$

$M_{DL}^{c} = -212^{1-k}$

$M_{LL} = -448^{1-k}$

$M_u = 1.3[DL + LL\,\tfrac{5}{3}]$

$= -1.3[212 + \tfrac{5}{3}(448)] = -1246^{1-k}$

$F = bd^2/12000$

$b = 22.0^{in}$

$d = 45 + 7\tfrac{1}{2} - 2''clr - \#6 - \#8/2 = 49.25^{in}$

$F = 22.0(49.25^2)/12000 = 4.45$

$K_u = M_u/F = 1246/4.45 = 280$

SP-17(73) $\rho = 0.0054 > \rho_{min} = 0.0033$

$a = 4.29\,in < 7''$ bot. flg. $\therefore$ OK

$A_s = \rho bd = 0.0054(22.0)49.25 = 5.85\,in^2$

Ref: "Design Handbook" ACI Pub SP-17(73)

Flexure 1.2—Coefficients for rectangular sections without compression reinforcement

$\rho = \omega \frac{f'_c}{f_y} = \frac{A_s}{bd}$

$\frac{c}{d} = 1.18 \frac{\omega}{\beta_1}$

$\frac{a}{d} = \beta_1 \frac{c}{d}$

$\beta_1 = 0.85$

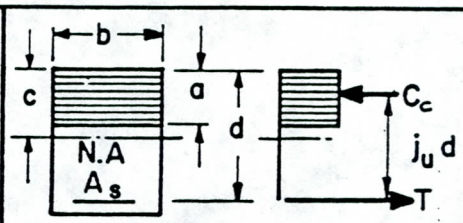
$K_u = \phi f'_c \omega (1 - .59\omega)$

$a_u = \phi f_y (1 - .59\omega)/12{,}000$

$A_s = \frac{M_u}{a_u d}$

$F = \frac{M_u}{K_u} = bd^2/12{,}000$

$j_u = 1 - \frac{a}{2d}$



A_s in square inches and M_u is in ft - kips

ω	K_u	f_y = 40,000		f_y = 50,000		f_y = 60,000		f_y = 80,000		c/d	a/d	j_u
		ρ	a_u	ρ	a_u	ρ	a_u	ρ	a_u			
.020	71	.0020	2.96	.0016	3.71	.0013	4.45	.0010	5.93	.028	.024	.988
.030	106	.0030	2.95	.0024	3.68	.0020	4.42	.0015	5.89	.042	.035	.982
.040	141	.0040	2.93	.0032	3.66	.0027	4.39	.0020	5.86	.056	.047	.976
.050	175	.0050	2.91	.0040	3.54	.0033	4.37	.0025	5.82	.069	.059	.971
.060	208	.0060	2.89	.0048	3.62	.0040	4.34	.0030	5.79	.083	.071	.965
.070	242	.0070	2.88	.0056	3.60	.0047	4.31	.0035	5.75	.097	.083	.959
.080	274	.0080	2.86	.0064	3.57	.0053	4.29	.0040	5.72	.111	.094	.953
.090	307	.0090	2.84	.0072	3.55	.0060	4.26	.0045	5.68	.125	.106	.947
.100	339	.0100	2.82	.0080	3.53	.0067	4.23	.0050	5.65	.139	.118	.941
.110	370	.0110	2.81	.0088	3.51	.0073	4.21	.0055	5.61	.153	.130	.935
.120	401	.0120	2.79	.0096	3.48	.0080	4.18	.0060	5.58	.167	.142	.929
.130	432	.0130	2.77	.0104	3.46	.0087	4.15	.0065	5.54	.180	.153	.923
.140	462	.0140	2.75	.0112	3.44	.0093	4.13	.0070	5.50	.194	.165	.917
.150	492	.0150	2.73	.0120	3.42	.0100	4.10	.0075	5.47	.208	.177	.912
.160	522	.0160	2.72	.0128	3.40	.0107	4.08	.0080	5.43	.222	.189	.906
.170	551	.0170	2.70	.0136	3.37	.0113	4.05	.0085	5.40	.236	.201	.900
.180	579	.0180	2.68	.0144	3.35	.0120	4.02	.0090	5.36	.250	.212	.894
.190	607	.0190	2.66	.0152	3.33	.0127	4.00	.0095	5.33	.264	.224	.888
.200	635	.0200	2.65	.0160	3.31	.0133	3.97	.0100	5.29	.278	.236	.882
.210	662	.0210	2.63	.0168	3.29	.0140	3.94	.0105	5.26	.292	.248	.876
.220	689	.0220	2.61	.0176	3.26	.0147	3.92	.0110	5.22	.305	.260	.870
.230	716	.0230	2.59	.0184	3.24	.0153	3.89	.0115	5.19	.319	.271	.864
.240	742	.0240	2.58	.0192	3.22	.0160	3.86	.0120	5.15	.333	.283	.858
.250	767	.0250	2.56	.0200	3.20	.0167	3.84	.0125	5.12	.347	.295	.853
.260	792	.0260	2.54	.0208	3.17	.0173	3.81	.0130	5.08	.361	.307	.847
.270	817	.0270	2.52	.0216	3.15	.0180	3.78	.0135	5.04	.375	.319	.841
.280	841	.0280	2.50	.0224	3.13	.0187	3.76	.0140	5.01	.389	.330	.835
.290	865	.0290	2.49	.0232	3.11	.0193	3.73			.403	.342	.829
.300	889	.0300	2.47	.0240	3.09	.0200	3.70			.416	.354	.823
.310	912	.0310	2.45	.0248	3.06	.0207	3.68			.430	.366	.817
.320	935	.0320	2.43	.0256	3.04	.0213	3.65			.444	.378	.811
.330	957	.0330	2.42	.0264	3.02					.458	.389	.805
.340	978	.0340	2.40	.0272	3.00					.472	.401	.799
.350	1000	.0350	2.38							.486	.413	.794
.360	1021	.0360	2.36							.500	.425	.788
.370	1041	.0370	2.35							.514	.437	.782

1. When $h_f / d \geq a/d$, beam is designed as rectangular section; when $h_f / d < a/d$, beam is designed as a T section (See Flexure 2).
2. Values of ρ above upper solid line are for percentages smaller than ρ min., ρ min. = $200/f_y$, See section 10.5, ACI 318-71.
3. Values of ρ below lower solid line would exceed ρ max. = 0.75 ρ_b. For values of ρ_{max} see values of $(\rho_w - 0.75\ \rho_f)$ max. in Flexure 2. See Section 10.3.2. ACI 318-71.
4. For deflection see Section 9.5 ACI 318-71 and "Deflection" chapter in this handbook.
5. Check for Crack Control where $f_y > 40{,}000$ See Section 10.6. ACI 318-71 and under chapter "Reinforcement" in this handbook.
6. The capacity reduction factor $\phi = 0.9$ has been included in the table values.
7. For use of this Table, see Flexure Examples 1 to 10, 15, 16

DISTRIBUTE OVER 1/2 GIRDER SPACING $= \frac{7'-10''}{2} = 3.92^{FT}$

$A_s = 5.85/3.92 = 1.49 in^2/FT$

TRY #8 @ 6" $A_s = 1.58 in^2/FT$

Final steel stress

TRY kd = 13.35 in

	A	y	Ay	Ay^2	I_o
7(16)	112	3.5	392	1372	457
6.35(7 + 2(7.5 − 6.35))	59.1	10.2	601	6114	198
$2[\frac{1}{2}(6.35^2)]$	40.3	9.1	368	3351	90
5.85(7)	41.0	49.25	2018	99,448	
	252.4		3380	110,285	745

$kd = 3380/252.4 = 13.39 in$ ∴ ok

$I = 110,285 + 745 - 252.4(13.39^2) = 63,777 in^4$

from N.A. to reinf = 49.25 − 13.39 = 35.86 in

steel stress due to LL+I

$f_s^{LL} = 449(12000)35.86 / 65777 (7) = 20,516$ psi

fatigue range $f_r = 21 + 0.8 (r/h)$
$= 21 + 0.8(0.3) = 23.4$ ksi
∴ OK

DUE to D.L

$f_s^{DL} = 212(12000)35.86 / 65777 (7) = 9708$ psi

$\Sigma f_s = 30,225$ psi

check flexural cracking [8.16.8.4]

$f_s \leq Z/(d_c A_c)^{1/3} \leq 0.6 f_y$

$Z = 170$ kli

$d_c = 2'\text{clr} + \#6 + \#8/2 = 3.25''$

$A_c = 2(3.25)6 = 39 \text{ in}^2$

$Z/(d_c A_c)^{\frac{1}{3}} = 170/(3.25(39))^{\frac{1}{3}} = 33.84$ ksi $< 0.6 f_y = 36$ ksi

30.3 ksi < 33.8 ksi ∴ OK

check compressive stress in Girder

Bottom

DUE to LL+I + DL^NC

$f_c^{LL} = 448(12000)\,13.39/65777 + 212(12000)\,13.32/65777$

$= 1094 + 518 = 1612$ psi

DUE to prestress

$f_c^{ps} = 464.7(1000)/560 + 464.7(1000)\,7.22/6186$

$= 830 + 542 = 1372$ psi

$\Sigma f_c = 2984$ psi $> 0.4 f_c = 2000$ psi

this stress will not actually occur because of tendon transfer length and due to the influence of positive moment induced by creep

Reduce the theoretical stress by shielding 6 STRANDS

e'_{ps}

4	@	$2\frac{1}{2}''$	=	10
6	@	$4\frac{1}{2}''$	=	27
2	@	$36\frac{1}{2}''$	=	73
2	@	$38\frac{1}{2}''$	=	77
2	@	$40\frac{1}{2}''$	=	81
16				268

$e'_{ps} = 20.27 - 268/16$

$= 3.52''$

$16(0.153)\,138.07 = 338^k$

$f_c^{ps} = 338000/560 + 338000(3.52)/6186 = 796$ psi

$\Sigma f_c = 2408$ psi

REDUCTION DUE to creep MOMENT

$M_r = 303^{1-k}$ ✓

$f_c^r = 303(12000)13.39 / 65777 = -740$ psi ✓

$\Sigma f_c = 1668$ psi < 2000 ∴ OK ✓

ADDITIONAL tensile stress @ MIDSPAN
DUE to Restraint MOMENT

$M = \frac{1}{2} M_r = 303/2 = 152^{1-k}$ ✓

$f_c^b = 152(12000) / 10383 = -175$ psi ✓

pg 20 ✓

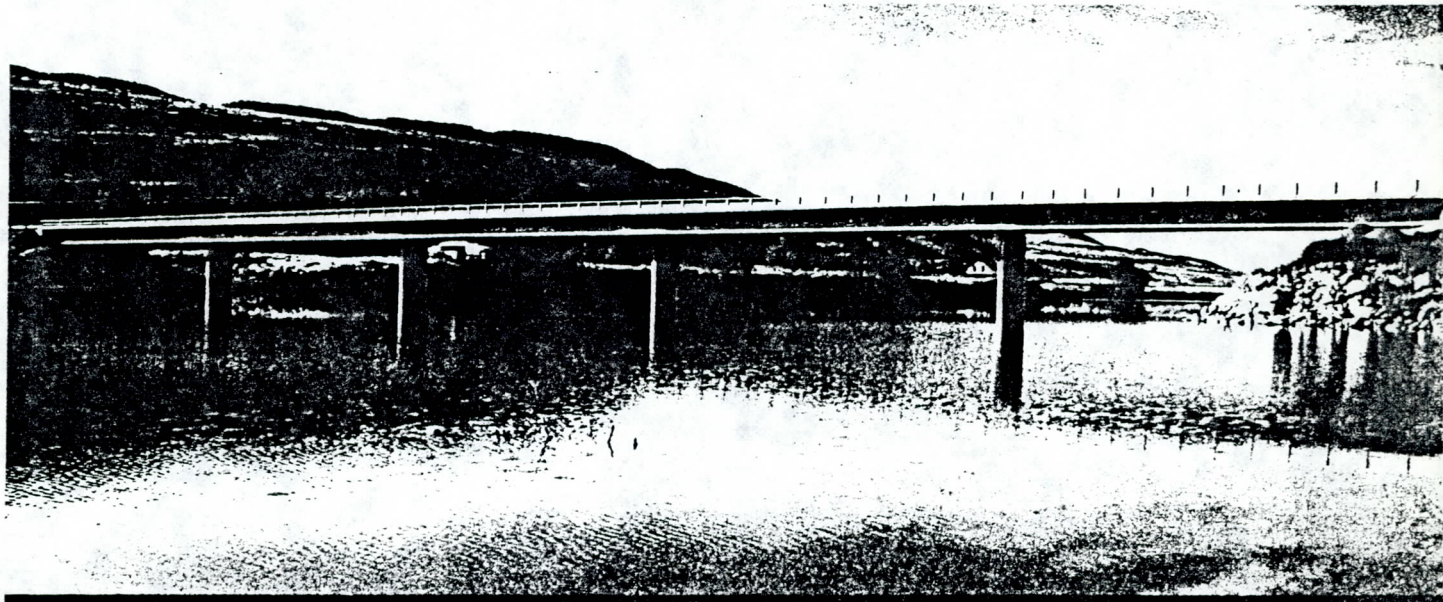
$+ 123$ psi

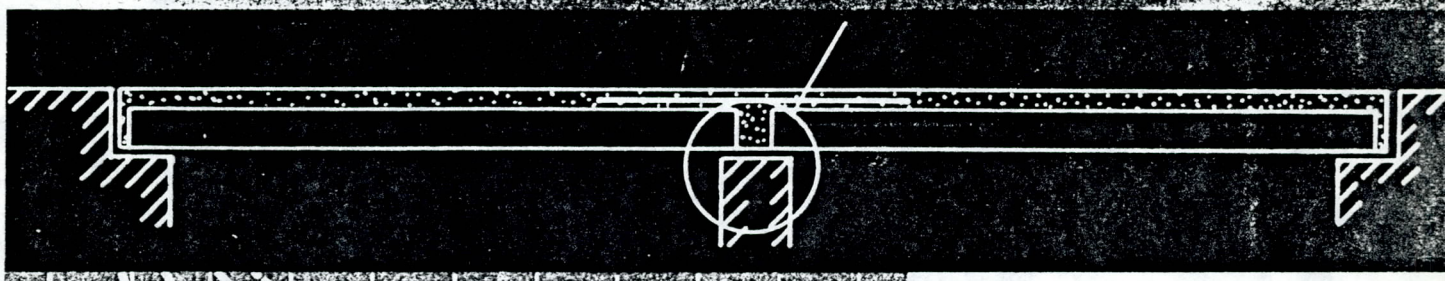
-52 psi $\ll 6\sqrt{f'_c} = 424$ psi

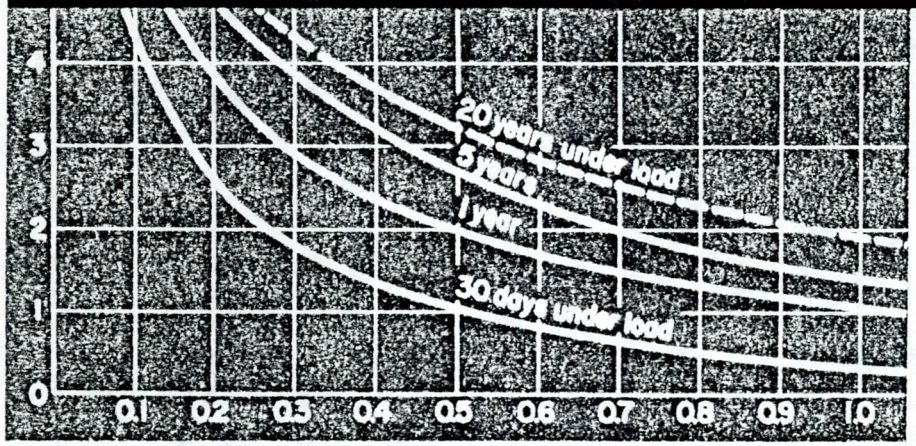
∴ OK ✓

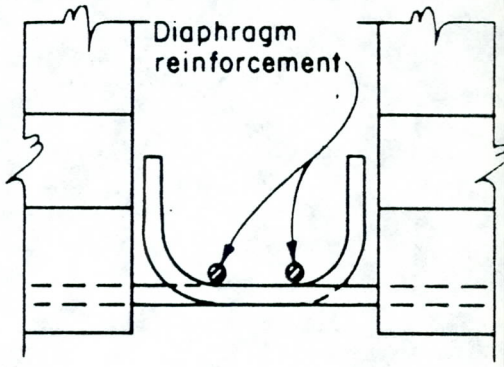
3 PRECAST CONCRETE prestressed units
Portland Cement Association
August 1969

Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders









PORTLAND CEMENT PCA ASSOCIATION

Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders

To achieve continuity in a bridge using precast, prestressed single span units requires design considerations that also include the effects of creep and shrinkage. A complete design example illustrates these considerations following a general discussion of the principles.

Continuous highway bridges with precast, prestressed girders have been built by a number of states. Examples of bridges of this type built by the states of Tennessee and California are presented in Figs. 1 and 2, respectively. The Big Sandy River Bridges in Tennessee were built in 1963-64. Performance has been excellent. The 639-ft. and 703-ft. continuous lengths of deck slab on these bridges are believed to be the longest built to date supported on precast, prestressed girders. The Los Penasquitos Bridge in California was a 1966 PCI award winner.

A primary reason for using continuity with precast, prestressed girders is the elimination of the maintenance costs associated with bridge deck joints and deck drainage onto the substructure. Continuity also improves the appearance and the riding qualities of this type of bridge. Due to the structural economy of continuous designs and the elimination of the deck joint details, some initial economic advantage may also be obtained.

The construction sequence for the type of bridge under consideration is shown in Fig. 3. Continuity is achieved for live load plus impact moments by use of non-prestressed reinforcement in the deck slab and in the diaphragms over piers. An extensive research program on this type of bridge was conducted by the Research and Development Laboratories of the Portland Cement Association during 1960-61. The results of this research, which provide most of the background for this publication, have been made available in a series of six PCA Development Department Bulletins[1-6]. While the above research and the applications of this procedure to date have been directed primarily towards bridges built with I-beams, the procedure applies equally well to bridges with box beams, tee beams, or other available shapes.

The first portion of this report contains a general description of the design features peculiar to this type of bridge and the background material necessary to make design calculations. The second portion consists of a design example.

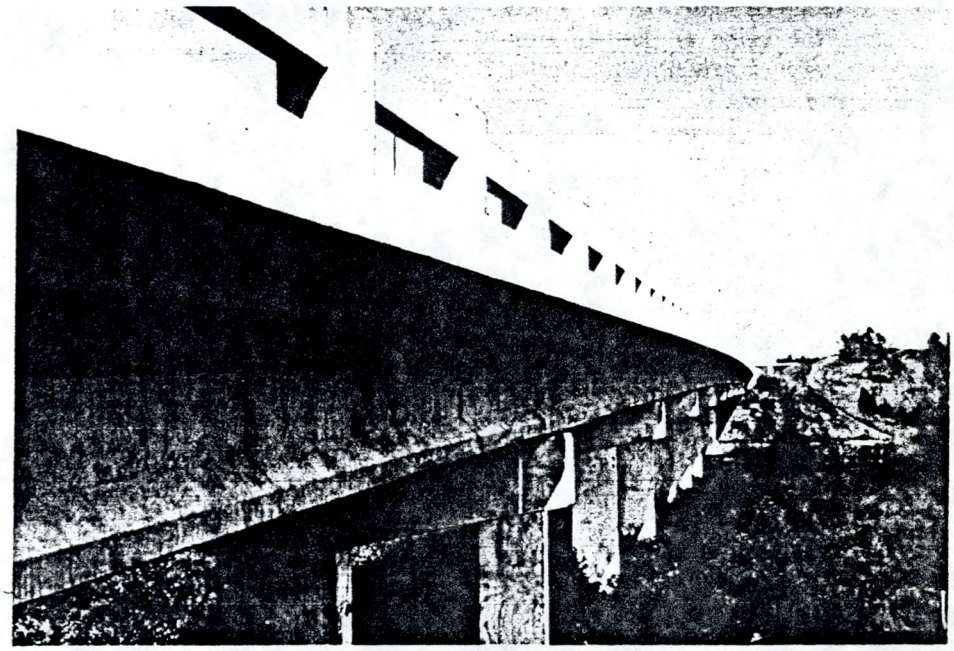


Fig. 1. One of the Big Sandy River Bridges, Tennessee

Part 1. Design Procedure

PRESTRESSING REINFORCEMENT

Except for the determination of the governing live load plus impact moment, the design of the prestressing reinforcement for continuous bridges follows the well-known procedure used for simple-span bridges[7]. The use of continuity permits a reduction of 5 to 15 percent in the required prestress force when compared to simple-span designs. The larger reductions occur in shorter span bridges where the live load plus impact moments are a large portion of the total design moment. Illustrative calculations for determining the governing live load plus impact moments and the required prestressing reinforcement are presented in the design example.

For bridges made up of a series of equal spans, the governing live load plus impact moment occurs at about the 0.4 or 0.5 point of the end spans, with the moment at the mid-point of interior spans being considerably less. Therefore, in some cases it may pay to make separate designs for the prestressing reinforcement in the interior and exterior spans. Where the designer has freedom in choosing the span layout, another alternative is to shorten the exterior spans to obtain the same amount of prestressing reinforcement in interior and exterior spans.

The positive restraint moments over piers, as described in the following section, also increase the positive moments at mid-span sections. When a section at mid-span is subject to the maximum service load moment plus the limiting value of the positive restraint moment at that section, it is recommended that the stress at the bottom face of the girder be limited to about 80 percent of the 28-day modulus of rupture of the concrete. The modulus of rupture is usually taken as $7.5\sqrt{f'_c}$. This requirement will usually be satisfied by the prestressing reinforcement supplied for gravity loads calculated as described in the paragraphs above.

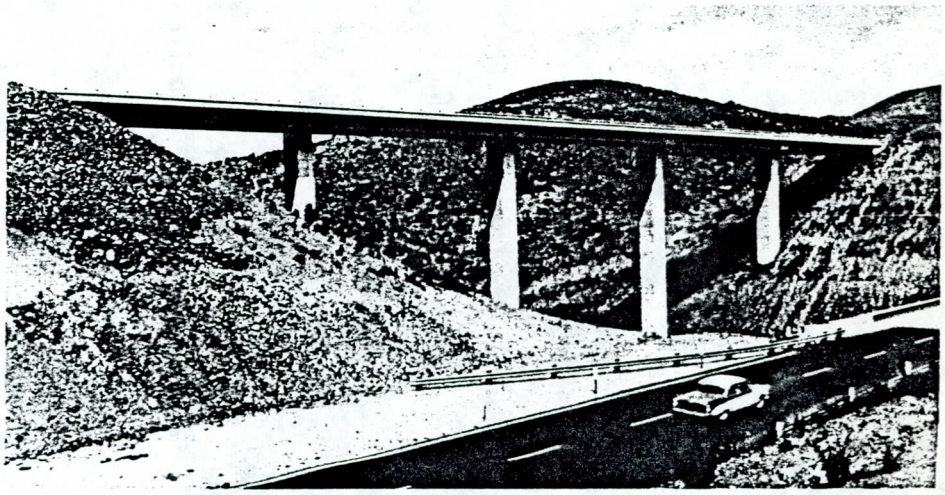


Fig. 2. Los Penasquitos Bridge, California

POSITIVE MOMENTS AT PIERS

One of the unique features of the type of design under consideration is that positive moments develop over piers due to creep in the prestressed girders, as well as due to the effect of live loads in remote spans. These positive moments are partially counteracted by negative moments resulting from differential shrinkage between the cast-in-place deck slab and the precast girders. The positive live load plus impact moments over piers due to loads in remote spans are calculated by conventional procedures.

The effect of creep. The deformations and restraint moments induced in a two-span continuous beam by creep due to prestress force are illustrated in Fig. 4. Since creep is time dependent, with the more rapid deformation occurring during the early stages of loading, the amount of positive restraint moment induced by the prestress force depends on the time when the continuity connection is made, as well as on the inherent creep potential of the concrete mix, the exposure conditions, and the volume to surface ratio of the prestressed member.

The most accurate method of obtaining creep data is by extrapolation from a number of laboratory samples prepared in advance from the actual mix to be used in the girders. In instances where many beams are to be produced, it may be possible to take observations on the initial beams produced, and to develop the necessary creep data from those observations. In most design work, however, it will suffice to rely on available research data. Such research[8,9] indicates that the basic creep value for loading at 28 days can be predicted from the elastic modulus according to the curves in Fig. 5. For design purposes, the 20-year creep curve can be regarded as the ultimate creep.

The ultimate creep value for loading at 28 days, from Fig. 5, must be adjusted to account for the age when the girders are prestressed and for the volume/surface ratio of the girders. The effects of reinforcement on creep can usually be ignored for a prestressed beam. The variation of creep with age at loading is given by the curve in Fig. 6[8,10]. For loading when the concrete is one day old, for example, creep would be 1.8 times the value from the 20-year curve in Fig. 5. The variation of creep with the volume/surface ratio is shown in Fig. 7[8,11]. The volume/surface ratio of the AASHO-PCI I-beams, and the related volume/surface ratio creep cor-

rection factor, are listed in Table 1. The amount of creep which will take place after the continuity connection is made can be found from Fig. 8[8,10]. For example, for a connection made at 28 days, about 40 percent of the creep strains has taken place, leaving the remaining 60 percent to develop moments in the connection.

It has been shown[5] that the effects of creep under prestress and dead load can be evaluated by an elastic analysis assuming that the girder and slab were cast and prestressed as a monolithic continuous girder. The moments obtained from this analysis are multiplied by a factor $(1-e^{-\phi})$ to allow for the effect of creep. The factor ϕ is the ratio of creep to elastic strain (ϵ creep/ϵ elastic). This factor can be evaluated as follows:

1. For a Type VI AASHO-PCI I-beam with an elastic modulus of 4×10^6 psi at release of strands, the creep for loading at 28 days can be obtained from the 20-year loading curve in Fig. 5 as 0.34×10^{-6} in./in./psi.
2. Assuming strand release two days after casting, the multiplication factor from Fig. 6 is about 1.7, and the adjusted creep becomes: $1.7 \times 0.34 \times 10^{-6} = 0.58 \times 10^{-6}$ in./in./psi.
3. The volume/surface ratio for a Type VI AASHO-PCI beam is 4.4 and the correction factor from Fig. 7 is 1.18. Therefore, the further adjusted creep becomes $1.18 \times 0.58 \times 10^{-6} = 0.67 \times 10^{-6}$ in./in./psi.
4. Assuming the moment connection is made at 28 days, Fig. 8 indicates that 40 percent of the ultimate creep will have occurred at the time the connection is made. The connection will have to be capable of developing moments that result from the remaining 60 percent, or, $0.60 \times 0.67 \times 10^{-6}$ in./in./psi $= 0.40 \times 10^{-6}$ in./in./psi.

$$\phi = \frac{\epsilon \text{ creep}}{\epsilon \text{ elastic}}$$

$$E = \frac{1}{\epsilon \text{ elastic}}$$

$$\phi = \epsilon \text{ creep} \times E$$

$$\phi = 0.40 \times 10^{-6} \times 4.0 \times 10^{6} = 1.60$$

Thus, for a connection at 28 days after prestressing, the creep strains still to occur are 1.60 times the original elastic strain.

5. Having evaluated ϕ, the value of $(1-e^{-\phi})$ can be taken directly from Fig. 9. This figure also contains a graph of the factor $(1-e^{-\phi})/\phi$ which is used to allow for the effect of creep in calculating the negative restraint moment due shrinkage.

Dead load creep partially counter-

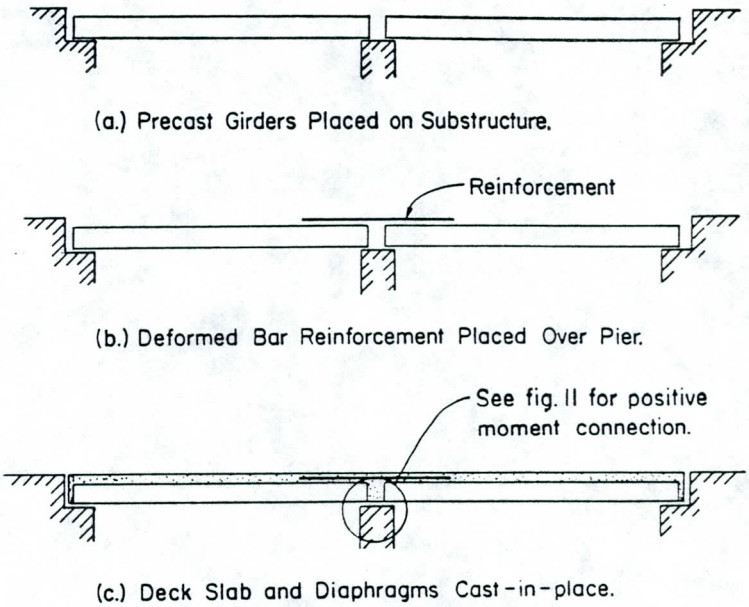


Fig. 3. Construction sequence for a two-span bridge with precast, prestressed girders made continuous for live loads

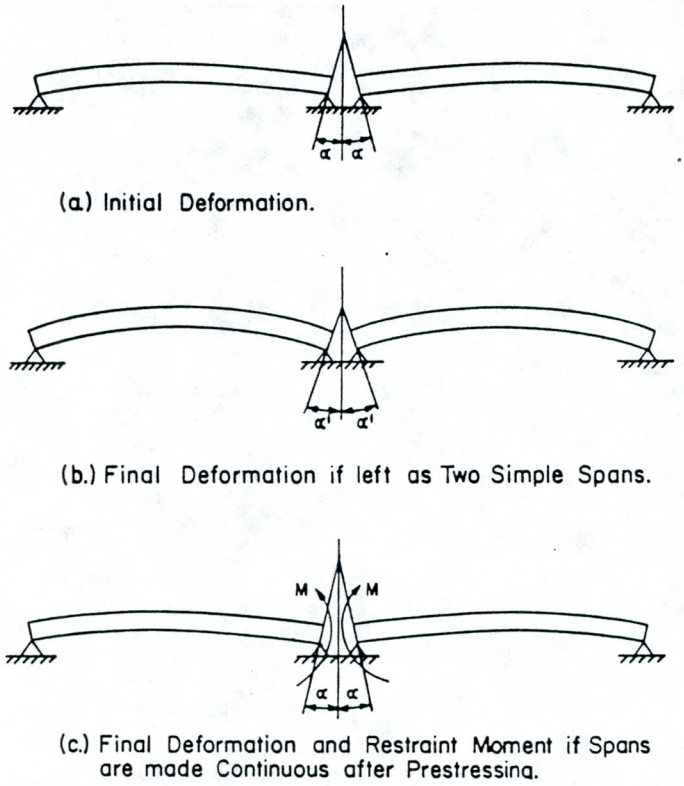


Fig. 4. Deformations and restraint moments in a two-span continuous beam caused by creep under prestress force

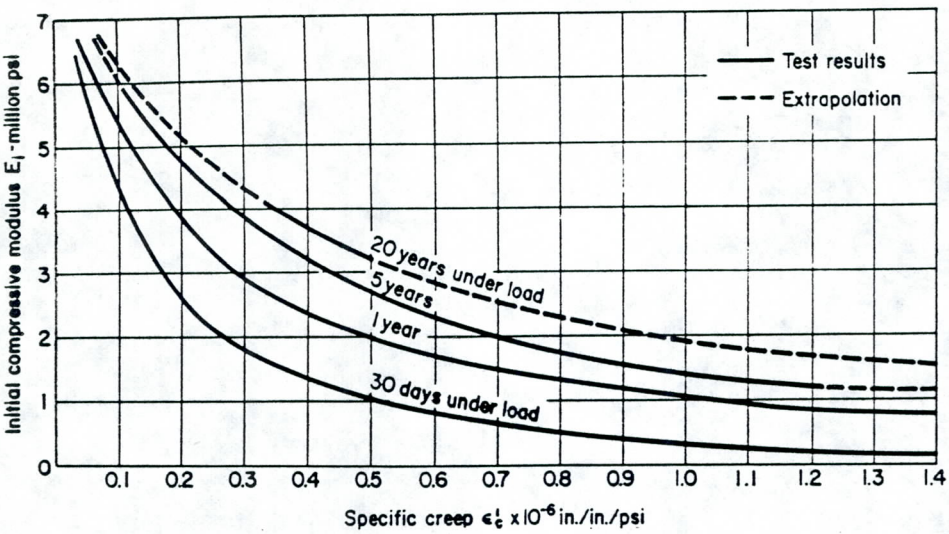


Fig. 5. Prediction of basic creep from elastic modulus

acts prestress creep because dead load causes tension in the precompressed bottom fibers. The amount of creep bending that occurs depends on the residual compressive stress gradient under dead load. For this reason, more creep bending is likely to occur on shorter span bridges where a larger percentage of the precompression remains in the bottom of the beam under dead load.

The effect of shrinkage. Shrinkage of the deck slab with respect to the girders causes negative restraint moments over piers that reduce the creep restraint moment. As shown in a PCA Development Department Bulletin[5], the restraint moments at the piers due to shrinkage are calculated as if the differential shrinkage moment were applied to the continuous composite girder along its entire length, assuming the girder to behave elastically. The differential shrinkage moment applied to the girder along its entire length is given by:

$$M_s = \epsilon_s E_b A_b \left(e'_2 + \frac{t}{2}\right)$$

where ϵ_s = differential shrinkage strain (as described below)

E_b = elastic modulus for the deck slab concrete

A_b = cross-sectional area of deck slab

$\left(e'_2 + \frac{t}{2}\right)$ = distance between the mid depth of the slab and centroid of composite section

t = slab thickness

The most reliable means of estimating shrinkage values is from observations on specimens made from the mix to be used in the structure. With such observations, the final values of shrinkage can be projected by assuming the shrinkage-time curve can be represented by the following equations[11]:

$$\epsilon_s = \frac{\epsilon_{su} T}{N_s + T}$$

where ϵ_s = measured shrinkage strain at any time T in days

ϵ_{su} = ultimate shrinkage strain at T = infinity

$N_s = 26.0\, e^{0.36 V/S}$

V/S is the volume/surface ratio of the member under consideration (see Table 1 for V/S ratios of the AASHO-PCI I-beams).

When test data are not available, the ultimate shrinkage under exposure at 50 percent relative humidity can be estimated[12,13] as 0.600×10^{-3}. This value should be corrected for very humid or very dry environments by multiplying by a humidity correction factor as presented in Fig. 10. The differential shrinkage strain between slab and girder for a given time lapse between castings can then be obtained by using the time-shrinkage relationship in Fig. 8. For example, with a 35-day lapse between casting of the girder and the slab, Fig. 8 indicates that about 0.42 of the girder shrinkage has taken place. Thus a slab exposed to 50 percent relative humidity would have $0.42 \times 0.600 \times 10^{-3}$ in./in. or 0.252×10^{-3} in./in. more shrinkage remaining than the girder. The value of 0.252×10^{-3} would be used for ϵ_s in the above equation for M_s. This value checks very closely the value of 0.245×10^{-3} in./in. obtained in the PCA research[5] using 10 ft. long free shrinkage specimens of the slab and girder and a time lag between castings of 35 days.

After calculating the negative elastic moments at the piers due to shrinkage, they must be modified for

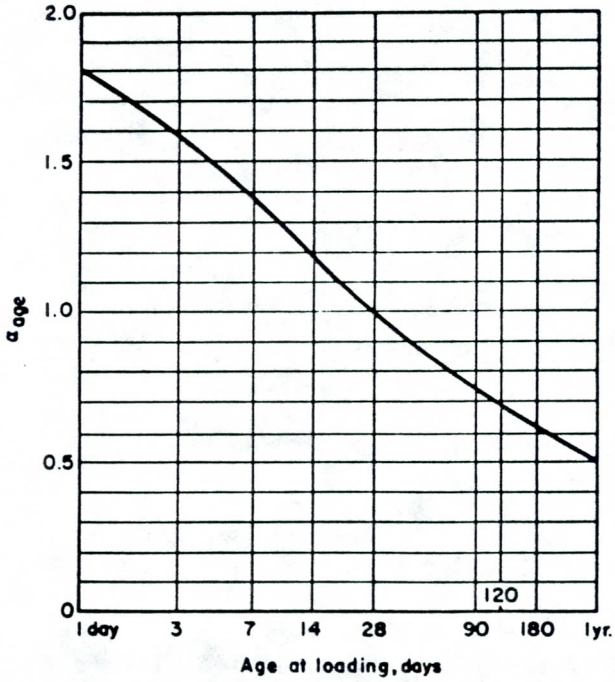


Fig. 6. Creep vs. age at loading

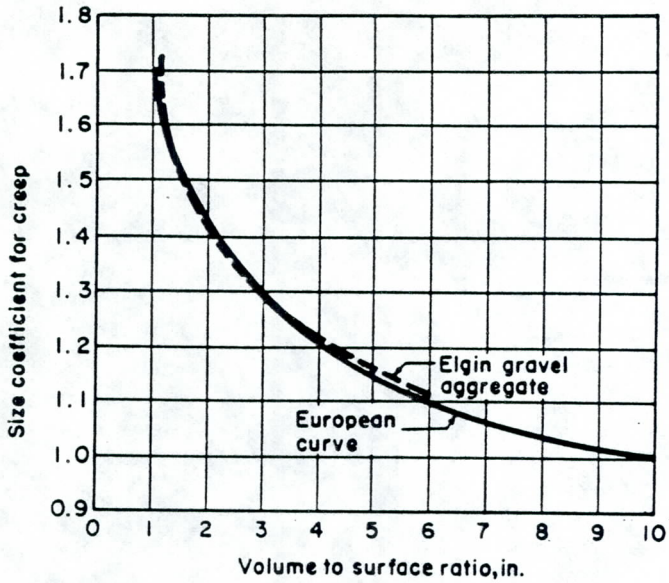


Fig. 7. Creep vs. volume-to-surface ratio

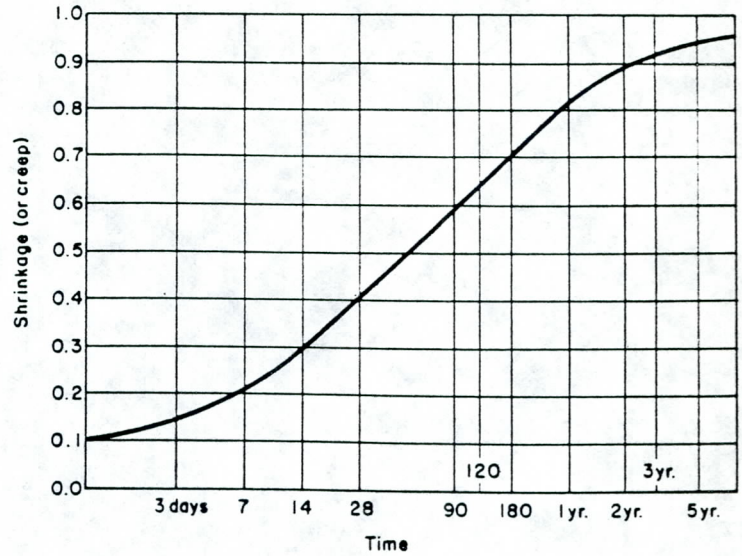


Fig. 8. Proportion of final shrinkage or creep vs. time

the effect of creep by multiplying by the factor $(1-e^{-\phi})/\phi$. A graph of this factor with respect to ϕ is presented in Fig. 9.

Summation of restraint moments. The final positive restraint moment over piers is calculated as:

$$M_r = (Y_c - Y_{DL})(1 - e^{-\phi}) - Y_s\left(\frac{1-e^{-\phi}}{\phi}\right) + Y_{LL}$$

where M_r = final restraint moment

Y_c = restraint moment at a pier due to creep under prestress force

Y_{DL} = restraint moment at a pier due to creep under dead load

Y_s = restraint moment at a pier due to differential shrinkage between the slab and girder

Y_{LL} = positive live load plus impact moment.

Details of the positive moment connection. In the PCA Development Department Bulletin on this subject[3], various connection details to resist the positive restraint moment were evaluated. The detail considered most practical is shown in Fig. 11.

In the tests run on this connection, the hooked bars were bent essentially at right angles and a relatively short horizontal projection was used to conserve on the width of the diaphragm. Although the connections developed substantial moments under static test loads, under fatigue testing (with a stress range of 20,000 psi) most of the connection bars failed after about 670,000 applications of the load. The bars fractured in a brittle manner at the knee of the hook. To avoid this type of failure, the live load plus impact stress range in the bars *at the point where the bend starts* should be limited to 50 percent of the fatigue strength[14]. A conservative value for this stress range is 12,000 psi[15]. Connections designed in accordance with the recommendations in the following para-

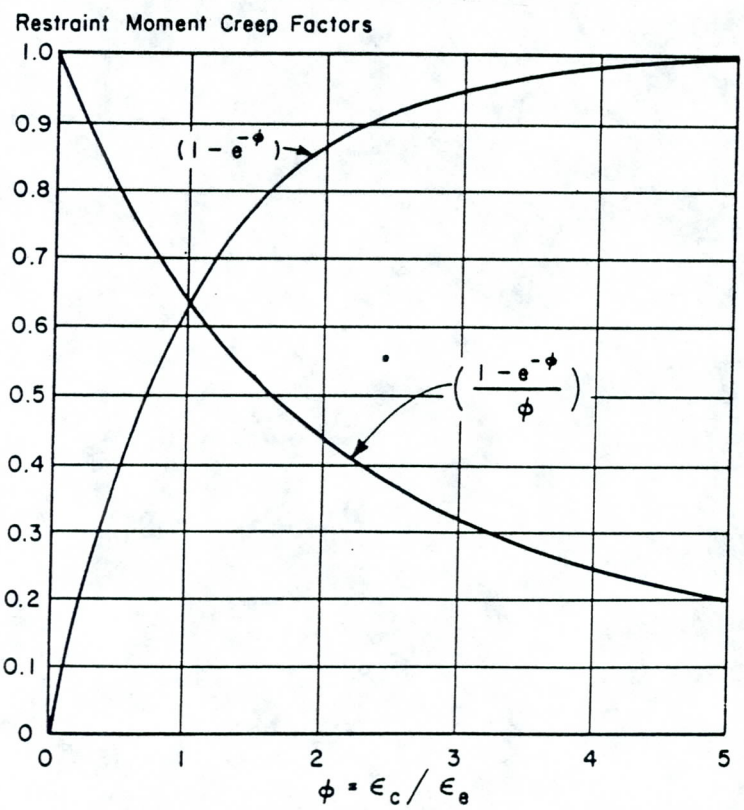


Fig. 9. Variations in restraint moment creep effect factors for both creep and shrinkage

Table 1. Creep data for AASHO-PCI I-beams

AASHO-PCI girder type	Volume/surface ratio in.	Creep volume/surface ratio correction factor (from Fig. 7)
I	3.0	1.28
II	3.4	1.25
III	4.0	1.20
IV	4.7	1.16
V	4.4	1.18
VI	4.4	1.18

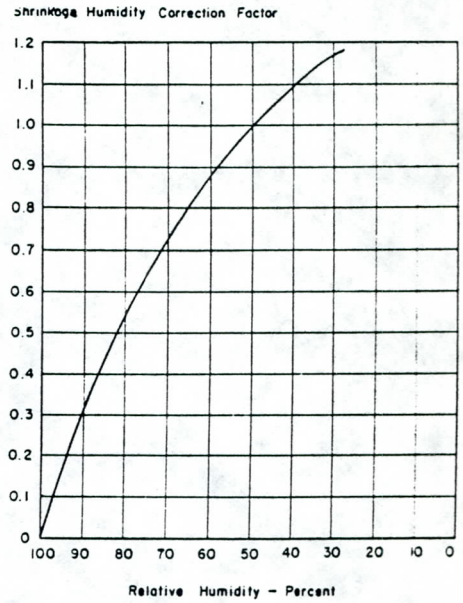


Fig. 10. Shrinkage humidity correction factor

graphs will usually conform to this stress range restriction.

The total length of embedment required for the hooked connection bars can be calculated considering bond to develop uniformly from the face of the beam around the bend to the end of the bar[16]. The unit bond stress should be 0.10 f'_c (maximum 350 psi) as prescribed in the AASHO Specifications[19]. The distance from the inside face of the precast girder to the inside face of the hook should be at least 12 times the bar diameter. The minimum bend radius should conform to Table 2.

Table 2. Minimum bar bend radii

Bar Size	Minimum Radii
#3, #4, #5	2½ bar diameters
#6, #7, #8	3 bar diameters

As determined in the tests at the PCA Laboratories, the restraint moment over the piers does not affect the factor of safety of the structure. For this reason, it was suggested that the positive moment continuity reinforcement be designed to work at 75 percent of its yield point when subject to the limiting value of the restraint moment due to creep and differential shrinkage together with the maximum positive moment due to live load plus impact in remote spans. However, when general research data are used to estimate creep and shrinkage coefficients, it is recommended that the design be based on the lower stress of 0.6 times the yield strength. This lower stress will reduce the live load plus impact stress range, and it will also provide more assurance against the possibility of diaphragm cracking.

Due to the amount of calculation involved in the design of a positive moment connection, as described above, it is suggested that standard details be developed for the various prestressed sections commonly used. A practical consideration in designing the positive moment connection is to detail it to facilitate the removal of the end girder form after prestressing. This may require that the connection bars be cast in the beam straight and then field bent after removal of the end form. However, in some areas, end forms have been developed to accommodate shop bent bars. If field bending is required, No. 6 connection bars should probably be the maximum size used. Consultation with local prestressed concrete fabricators is recommended to develop practical and economical positive moment connection details for use in a particular area. The connection and bearing details for the Big Sandy River Bridges are shown in Fig. 12.

NEGATIVE MOMENT REINFORCEMENT

Since most of the dead load moments are carried by the beam acting as a simple span, the negative design moments over piers are those due to live load plus impact. In some designs, the dead load applied after continuity is achieved may also be considered in the negative design moment. The recommended design procedure for proportioning negative moment reinforcement is to use ultimate strength procedures with a load factor of 2.5 on live load and 1.5 on dead load. The effect of initial precompression due to prestress in the precast girders may be neglected

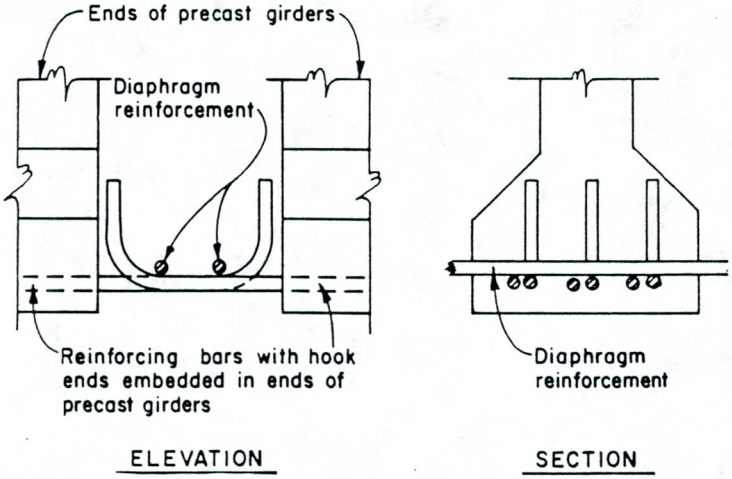


Fig. 11. Positive restraint moment connection details

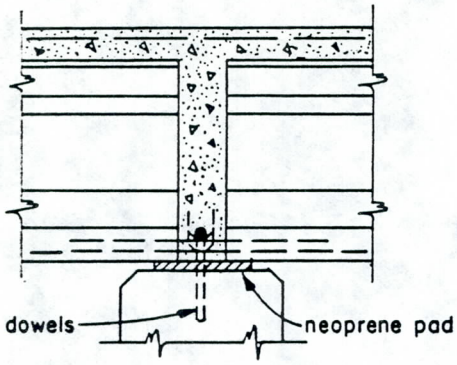


Fig. 12. Bearing and connection details at piers, Big Sandy River Bridges, Tennessee

in the negative moment computation of ultimate strength if the maximum precompression stress is less than 0.4 f'_c and the continuity reinforcement is less than 1.5 percent[1]. A design aid to facilitate determination of the required steel percentage is presented in Fig. 13[17]. Use of this aid is illustrated in the design example.

It will usually be found that the depth of the compression block will be less than the thickness of the bottom flange of the precast girder. For this reason, the negative moment reinforcement required can be determined by assuming the beam to be a rectangular section with a width equal to the bottom flange width of the girder. Due to the lateral restraint of the diaphragm concrete, ultimate negative compression failure in the PCA tests[1] always occurred in the girders, even though the diaphragm concrete strength was about 2000 psi less than that of the girder concrete. For this reason, it is recommended that the negative moment reinforcement be designed using the compressive strength of the girder concrete.

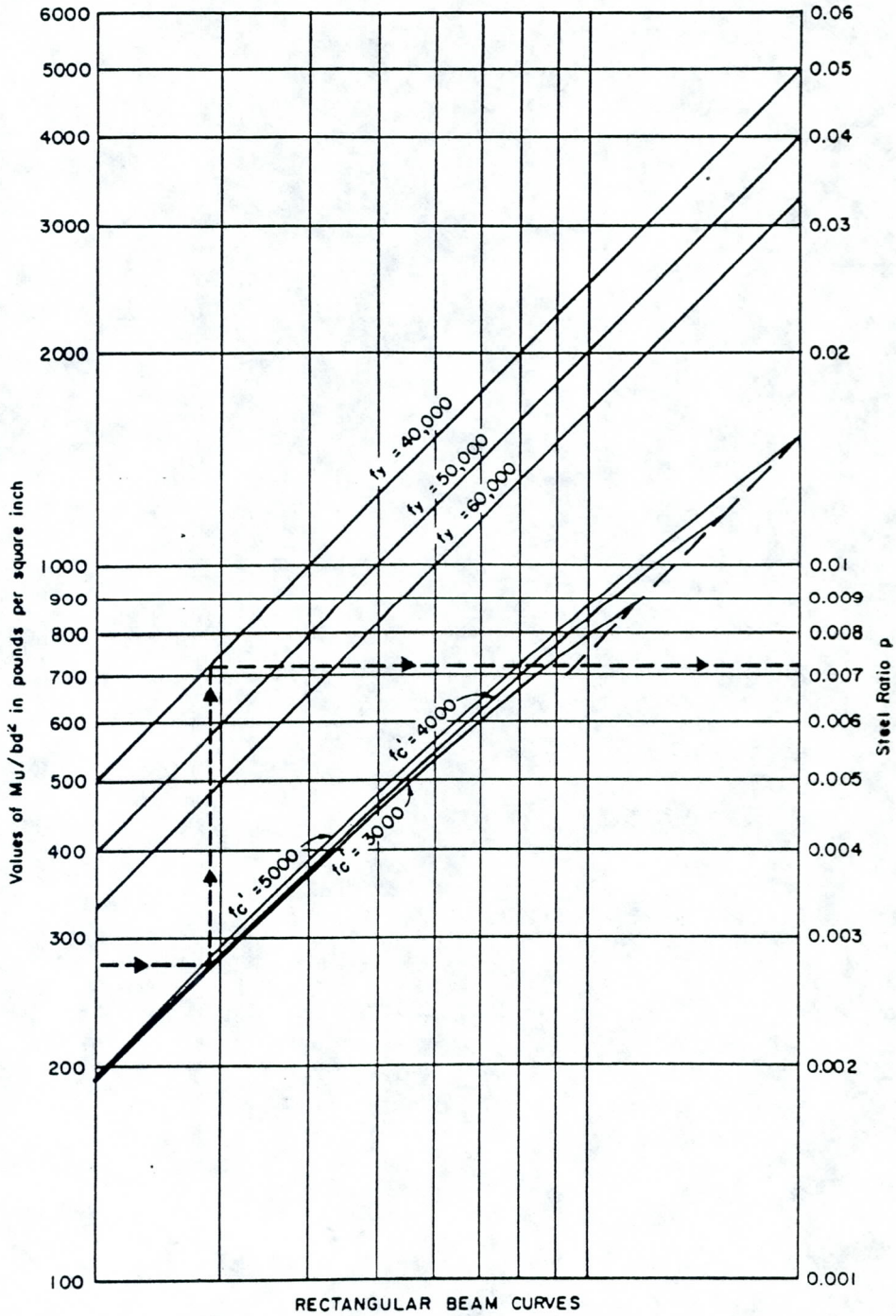


Fig. 13. Design aid for negative moment reinforcement

COMPRESSIVE STRESS IN GIRDERS AT PIERS

A point that concerns some designers is the high compression stresses that might theoretically occur due to addition of stresses due to prestress and negative live load bending over the supports. The PCA studies indicate that these stresses do not affect the ultimate strength for the following reasons:

1. Stress relief is obtained through inelastic strains.
2. Full compression does not occur at beam ends due to the strand transfer length.
3. Diaphragms strengthen the area by providing lateral restraint.

In addition, the effect of the negative live load plus impact moment is reduced by the positive creep restraint moment. However, when considered essential, the theoretical precompression can be reduced by raising the strand center of gravity and/or by blanketing some of the strands at the girder ends. Tests of girders with blanketed prestressing strands have been made at the PCA Laboratories[18]

SHEAR

The shear tests of continuous beams conducted at the PCA Laboratories[4] revealed that the present shear provisions in the American As-

sociation of State Highway Officials (AASHO) Specifications for Highway Bridges[19] provide a conservative estimate of the shear capacity of a continuous beam. However, on continuous beams the formulas must be applied over the full length of the beams rather than only over the middle half of the spans, as is usually the case for simply supported girders.

The horizontal shear stress between the cast-in-place slab and the precast girder was also investigated at the PCA Laboratories[4,5]. These tests found the horizontal shear stresses between slab and beam permitted by the AASHO Specifications to be very conservative.

Part 2. Design Example

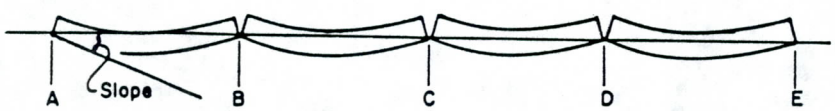
DERIVATION OF GENERAL FORMULAS—FOUR EQUAL CONTINUOUS SPANS

The design example is for a bridge with four equal spans of 130 ft. Prior to the detailed calculations, development of general formulas for restraint moments due to shrinkage and creep is presented. Similar developments for bridges with any number of equal or unequal spans can be made by use of the same procedure or any other consistent procedure for the analysis of indeterminant structures. The conjugate beam theory is used to calculate the various fixed-end restraint moments. The final restraint moments are then obtained by moment distribution.

Shrinkage restraint moments. As previously discussed, differential shrinkage between the deck slab and the prestressed beam causes the following moment on the composite section:

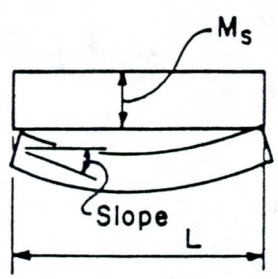
$$M_s = \epsilon_s E_b A_b \left(e'_2 + \frac{t}{2} \right)$$

If the bridge were composed of 4 equal simple spans, this moment would cause bending as shown below:



The slope at the ends of the simple-span beams may be obtained as the reaction of the ML/EI diagram:

$$\text{Slope} = \frac{M_s L}{2\,EI}$$

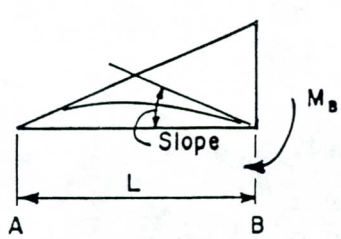


Apply moments at B, C and D to return the beam ends to a horizontal position:

End Span

$$\text{Slope at B} = -\frac{M_B L}{2\,E\,I} \times \frac{2}{3}$$

$$= -\frac{M_B L}{3\,E\,I}$$

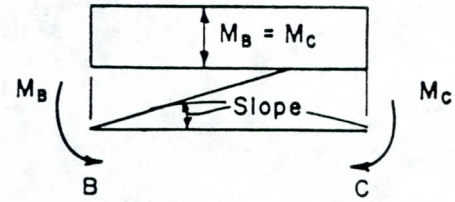


Set slope equal to slope due to shrinkage:

$$-\frac{M_B L}{3EI} = \frac{M_s L}{2EI}$$

$$M_B = -\frac{3}{2} M_s$$

Interior Span



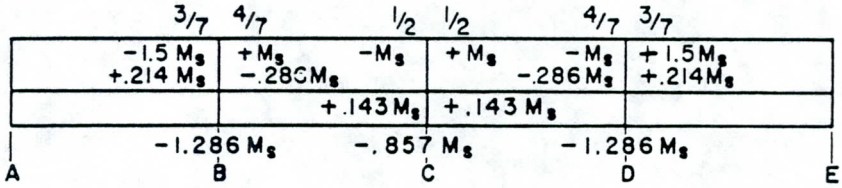
Slope at B and C $= -\dfrac{M_B L}{2EI}$

Set slope equal to slope due to shrinkage:

$$-\frac{M_B L}{2EI} = \frac{M_s L}{2EI}$$

$$M_B = -M_s = M_C$$

Since different moments are required to bring the various beams to a horizontal position, the moments must be distributed to obtain the final shrinkage restraint moments.

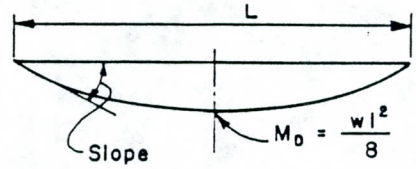


Final shrinkage restraint moments (not reduced by the creep effect factor) are $-1.286\ M_s$ at B and D, and $-0.857\ M_s$ at C.

Creep restraint moments. The creep restraint moment is broken into two components, one due to dead load and the other due to prestress.

Dead Load Creep Restraint Moment

Slope $= \dfrac{M_D L}{3EI}$ ✓

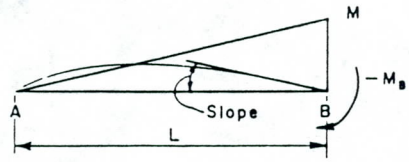


Apply moments to return beam ends to horizontal:

End Span

$$-\frac{M_B L}{3EI} = \frac{M_D L}{3EI}$$

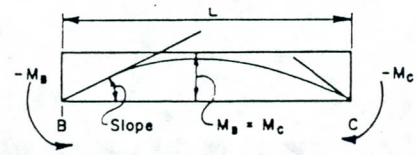
$$M_B = -M_D$$



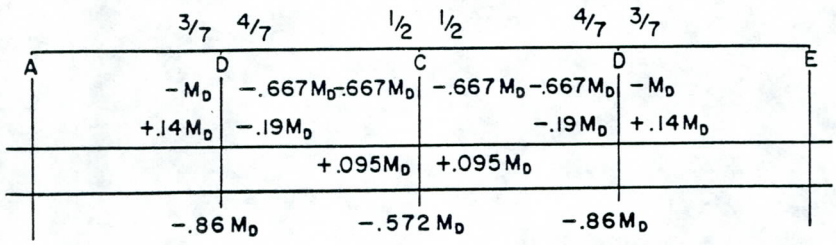
Interior Span

$$-\frac{M_B L}{2EI} = \frac{M_D L}{3EI}$$

$$M_B = 2/3\ M_D = M_C$$



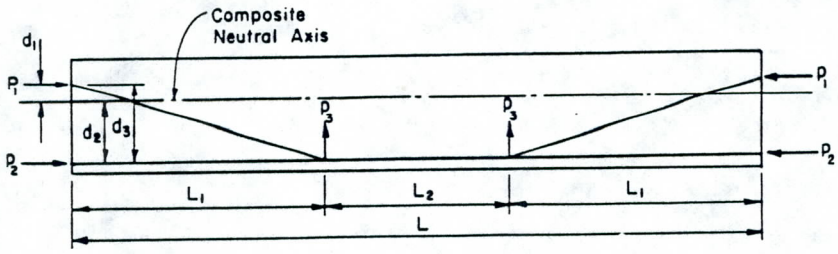
Distribute the unequal end moments due to creep under dead load:



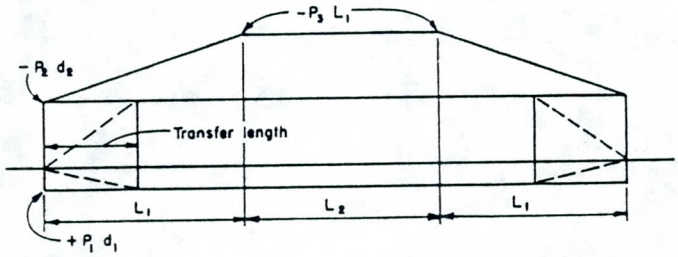
Final dead load creep restraint moments (not reduced by the creep effect factor) are $-0.86\ M_D$ at B and D, and $-0.572\ M_D$ at C.

Prestress Creep Restraint Moment

The creep under the prestress force depends on the location of the various forces with respect to the center of gravity of the composite section. The tendons will be replaced in the calculations by the equivalent forces exerted on the member.

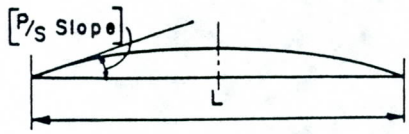


The moment diagram by parts for the above forces is as follows:



Moments might more realistically be considered to vary from zero to full value along the transfer length (dotted lines). This would reduce the creep under the prestress force. Also, any blanketed strand could be disregarded for the length of the blanketing. For convenience, the rectangular moment diagrams will be used in the following.

Slope due to prestress (abbreviated P/S)



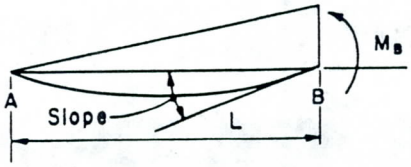
$$[\text{P/S Slope}] = \frac{P_1\, d_1 L}{2\,E\,I} - \frac{P_2\, d_2 L}{2\,E\,I} - \frac{P_3\, L_1^2}{2\,E\,I} - \frac{P_3\, L_1 L_2}{2\,E\,I}$$

Apply moments to bring beam ends at B, C and D to horizontal:

End Span

$$\frac{M_B L}{3\,E\,I} = -\ [\text{P/S Slope}]$$

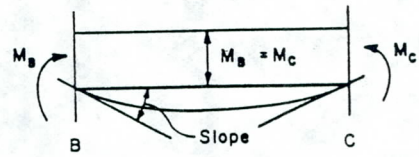
$$M_B = -\frac{3\,E\,I}{L}[\text{P/S Slope}]$$



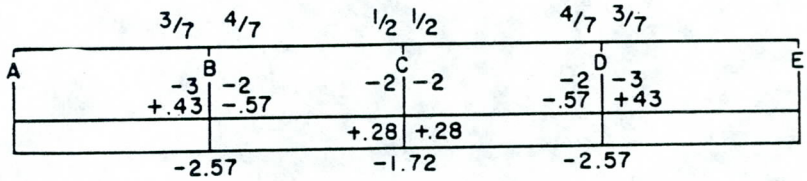
Interior Span

$$\frac{M_B L}{2EI} = -\,[\text{P/S Slope}]$$

$$M_B = M_C = -\frac{2EI}{L}[\text{P/S Slope}]$$



Distribute moments (for convenience distribute −3 and −2):



Final creep restraint moments due to prestress (not reduced by creep effect factor) are $\frac{-2.57\,EI}{L}$ [P/S Slope] at B and D, and $\frac{-1.72\,EI}{L}$[P/S Slope] at C. Recognizing that the value of [P/S Slope] will be negative, the prestress restraint moment will then have a positive value.

Summing up the various restraint moments and multiplying by the appropriate factors, the final restraint moments due to creep and shrinkage can be calculated as:

$$\text{at B and D} = \left(\frac{-2.57\,EI}{L}[\text{P/S Slope}] - 0.86\,M_D\right)(1 - e^{-\phi})$$

$$- (1.286\,M_s)\left(\frac{1 - e^{-\phi}}{\phi}\right)$$

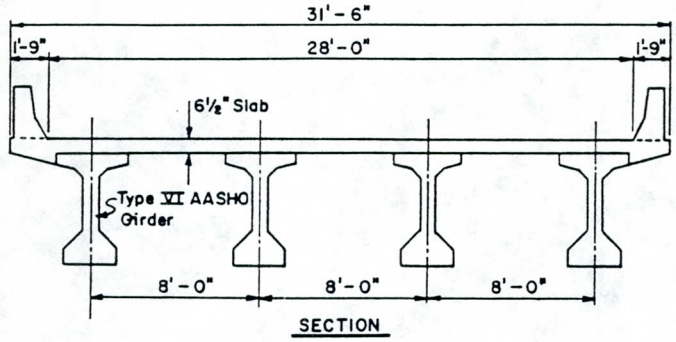
$$\text{at C} = \left(\frac{-1.72\,EI}{L}[\text{P/S Slope}] - 0.572\,M_D\right)(1 - e^{-\phi})$$

$$- (0.857\,M_s)\left(\frac{1 - e^{-\phi}}{\phi}\right)$$

For a given value of ϕ, the values of $(1 - e^{-\phi})$ and $\left(\frac{1 - e^{-\phi}}{\phi}\right)$ may be taken directly from Fig. 9.

SAMPLE CALCULATIONS FOR A BRIDGE WITH FOUR EQUAL 130-FT. SPANS

This design example for the interior girder of a bridge with four equal 130-ft. spans was chosen largely to illustrate that this type of construction is applicable to the entire range of bridge spans for which precast, prestressed girders would usually be considered. More numerous applications will undoubtedly occur for shorter spans. In states which have laws prohibiting highway transportation of the longer and heavier precast girders, the girders may be cast in segments and connected at the bridge site. A comprehensive study of methods of connecting precast segments to achieve long spans has been published by the Prestressed Concrete Institute[20].



SECTION

Design criteria:

	Concrete Strength	E
Prestressed beam at release:	4000 psi	3.66×10^6 psi
Prestressed beam at 28 days:	5000 psi	4.05×10^6 psi
Deck slab and diaphragm:	4500 psi	3.88×10^6 psi

Loading: AASHO HS20-44

Prestressing strand: 1/2" ϕ, 270 k

Area = 0.153 in.^{2}

Final force (after losses) = 23.6 k/strand

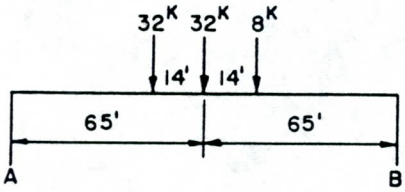
Section properties:

Prestressed beam		Composite girder (interior)	
Area =	1085 in.^{2}	Area =	1676 in.^{2}
$\bar{y}$ =	36.38 in.	STC =	59,977 in.^{3}
I =	733,320 in.^{4}	SBC =	26,266 in.^{3}
ST =	20,587 in.^{3}	STSC =	46,252 in.^{3}
SB =	20,157 in.^{3}	I =	1,313,811 in.^{4}

Design of prestressing reinforcement

Impact = 50/255 = 19.6%

$$LL = \frac{8.0}{5.5 \times 2} = 0.727 \text{ lanes}$$



Moment at 0.5 Span AB:

Using influence lines for 4 equal continuous spans:

$0.1517 \times 32 \times 130 =$ 631 ft. kips
$0.1998 \times 32 \times 130 =$ 830
$0.1449 \times 8 \times 130 =$ 150
Total for 1 lane = 1611 ft. kips

LL + I moment at
0.5 Span AB = $1.196 \times 0.727 \times 1611 = 1403$ ft. kips

Dead load moments at 0.5 Span AB:

Beam = $1085/144 \times 0.150 \times 130^2/8$ = 2390 ft. kips
Slab = $96 \times 6.5/144 \times 0.150 \times 130^2/8$ = 1372
Diaphragm = $3.30 \times 130/3$ = 143
M_D = 3905 ft. kips

Stresses at 0.5 Span AB

	Top	Bottom
Slab	802 psi	- ~~1425~~ psi - 817
Beam	1395	~~817~~ - 1425
Diaphragm	83	- 86
LL + I	282	- 643
	2562 psi	- 2971 psi
Allowable tension	~~212~~	- 212
	~~2774 psi~~	- 2759 psi
	Compression	Tension

Assume eccentricity of strand center of gravity from bottom of beam = 4.70 in. Eccentricity from center of gravity:

$$36.38 - 4.70 = 31.68 \text{ in.}$$

Calculate force required to give a bottom flange stress of 2759 psi:

$$2759 = F/1085 + F \times 31.68/20{,}157$$
$$F \quad = 1{,}110{,}000 \text{ lb.}$$

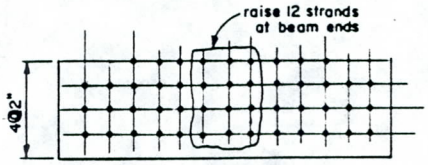
26,260

Strands required = 1,110,000/23,600 = 47.1 strands

Use 48 − 1/2″ ϕ 270 k strands

Check c.g. of strands compared to assumed value:

13 at 2 =	26
13 at 4 =	52
13 at 6 =	78
9 at 8 =	72
48	228



228/48 = 4.75 in.

(close enough to assumed value)

48 strands at 23.6 = 1132 kips
36 strands at 23.6 = 850 kips
12 strands at 23.6 = 283 kips

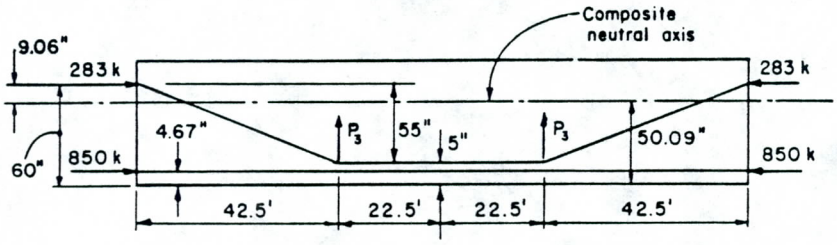
Check eccentricity at girder ends for no tension under working loads:

$$0 = 1132/1085 - 1132e/20{,}587$$
$$e = 19 \text{ in. (17.38 in. from bottom)}$$

Center of gravity of 12 harped strands must be about 55 in. from the bottom to provide this eccentricity. Raise center of gravity of 12 strands 60 in. at ends to reduce compressive stress in bottom fibers.

Design for positive moments at piers

Prestress Creep Restraint Moments (not adjusted by creep effect factor)



$P_3 = 55/42.5 \times 12 \times 283 = 30.5$ kips

$$[\text{P/S Slope}] = \frac{P_1 d_1 L}{2EI} - \frac{P_2 d_2 L}{2EI} - \frac{P_3 L_1^2}{2EI} - \frac{P_3 L_1 L_2}{2EI}$$

$$\frac{P_1 d_1 L}{2EI} = 283 \times 9.91 \times 130 \times 12/2 \quad = +\ 2{,}190{,}000 \text{ in. kips}$$

$$-\frac{P_2 d_2 L}{2EI} = -850 \times 45.42 \times 130 \times 12/2 = -\ 30{,}100{,}000$$

$$-\frac{P_3 L_1^2}{2EI} = -30.5 \times 42.5^2 \times 144/2 \quad = -\ 3{,}960{,}000$$

$$-\frac{P_3 L_1 L_2}{2\,E\,I} = -30.5 \times 42.5 \times 45 \times 144/2 \;= -\;\underline{4{,}200{,}000}$$

$$[\text{P/S Slope}] \qquad = -\frac{36{,}070{,}000}{E\,I}\text{in. kips}$$

$$\text{at B} = \frac{-2.57\,EI}{130 \times 12}\left[-\frac{36{,}070{,}000}{E\,I}\right] = 59.4 \times 10^6 \text{ in. lb.} = 4960 \text{ ft. kips}$$

$$\text{at C} = \frac{-1.72\,EI}{130 \times 12}\left[-\frac{36{,}070{,}000}{E\,I}\right] = 39.8 \times 10^6 \text{ in. lb.} = 3320 \text{ ft. kips}$$

Dead Load Creep Restraint Moments (not adjusted by creep effect factor)

at B = −0.86 × 3905 = −3360 ft. kips

at C = −0.572 × 3905 = −2230 ft. kips

Shrinkage Retraint Moments (not adjusted by creep effect factor)

Assuming continuity connection made 28 days after prestress release, exposure at 50 percent relative humidity, and ultimate shrinkage = 0.0006 in./in.:

ϵ_s = 0.0006 × 0.4 × 1.0 = 0.00024 in./in.
(factors from Figs. 8 and 10)

$$M_s = \epsilon_s \times E_b \times A_b \left(e_2' + \frac{t}{2}\right)$$

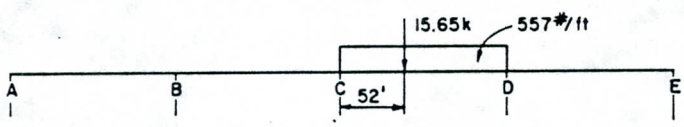
$$M_s = 0.00024 \times 3.88 \times 10^6 \times 96 \times 6.5\,(21.91 + 3.25)$$
$$M_s = 14{,}600 \text{ in. kips} = 1215 \text{ ft. kips}$$

The elastic shrinkage restraint moments are then:

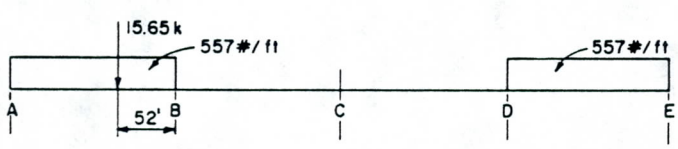
at B = −1.286 × 1215 = −1565 ft. kips

at C = −0.857 × 1215 = −1040 ft. kips

Live Load Positive Moments



Using influence lines for 4 equal continuous spans, the positive LL + I moment at B is calculated to be = 169.5 ft. kips



The LL + I moment for this loading at C = 392 ft. kips

Evaluation of Creep Factors

From Fig. 5, for an elastic modulus at strand release = 3.66×10^6 psi, specific creep at 20 years = 0.41×10^{-6} in./in./psi.

For release of the prestress force at 2 days, the value of specific creep is adjusted by Fig. 6 to:

$1.66 \times 0.41 \times 10^{-6} = 0.68 \times 10^{-6}$ in./in./psi

From Fig. 7 and Table 1, the volume/surface ratio correction revises the specific creep to:

$1.18 \times 0.68 \times 10^{-6} = 0.802 \times 10^{-6}$ in./in./psi

From Fig. 8, at 28 days 0.4 of the creep has occurred, leaving 0.6 to occur or:

$$0.6 \times 0.802 \times 10^{-6} = 0.481 \times 10^{-6} \text{ in./in./psi}$$

The value of $\phi = \epsilon_c/\epsilon_e$ for the structure with a connection made at 28 days would then be:

$$\phi = \epsilon_c/\epsilon_e = \epsilon_c \times E = 0.481 \times 10^{-6} \times 4.05 \times 10^{6} = 1.95$$

From Fig. 9, for $\phi = 1.95$:

$$(1 - e^{-\phi}) = 0.88$$

$$\left(\frac{1 - e^{-\phi}}{\phi}\right) = 0.45$$

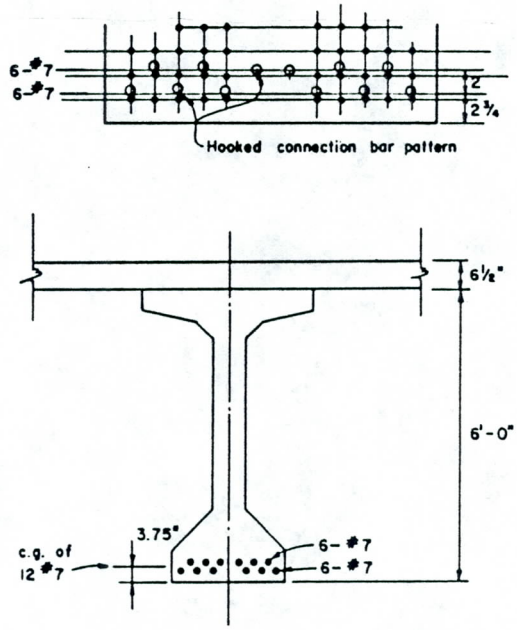
Summation of Positive Restraint Moments at Piers

Shrinkage plus creep:
at B = (4960 − 3360) × 0.88 − 1565 × 0.45 = 705 ft. kips
at C = (3320 − 2230) × 0.88 − 1040 × 0.45 = 491 ft. kips
Add LL + I:
at B = 705 + 169.5 = 874.5 ft. kips
at C = 491 + 392 = 883 ft. kips

Design of the Positive Moment Connection

To design for the 883 ft. kips moment at C, use intermediate grade reinforcement with an allowable stress = $0.6 \times f_y = 0.6 \times 40$ = 24 ksi. Obtain approximate area of steel required using an estimate of jd = 68 in.:

$A_s = M/f_s jd$
A_s = 883,000 × 12/24,000 × 68 = 6.50 sq. in.
Try 12 No. 7 bars, A_s = 7.20 sq. in.



Assume kd = 8.76 in. (from trial computation)

Section	A	y	Ay	Ay²	I_o
Slab, 96 × 6.5 × 0.96	600	− 3.25	−1950	6,330	2530
Top flange, 2.26 × 42	95	− 7.63	− 725	5,530	—
Steel, 7.20 × 7.16	51.6	−74.75	−3860	289,000	—
Totals	746.6		6535	300,860	2530

$$\frac{6535}{746.6} = 8.75 \text{ in. (ok)}$$

$$\begin{aligned} 300{,}860 + 2530 &= \quad 303{,}390 \\ -746.6 \times 8.75^2 &= \quad -57{,}200 \\ \hline I &= \quad 246{,}190 \text{ in.}^4 \end{aligned}$$

Distance from neutral axis to steel = 78.5 − 8.75 − 3.75 = 66.0 in.

$$\text{Steel stress} = \frac{883{,}000 \times 12 \times 66.0}{246{,}190} \times 7.16 = 20{,}300 \text{ psi}$$

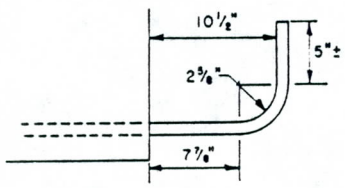
Since the stress is less than 24,000 psi the design is satisfactory. However, in view of the low stress, consideration might be given to reducing the size or number of bars in the connection.

Embedment Length and Bend Details

Allowable bond stress = 0.10 × 4500 = 450 psi, use 350 psi maximum. Perimeter of a No. 7 bar = 2.749 in.

Assume distance from the end face of the precast girder to the inside face of the hook = 12 × bar diameter = 12 × 7/8 = 10.5 in.

Minimum hook radius = 3 × 7/8 = 2⅝ in.



Required embedment length

$$= \frac{20{,}300 \times 0.60}{350 \times 2.749} = 12.2 \text{ in.}$$

1/4 bend = 4.80 in.

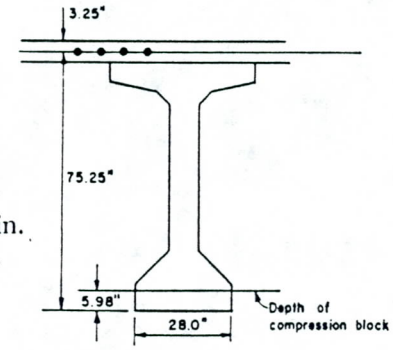
7.875 in. + 4.80 in. = 12.675 in.
12.2 − 12.7 = no extension required
Use 5 in. ± extension—gives 18 in. embedment

To ensure that bars extend into the part of the beam where the prestress force is effective, and to avoid terminating all connection bars at one point, extend 6 bars 3 ft. 0 in. into the beam, and extend the other 6 bars 4 ft. 6 in. into the beam.

Design negative moment reinforcement over piers. The live load plus impact negative moments over piers are calculated as 1499 ft. kips at B, and 1359 ft. kips at C. Design the negative moment reinforcement at B.

Ultimate negative moment = 2.5 × 1499 = 3740 ft. kips

$$\frac{M_u}{bd^2} = \frac{3{,}740{,}000 \times 12}{28 \times 75.25^2} = 283$$



From Fig. 13 (the arrows in Fig. 13 apply to this design example):

$p = 0.0074$

$A_s = 0.0074 \times 28 \times 75.25 = 15.6$ sq. in.
Try 16 No. 9 bars, $A_s = 16.00$ sq. in.

Check ultimate moment provided:

$$a = \frac{A_s f_y}{0.85 f'_c b}$$

$$a = \frac{16 \times 40{,}000}{0.85 \times 4500 \times 28}$$

$a = 5.98$ in.

Note depth of compression block < bottom flange thickness.

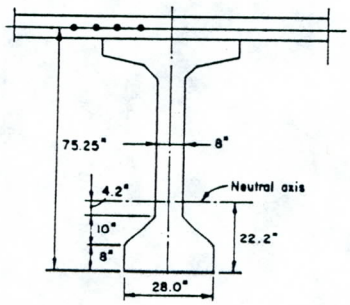
$$M_u = A_s f_y \left(d - \frac{a}{2}\right) = 16.00 \times 40{,}000\ (75.25 - 2.99)$$

$M_u = 46{,}300{,}000$ in. lb. = 3850 ft. kips

M_u provided > required 3740 ft. kips.

Compressive stress in girders at piers. From a trial calculation, assume $kd = 22.2$ in.

$$n = E_s/E_c = \frac{29 \times 10^6}{4.05 \times 10^6} = 7.16$$



Section	A	y	Ay	Ay²	I_o
16 × 7.16 (steel)	114.2	−75.25	−8600	647,000	—
8 × 28	224	− 4	− 896	3,580	1195
10 × 18	180	−11.63	−2095	24,400	1221
8 × 4.2	33.6	−20.1	− 640	12,850	—
Totals	551.8		−12,231	687,830	2416

$$\text{N.A} = \frac{12{,}231}{551.8} = 22.2 \text{ in.}$$

$$\begin{aligned} 687{,}830 + 2416 &= \quad 690{,}246 \\ -551.8 \times 22.2^2 &= \quad -272{,}000 \\ I &= \quad 418{,}246 \text{ in.}^4 \end{aligned}$$

$$f_c\,(\text{LL} + \text{I}) = \frac{1{,}499{,}000 \times 12 \times 22.2}{418{,}246} = 952 \text{ psi}$$

$$f_c\,(\text{prestress}) = \frac{1{,}132{,}000}{1085} + \frac{1{,}132{,}000 \times 18.88 \times 36.38}{733{,}320} = 2105 \text{ psi}$$

$$= 3057 \text{ psi}$$

This stress will not actually occur because of the tendon transfer length. The stress is also reduced by the positive moment induced by creep. The theoretical stress can be reduced to 2000 psi by blanketing 14 bottom strands for a short distance.

Tensile stresses in girders at midspan. In span AB, the positive creep restraint moment at midspan would be $0.5 \times 705 = 352.5$ ft. kips. This moment causes a tension of:

$$f_c = \frac{352{,}500 \times 12}{26{,}266} = 161 \text{ psi}$$

Adding the tension due to gravity loads of about 94 psi, the total tensile stress becomes 255 psi. The permissible value to avoid cracking would be $0.8 \times 7.5\sqrt{5000} = 426$ psi. Since the tensile stress is much less than the allowable, no additional prestressing reinforcement is required.

In span BC, the positive creep restraint moment at midspan is

$\frac{491 + 705}{2} = 598$ ft. kips. This causes a tensile stress of 273 psi.

Since there is no tensile stress in this span under gravity loads, this section is also satisfactory for tensile stresses due to creep.

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PORTLAND CEMENT ASSOCIATION

An organization of cement manufacturers to improve and extend the uses of portland cement and concrete through scientific research, engineering field work, and market development.

5420 Old Orchard Road, Skokie, Illinois 60076

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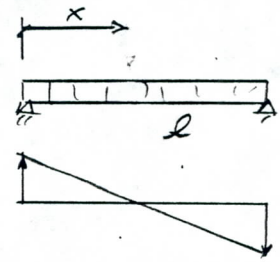
EB014.01E

Simple Beam shear

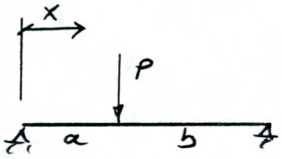
$\ell = 59'-0$

Uniform Load

$$V_x = w\left(\frac{\ell}{2} - x\right)$$

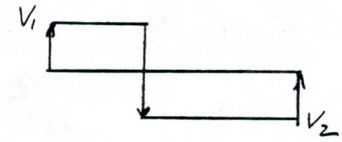


Point Load



$V_1 = Pb/\ell$

$V_2 = Pa/\ell$



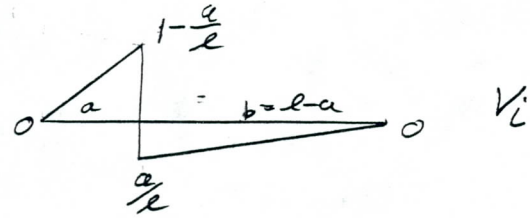
DEAD LOADS

$w_{DL}^{NC} = 1.35$ klf/Girder

$P_{DL}^{NC} = 1.76^k$/DIAP /Girder @ midspan

$w_{DL}^{C} = 0.47$ klf/Girder

Live Load



HS20-44

$I = 1.272$

$DF = 0.712$

SHEAR REINFORCING

[AASHTO 9.20]

$$V_u \le \phi(V_c + V_s) \qquad \therefore V_s = V_u/\phi - V_c$$

$$\phi = 0.90$$

concrete strength

use lesser of:

$$V_{ci} = 0.6\sqrt{f'_c}\, b'd + V_d + \frac{V_i M_{cr}}{M_{max}} \ge 1.7\sqrt{f'_c}\, b'd$$

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3 f_{pc})b'd + V_p$$

$$d \ge 0.8h$$

$$M_{cr} = \frac{I}{y_t}(6\sqrt{f'_c} + f_{pe} - f_d)$$

steel strength

$$V_s = A_v f_{sy} d/s \le 8\sqrt{f'_c}\, b'd$$

$$A_{v_{min}} = 50 b' s / f_{sy} = 50(7)12/40000$$

$$= 0.11 \text{ in}^2/\text{FT}$$

@ support + $h/2$

$$x = \frac{45/2}{12} = 1.875^{ft}$$

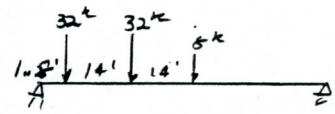
$$V_{DL}^{NC} = 1.35\left(\frac{59}{2} - 1.875\right) + \frac{1.76}{2} = 38.2^{k}$$

$$V_{DL}^{C} = 0.47\left(\frac{59}{2} - 1.875\right) = 13.0^{k}$$

$$\frac{13.0^{k}}{31.2^{k}} = V_d$$ simple beam

$$V_{LL}^{TRUCK} = \frac{1}{59}\left[32(57.125 + 43.125) + 8(29.125)\right]$$

$$= 58.3^{k}$$



V_{LL}^{LANE} = Not critical

$$V_{LL} = 58.3(1.272)\,0.712 = 52.8^{k}/\text{Girder}$$

$$V_u = 1.3\left[V_{DL} + \tfrac{5}{3}V_{LL+I}\right]$$

$$= 1.3\left[51.2 + \tfrac{5}{3}(52.8)\right] = 181^{k}$$

FIND V_c

$$d = 45 + 7.5 - 20.27 + \left[\left(\frac{16.50 - 7.22}{23.5}\right)1.875 + 7.22\right] = 40.19$$

7.96

$< 0.8h$
$= 42^{in}$

$d = 42^{in}$ composite

$P_{ps} = (16 \text{ strand})(0.153 \text{ in}^2/\text{str})\left[138.07 \text{ ksi}\left(\frac{1.875(12)}{50(0.50)}\right)\right]$

6 strands shielded — reduction due to bend

$= 304.2^k$ ✓

$f_{pe} = 304.2(1000)/560 + 304.2(1000)7.96/6186 = 935 \text{ psi}$ ✓ C

$f_d = M_{DL}/S_b$

$M_{DL}^{NC} = 1.35(1.875)/2(59-1.875) + 1.76(1.875)/2 = 74^{'k}$ ✓

$f_d = 74(12000)/6186 = 144 \text{ psi}$ T ✓

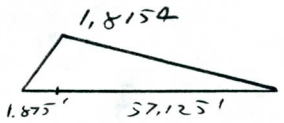
$I/y_T = S_b^c = 10{,}383 \text{ in}^3$ ✓

$M_{cr} = 10383(6\sqrt{5000} + 935 - 144)/12000 = 1052^{'k}$ ✓

$M_{DL}^c = 0.47(1.875)/2(59-1.875) = 25.2^{'k}$ ✓

$M_u = [32(1.8154 + 1.3705) + 8(0.9256)]1.272(0.712) = 99^{'k}$ ✓

$M_u = 1.3[74 + 25.2 + \frac{2}{3}(99)] = 343^{'k}$ ✓



M_i

$M_{MAX} = 343 - 1.3(74) = 247^{'k}$ ✓

$V_i = 1.3(13.0 + \frac{2}{3}(52.8)) = 131.3^k$ ✓

$V_{ci} = 0.6\sqrt{5000}(7)42/1000 + 52.1 + 131.3(1052)/247$

$= 623^k \geq 1.7\sqrt{3000}(7)42/1000 = 35.3^k$ ✓

$V_{ci} = 623^k$ ✓

Find V_{cw}

$$V_{ps} = 304.2\left(\frac{16.50 - 7.22}{23.5(12)}\right) = 10.^{k}$$

$$c = \underset{\text{cgc composite}}{35.05} - \underset{\text{non-comp}}{20.27} = 14.78^{in}$$

$$f_{pc} = P/A - Pec/I^{nc} + M^{nc}c/I$$

$$= 304.2(1000)/560 - 304.2(1000)(7.96)(14.78)/125390 + 74(12000)(14.78)/125390$$

$$= +362 \text{ psi}$$

$$V_{cw} = [3.5\sqrt{5000} + 0.3(362)]\,7(42)/1000 + 10 = 115^{k}$$

$$\therefore V_c = 115^{k}$$

$$V_s = V_u/0.90 - V_c = 181/0.9 - 115 = 86.1^{k}$$

$$8\sqrt{f'_c}\,b'd = 166^{k} > V_s \quad \therefore \text{ok}$$

$$V_s > 4\sqrt{f'_c}\,b'd = 83^{k}$$

$$A_v = 86.1(12)/40(42) = 0.62 \text{ in}^2/\text{ft} > A_{v\,min}$$

use $S_{max} = 12^{in}$

#5 - DBL. Leg stirrups @ 12^{in} $A_v = 0.62$ in²/ft

pg 56 to 57 Not used

Alternate method

AASHTO 1979 interim ✓

$$V_c = 0.06 f'_c b'_j d \leq 180 b'_j d$$

$$j = 1 - 0.6 \omega_{ps}$$

$$\omega_{ps} = \frac{A_{ps}}{bd} \frac{f^*_{su}}{f'_c}$$ ✓

$$f^*_{su} = f_{py}\left(1 - 0.5 \frac{A_{ps}}{bd} \frac{f_{py}}{f'_c}\right)$$

$$A_v = \frac{(V_u - \phi V_c)s}{2 f_{sy} j d} \geq \frac{100 b's}{f_{sy}} = \frac{100(7)12}{40000}$$

$$= 0.21 \text{ in}^2/\text{FT}$$ ✓

For $V_u = 181^k$

$d = 45 + 7.5 - 20.27 + 7.96 = 40.19^{in}$

$\rho_{ps} = \frac{16(0.153)}{80.55(40.19)} = 0.00076$

$f_{su}^* = 270\left(1 - 0.5(0.00076)\frac{270}{5}\right) = 264.5^{ksi}$

$\omega_{ps} = 0.00076(264.5/5) = 0.040$

$j = 1 - 0.6(0.040) = 0.976$

$V_c = 0.06(5)7(0.976)40.19 = 82.4^k$ $\quad 180\,b'jd = 49.1^k$

$V_c = 49.1^k$

$A_v = \frac{(181 - 0.90(49.1))12}{2(40)0.97(40.19)}$

$= 0.53\ in^2/ft < 0.62\ in^2/ft$

$\angle$ pg 55

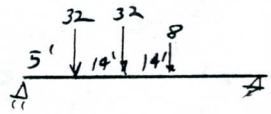
@ $x = 5'$

$V_{DL}^{NC} = 1.35(59/2 - 5) + 1.76/2 = 34^k$ ✓

$V_{DL}^{C} = 0.47(59/2 - 5) = 11.5^k$ simple

$45.5^k = V_d$ ✓

$V_{LL+I}^{TRUCK} = 1/59\,[32(54+40) + 8(26)]\,1.272(0.712)$



$= 49^k/\text{Girder}$ ✓

$V_u = 1.3[45.5 + 5/3\,(49)] = 165^k$ ✓

$V_i = 1.3[11.5 + 5/3\,(49)] = 121^k$ ✓

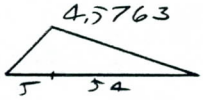
$M_{DL}^{NC} = 1.35(5)/2\,(59-5) + 1.76(5)/2 = 187^{'-k}$

$M_{DL}^{C} = 0.47(5)/2\,(59-5) = 63^{'-k}$

$250^{'-k}$ ✓

$M_{LL+I}^{TRUCK} = 1.272(0.705)[32(4.5763 + 3.3898) + 8(2.2034)]$

$= 244^{'-k}/\text{Girder}$ ✓



M_i

$M_u = 1.3(250 + 5/3\,(294)) = 854^{'-k}$ ✓

$M_{MAX} = 1.3(63 + 5/3\,(294)) = 611^{'-k}$ ✓

$$d = 45 + 7.5 - 20.27 + \left[\underbrace{\left(\frac{16.50 + 7.22}{23.5} \right) 5.00}_{9.19^{in}} + 7.22 \right]$$

$$= 41.42^{in} \quad < 0.8h = 42^{in}$$

$$\therefore d = 42^{in}$$

$$P_{ps} = 22(0.153)138.07^{ksi} = 464.7^{k}$$

$$e = 9.19^{in}$$

$$f_{pe} = 464.7(1000)/560 + 464.7(1000)9.19/6186 = 1520^{psi} \quad C$$

$$f_d = M_{DL}^{NC}/S_b = 187(12000)/6186 = 363^{psi} \quad T$$

$$I/y_T = S_b^c = 10383^{in^3}$$

$$M_{cr} = 10383\left[6\sqrt{5000} + 1520 - 363\right]/12000 = 1368^{in\text{-}k}$$

$$V_{ci} = 0.6\sqrt{5000}(7)42/1000 + 45.5 + 121(1368)/611$$

$$= 329^{k} \geq 1.7\sqrt{5000}(7)42/1000 = 35.3^{k}$$

$$V_{ci} = 329^{k}$$

$$V_p = 464.7\left(\frac{16.50 - 7.22}{23.5(12)}\right) = 15.3^k$$

$$e = 33.05 - 20.27 = 14.78^{in}$$

$$f_{pc} = 464.7(1000)/560 - 464.7(1000)\,9.19(14.78)/125390 + 187(12000)(14.78)/125390$$

$$= 830 - 303 + 265 = 591 \text{ psi}$$

$$V_{cw} = [3.5\sqrt{3000} + 0.3(591)]\,7(42)/1000 + 15.3$$

$$= 140.2^k$$

$$V_c = 140.2^k$$

$$V_s = V_u/\phi - V_c = 165/0.90 - (140.2) = 43.1^k$$

$$A_v = 43.1(12)/40(42) = 0.31 \text{ in}^2/\text{FT} > A_{v\,min}$$

#5 - DBL stirrup @ 12"

@ $x = 10'$

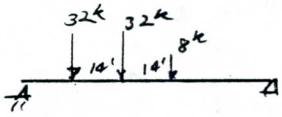
$V_{DL}^{NC} = 1.35\left(\frac{59}{2} - 10\right) + \frac{1.76}{2} = 27.2^k$ ✓

$V_{DL}^{C} = 0.47\left(\frac{59}{2} - 10\right) = 9.2^k$

$36.4^k = V_d$ simple span ✓

$V_{LL+I}^{Truck} = \frac{1}{59}\left[32(49+35)+8(21)\right] 1.272(0.712)$

$= 43.8^k/\text{Girder}$ ✓



$V_u = 1.3\left[36.4 + \frac{5}{3}(43.8)\right] = 142^k$ ✓

$V_i = 1.3\left[9.2 + \frac{5}{3}(43.8)\right] = 107^k$ ✓

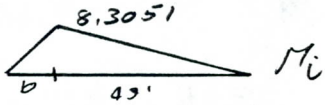
$M_{DL}^{NC} = \frac{1.35(10)}{2}(59-10) + \frac{1.76(10)}{2} = 340'^k$ ✓

$M_{DL}^{C} = \frac{0.47(10)}{2}(59-10) = 115$

$455'^k$ ✓

$M_{LL+I}^{Truck} = 1.272(0.712)\left[32(8.3051 + 5.9322) + 8(3.5593)\right]$

$= 438'^k$ ✓



$M_u = 1.3\left[435 + \frac{5}{3}(438)\right] = 1541'^k$ ✓

$M_{MAX} = 1.3\left[115 + \frac{5}{3}(438)\right] = 1099'^k$ ✓

$$d = 45 + 7.5 - 20.27 + \underbrace{\left[\left(\frac{16.50 - 7.22}{23.5}\right)10.00 + 7.22\right]}_{11.17\,in}$$

$$= 43.4^{in} > 0.8h = 42$$

$$\therefore d = 43.4\,in$$

$$P_{ps} = 464.7^{k} \qquad e_{ps} = 11.17\,in$$

$$f_{pe} = P/A + Pe/S = 464.7(1000)/560 + 464.7(1000)11.17/6186$$

$$= 830 + 839 = 1669\ psi \quad C$$

$$f_d = M_{DL}^{NC}/S = 340(12000)/6186 = 660\ psi \quad T$$

$$I/y_T = 10383\ in^3 \quad \text{composite}$$

$$M_{cr} = 10383\left[6\sqrt{5000} + 1669 - 660\right]/12000 = 1240^{1-k}$$

$$V_{ci} = 0.6\sqrt{5000}(7)43.4/1000 + 36.4 + 107(1240)/1000$$

$$= 170.0^{k} \geq 1.7\sqrt{5000}(7)43.4/1000 = 36.5^{k}$$

$$\therefore V_{ci} = 170.0^{k}$$

$$V_p = 464.7^k \left(\frac{16.50 - 7.22}{23.5(12)} \right) = 15.3^k$$

$$C = 35.05 - 20.27 = 14.78^{in}$$

$$f_{pc} = P/A - P e_{ps} c / I + M_{DL}^{NC} c / I$$

$$I/c = 125390 / 14.78 = 8483.8$$

$$f_{pc} = 464.7(1000)/560 - 464.7(1000)\,11.17/8483.8 + 340(12000)/8483.8$$

$$= 830 - 612 + 481 = +699 \text{ psi}$$

$$V_{cw} = [3.5\sqrt{5000} + 0.3(699)]\,7(43.4)/1000 + 15.3$$

$$= 154.2^k$$

$$\therefore V_c = 154.2^k$$

$$V_s = V_u - \phi V_c = 142/0.9 - 154.2 = 3.6^k$$

$$A_v = 3.6(12)/40(43.4) = 0.03 \text{ in}^2/\text{FT} < A_{v_{min}}$$

Use #5 DBL Leg stirrup @ 12 in

$X = 15^{ET}$

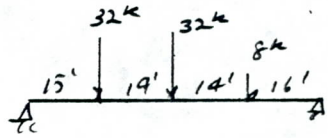
$V_{DL}^{NC} = 1.35(57/2 - 15) + 1.76/2 = 20.5^k$

$V_{DL}^{c} = 0.47(57/2 - 15) = 6.8$

$27.3^k = V_d$

$V_{LL+I}^{TRUCK} = 1/57[32(44+30) + 8(16)]\,1.272\,(0.712)$

$= 38.3^k$



$V_u = 1.3[27.3 + 5/3\,(38.3)] = 118.5^k$

$V_i = 1.3[6.8 + 5/3\,(38.3)] = 92^k$

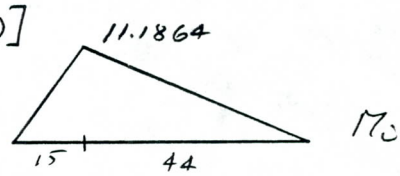
$M_{DL}^{NC} = 1.35(15)/2\,(57-15) + 1.76(15)/2 = 459^{'-k}$

$M_{DL}^{c} = 0.47(15)/2\,(57-15) = 155^{'-k}$ simple

$614^{'-k}$

$M_{LL+I}^{TRUCK} = 1.272(0.712)[32(11.1864 + 7.6271) + 8(4.0678)]$

$= 575^{'-k}$/Girder



$M_u = 1.3[614 + 5/3\,(575)] = 2044^{'-k}$

$M_{max} = 1.3[155 + 5/3\,(575)] = 1447^{'-k}$

$$d = 45 + 7.5 - 20.27 + \left[\underbrace{\left(\frac{16.50 - 7.22}{23.5}\right)15 + 7.22}_{13.14\ in}\right]$$

$$= 45.37\ in > 0.8h = 42$$

$$P_{ps} = 464.7^k \qquad e_{ps}\ 13.14\ in$$

$$f_{pe} = 464.7(1000)/560 + 464.7(1000)13.14/6186 = 1817\ psi\ C$$

$$f_d = 459(12000)/6186 = 890\ psi\ T$$

$$I/y_T = 10383\ in^3 \quad \text{composite}$$

$$M_{cr} = 10383[6\sqrt{5000} + 1817 - 890]/12000 = 1169'^k$$

$$V_{ci} = 0.6\sqrt{5000}(7)45.37/1000 + 27.3 + 92(1169)/7447$$

$$= 115.0^k > 1.7\sqrt{5000}(7)45.37/1000 = 38.2^k$$

$$\therefore V_{ci} = 115.0^k$$

$$V_p = 464.7\left(\frac{16.50 - 7.22}{23.5(12)}\right) = 15.3^k$$

$$c = 35.05 - 20.27 = 14.78^{in}$$

$$f_{pc} = 464.7(1000)/560 - 464.7(1000)\,13.14\,(14.78)/125390 + 459(12000)\,14.78/125390$$

$$= 830 - 720 + 649 = 759 \text{ psi} \quad c$$

$$V_{cw} = [3.5\sqrt{5000} + 0.3(759)]\,7(45.37)/1000 + 15.3$$

$$= 166.2^k$$

$$\therefore V_c = 115.0^k$$

$$V_s = 117.6/0.90 - 115.0 = 16^k$$

$$A_v = 16(12)/40(45.37) = 0.11 \text{ in}^2 = A_{v\,min}$$

use #5 DBL stirrup @ 12"

@ X = 23.50'

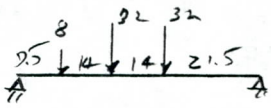
@ HOLD-DOWN

$V_{DL}^{NC} = 1.35(59/2 - 23.50) + 1.76/2 = 9.0^k$ ✓

$V_{DL}^{C} = 0.47(59/2 - 23.50) = 2.8$

$11.8^k = V_d$ ✓

$V_{LL+I}^{TRUCK} = [1/59[32(21.5+35.5)+8(49.5)]-8]\,1.272(0.712)$



$= 26.8^k$ ✓

$V_u = 1.3[11.8 + 5/3(26.8)] = 73^k$ ✓

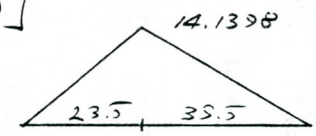
$V_i = 1.3[2.8 + 5/3(26.8)] = 61.7^k$ ✓

$M_{DL}^{NC} = 1.35(23.5)/2\,(59 - 23.5) + 1.76(23.5)/2 = 584\ ^{'-k}$

$M_{DL}^{C} = 0.47(23.5)/2\,(59 - 23.5) = 196$ simple

$780\ ^{'-k}$ ✓

$M_{LL+I}^{TRUCK} = 1.272(0.712)[32(14.1398 + 8.5636) + 8(5.7161)]$



$= 699\ ^{'-k}$ ✓

$M_u = 1.3[780 + 5/3(699)] = 2529\ ^{'-k}$ ✓

$M_{max} = 1.3[196 + 5/3(699)] = 1769\ ^{'-k}$ ✓

$d = 45 + 7.5 - 20.27 + 16.50 = 48.73$ in ✓

$P_{ps} = 464.7^k$ ✓ $e_{ps} = 16.50^{in}$ ✓

$f_{pe} = 464.7(1000)/560 + 464.7(1000)\,16.50/6186$

$= 830 + 1240 = 2070$ psi C ✓

$f_d = 584(12000)/6186 = 1135$ psi T ✓

$I/y = 10383$ in^3 composite ✓

$M_{cr} = 10383\,[6\sqrt{5000} + 2070 - 1135]/12000 = 1176^{'-k}$ ✓

$V_{ci} = 0.6\sqrt{5000}\,(7)\,48.73/1000 + 11.8 + 61.7(1176)/1769$

$= 67.3^k \geq 1.7\sqrt{5000}\,(7)\,48.73/1000 = 41^k$ ✓

$V_{ci} = 67.3^k$ ✓

$V_p = 0$ ✓

$c = 33.05 - 20.27 = 14.78''$ ✓

$f_{pc} = 464.7(1000)/560 - 464.7(1000)(16.50)14.78/125390 + 584(12000)14.78/125390$

$= 830 - 904 + 826 = 752 \text{ psi } C$ ✓

$V_{cw} = [3.5\sqrt{5000} + 0.3(752)]\,7(48.73)/1000 + 0$

$= 161.4^k$ ✓

$\therefore V_c = 67.3^k$ ✓

$V_s = 73/0.90 - 67.3 = 14^k$ ✓

$A_v = 14(12)/40(48.73) = 0.09 \text{ in}^2/\text{FT} < A_{v_{min}}$ ✓

<u>Use #5 DBL stirrups @ 12"</u> ✓

check alternate method

$$\rho_{ps} = \frac{A_{ps}}{bd} = \frac{22(0.153)}{80.55(48.73)} = 0.00086 \checkmark$$

$$f_{su}^* = f_{py}\left(1 - 0.5\rho_{ps}\frac{f_{py}}{f'_c}\right) = 270\left(1 - 0.5(0.00086)\frac{270}{5}\right)$$

$$= 263.7 \text{ ksi}$$

$$\omega_{ps} = \rho_{ps}\frac{f_{su}^*}{f'_c} = 0.00086\,\frac{263.7}{5} = 0.045 \checkmark$$

$$j = 1 - 0.6\,\omega_{ps} = 1 - 0.6(0.045) = 0.973 \checkmark$$

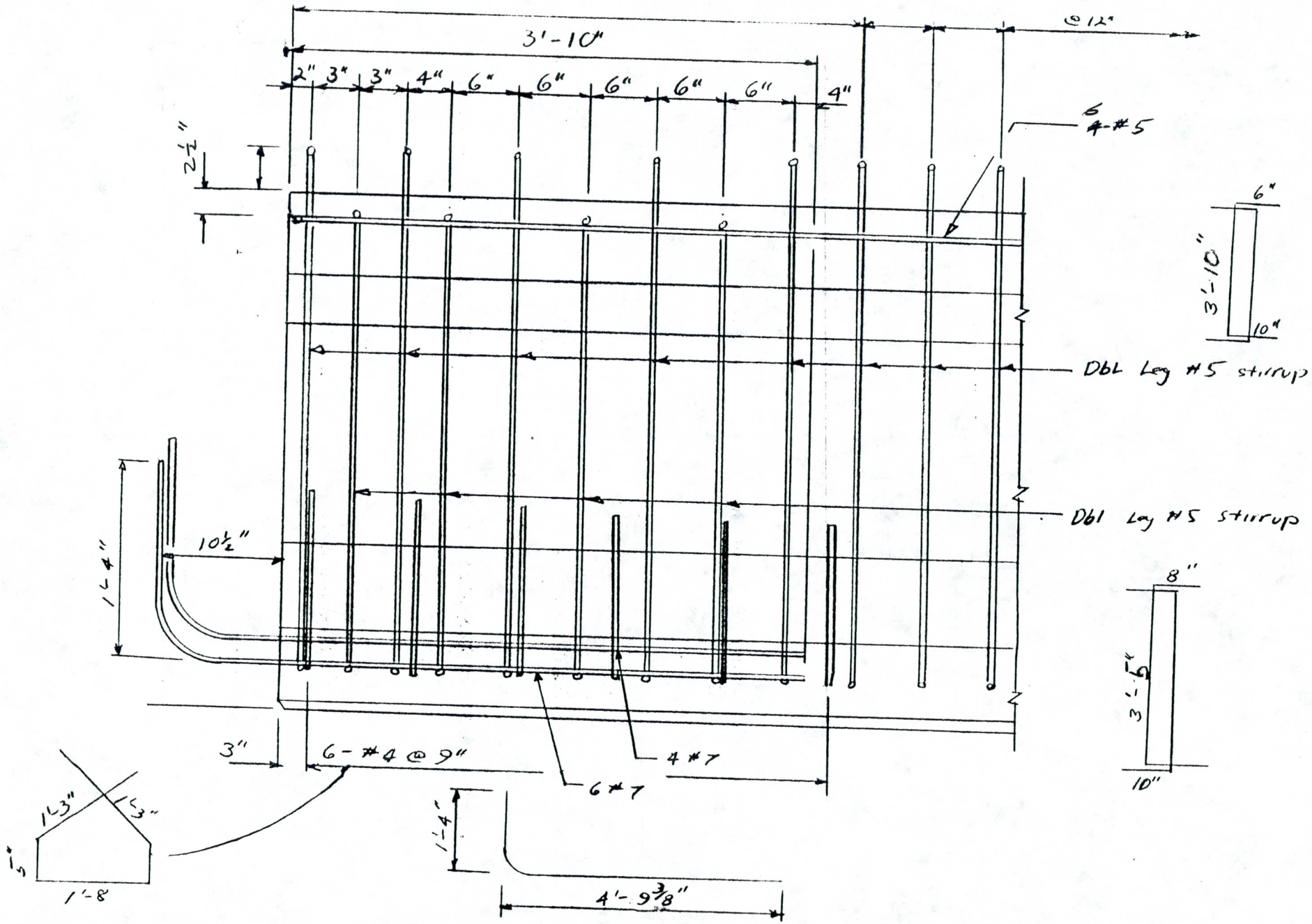
$$V_c = 0.06\,f'_c\,b'_w\,j\,d = 0.06(5)7(0.973)48.73$$

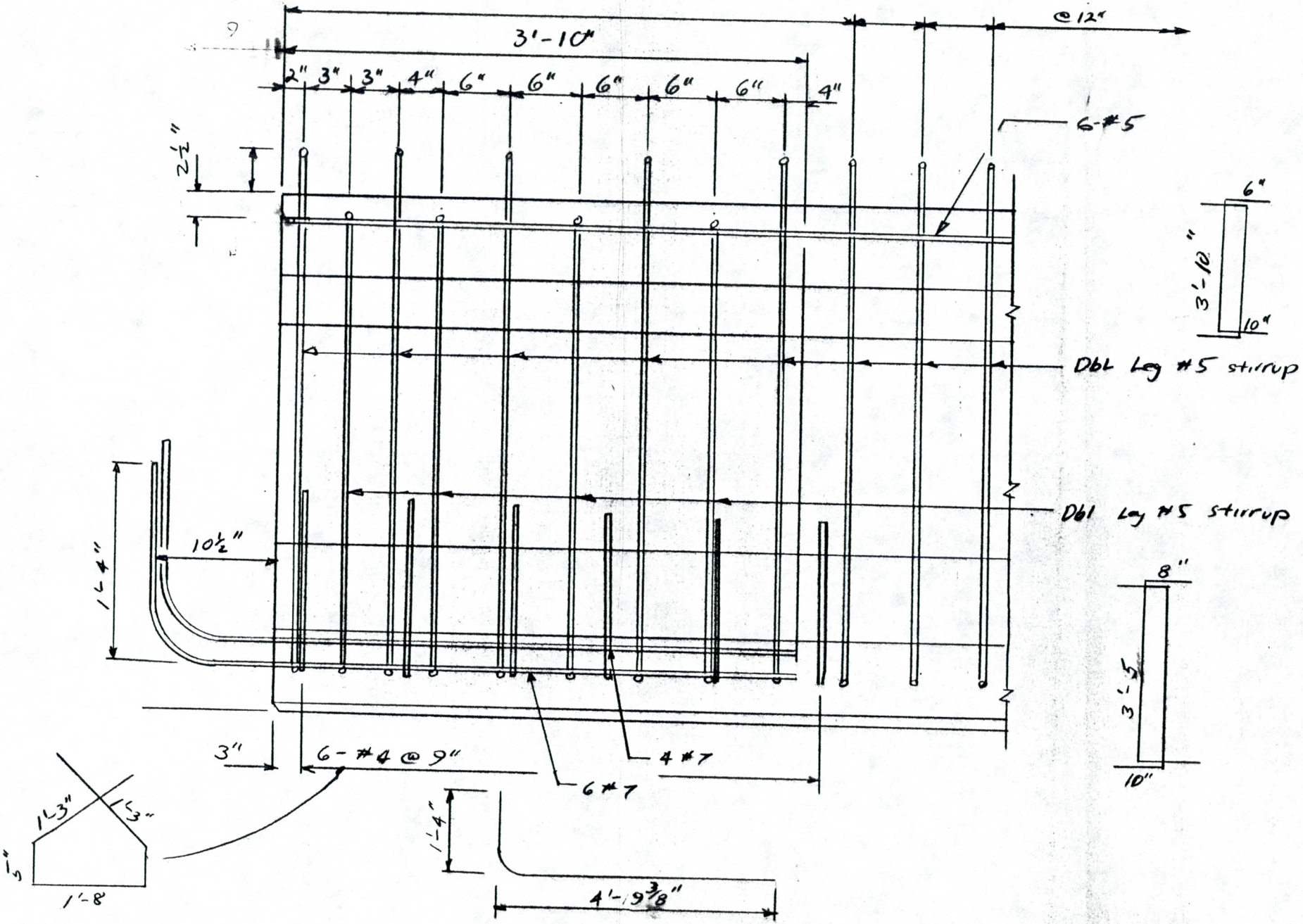
$$= 99.5^k > 180\,b'_w\,j\,d = 59.7^k$$

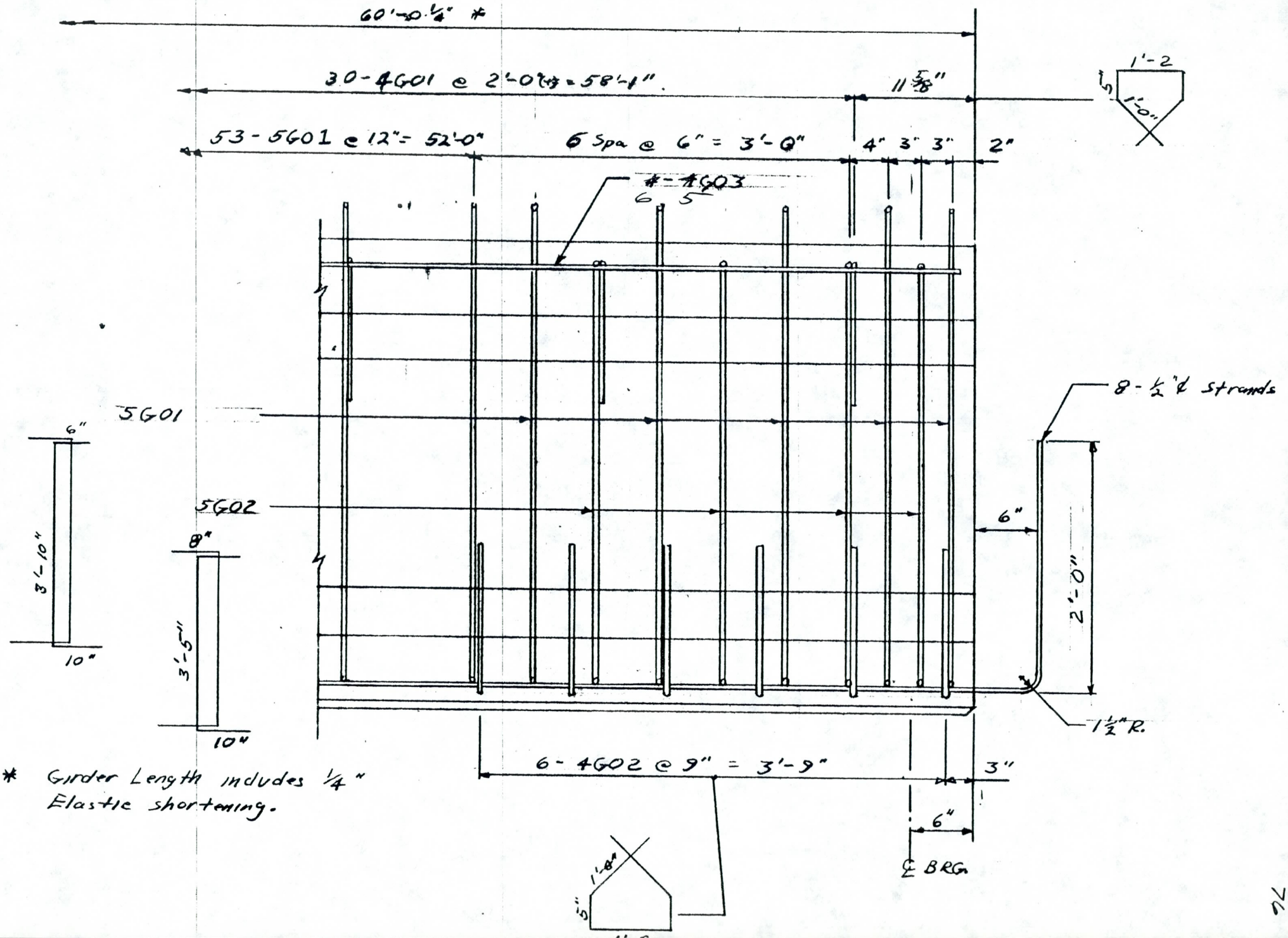
$$V_c = 59.7^k \checkmark$$

$$A_v = \frac{(73 - 0.9(59.7))12}{2(40)0.973(48.73)} = 0.06 \text{ in}^2/\text{FT} \checkmark \quad \& A_{v_{min}} = 0.2$$

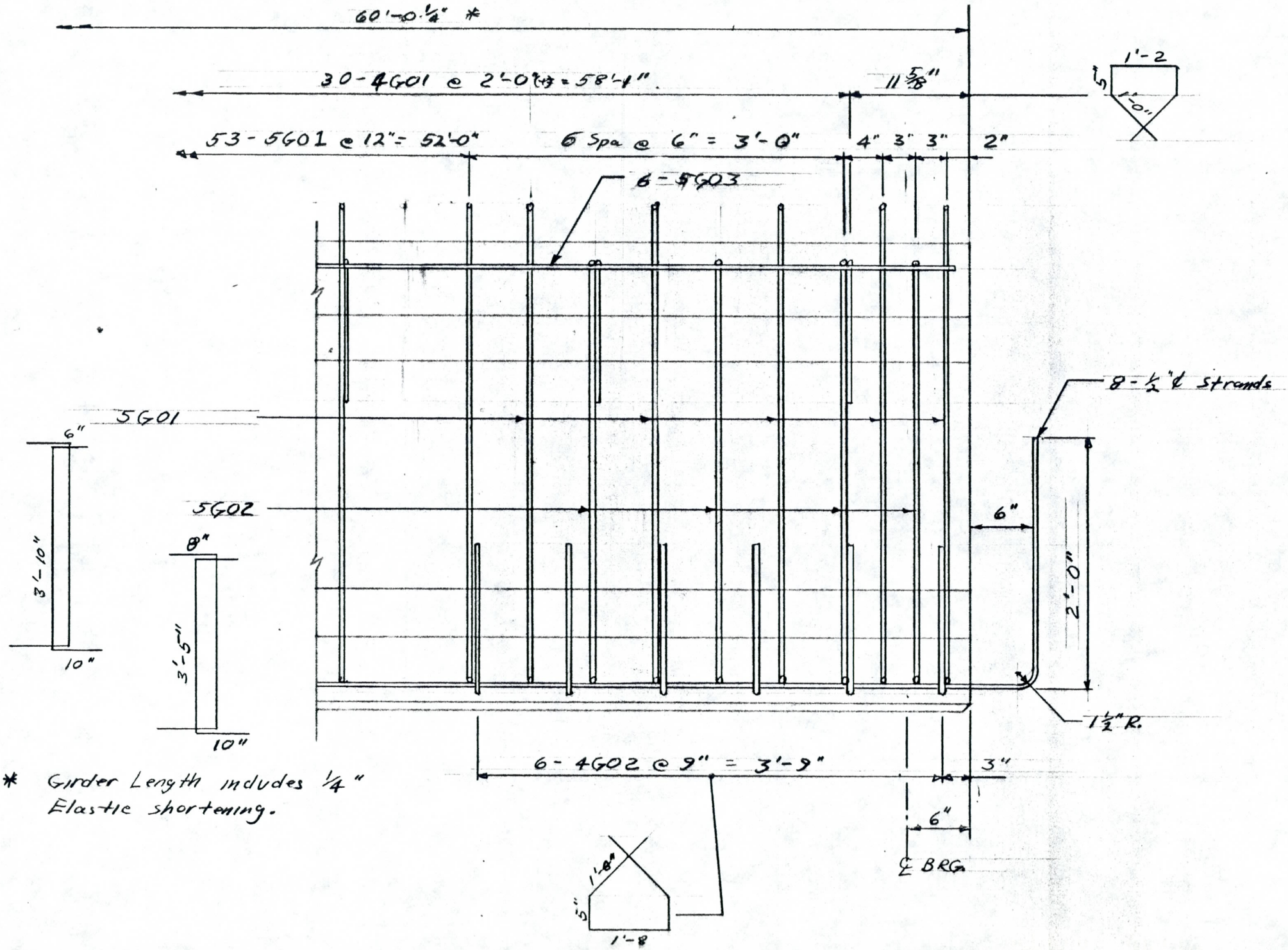
Use #5 @ 12" DBL Leg stirrup ✓


3'-10"
@ 12"
2" 3" 3" 4" 6" 6" 6" 6" 6" 4"
2½"
6 #5
6"
3'-10"
10"
Dbl Leg #5 stirrup
Dbl Leg #5 stirrup
1'-4"
10½"
8"
3'-5"
10"
3"
6 - #4 @ 9"
4 #7
6 #7
1'-3"
1'-3"
1'-8
1'-4"
4'-9⅜"


3'-10"
@12"
2" 3" 3" 4" 6" 6" 6" 6" 6" 4"
2½"
6-#5
6"
3'-10"
10"
Dbl Leg #5 stirrup
Dbl Leg #5 stirrup
1'-4"
10½"
8"
3'-5"
10"
3"
6-#4 @ 9"
4 #7
6 #7
1'-3"
1'-3"
5"
1'-8"
1'-4"
4'-9⅜"



* Girder Length includes 1/4" Elastic shortening.



* Girder Length includes 1/4" Elastic shortening.

PRECAST, PRESTRESSED GIRDER NOTES

PRESTRESSING STEEL SHALL BE 1/2 INCH DIAMETER HIGH-STRENGTH SEVEN-WIRE STRESS-RELIEVED STEEL STRAND, GRADE 270 CONFORMING TO THE REQUIREMENTS OF SECTION 506.3 OF THE CONSTRUCTION SPECIFICATIONS.

STRESS-RELIEVED LOW-RELAXATION SEVEN-WIRE STEEL STRAND, GRADE 270 MAY BE SUBSTITUTED IN LIEU OF THE ABOVE PROVIDED EQUAL PRESTRESS FORCE IS FURNISHED.

THE PRESTRESSING STRANDS SHALL BE TENSIONED TO AN INITIAL FORCE OF 28.92 KIPS PER STRAND AFTER ANCHORAGE SET HAS TAKEN PLACE. A MAXIMUM JACKING FORCE OF 33.05 KIPS PER STRAND SHALL BE ALLOWED PRIOR TO ANCHOR SET. AFTER TRANSFER OF THE PRESTRESSING FORCE TO THE GIRDER AND ELASTIC SHORTENING OF THE GIRDER HAS OCCURRED, A FORCE OF 25.89 KIPS PER STRAND SHALL REMAIN WITH A FINAL FORCE AFTER ALL LOSSES HAVE OCCURRED OF 21.12 KIPS PER STRAND. THE CONTRACTOR SHALL SUBMIT TO THE ENGINEER FOR APPROVAL ALL STRESSING, LOSSES AND ELONGATION CALCULATIONS PRIOR TO COMMENCEMENT OF WORK.

THE PRECAST, PRESTRESSED CONCRETE BRDGE GIRDER SHALL CONFORM TO THE STANDARD TYPE III AASHTO BRIDGE GIRDER WITH THE FOLLOWING PROPERTIES:

CROSS SECTIONAL AREA	A = 560 SQ. IN.
MOMENT OF INERTIA	I = 125,390 IN.^{3}

THE GIRDER CONCRETE SHALL BE CLASS AA HAVING A MINIMUM COMPRESSIVE STRENGTH AT 28 DAYS OF 5,000 POUNDS PER SQUARE INCH. THE CONCRETE SHALL HAVE REACHED A MINIMUM COMPRESSIVE STRENGTH OF 4,000 POUNDS PER SQUARE INCH PRIOR TO TRANSFER OF THE PRESTRESSING FORCE TO THE CONCRETE.

ALL GIRDERS SHALL BE CAST ON CONCRETE BASED PALLETS AND IN APPROVED METAL FORMS.

TRANSFER OF THE PRESTRESSING FORCE TO THE CONCRETE SHALL BE IN SUCH A MANNER AS TO MAINTAIN FORCES SYMMETRICAL ABOUT THE CENTER-LINE OF THE GIRDER. THE CONTRACTOR SHALL SUBMIT TO THE ENGINEER FOR APPROVAL THE METHOD AND SEQUENCE OF TRANSFER PRIOR TO COMMENCEMENT OF WORK.

THE CONTRACTOR SHALL SUBMIT TO THE ENGINEER FOR APPROVAL THE LOCATION AND TYPE OF LIFTING DEVICES AND PROCEDURE USED TO TRANSPORT AND ERECT THE GIRDERS. DURING STORAGE AND TRANS-PORT, THE GIRDER SHALL BE SUPPORTED ONLY AT THE CENTERLINE OF BEARING.

AFTER TRANSFER OF THE PRESTRESSING FORCE TO THE GIRDER, ALL STRANDS NOT EXTENDING INTO THE DIAPHRAGM SHALL BE CUT OFF FLUSH AND THE ENDS COATED WITH AN APPROVED BITUMINUS OR EPOXY RESIN.

THE ENDS OF THE GIRDER SHALL BE CAST SO AS TO BE VERTICAL WHEN ERECTED.

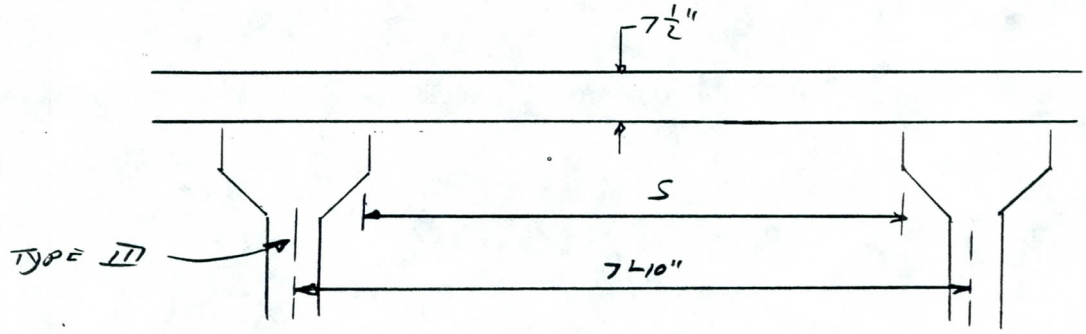
TOPS OF ALL GIRDERS SHALL BE LEFT ROUGH. AT THE TIME OF INITIAL CONCRETE SET, ALL LAITENCE SHALL BE REMOVED WITH A COARSE WIRE BRUSH AND THE TOP OF THE GIRDER ROUGHENED TO AN APPROXIMATELY 1/4 INCH AMPLITUDE.

AT THE CONTRACTOR'S OPTION, ONE HIGH-STRENGTH SEVEN-WIRE STEEL STRAND 3/8 INCH DIAMETER OR LARGER STRESSED TO A TENSION OF APPROXIMATELY 12,000 POUNDS EACH MAY BE USED IN LIEU OF EACH 5G03 BAR.

STIRRUPS 5G01 AND 5G02 SHALL BE PLACED AND TIED TO THE TOP OF THE BOTTOM ROW OF PRE-STRESSED STRANDS. BAR 4G02 SHALL BE PLACED AND TIED TO THE BOTTOM OF THE BOTTOM ROW OF STRANDS AND SHALL ENCLOSE THE STRAIGHT STRANDS.

THE CONTRACTOR SHALL SUMBIT TO THE ENGINEER FOR APPROVAL PRIOR TO COMMENCEMENT OF WORK 2 COPIES OF SHOP DRAWINGS AND CALCULATIONS. THE DRAWINGS AND THE CALCULATIONS SHALL SHOW THE METHOD AND SEQUENCE OF GIRDER CONSTRUC-TION, INCLUDING BUT NOT LIMITED TO FORMING, CASTING, STRESSING, CURING, STORAGE, TRANS-PORTING, AND ERECTION OF THE GIRDER.

Deck slab Design



$S = (7'-10)12 - (1'-4)12 =$ [3.24.1.2]

$= 94 - 16 = 78^{in} = 6.50^{FT}$ ✓

main reinforcing perpendicular to traffic

$M_{LL} = \left(\frac{S+2}{32}\right)P_{20}$ ✓ [AASHTO 3.24.3.1 (EQ 3-15)]

$P_{20} = 16^{k}$ ✓

$M_{LL} = \left(\frac{6.5+2}{32}\right)16 = 4.25^{1-k}/FT \; (0.8) \; 1.30$ ← continuity ✓

$M^{\pm}_{LL+I} = 4.42^{1-k}/FT$ say $4.50^{1-k}/FT$ ✓ ↑ impact LIVE LOAD + IMPACT

DEAD LOAD

SLAB $7\frac{1}{2}/12\,(0.150) = 0.094$ ksf
FWS $= 0.025$ ksf

$M_{DL} = \frac{wl^2}{8} = (0.094+0.025)\,6.5^2/8 = 0.63^{1-k}/FT$ ✓

consv

under median

$w = \frac{9}{12}(0.150) = 0.113$ ksf ✓

$M = 0.113(6.5)^2/8 = 0.60$ 'k ✓

total $M = 0.60 + 0.63 + 4.50 = 5.73$

say 5.75 'k/FT ✓

$A_s = M/ad$

$d = 7\frac{1}{2}'' - 2''clr - \#7/2 = 5.06$ in ✓ #7's

5.12 in #6's

$k = 1/(1 + f_s/nf_c)$

$n = E_s/E_c = 28(10^6)/57000\sqrt{4000} = 7.76$

say 8 ✓

$f'_c = 4000$ psi $f_s = 20$ ksi ✓ #3 to #6

$f_c = 1400$ psi ✓

$k = 1/(1 + 20/8(1.4)) = 0.359$ ✓

$j = 1 - k/3 = 0.880$ ✓

$a = f_s j/12000 = 20000(0.88)/12000 = 1.47$ ✓

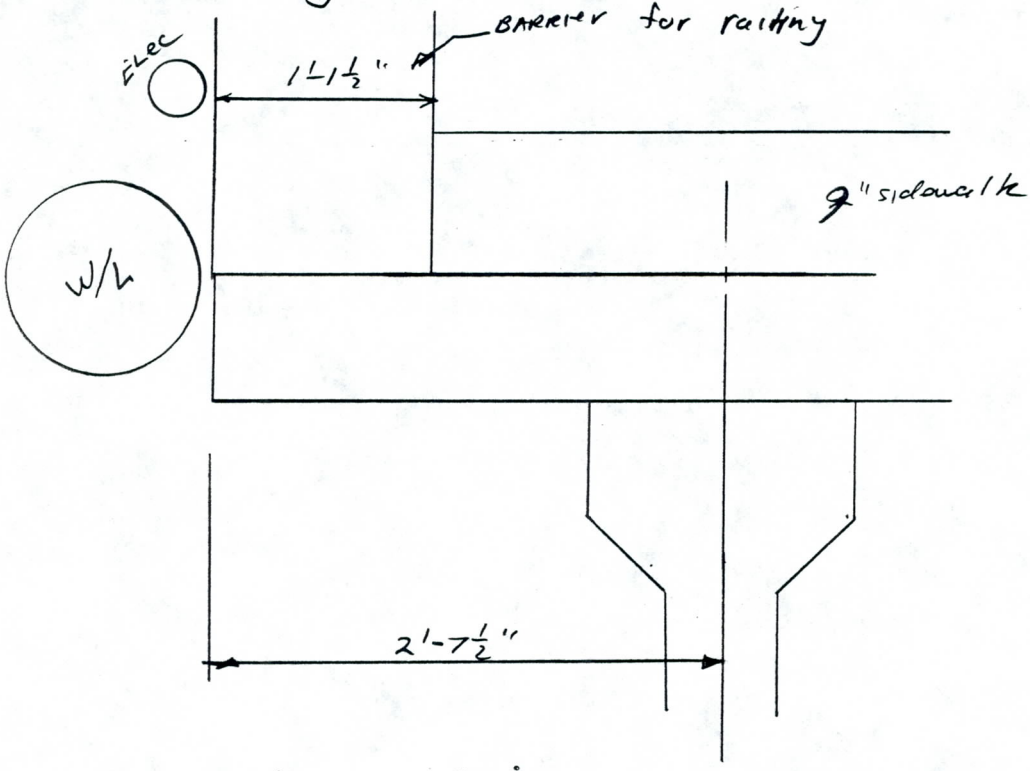
5.12 #6's < 0.77

$A_s = 5.75/1.47(5.06) = 0.77$ in²/FT ✓

Use #6 @ 6" ✓

A = No 3

check over hang



$S = (2'-7\tfrac{1}{2})12 - 8'' = 23.5''$ say $24^{IN} = 2'-0$

SLAB	$\frac{7\frac{1}{2}}{12}(0.150)$	= 0.094 ksf	✓
Barrier		= 0.388 klf	✓
Water Line		= 0.120 klf	✓
ELEC.		= 0.032 klf	✓
SIDEWALK	$\frac{9}{12}(0.150)$	= 0.113 klf	✓

$$M_{DL} = \frac{0.094(2^2)}{2} + \frac{0.113(2-1.125)^2}{2} + 0.388\left(2 - \frac{1.125}{2}\right)$$

$$+ (0.120 + 0.032)2$$

$$= 0.188 + 0.043 + 0.558 + 0.304$$

$$= 1.093^{'-k}/FT \quad < \quad 0.60 + 0.63 = 1.23^{'-k} \checkmark$$

∴ use #6 @ 6" ✓

Distribution REINFORCING [3.24.10]

$220/\sqrt{S} \leq 67\%$ ✓

$220/\sqrt{6.5} = 86.3\%$ => use 67% ✓

$0.77\ in^2/FT\ (0.67) = 0.52\ in^2/FT$ ✓
in middle half

$S/2 = 6.5/2 = 3.25'$

use #5 @ 7" ✓ $A_s = 0.33\ in^2/FT$

FULL width

Development Length [8.25]

$l_d = 0.04 A_b f_y / \sqrt{f'_c} \geq 12^{in}$

#6 $l_d = 0.04(0.44)\,40000 / \sqrt{4000} = 11.1^{in}$ use 12" ✓

#5 $l_d = 0.04(0.44)\,40000 / \sqrt{4000} = 7.8^{in}$ => 12" ✓

splice Length [8.33]

Class C splice $= 1.7\,l_d$

#6 $l_s = 20.4$ use 24^{in} min ✓

#5 $l_s = 20.4$ use 24^{in} min ✓

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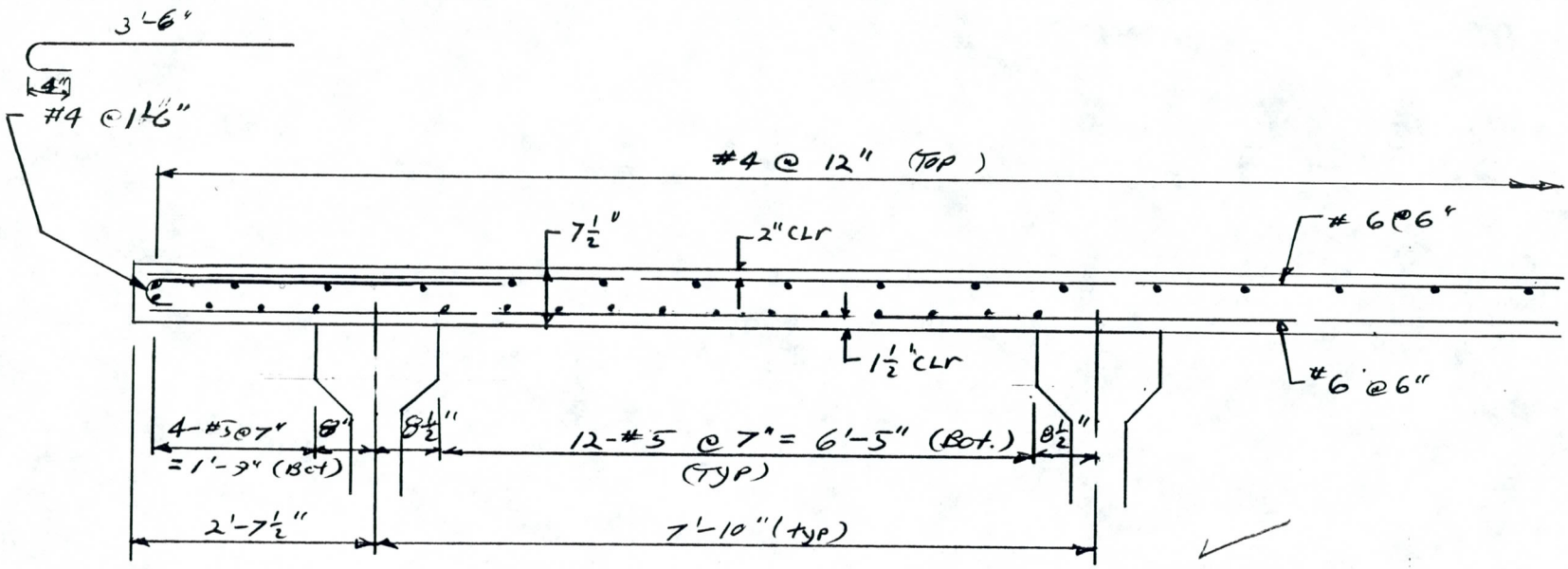
Description

Calculated by RLC Date 26 Nov 84 Sheet No. 84

Checked by G.G.H. Date 7 DEC 84 of



TYPICAL Slab Section
@ MIDSPAN & Abutment

over pier use #8 @ 6" top

DECK PLACING NOTES

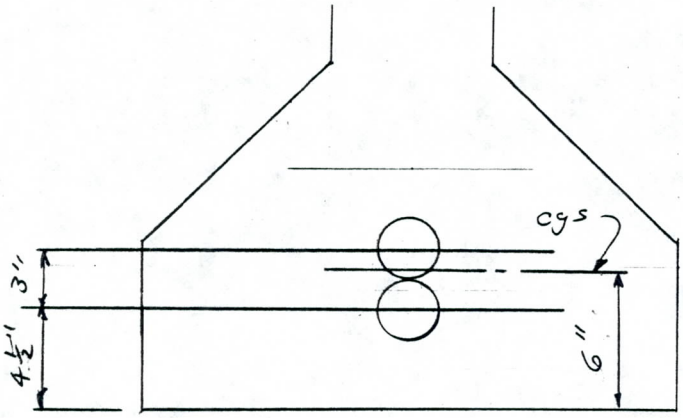
LONGITUDINAL CONSTRUCTION JOINTS SHALL NOT BE PERMITTED OTHER THAN ALONG THE CONSTRUCTION CENTERLINE.

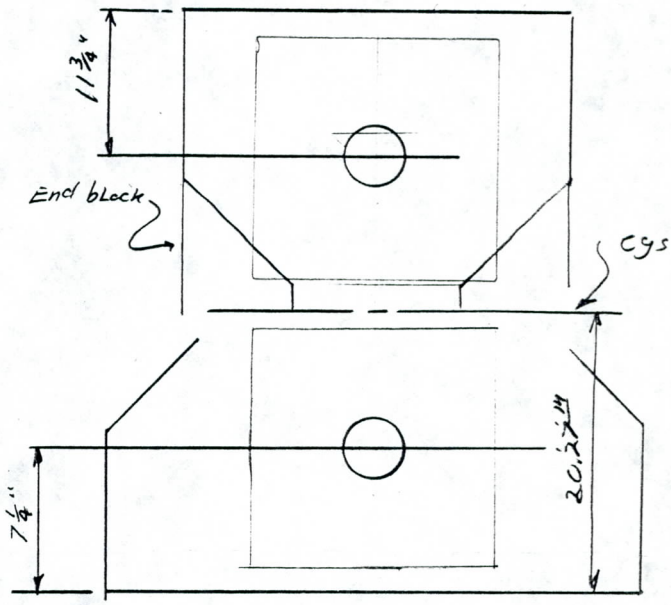
TRANSVERSE CONSTRUCTION JOINTS SHALL BE LOCATED IN ACCORDANCE WITH THE DECK PLACING SEQUENCE. ALL (1) PLACEMENTS SHALL BE MADE BEFORE (2) PLACEMENTS ARE STARTED AND ALL (3) PLACEMENTS MADE BEFORE (4) PLACEMENTS ARE STARTED.

ALL REINFORCEMENTS FOR EACH BRIDGE HALF SHALL BE IN PLACE PRIOR TO BEGINNING DECK CONCRETE PLACING. REINFORCING FROM EAST HALF SHALL EXTEND THROUGH CONSTRUCTION JOINT.

Post-Tensioned Alternative

Assume 2 tendons
11 - $\frac{1}{2}$"ϕ strands each tendon





@ midspan
e_{ps} = 14.27"

@ Ends
e_{ps} = 0.00"

DESIGN MOMENTS @ MIDSPAN
simple beam pg 8

M_{DL}^{NC} = 613 '-k

M_{DL}^{C} = 205 '-k

M_{LL+I} = 708 '-k

} 911 '-k

GIRDER PROPERTIES

NON-COMPOSITE pg. 12

$A = 560 \text{ in}^2$ $y_b = 20.27^{in}$

$S_b = 6186 \text{ in}^3$ $S_t = 5070 \text{ in}^3$

COMPOSITE pg 14

$S_b^c = 10{,}383 \text{ in}^3$ $S_t^c = 36{,}577 \text{ in}^3$

$y_b = 35.05^{in}$

ASSUME

$\frac{1}{2}$"ϕ 270 k STRAND

TOTAL LOSSES = 45 ksi

$F_f = 22 \text{ str.} (0.153 \text{ in}^2/\text{str}) (189 - 45) = 484.7^k$

Check stresses @ MIDSPAN

$$P/A \mp Pe/S \pm M^{DL}/S \pm \frac{M_{DL}^c + M_{LL}^c}{S} = f \qquad e_{ps} = 14.27^{in}$$

TOP	866	−	1364	+	1451	+	299	= +1252 psi	< 2000 psi
BOT	866	+	1118	−	1189	−	1053	= −258 psi	> −424 psi

∴ OK

USE $f'_c = 5000$ psi

$f'_{ci} = 4000$ psi @ TRANSFER

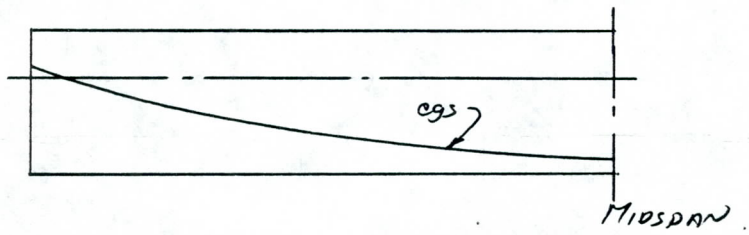
FIND PRESTRESS LOSSES

[AASHTO 9.16]

ASSUME:

½"ϕ 270K STRANDS
TENSIONING FROM BOTH ENDS
GALVANIZED RIGID STEEL DUCTS

FRICTION LOSSES



$K = 0.0002$
$\mu = 0.25$

$\alpha = TAN^{-1}\left(2y/x\right)$

$y = 20.27 - 6 = 14.27$ in

y_{ps} end, y_{ps} ℄

$x = 59/2\,(12) = 354$ in

$\therefore \alpha = TAN^{-1}\left(2(14.27)/354\right) = 0.0806$ RAD

$KL + \mu\alpha = 0.0002\,(59/2) + 0.25\,(0.0806) = 0.026 < 0.3$

$\therefore T_0 = T_x\,(1 + KL + \mu\alpha)$ [AASHTO 9.2]

$\therefore T_x = T_0/(1 + 0.026) = 0.97\,T_0$

ASSUMING INITIAL JACKING STRESS $= 0.8\,f_{pu}$
$= 0.8\,(270) = 216$ ksi

$T_x = 216\,(0.97) = 209.5$ ksi or a LOSS of 6.5 ksi

Assuming maximum stress in after anchor set (before elastic shortening
$= 0.70 f_{pu} = 189$ ksi

FIND ELASTIC SHORTENING LOSS

stress tendons in sequence to 189 ksi

$P_{tendon} = 189\text{ksi}\,(11\text{str})0.153 = 318.1^k$ / TENDON
$\times 2$ tendons
636.2^k @ End

$ES = 0.5 \frac{E_s}{E_{ci}} f_{cir}$

By stressing in sequence one tendon will have losses due to elastic shortening while the 2nd tendon will experience no elastic shortening losses. It is accurate enough to determine the losses assuming all tendons are stressed and using half the value for an average per tendon; hence the 0.5 factor

Assume a 6 ksi ES loss
6.5 ksi friction loss
12.5 ksi

$P_o = (189 - 12.5)\,22\,(0.153) = 594.1^k$ @ midspan

BEAM self-weight = 583 plf

$M_{DL}^{BEAM} = wl^2/8 = 0.583(39^2)/8 = 254'-k$

$f_{cir} = P_o/A + P_o e_{ps}^2/I - M_{DL}^{BEAM} e_{ps}/I$

$I = 125,390 \text{ in}^4$ (p. 100) $A = 560 \text{ in}^2$

$e_{ps} = 20.27 - 6 = 14.27$ in @ midspan

$f_{cir} = 594.1(1000)/560 + 594.1(1000)14.27^2/125390 - 254(12000)14.27/125390$

$= 1061 + 965 - 347 = 1679$ psi

$E_s = 28 \times 10^6$ psi

$E_{ci} = 33 w^{3/2}\sqrt{f'_{ci}} = 33(145^{3/2})\sqrt{4000} = 3.64 \times 10^6$ psi

$\therefore \underline{ES} = 0.5 \frac{28(10^6)}{3.64(10^6)} 1679 = \underline{6458 \text{ psi}}$ ≈ 6000 psi ∴ ok

SHRINKAGE

POST TENS

$SH = 0.8(17000 - 150RH)$

40% RELATIVE HUMIDITY

$= 8800$ psi

Creep of Concrete

$CR_c = 12 f_{cir} - 7 f_{cds}$

$f_{cds} = M_{DL}^{NC} e_{ps} / I + M_{DL}^{C} e_{ps}^{C} / I^{C}$

$M_{DL}^{NC} = 360$ '-k $\quad M_{DL}^{C} = 205$ '-k $\quad$ pg 18

$e_{ps} = 14.27$ in $\quad I = 125390$ in^4

$e_{ps}^{C} = 35.05 - 6 = 29.05$ in $\quad I^{C} = 363,941$ in^4

$f_{cds} = \left[360(14.27)/125390 + 205(29.05)/363941 \right] 12000$

$= 492 + 196 = 688$ psi

$\therefore CR_c = 12(1679) - 7(688) = 15,332$ psi

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Relaxation of Prestressing Steel

$CR_s = 20{,}000 - 0.3FR - 0.4ES - 0.2(SH + CR_c)$

$= 20\,000 - 0.3(6500) - 0.4(6458) - 0.2(8800 + 15332)$

$= \underline{10{,}640 \text{ psi}}$

@ Midspan

FR	=	6 500
ES	=	6 458
SH	=	8 800
CR_c	=	15,332
CR_s	=	10,640
TL	=	47,730 psi

$F_o = 189 - 6.5 - 6.458 = 176.04$ ksi (FR)

$F_f = 189 - 47.73 = 141.27$ ksi (τ_{TL})

$P_o = 592.6^k$

$P_f = 475.5^k$

@ End

ES	=	6458
SH	=	8800
CR_c	=	15 332
CR_s	=	10 640
TL	=	41,230 psi

$F_o = 189 - 6.458 = \cancel{186.54}$ 182.54 ksi

$F_f = 189 - 41.23 = 147.77$ ksi

$P_o = 627.9^k$ (614.4^k)

$P_f = 497.4^k$

Check Anchor Set Losses

$ANC = \Delta_a E_s / L$

ref: "Design of Prestressed Concrete Structure", T.Y. Lin 3rd Ed, p. 104

Assume $\Delta_a = \frac{5}{8}$ in anchorage set

$E_s = 28 \times 10^6$ psi

$L = 59 (12) = 708$ in

$\therefore ANC = \frac{5/8 (28)10^6}{708} = 24,718$ psi

maximum STRAND stress

$= 189 + 24.72 = 213.72$ ksi $< 0.8 f_{pu} = 216$ ksi

$\therefore$ OK

Concrete stresses

@ MIDSPAN

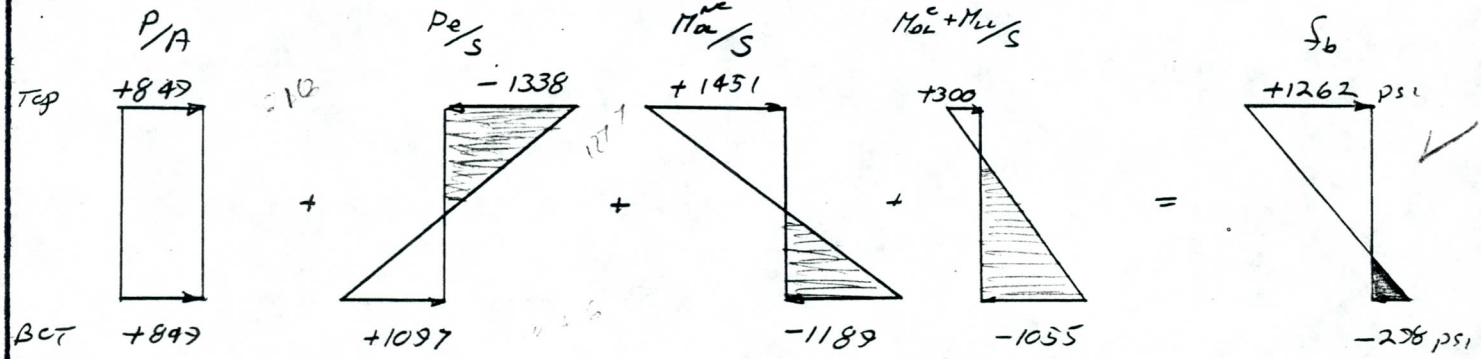
@ service LOAD

$P_f = 475.5^k$ $\qquad e_{ps} = 14.27^{in}$

$M_{DL}^{nc} = 613^{1-k}$

$M_{DL}^{c} = 205^{1-k}$ $\qquad$ pg 8 simple beam

$M_{LL+I} = 708^{1-k}$



$+1262\,psi < 0.4 f'_c = 2000\,psi$

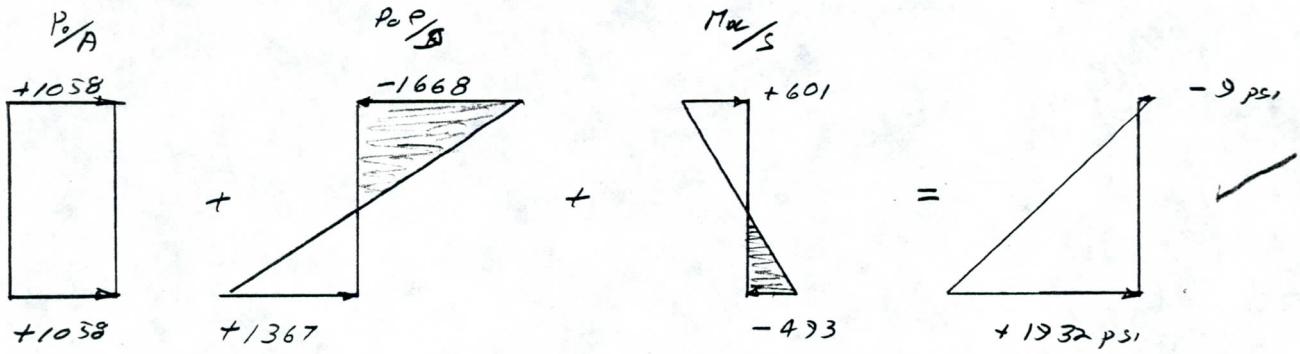
$-298\,psi > -6\sqrt{f'_c} = -424\,psi$ $\quad \therefore$ OK

$\cong -4.2\sqrt{f'_c}$

@ TRANSFER

$P_o = 592.6^k$ ✓ $e_{ps} = 14.27^{in}$ ✓

$M_{DL}^{BEAM} = 254^{'-k}$ ✓



$-9 \cong 0$ psi

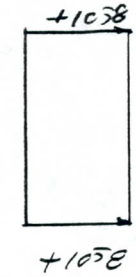
$1932 < 0.6 f'_{ci} = 2400$ psi

∴ Ok ✓

@ End

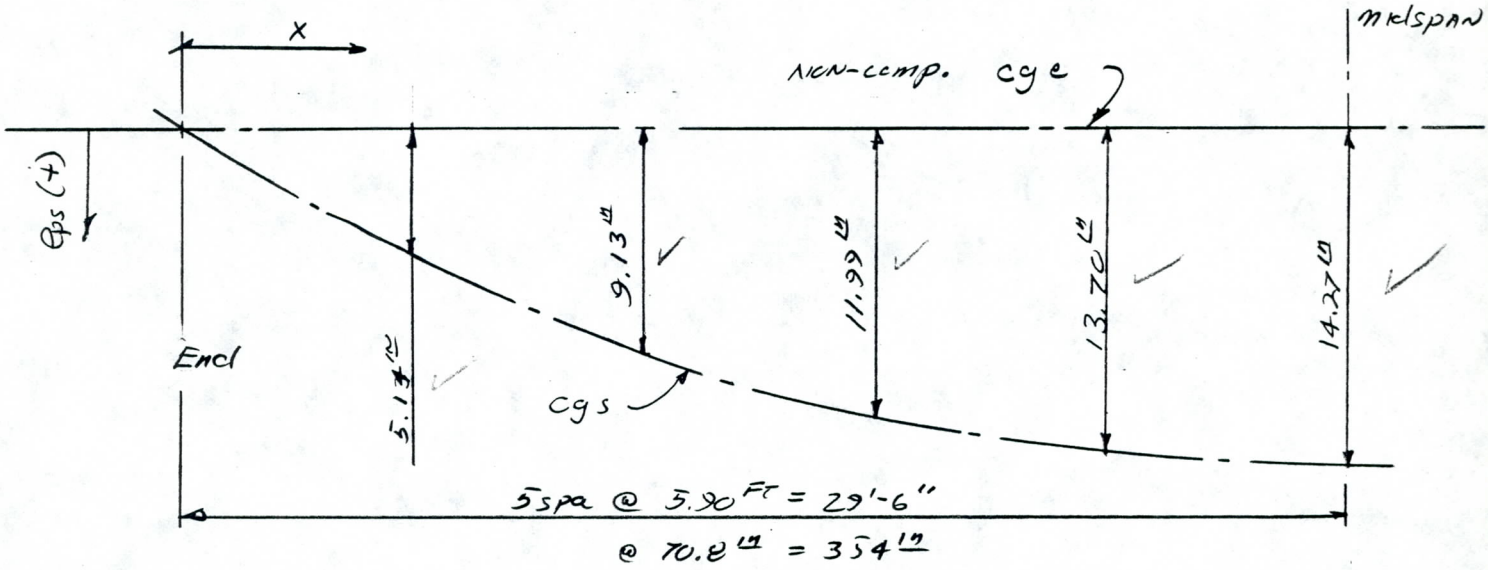
@ transfer

$P_o = 592.6^k$ $e_{ps} = 0^{in}$



✓

Center of Gravity of Prestressing



PARABOLIC CURVE

$$y(x) = ax^2 + bx + c \qquad y'(x) = 2ax + b$$

$$e_{ps}(0) = 0 = 0 + 0 + c \qquad \therefore \; c = 0$$

$$e_{ps}(354) = 14.27 = a(354^2) + b(354)$$

$$e'_{ps}(354) = 0 = 2a(354) + b$$

$$\therefore b = -708a$$

$$14.27 = 125316a + 354(-708a)$$

$$\therefore a = -0.0001139$$
$$b = 0.0806215$$

$$\therefore e_{ps}(x) = -0.0001139x^2 + 0.0806215x \quad \text{(inches)}$$

$$e'_{ps}(x) = -0.0002278x + 0.0806215$$

Check Ultimate strength

AASHTO 9.17

REQUIRED

$$M_u = 1.3\left[M^{DL} + \tfrac{5}{3} M^{LL+I}\right]$$

$$= 1.3\left[613 + 205 + \tfrac{5}{3}(708)\right] = \underline{2600}^{\text{FT-K}}$$

PROVIDED

$$\rho_{ps} = A_{ps}/b'd$$

$A_{ps} = 22(0.153) = 3.366 \text{ in}^2$

$b' = 80.55$ in (Rectangular section)

$d = 45 + 7\tfrac{1}{2} - 6 = 46.50^{\text{in}}$

$$\therefore \rho_{ps} = 3.666/80.55(46.50) = 0.000979$$

$$f_{su}^* = f_{pu}\left(1 - 0.5\frac{\rho_{ps} f_{pu}}{f'_c}\right)$$

$$\rho_{ps}\frac{f_{pu}}{f'_c} = \omega = 0.000979\left(\frac{270}{5}\right) = 0.053 < 0.30$$

maximum steel OK (9.18.1)

$$f_{su}^* = 270(1 - 0.5(0.053)) = 262.9 \text{ ksi}$$

$$1.4\, d\rho_{ps} f_{su}^*/f'_c = 1.4(46.50)(0.053) = 3.45^{\text{in}} < 7.50^{\text{in}}$$

rectangular section

$M_u = A_{ps} f^*_{su} d (1 - 0.6\,\omega)$

$= 3.666(262.9)\,46.50\,(1 - 0.6\,(0.053))/12$

$= \underline{3616}^{FT\text{-}k}$ ✓ $> 2600^{FT\text{-}k}$ $\therefore$ ok

check cracking moment

[AASHTO 9.18.2]

$f_{cr} = P/A + Pe/S_b - M_{cr}/S_b'$

$f_{cr} = -7.5\sqrt{f'_c} = -7.5\sqrt{5000} = -530$ psi ✓

$P = 475.5^k$ ✓

$-530 = 475500/560 + 475500(14.27)/6186 - 12000\,M_{cr}/10383$

$\therefore M_{cr} = [\,849 + 1097 + 330\,]\,10383/12000 = 2142^{1\text{-}k}$

$1.2\,M_{cr} = 2570^{1\text{-}k} < 3616^{1\text{-}k}$ $\therefore$ ok ✓

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Job ____
Description ____
Calculated by ____ Date ____ Sheet No. ____
Checked by ____ Date 2 JAN 85 of ____

x/L	P	e_{ps}	P/A	P_{eps}/S		M_{DL}^{NC}/S		$M_{DL}^{C}+M^{LLZ}/S$		Σf_c	
				TOP	BOT	TOP	BOT	TOP	BOT	TOP	BOT
	(KIPS)	IN	PSI	PSI	PSI	PSI	PSI	PSI	PSI	PSI	PSI
Simple Beam —											
0	497.4	0	888	—	—	—	—	—	—	888	888
0.2	488.6	9.13	873	-880	721	914	-749	205	-722	1112	123
0.4	479.8	13.70	857	-1297	1063	1385	-1135	294	-1036	1239	-251
0.5	475.5	14.27	849	-1338	1097	1451	-1189	300	-1055	1262	-298
2-span continuous											
0	497.4	0	888	—	—	—	—	—	—	888	888
0.2	488.6	9.13	873	-880	721	914	-749	178	-626	1085	219
0.4	479.9	13.70	857	-1297	1063	1385	-1135	231	-812	1176	-27
0.5	475.5	14.27	849	-1338	1097	1451	-1189	222	-784	1184	-27
0.6	479.9	13.70	857	-1297	1063	1385	-1135	195 / -40	-685 / +141	1140 / 905	100 / 926
0.8	488.6	9.13	873	-880	721	914	-749	70 / -78 / 27	-245 / +344	977 / 829 / 810	600 / 1189
1.0	497.4	0	888	—	—	—	—	-217	+763	+671	1651

Camber and Deflection

@ Transfer

DUE to beam self-weight

$$\Delta_{DL}^{BEAM} = 5wl^4 / 384EI \quad @ \text{ midspan}$$

$w = 583 \text{ plf} = 48.58 \text{ pli}$

$E = E_{ci} = 33(145^{3/2})\sqrt{4000} = 3.64 \times 10^6 \text{ psi}$

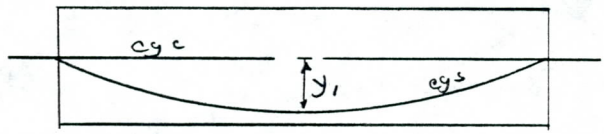
$I = 125{,}390 \text{ in}^4$

$l = 59^{ft} = 708 \text{ in}$

$$\Delta_{DL}^{BEAM} = 5(48.58)\,708^4 / 384(3.64)10^6(125390) = 0.348^{in} \downarrow$$

DUE to post-tensioning

$$\Delta_{ps} = \frac{l^2}{8EI}\left(\frac{5}{6}M_1\right)$$



$M_1 = Fy_1$

$F = 592.6^k \qquad y_1 = e_{ps} = 14.27^{in}$

$$\Delta_{ps} = \frac{708^2}{8(3.64)10^6(125390)}\left[\frac{5}{6}(592.6)10^3(14.27)\right] = 0.967^{in} \uparrow$$

TOTAL CAMBER AT TRANSfer

$$\Delta_t = 0.967\uparrow + 0.348\downarrow = 0.619^{in} \uparrow$$
$$\approx \tfrac{5}{8}^{in}$$

Total camber @ Erection

ref: PCI Design Handbook, pg 3-27

$$\Delta_{er} = 1.85(0.348\downarrow) + 1.80(0.967\uparrow) = 1.097^{in} \uparrow$$
$$\approx 1\tfrac{1}{8}^{in}$$

DUE to composite topping

$$\Delta_{DL}^{c} = 0.435^{in} \downarrow \quad (pg\ 28)$$

$A_b = \frac{F_b}{s}$

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Description

Calculated by RLC Date 10 Dec 84 Sheet No. 1/6

Checked by G.G.H. Date 28 Dec 84 of

End Anchorage

[AASHTO 9.21, 9.26]

Develope 95% of ultimate strength of prestressing steel

ref: "Design of Post-Tensioned Girder Anchorage Zones" W.C. Stone & J.E. Breen, Center for Transportation Research, University of Texas, 1981
"Design of Prestressed Concrete Structures", 3rd ed T.Y. Lin

Find required Bearing Area for Anchor plate

f_{cb} = 3000 psi @ service load [9.15.2.4]
< 0.9(4000) = 3600 OK

(497.4/2)

F = (11 str.) 0.153 in²/str (147.8 ksi) = 248.7 k/Tendon

$A_b = 248.7(1000)/3000 = 82.9 in^2 \Rightarrow$ 9.11 in sq.
10.3 in ϕ

Assume a 10" sq. Plate

- Due to width of required Bearing Plate use End Block Details

Needed per 9.21.1

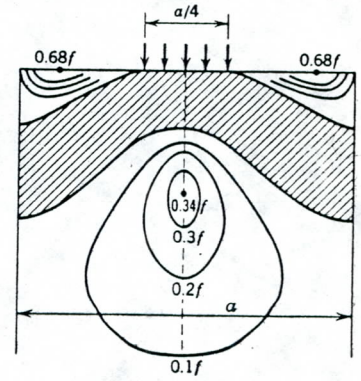
Check transverse tension @ End Block

per T.Y. Lin

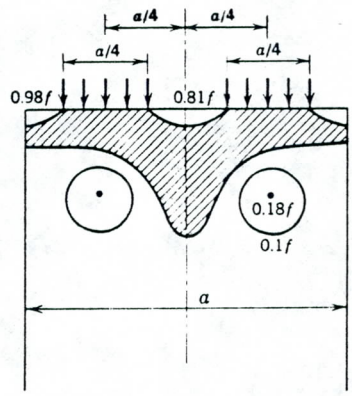
$f = F/A$

$F = 22\,str\,(0.153\,in^2/str)\,270\,(0.95)$

$= 863.4^k$ ✓



(c) One Distributed Load



(b) Two Distributed Loads

pg 208

IF $a = 45^{in}$ ✓

$a/4 = 11\frac{1}{4}'' \cong 10''$ ok

$A = 560\,in^2 + 2\left[4.5(18) + \frac{1}{2}(4.5^2)\right]$ X

$560 + 2\left[4.5(18) + 2\left(\frac{1}{2}\right)(4.5^2)\right]$

$= \cancel{751}\,in^2$ 771.5

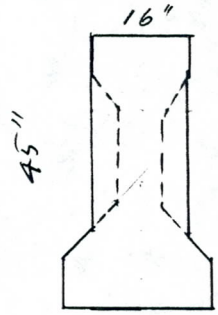
$f = 863.4(10^3)/\cancel{751} = \cancel{1150}$ psi 1119 psi

from figure (c)

IF $a = 16^{in}$, $a/4 = 4^{in}$

$0.34f = 0.34(\cancel{1150}) = 391$ psi 1119 380

maximum tensile stress



Assume tensile stress is distributed over a 2 in Length ($\approx 16/8$)

$F = \cancel{391}\ 380$ psi (16 in width) 2 in $= \cancel{12,512}\ 12,160$ lb

for $f_s = 20$ ksi

$A_s = \cancel{12.51}\ 12.16 / 2c = \cancel{0.63}\ 0.61$ in^2 in 2 in

or $\cancel{3.75}\ 3.66$ in^2/FT

TRY ½"ϕ x 12"ϕ spiral @ 2 in pitch x 3'-0"
and #5 @ 4 in
($A_s = 4.26$ in^2/FT)

per W. C. Stone
pg 105 TO 115

Find Cracking Load

$$P_{cr} = t\left[\frac{f_{sp}}{24}(38a-120) - \frac{t}{81}\left(26 - 252\left(\frac{e}{a}\right)f_{sp}\right) - \frac{103}{9}\left(\frac{e}{a}\right) - 7\right] + 39a' + \frac{1}{5}f_{sp}\left[166 - 975\left(\frac{a'}{t}\right)^2\right] - 9.1$$

e = tendon eccentricity = $\pm$ 13 in

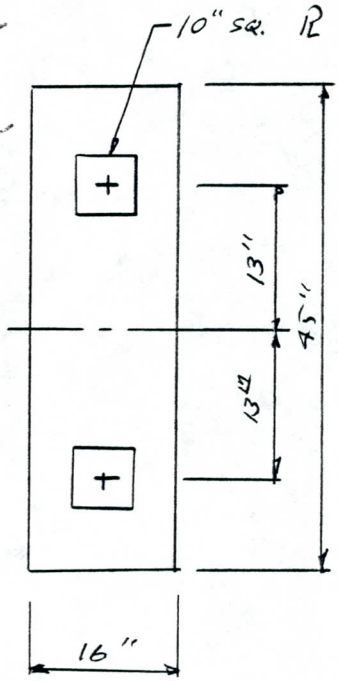
$2a$ = section ht = 45 in (a = 22.5 in)

$2a'$ = width of anchor PL = 10 in (a' = 5")

t = section thickness = 16 in

f_{sp} = split tensile strength
$\cong 6.5\sqrt{f'_c}$ = 0.411 ksi

θ = tendon inclination
$\cong \tan^{-1}(0.0806215)$ = 4.6° (avg)



$0.05 \le t/2a \le 0.25$

$t/2a = 16/45 = 0.36 > 0.25$

use $t = 45(0.25) = 11.25$ in

$$P_{cr} = 11.25\left[\frac{0.411}{24}(38(22.5)-120) - \frac{11.25}{81}\left(2(4.6) - 252\left(\tfrac{13}{22.5}\right)0.411\right) - 103/9\left(\tfrac{13}{22.5}\right) - 7\right] + 39(5) + \tfrac{1}{5}(0.411)\left[166 - 975\left(\tfrac{5}{11.25}\right)^2\right] - 9.1$$

$$= 11.25[12.587 + 7.034 - 6.612 - 7] + 195 - 2.186 - 9.1$$

$$P_{cr} = 67.60 + 195 - 2.186 - 9.1 = 251.3^k$$

$$1.10 f_{pu} A_{sp} = 1.10(270)11(0.153) = 500^k$$

$500^k > 251.3^k$

$\therefore$ section does not meet cracking strength requirement

supply spiral reinforcement

$$P'_{cr} = (2.03 - 0.032e)P_{cr}$$

$$= (2.03 - 0.032(4.6))251.3 = 473.1^k$$

$$\approx 1.04 f_{pu} A_{sp} \geq 0.95 f_{pu} A_{sp}$$

[AASHTO 9.26]

SAY OK

$$A_{sp} = \frac{f_1 - 0.6 f'_{ci}}{5 f_y} D_s$$

$f_1 = 4P_u / \pi D^2$

$f'_{ci} = 4000$ psi

$f_y = 40000$ psi

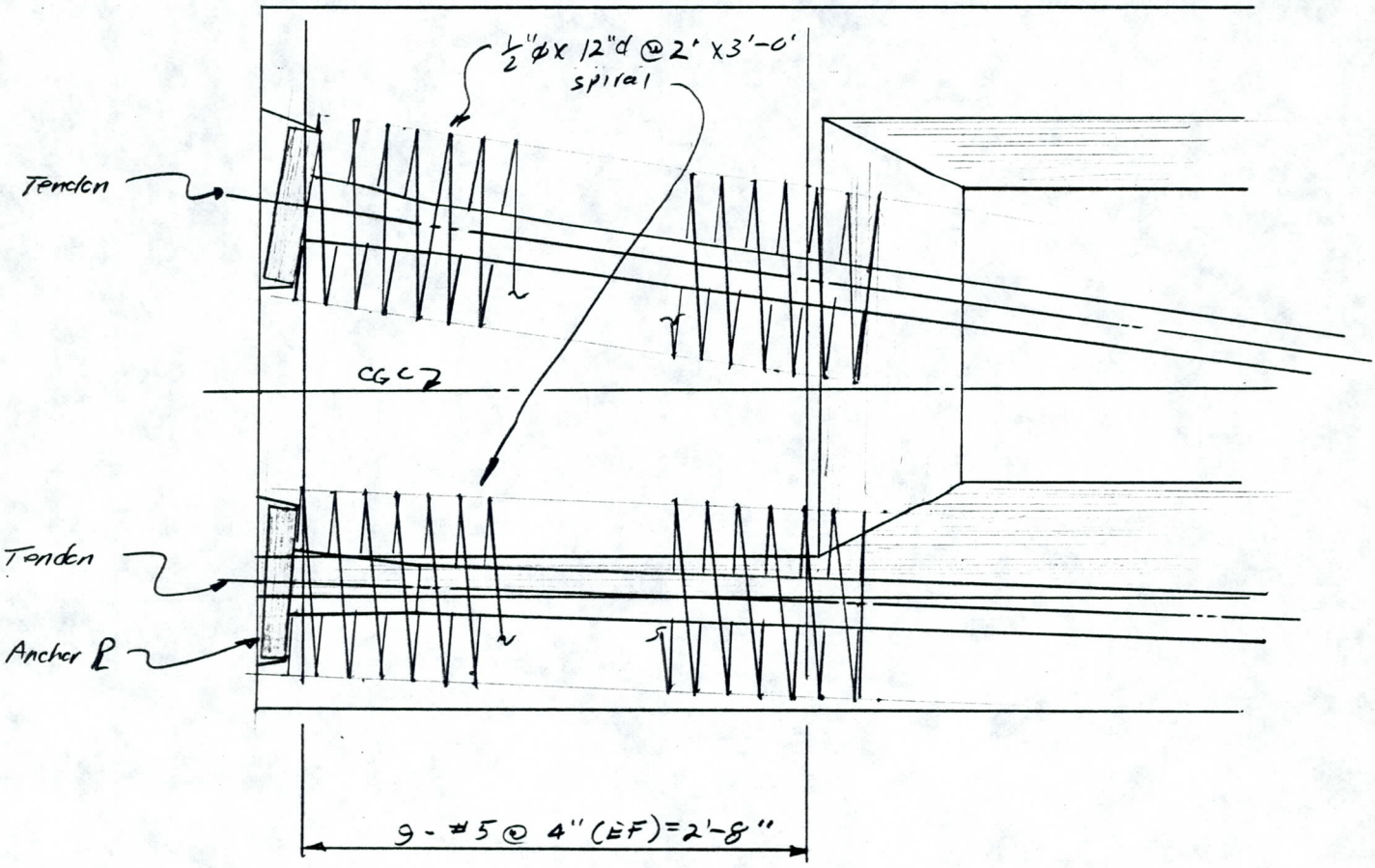
$P_u = 500^k$

$f_1 = 4(500)10^3 / \pi D^2 = 636{,}620 / D^2$

D	f_1	s = 2"	s = 3"	s = 1½"
12"	4421 psi	0.24 in²	0.36 in²	0.18 in²
9"	7860 psi	0.42 in²	0.74 in²	

for $A_{sp} = 0.20 \text{ in}^2$, $D_1 = 12''$ ∴ $s = 1.65^{in}$

use a ½"ϕ x 12"ϕ spiral @ 1½" pitch x 3'-0"
($A_s = 3.20 \text{ in}^2/\text{FT}$)
≤ ~~3.75~~ 3.66 in²/FT pg 118 OK



∴ Contractor shall verify Anchorage design and reinforcing based on his specific product

DESIGN OF POST-TENSIONED GIRDER ANCHORAGE ZONES

W. C. Stone and J. E. Breen

RESEARCH REPORT 208-3F

PROJECT 3-5-77-208

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

JUNE 1981

suggested in Sec. 2.3.3, an effective bearing stress design criterion for post-tensioned anchorages is:

$$f_{b_{all}} = 0.8\, f'_{c_i} \sqrt{A_2/A_1} \leq 1.33\, f'_{c_i} \qquad (3.18)$$

where f_b = permissible concrete bearing stress under the anchor plate of post-tensioning tendons

A_1 = bearing area of anchor plate

A_2 = maximum area of the portion of the anchorage surface that is geometrically similar to, and concentric with, the area of the anchor plate

f'_{c_i} = compressive strength of concrete at time of initial prestress.

3.6 Suggested Code or Specification Requirements

The general design criteria and recommendations contained in Secs. 3.4 and 3.5 are difficult to reduce to simple, concise language suitable for direct inclusion in regulations such as the AASHTO Specifications or the ACI Building Code. The provisions are best expressed as general performance requirements in the Specification or Code but with accompanying commentary indicating possible ways of satisfying the performance requirements.

Section 3.6.1 contains suggested performance requirements and Section 3.6.2 provides more detailed commentary text to assist designers and fabricators in meeting these requirements.

3.6.1 Code Provisions

A.0 Notation

A_{ps} = nominal area of post-tensioning tendon (in.^{2})

f_{pu} = specified tensile strength of prestressing tendons (psi)

A.1 Post-Tensioned Tendon Anchorage Zones

A.1.1 Reinforcement shall be provided where required in tendon anchorage zones to resist bursting, splitting, and spalling forces. Regions of abrupt change in section shall be adequately reinforced.

A.1.2 End blocks shall be provided where required for support bearing or for distribution of concentrated prestressing forces.

A.1.3 Post-tensioning anchorages and supporting concrete shall be designed to resist maximum jacking forces for strength of concrete at time of prestressing.

A.1.4 Post-tensioning anchorage zones shall be designed such that the minimum load producing cracking along the tendon path shall be at least equal to $1.10\ f_{pu}A_{ps}$.

A.1.5 Post-tensioning anchorage zones shall be designed such that their minimum strength shall be at least equal to $1.60\ f_{pu}A_{ps}$.

A.1.6 Supplementary anchorage zone reinforcement required for control of cracking or development of minimum strength may consist of passive reinforcement such as spirals or orthogonal closed hoops or mats. Active reinforcement such as lateral post-tensioning may be used.

A.1.7 Supplementary reinforcement such as spirals shall be provided to resist web face rupture in regions of high tendon curvature when multiple strand tendons are used.

A.1.8 Unless structural adequacy is demonstrated by comprehensive tests or a more comprehensive analysis, anchorage bearing stress at $1.1\ f_{pu}A_{ps}$ shall not exceed

$$f_b = 0.8f'_{c_i}\sqrt{A_2/A_1} \leq 1.33f'_{c_i}$$

where f_b = maximum concrete bearing stress under the anchor plate of post-tensioning tendons

A_1 = bearing area of anchor plate

A_2 = maximum area of the portion of the anchorage surface that is geometrically similar to, and concentric with, the area of the anchor plate.

f'_{c_i} = compressive strength of concrete at time of initial prestress.

3.6.2 Commentary

C.A.1 The general problems of anchorage of post-tensioned tendons are significantly different from the development of pretensioned reinforcement. Items concerning pretensioned element anchorage zones such as now included in AASHTO Sec. 1.6.15 should be put in a separate section.

C.A.1.1 This general performance statement alerts the user to the fact that the actual stresses around post-tensioning anchorages may differ substantially from those obtained by means of usual engineering theory of strength of materials. Consideration must be given to all factors affecting bursting, splitting, and spalling stresses. A refined strength analysis should be used whenever possible considering both the cracking and ultimate limit states.

C.A.1.2 Where convenient, widening of the anchorage region to distribute the high localized forces is an effective way of reducing bursting and spalling stresses and raising the cracking and ultimate capacities. The effect of increased width is indicated in Eq. A in Sec. A.1.4.

C.A.1.3 In application of all anchorage zone design the level of prestress applied and the concrete strength at time of application must be considered. This is particularly important with stage prestressing.

C.A.1.4 It is highly desirable that the anchorage zone remain uncracked at service levels to protect this vital area from corrosive and freeze-thaw deterioration. This can be ensured by proportioning the anchorage zone so that the cracking load is greater than any anticipated stressing load. In this proportioning the anchor zone can be designed to remain crack free without supplementary anchorage zone reinforcement by use of Eqs. A through D. The zone can be designed to remain surface crack free through provision of supplementary reinforcement which will raise the level of the cracking loads as indicated by Eqs. E through G. The service load level specified $1.10\ f_{pu}A_{ps}$ contains allowances for jacking errors, material tolerances, and a margin of variability.

C.A.1.4.1 Cracking Loads. The cracking load for thin web post-tensioned sections without supplementary anchorage zone reinforcement can be determined for certain conditions as:

$$P_{cr} = t\left[\frac{f_{sp}}{24}(38a - 120) - \frac{t}{81}[2\theta - 252(e/a)f_{sp}] - \frac{103}{9}(e/a) - 7\right] \quad \text{(A)}$$
$$+ 39a' + \frac{f_{sp}}{5}[166 - 975(a'/t)^2] - 9.1$$

where P_{cr} = cracking load in kips

e = tendon eccentricity (in.)

$2a$ = section height (in.)

$2a'$ = width of anchor plate (assumed square, in.)

t = section thickness (in.)

θ = tendon inclination at loaded face (degrees)

f_{sp} = split cylinder tensile strength (ksi) may be estimated as $6.5\sqrt{f'_c}$ psi)

All variables are illustrated in Fig. A.1 (Fig. 3.1 in text, not repeated). Limitations on the use of Eq. A assume

a. e, θ are both positive as defined in Fig. A.1
b. $0.05 \leq t/2a \leq 0.25$
c. anchors are assumed square, plate type
d. single tendon anchored in the web.

The equation can be easily extended to other practical applications as shown in Fig. A.2 (Fig. 3.2 in text, not repeated).

For sections which do not meet the above criteria cracking loads can be obtained using three-dimensional finite element analysis techniques, or by comprehensive physical tests.

The cracking load can be calculated from a three-dimensional finite element computer analysis which has been calibrated to extensive physical tests. One such calibration indicates:

1. The maximum spalling strain (transverse tensile strain parallel to the loaded face) at the anchor plate edge must be calculated. For most cases this will require a detailed mesh refinement in the vicinity of the anchor plate edge following a preliminary analysis with a coarse grid. This is particularly necessary for inclined tendon blockouts with square corners. Anchorage zone reinforcement need not be modeled for this analysis.

2. The peak spalling strain corresponding to a load of 1 kip should be computed. The approximate cracking load (for a section without supplementary reinforcement) can be calculated as follows:

$$P_{cr} = \frac{\epsilon_{cr}}{\epsilon_1 \text{ kip(FEM)}} \qquad \text{(B)}$$

where P P_{cr} = cracking load (kips)

ϵ_{cr} = threshold cracking strain ($\mu\epsilon$)

ϵ_1 kip(FEM) = peak spalling strain at plate edge from program with unit post-tension load of 1 kip.

Calibration studies indicate that appropriate values of ϵ_{cr} are 172 $\mu\epsilon$ for plate anchors with straight tendons and 1092 $\mu\epsilon$ for plate anchors with inclined tendons in which a right angle blockout is used.

For other than plate bearing-type anchorages, the cracking loads obtained from Eqs. A and B should be modified as follows:

Conical Anchor $P_{cr} = 0.61\ P_{cr}$ (plate) (C)

Bell Anchor $P_{cr} = 1.08\ P_{cr}$ (plate) (D)

These coefficients apply only when the anchorages present approximately the same projected bearing area.

In any physical tests to determine cracking loads, the conditions to be expected during construction of the actual structure must be replicated as precisely as possible. These include the effects of tendon eccentricity, inclination, curvature and multiple strands, as well as anchor size, section width and height, and supplementary reinforcement.

C.A.1.4.2 Effect or Reinforcement on Cracking --Cracking loads as calculated from Eqs. A through D represent the minimum value to be expected for a section with no supplementary reinforcing in the anchorage zone. The addition of supplementary reinforcing will raise both the cracking and ultimate load. For sections provided with spiral, orthogonal, or active reinforcement designed in accordance with A.1.4.6, the cracking load can be determined as

Spiral Reinforcement: $P'_{cr} = (2.03 - 0.032\theta)P_{cr}$ (E)

Orthogonal Reinforcement: $P'_{cr} = (1.61 - 0.019\theta)P_{cr}$ (F)

Active Reinforcement: $P'_{cr} = (2.37 - 0.0372\theta)P_{cr}$ (G)

where P'_{cr} = cracking load for the reinforced section (kips)

θ = angle of tendon inclination (degrees)

P_{cr} = cracking load for the unreinforced section as calculated above (kips)

C.A.1.5 The proper development of the post-tensioning force in unbonded tendons and prior to completion of grouting in bonded tendons is completely dependent on proper anchorage of the tendons. The anchorage capacity must be greater than any anticipated tendon load with a reasonable factor of safety. The capacity specified 1.60 $f_{pu}A_{ps}$ contains allowance for tendon tolerances, actual strength range rather than guaranteed minimum strength, and a margin of safety against the explosive type failure which would occur if an anchorage zone failed.

The ultimate load for sections without supplementary anchorage zone reinforcement is conservatively assumed to be equal to the cracking load. With the addition of reinforcement designed according to A.1.6 the ultimate load will be:

No Supplementary Reinforcement: $P_{ult} = P_{cr}$ (H)

Spiral Reinforcement: $P_{ult} = (3.18 - 0.053\theta)P_{cr}$ (I)

Orthogonal Reinforcement: $P_{ult} = (1.71 - 0.017\theta)P_{cr}$ (J)

Active Reinforcement: $P_{ult} = (3.89 - 0.0640\theta)$ (K)

where P_{ult} = ultimate load for the reinforced section (kips)

θ = angle of tendon inclination (degrees)

P_{cr} = cracking load for the unreinforced section as calculated above.

C.A.1.6 In order to obtain the strength increase indicated in Eqs. E through K, supplementary anchorage zone reinforcement must meet the following minimum requirements.

C.A.1.6.1 Spiral Reinforcement--Spiral confinement must be adequate to resist cracking and fully develop the anchorage. To

ensure a sturdy unit the minimum spiral wire diameter is 1/4 in. Minimum spiral area is

$$A_{sp} \geq \frac{f_1 - 0.6f'_{c_i}}{5f_y} Ds \geq 0.05 \text{ si}$$

where A_{sp} = spiral wire cross-sectional area, in.^{2}

f_1 = post-tension load divided by the area confined by the spiral = $4P_u/\pi D^2$, psi

f'_{c_i} = specified concrete compressive strength at time of stressing, psi

D = overall diameter of spiral, in.

s = pitch of spiral, in.

f_y = spiral yield strength.

In thin webs, the spiral diameter, D, should be as large as possible while still satisfying cover requirements. In general, the spiral diameter should be the maximum linear dimension of the anchor projected bearing surface (the diagonal for square anchor plates). Spiral pitch should be as small as possible but must allow for concrete placement. The spiral should begin at the anchor plate and have a minimum length of twice the anchor plate depth or width, whichever is larger.

C.A.1.6.2 Orthogonal Reinforcement--While spiral reinforcement is usually superior to orthogonal reinforcement, in some applications an orthogonal grid of closely spaced closed stirrups or a mesh of orthogonal bars may be used. The minimum area of bars in such closed stirrups or meshes should be calculated using the expression given in A.1.6.1 with the minimum lateral dimension of the orthogonal closed stirrup or mesh substituted for D and the stirrup spacing substituted for s.

C.A.1.6.3 Active Reinforcement--Lateral post-tensioning (LPT) is highly effective as active reinforcement. Such reinforcement should be designed on the following basis:

1. LPT tendons should be placed as close as possible to the loaded face and should extend throughout the height of the web.

2. LPT tendons should produce a minimum lateral precompression in the anchor zone of 100 psi after losses. Initial stressing should provide 150 to 200 psi. The nominal effective area for stress calculation should be taken as the web thickness times a length equal to half the section height.

3. LPT tendons should be placed in pairs equidistant from the tendon centerline to minimize lateral moments in the web.

4. LPT tendons should be grouted and should utilize the most positive seating load lock-off mechanism available.

C.A.1.7 Reinforcement for Multistrand Effects--For post-tensioning applications with significant tendon curvatures and with multiple strand tendons, a side face failure mechanism may govern the failure of the section. Any time a loaded tendon follows a curved path, normal and friction forces are set up along the length of the duct. In regions of small radius of curvature lateral forces due to the flattening out of the multi-strand tendon under stressing loads can cause tendon path cracking at loads below those which would initiate cracking in the anchorage zone proper. Such cracking will be likely if

$$P_{des} \geq P_o = \frac{2\phi \sqrt{f'_{c_i}}\, CR\pi\alpha}{90(1 - \cos \alpha)}$$

or

$$R_{min} \leq R_o = \frac{90P(1 - \cos \alpha)}{\pi\alpha C 2\phi \sqrt{f'_{c_i}}}$$

where P_{des} = P = the minimum cracking design load ($1.10\ f_{pu}A_{ps}$)

P_o = side face cracking load

ϕ = strength reduction factor for shear = 0.85

f'_{c_i} = compressive strength of concrete at time of stressing, psi

C = minimum concrete cover on one side of duct

R = minimum radius of curvature of tendon, in.

α = 1/2 the duct loaded arc angle, degrees (but not more than 90°)

If $P_{des} \geq P_o$ or $R_{min} \leq R_o$ then supplementary reinforcement will be required in the region where $R \leq R_o$. Since the region of minimum radius of curvature is typically some distance removed from the anchorage zone (and the benefit of the supplemental reinforcement there) additional reinforcement must be provided. This can be accomplished most efficiently through the use of spiral reinforcement designed as follows:

1. The radius of curvature along the tendon profile is calculated as:

$$R = \frac{\left[1 + (dx/dz)^2\right]^{3/2}}{\left|d^2x/dz^2\right|}$$

where x is the dependent vertical variable and
z is the longitudinal variable.

Most tendon profiles can be defined by the equation

$$x = Az^3 + Bz^2 + Cz + D$$

The minimum radius of curvature R can thus be calculated.

2. Given the internal diameter of the tendon duct and the number of strands used, make a scale drawing of the duct with all strands placed as close as possible to the concave side of the duct as would occur when the stressing load is applied. Draw two tangential lines from the center of the duct, to the outside of the outermost strand, as in Fig. A.3 (Fig. 3.4 in text, not repeated). This defines α. The area of spiral required is then

$$A_{sp} = \frac{45Ps(1 - \cos \alpha)}{\pi \alpha R 0.6 f_y} \geq 0.05 \text{ sq. in.}$$

General spiral proportioning should follow the requirements in Sec. A.1.6.1. The spiral should extend throughout those regions where $R \leq R_o$ but at least 2t (where t = web thickness) to either side of the point of minimum radius of curvature. Such spiral reinforcement designed for multistrand cracking need not be placed in areas where equivalent or stronger primary anchorage zone reinforcement has already been supplied.

C.A.1.8 Bearing Stress--In many cases the adequacy of anchorage assemblies will have been demonstrated by comprehensive tests or analyses. However, in other cases it is desirable to have a relatively simple method to proportion the size of bearing plates. Comprehensive tests and analyses show the tendon anchorage cracking load is relatively insensitive to bearing area and bearing stress. However, the confinement provided by concrete surrounding the bearing plate does increase the cracking load somewhat. The value of allowable bearing stress given in

SHEAR REINFORCING

[AASHTO 9.20]

· REFER TO PAGES 51 to 76, PRECAST, Prestressed BEAM DESIGN

@ support + h/2 (pg 53, 54, 55)

$X = 1.875^{FT} = 22.5^{IN}$

$V_{DL}^{NC} = 38.2^k$

$V_{DL}^{C} = 13.9^k$

$52.1^k = V_d$

$V_{LL+I} = 52.8^k$

$V_u = 1.3[52.1 + 5/3(52.8)] = 182^k$

$V_i = 1.3[13.9 + 5/3(52.8)] = 132^k$

$M_{DL}^{NC} = 74^{'-k}$

$M_{LL+I} = 99^{'-k}$

$M_{DL}^{C} = 25.2^{'-k}$

$99.2^{'-k}$

$M_u = 1.3[99.2 + 5/3(99)] = 343^{'-k}$

$M_{MAX} = 1.3[25.2 + 5/3(99)] = 247^{'-k}$

$e_{ps}(22.5) = -0.0001139(22.5^2) + 0.0806215(22.5) = 1.76^{IN}$

$d = 45 + 7.5 - 20.27 + 1.76 = 34^{IN} < 0.8h = 42^{IN}$

$d = 42^{IN}$

$P_{ps} = 497.4^k$

$f_{pe} = P/A + Pe_{ps}/S$

$= [497.4/560 + 497.4(1.76)/6186]\,10^3 = +1030$ psi

$f_d = M_{DL}/S = 74(12000)/6186 = -144$ psi

$M_{cr} = S_b^t\,(6\sqrt{f'_c} + f_{pe} - f_d)$

$= 10383(6\sqrt{5000} + 1030 - 144)/12000 = 1134^{1-k}$

$V_{ci} = 0.6\sqrt{f'_c}\,b'd + V_d + \frac{V_i M_{cr}}{M_{MAX}}$

$= 0.6\sqrt{5000}(16)(42)/1000 + 52.1 + 132(1134)/247 = 687^k$

$e'_{ps}(22.5) = -0.0002278(22.5) + 0.0806215 = 0.0755$

$V_p = 497.4^k(0.0755) = 37.6^k$

$c = 14.78^{in}$ (pg 55)

$f_{pc} = P/A - Pec/I + M_a^{nc}c/I$

$= [497.4/560 - 497.4(1.76)14.78/125390 + 74(12)14.78/125390]\,10^3$

$= 888 - 103 + 105 = 890$ psi

$$V_{cw} = [3.5\sqrt{f'_c} + 0.3 f_{pc}]\, b'd + V_p$$

$$= [3.5\sqrt{5000} + 0.3(\text{[illegible]})]\, 16(42) + 37.6 = 383.3^k$$

$$V_{cw} < V_{ci}$$

$$\therefore V_c = 383^k$$

$$2V_u > \phi V_c = 345^k > V_u$$

$\therefore$ supply min Reinf. $= 0.11 in^2/ft$

@ X = 5' (pg. 62 -64)

$V_{DL}^{nc} = 34^k$

$V_{LL+I} = 49^k$

$V_{DL}^{c} = 11.5^k$

$45.5^k = V_d$

$V_u = 1.3[45.5 + 5/3(49)] = 165^k$

$V_i = 1.3[11.5 + 5/3(49)] = 121^k$

$M_{DL}^{nc} = 187'^{-k}$

$M_{LL+I} = 244'^{-k}$

$M_{DL}^{c} = 63'^{-k}$

$250'^{-k}$

$M_{MAX} = 1.3[63 + 5/3(244)] = 611'^{-k}$

$e_{ps}(60) = -0.0001139(60^2) + 0.0806215(60) = 4.43^{in}$

$e'_{ps}(60) = -0.0002278(60) + 0.0806215 = 0.0670$

$d = 0.8h = 42^{in}$

$P_{ps} = -\frac{497.4 - 475.5}{29.5}(5) + 497.4 = 493.7^k$

$f_{pe} = 493.7(10^3)/560 + 493.7(10^3)4.43/6186 = 1235$ psi

$f_{d} = -187(12000)/6186 = -363$ psi

$M_{cr} = 10383[6\sqrt{5000} + 1235 - 363]/12000 = 1121.6^{k-ft}$

$V_{ci} = 0.6\sqrt{5000}\,(7)42/1000 + 45.5 + 121(1121.6)/611$

$= 280^k$

$V_p = 493.7\,(0.0670) = 33.1^k$

$c = 14.78^{in}$

$f_{pc} = 493.7(1000)/560 - 493.7(1000)\,4.43(14.78)/125390 + 187(12000)\,14.78/125390$

$= 882 - 258 + 265 = 888$ psi

$V_{cw} = [3.5\sqrt{5000} + 0.3(888)]\,7(42)/1000 + 33.1$

$= 184.2^k$

$V_{cw} < V_{ci}$

$\therefore V_c = 184.2^k$

$V_s = V_u/\phi - V_c = 165/0.9 - 184.2 = -0.8^k$

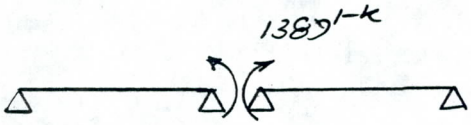
$\therefore$ supply $A_{v\,min} = 0.11\ in^2/FT$

$\therefore$ from previous, and by inspection
use shear design from precast, prestressed design.

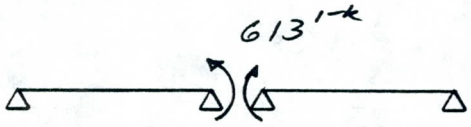
Check Continuity

p.p. 31 - 30

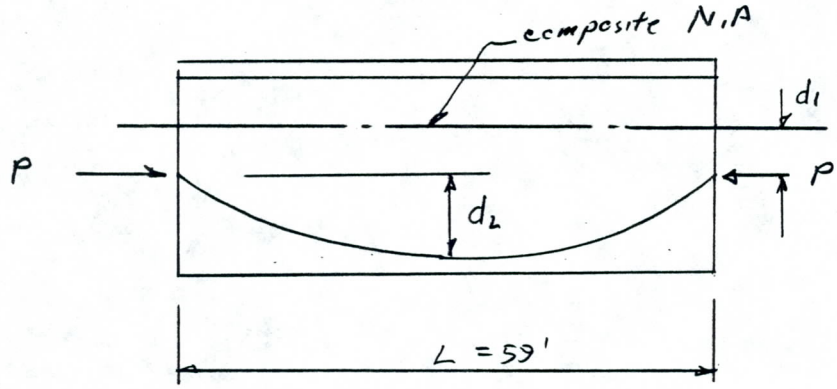
SHRINAGE (pg 33)



DEAD LOAD CREEP



Prestressed Creep



$d_1 = 33.05 - 20.27 = 14.78$ in

$d_2 = 14.27$ in

slope due to prestress

$$[P/S \text{ slope}] = -8Pd_2 \ell / 24EI - Pd_1 \ell / 2EI$$

Assume 75% of long term stress losses @ time of connection

$\therefore$ end $189 - 0.75(41.23) = 158$ ksi

midspan $189 - 6.5 - 0.75(41.23) = 151.6$ ksi

Avg $f_{se} = \frac{1}{2}(158 + 151.6) = 154.8$ ksi

$P = 22(0.153)154.8 = 521^k$

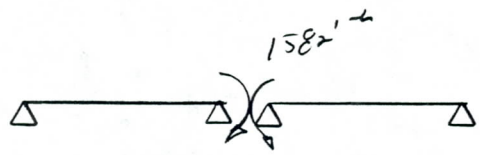
$-8Pd_2\ell/24 = -8/24\,(521)14.27(59)12 = -1,754,582$

$-Pd_1\ell/2 = -\frac{1}{2}(521)14.78(59)(12) = -2,725,935$

$\Sigma -4,480,517$

$[P/S \text{ slope}] = -4,480,517/EI$

$\therefore M_B = \frac{-3EI}{L}[P/S \text{ slope}] = 1582^{\prime-k}$



Creep Factors

pg 37 - 39

$\phi = 2.05$

$(1 - e^{-\phi}) = 0.83$

$(1 - e^{-\phi}/\phi) = 0.47$

Summation of Restraint Moments

$$M_r = (M_{ps} - M_{DL})(1 - e^{-\phi}) - M_s\left(\frac{1 - e^{-\phi}}{\phi}\right)$$

$$= (1582 - 613)0.83 - 1389(0.47)$$

$$= 151^{'-k} < 303^{'-k}$$ (pg 39)

∴ previous design OK

PRECAST, POST-TENSIONED GIRDER NOTES

POST-TENSIONING TENDONS SHALL BE A MULTI-STRAND SYSTEM UTILIZING HIGH-STRENGTH, SEVEN-WIRE STRESS -RELIEVED STEEL STRAND, GRADE 270, 1/2-INCH OR 0.6-INCH DIAMETER CONFORMING TO THE REQUIREMENTS OF SECTION 506.3 OF THE CONSTRUCTION SPECIFICATIONS.

STRESS-RELIEVED LOW-RELAXATION SEVEN-WIRE STEEL STRAND, GRADE 270 MAY BE SUBSTITUTED IN LIEU OF THE ABOVE PROVIDED EQUAL PRESTRESS FORCE IS FURNISHED.

THE POST-TENSIONING SHALL SUPPLY AN INITIAL FORCE OF 614.4 KIPS PER GIRDER AT THE JACKING END AFTER AN EQUIVALENT ANCHOR SET OF 5/8 INCH. A FINAL FORCE OF 497.4 KIPS PER GIRDER AT THE JACKING END AFTER ALL LOSSES HAVE OCCURRED SHALL BE MAINTAINED. DESIGN IS BASED ON:

μ (FRICTION CURVATURE COEFFICIENT) = 0.25
K (FRICTION WOBBLE COEFFICIENT) = 0.0002

THE CONTRACTOR SHALL SUBMIT ELONGATION AND JACKING CALCULATIONS BASED ON $\mu\alpha + KL = 0.026$ AND AN INITIAL STRESS AT THE C.L. OF THE GIRDER SPAN OF 0.965 TIMES THE INITIAL JACKING END FORCE PRIOR TO COMMENCEMENT OF WORK.

EACH TENDON SHALL BE STRESSED FROM EACH END OF THE GIRDER.

TENDON DUCTS SHALL BE OF RIGID GALVANIZED METAL. DUCTS SHALL BE TIED SECURELY TO THE VERTICAL STIRRUPS AT A MAXIMUM SPACING OF 4'-0" TO PREVENT DISPLACEMENT DURING CONCRETING.

CENTER OF GRAVITY OF PRESTRESSING STEEL IS PARABOLIC BETWEEN ANCHORAGE POINTS AS SHOWN ON THE PLANS.

THE PRECAST, PRESTRESSED CONCRETE BRDGE GIRDER SHALL CONFORM TO THE STANDARD TYPE III AASHTO BRIDGE GIRDER WITH THE FOLLOWING PROPERTIES:

CROSS SECTIONAL AREA	A = 560 SQ. IN.
MOMENT OF INERTIA	I = 125,390 IN.^{3}

THE GIRDER CONCRETE SHALL BE CLASS AA HAVING A MINIMUM COMPRESSIVE STRENGTH AT 28 DAYS OF 5,000 POUNDS PER SQUARE INCH. THE CONCRETE SHALL HAVE REACHED A MINIMUM COMPRESSIVE STRENGTH OF 4,000 POUNDS PER SQUARE INCH PRIOR TO TRANSFER OF THE PRESTRESSING FORCE TO THE CONCRETE.

ALL GIRDERS SHALL BE CAST ON CONCRETE BASED PALLETS AND IN APPROVED METAL FORMS.

THE CONTRACTOR SHALL SUPPLY SUFFICIENT GROUTING INLETS, VENT PIPES, AND DRAINS TO ALLOW PROPER GROUTING OF THE DUCTS AND TO AVOID ACCUMULATION OF WATER DURING CONSTRUCTION.

REINFORCING STEEL MAY BE ADJUSTED DURING INSTALLATION OF THE PRESTRESSING DUCTS AS REQUIRED TO PROVIDE PLANNED CLEARANCES FOR THE METAL CONDUITS, ANCHORAGES, JACKS AND EQUIPMENT SUBJECT TO APPROVAL BY THE ENGINEER. THE WORKING DRAWINGS SHALL INCLUDE ANY ADDITIONS OR REARRANGEMENT OF THE REINFORCING STEEL FROM THAT SHOWN ON THE PLANS.

BEARING PLATES SHALL BE PLACED TIGHTLY AGAINST THE FORMS WHICH SHALL BE BRACED AND ANCHORED TO SUPPORT THEIR WEIGHT.

THE CONTRACTOR SHALL SUBMIT TO THE ENGINEER FOR APPROVAL THE LOCATION AND TYPE OF LIFTING DEVICES AND PROCEDURE USED TO TRANSPORT AND ERECT THE GIRDERS. DURING STORAGE AND TRANSPORT, THE GIRDER SHALL BE SUPPORTED ONLY AT THE CENTERLINE OF BEARING.

TOPS OF ALL GIRDERS SHALL BE LEFT ROUGH. AT THE TIME OF INITIAL CONCRETE SET, ALL LAITENCE SHALL BE REMOVED WITH A COARSE WIRE BRUSH AND THE TOP OF THE GIRDER ROUGHENED TO AN APPROXIMATELY 1/4 INCH AMPLITUDE.

THE CONTRACTOR SHALL SUMBIT TO THE ENGINEER FOR APPROVAL PRIOR TO COMMENCEMENT OF WORK 2 COPIES OF SHOP DRAWINGS AND CALCULATIONS. THE DRAWINGS AND THE CALCULATIONS SHALL SHOW THE METHOD AND SEQUENCE OF GIRDER CONSTRUCTION, INCLUDING BUT NOT LIMITED TO FORMING, CASTING, STRESSING, CURING, STORAGE, TRANSPORTING, AND ERECTION OF THE GIRDER.