75TH AVE. SOUTH OF BELL ROAD OVER ACDC

ROYDEN
ENGINEERING
COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75 TH AVE
MCFCD

SHT. COVER OF
BY AGD
DATE OCT. 1984

TSR

75 TH AVE. SOUTH OF BELL RD.
OVER A.C.D.C.

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ROYDEN
ENGINEERING
COMPANY
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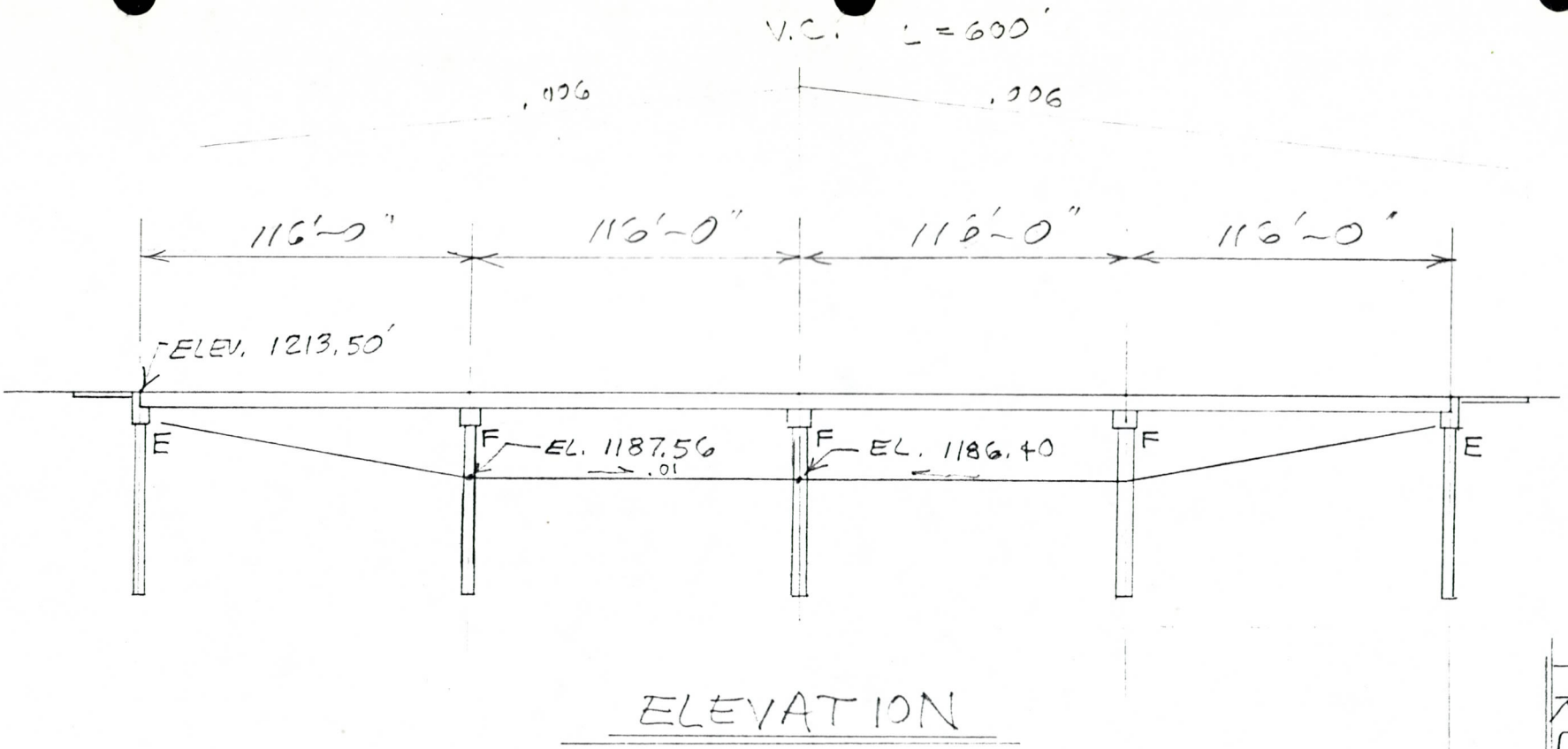
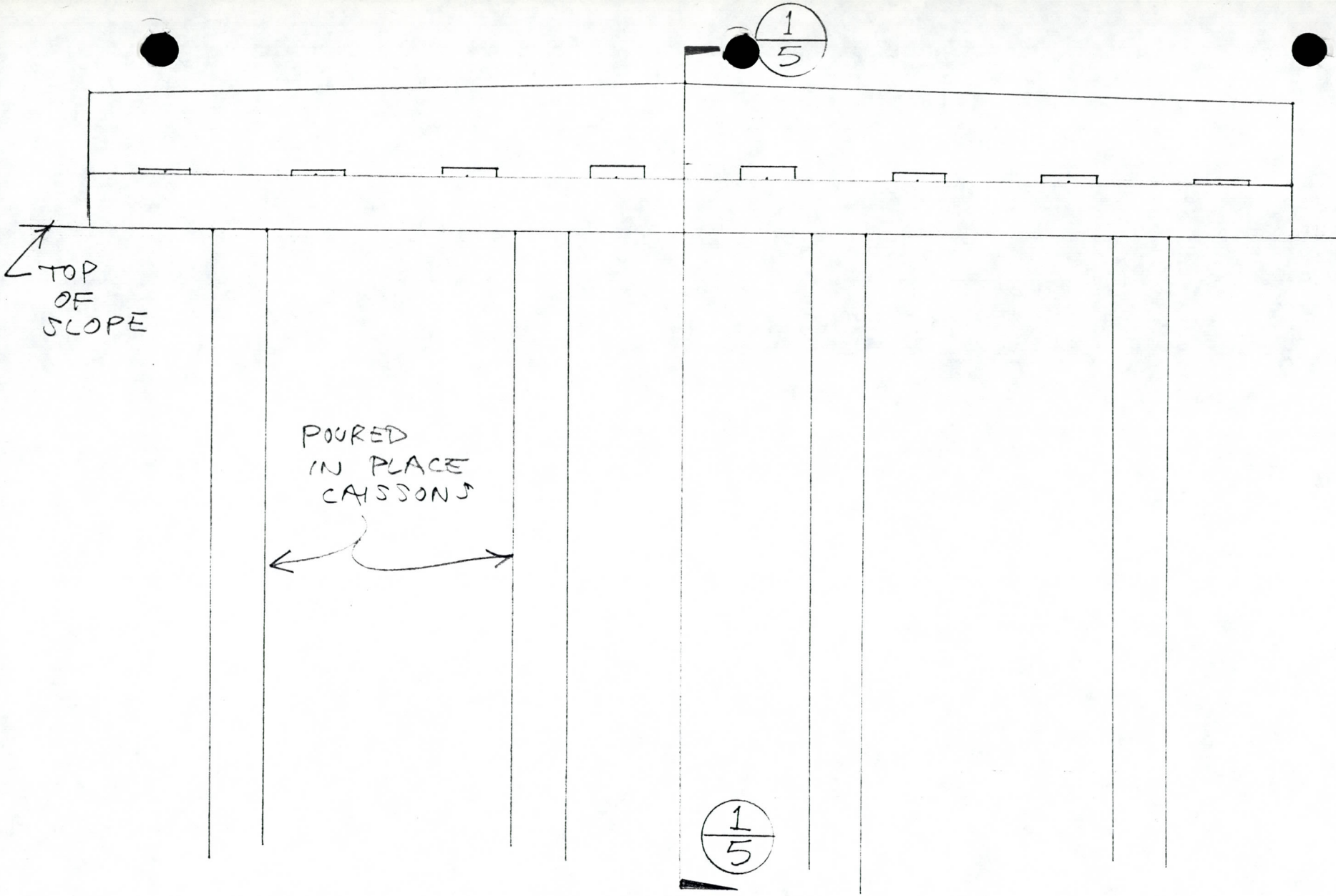
JOB NO. 1040
DESC. 75 TH. AVE
MCFCD

SHT. 1 OF 5
BY AGD
DATE JULY 13, 1984
73R

PRELIMINARY
CONCEPT



ELEVATION

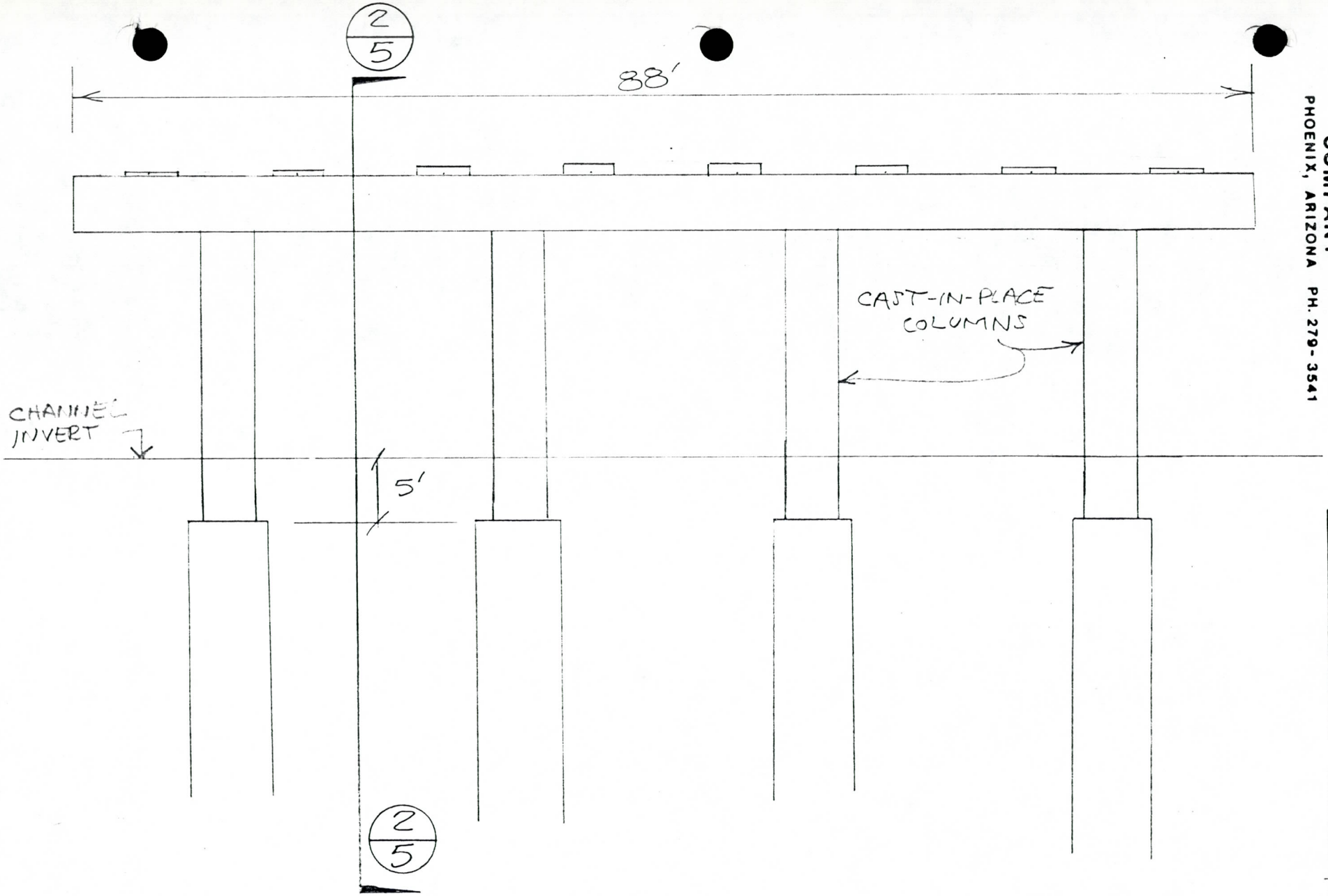


ELEV. ABUTMENTS 1 & 2

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1240
DESC. 75 TH AVE MCFCD

SHT. 3 OF 5
BY AGD
DATE JULY 13, 1984
TSR

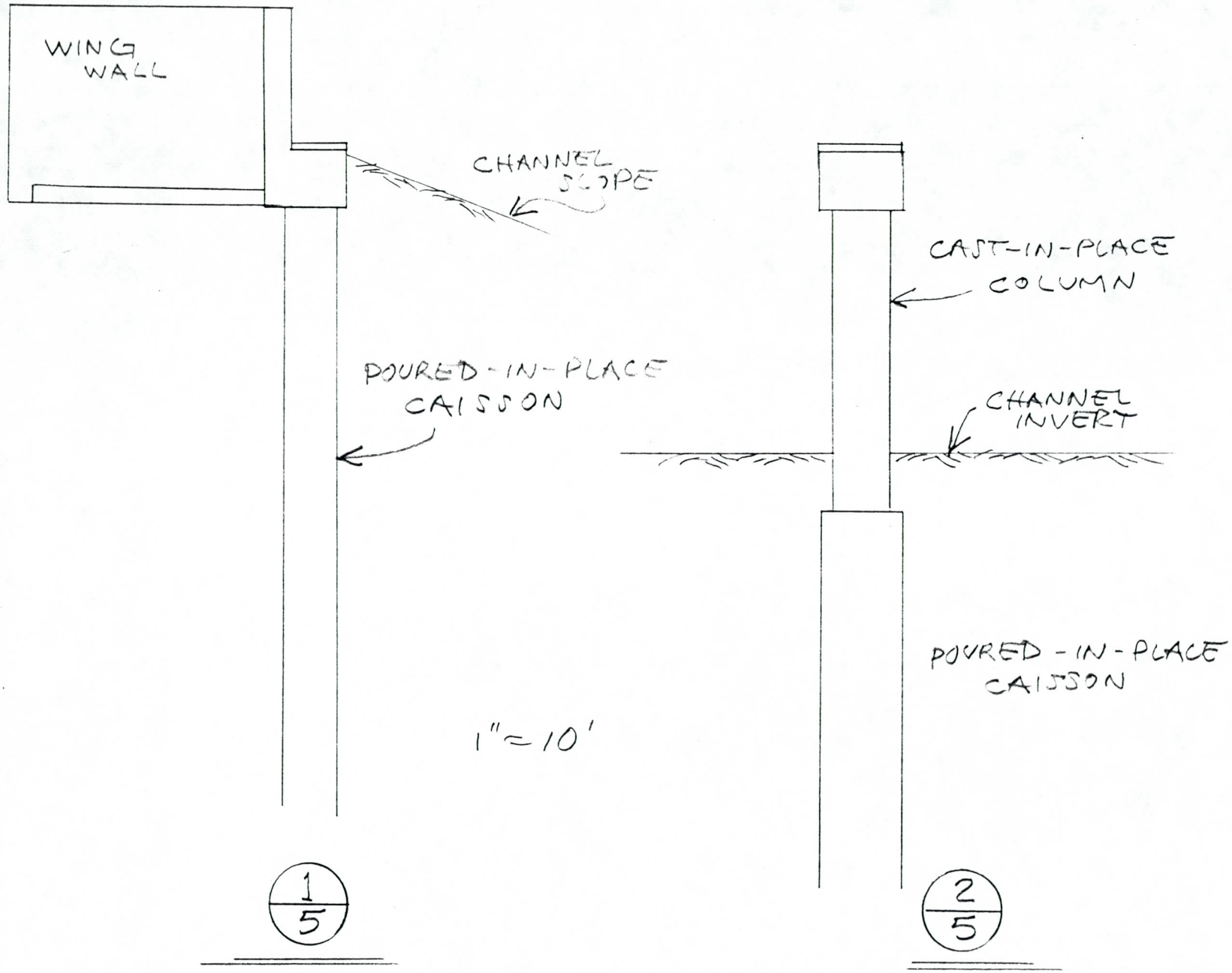


ELEV. PIERS 1, 2 & 3

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75TH AVE MCFCD

SHT. 4 OF 5
BY AGD
DATE JULY 13, 1984
75R



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JOB NO. 1040
DESC. 75 TH AVE MCFCD

SHT. 5 OF 5
BY AGD
DATE JULY 13, 1984
TSR

ESTIMATE OF SCOUR DEPTH

FHWA METHOD

REF. "HIGHWAYS IN THE RIVER ENVIRONMENT"

PIER DIAMETER $a = 5.0$ FT.

WATER DEPTH $Y_1 = 18.0$ FT

WATER VELOCITY $V_1 = 5$ FT/SEC

FROUDE NUMBER

$$F_{r1} = \frac{V_1}{\sqrt{g Y_1}}$$

$$= \frac{5.0}{\sqrt{(32.2)(18)}} = 0.208$$

$$\frac{Y_s}{Y_1} = 2.0\left(\frac{a}{Y_1}\right)^{0.65} F_{r1}^{0.43} \quad [\text{EQ. } 6.5.11]$$

$$= 2.0\left(\frac{5.0}{18.0}\right)^{0.65} (.208)^{0.43}$$

$$= (.8698)\ (.509) = 0.4427$$

$$Y_s = 18.0(.4427) = 7.97 \text{ FT}$$

$$\text{MAX. SCOUR} = 1.3(Y_s) = 10.36 \text{ FT}$$

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040 SHT. S-2 OF 5
DESC. 75TH AVE MCFCD BY AGD
DATE JULY 31, 1984 TSR

ESTIMATE OF SCOUR DEPTH

FOR EFFECTIVE PIER DIAMETERS OF VARIOUS WIDTHS:

$$\text{MAX. SCOUR} = (1.3)\,Y_1\,(2)\left(\frac{a}{Y_1}\right)^{0.65}\left(F_{r_1}\right)^{0.43}$$

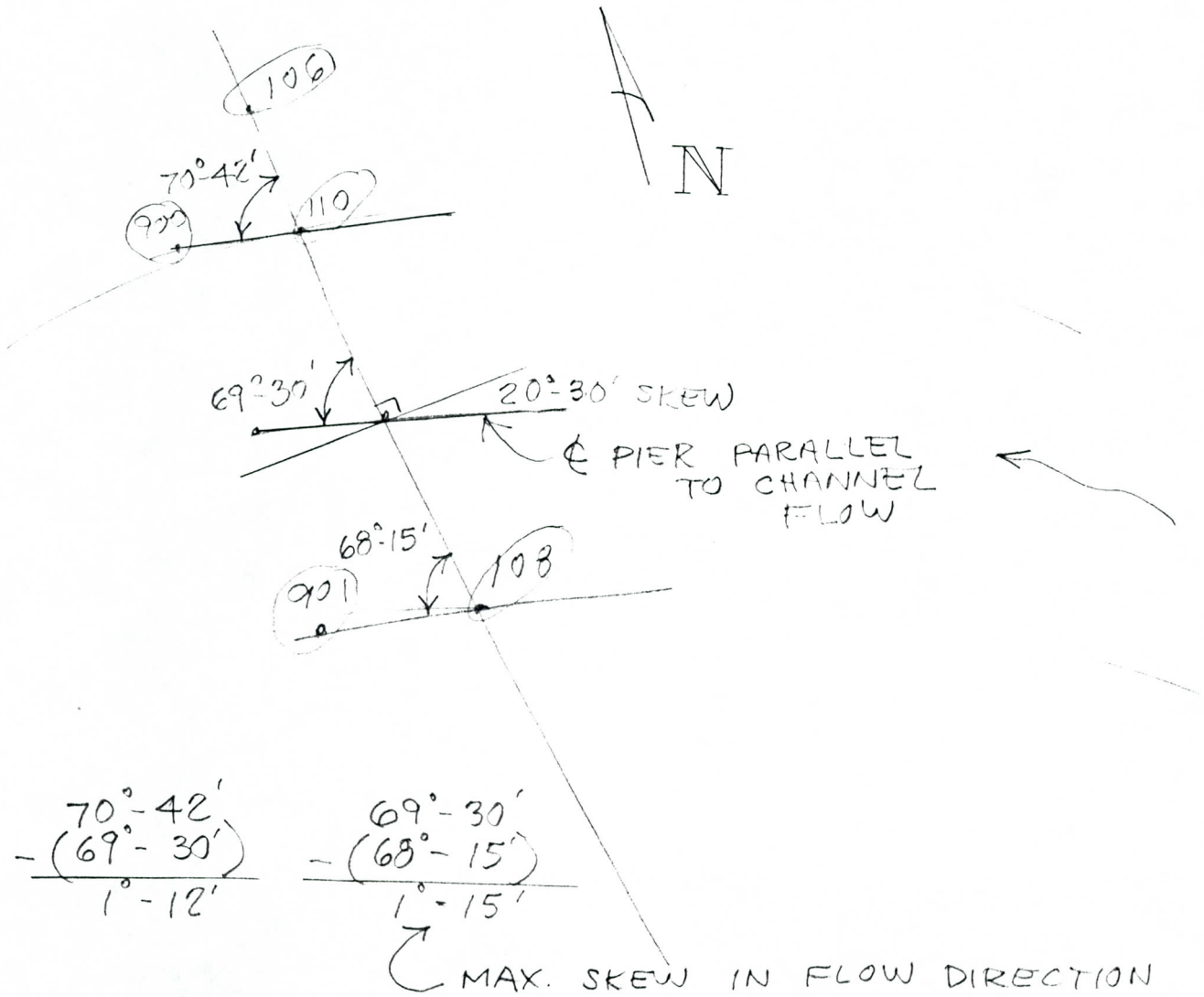
$$= (1.3)(18)(2)\left(\frac{a}{18}\right)^{0.65}\left(\frac{5.0}{\sqrt{32.2(18)}}\right)^{0.43}$$

$$= 23.821\left(\frac{a}{18}\right)^{0.65}$$

	a (FT)	MAX. SCOUR (FT)
	6	11.6635612
	6.5	12.2864535
	7	12.8927804
	7.5	13.4841223
4'Ø + 2 + 2 →	8	14.0618141
	8.5	14.6269967
5'Ø + 2 + 2 →	9	15.1806544
	9.5	15.7236441
	10	16.2567176
	10.5	16.780539
	11	17.2956985
	11.5	17.8027239
3 x 4'Ø →	12	18.3020893
	12.5	18.7942227
	13	19.2795121
	13.5	19.7583107
	14	20.230941
	14.5	20.6976988
3 x 5'Ø →	15	21.1588558

NOTE: THE ABOVE DEPTHS DO NOT INCLUDE SCOUR INCREASES DUE TO PIER ℄ BEING SKEWED TO DIRECTION OF CHANNEL FLOW.

```
(   2) ROYDEN ENGINEERING CO.
(   3) 75TH AVE BRIDGE OVER ACDC
(   4) 9/4/84
(   5) $
(   6) $SKEW ANGLE CALCULATION
(   7) $
(   8) RES FCD 1 999
(   9) LAN 101 110 900 100 90 00 00
(  10) LAN 101 108 901 100 90 00 00
(  11) ANG 900 110 106
(  12) ANG 901 108 106
(  13) SAVE FCD 1 999
(  14) FINISH
```



$$\begin{array}{r} 70^\circ - 42' \\ -(69^\circ - 30') \\ \hline 1^\circ - 12' \end{array} \qquad \begin{array}{r} 69^\circ - 30' \\ -(68^\circ - 15') \\ \hline 1^\circ - 15' \end{array}$$

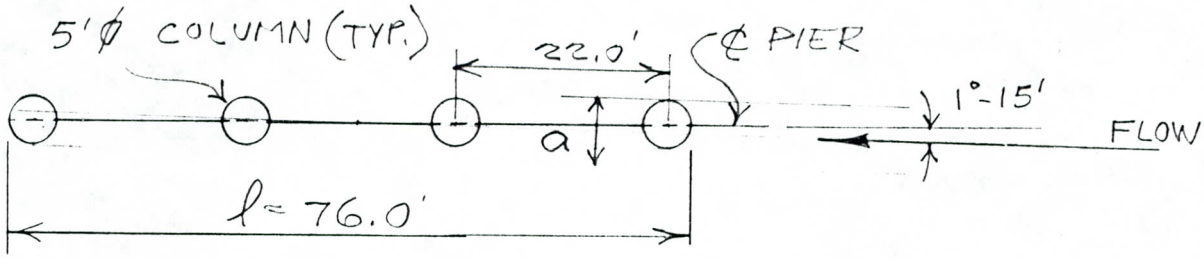
TBR


```
ROYDEN ENGINEERING CO.
75TH AVE BRIDGE OVER ACDC
9/4/84


(   5) $
(   6) $SKEW ANGLE CALCULATION
(   6) $
(   7) $

 (   8) ***  COMMAND  RES FCD 1 999  ***
 32  POINTS RESTORED

 (   9) ***  COMMAND  LAN 101 110 900 100 90 00 00  ***
PT= 900 YCOORD= 957874.0973 XCOORD= 407796.4709

 (  10) ***  COMMAND  LAN 101 108 901 100 90 00 00  ***
PT= 901 YCOORD= 957646.2458 XCOORD= 407804.4859

 (  11) ***  COMMAND  ANG 900 110 106  ***
ANGLE FORMED BY POINTS  900  TO  110  TO  106  =  70 41 55.892

 (  12) ***  COMMAND  ANG 901 108 106  ***
ANGLE FORMED BY POINTS  901  TO  108  TO  106  =  68 14 37.608

 (  13) ***  COMMAND  SAVE FCD 1 999  ***
 34   POINTS SAVED

END COGO
```

ESTIMATE OF SCOUR DEPTH

CORRECTION FOR SKEWED FLOW DIRECTION

MAX. SKEW ANGLE IS 1°-15'



a = COLUMN DIAMETER PLUS X

$$X \approx 76.0'(\sin 1°15') \approx 76.0'(\tan 1°15') = 1.66'$$

$$a = 5.0 + 1.66' = 6.66'$$

USE 9.0' TO COMPENSATE FOR DEBRIS

$$\ell/a = 76/9 = 8.44$$

MULTIPLIER FACTOR FOR SCOUR

ANGLE OF SKEW	$\ell/a = 8$	$\ell/a = 8.44$	$\ell/a = 12$
0°	1.00	1.00	1.00
1°-15'	1.08	1.09	1.13
15°	2.00		2.50

ESTIMATED SCOUR = 15.18 (1.09) = 16.55'

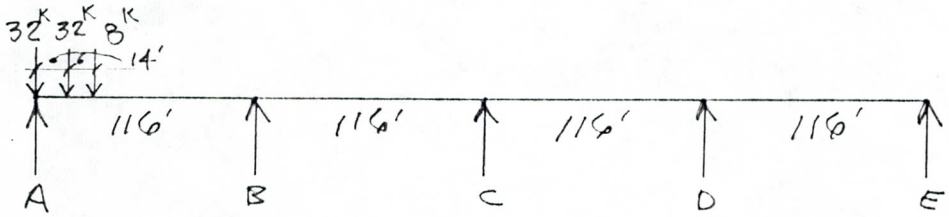
ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040 SHT. V-1 OF 6
DESC. 75 TH AVE MCFCD BY AGD
DATE JULY 18, 1984
TSR

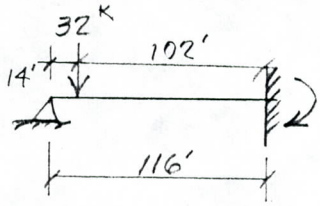
VERIFICATION OF COMPUTER PRINT-OUT FOR SHEAR & MOMENT

IMPACT FACTOR

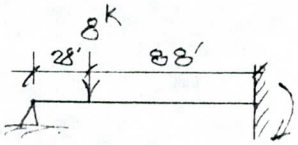
$$I = \frac{50}{116+125} = 0.207$$



FIXED END MOMENT



$$M = \frac{(32)(14)(102)(14+116)}{2(116)^2} = 220.73 \text{ K-FT}$$

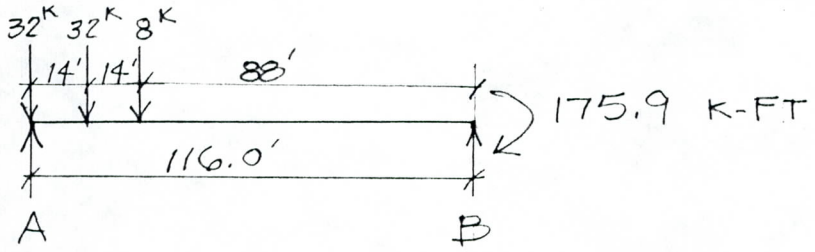


$$M = \frac{(8)(28)(88)(28+116)}{2(116)^2} = 105.47 \text{ K-FT}$$

326.20 K-FT

	A	B		C		D		E
		.43	.57	.50	.50	.57	.43	
F.E.M.	0	+326.2	0	0	0	0	0	0
BAL		-140.3	-185.9					
C.O.				+93.0				
				-46.5	-46.5			
			+23.3			+23.3		
		-10.0	-13.3			-13.3	-10.0	
				+6.7	+6.7			
				-6.7	-6.7			
TOTAL	0	+175.9	-175.9	+46.5	-46.5	+10.0	-10.0	0

VERIFICATION OF COMPUTER PRINT-OUT FOR SHEAR & MOMENT



$\Sigma M_B = 0 \quad +\circlearrowleft$

$$175.9 + 116 V_A = 88(8) + 102(32)$$

$$V_A = 32.69^K$$

$$R_A = (32.69 + 32)\,1.207 = 78.08 \text{ K}$$

NOTE: THIS IS THE SAME PRINT-OUT VALUE FOR MAXIMUM SHEAR INCLUDING IMPACT FOR AN AASHTO TRUCK POSITIONED AT THE LEFT END OF MEMBER 1.

ROYDEN
ENGINEERING
COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040 SHT. V.3 OF 6
DESC. 75 TH AVE MCFCD BY AGD
DATE JULY 18, 1984
T3R

00057:0LL NO. 1. NEGATIVE LIVE LOAD MOMENT ENVELOPE AND ASSOCIATED SHEARS

	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
00058:0MEM											
00059: NO											
00060:0 1	0.	-77.	-154.	-231.	-308.	-385.	-462.	-539.	-616.	-998.	-1711.
00061: SHEAR	0.0	-6.6	-6.6	-6.6	-6.6	-6.6	-6.6	-6.6	-6.6	-26.3	-72.6
00062:0 2	-1712.	-1039.	-753.	-689.	-665.	-640.	-616.	-591.	-628.	-899.	-1546.
00063: SHEAR	70.1	20.9	11.0	2.1	2.1	2.1	2.1	2.1	-9.0	-43.5	-67.1
00064:0 3	-1546.	-899.	-628.	-591.	-616.	-640.	-665.	-689.	-753.	-1039.	-1712.
00065: SHEAR	67.1	43.5	9.0	-2.1	-2.1	-2.1	-2.1	-2.1	-11.0	-20.9	-70.1
00066:0 4	-1712.	-998.	-616.	-539.	-462.	-385.	-308.	-231.	-154.	-77.	0.
00067: SHEAR	72.6	26.3	6.6	6.6	6.6	6.6	6.6	6.6	6.6	6.6	0.0

00068:1ACCOUNT PAGE 6
00069: -- E N G I N E E R I N G C O M P U T E R C O R P O R A T I O N -- TUE, JUL 17 1984

00070:0LL NO. 1. POSITIVE LIVE LOAD MOMENT ENVELOPE AND ASSOCIATED SHEARS

	LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
00071:0MEM											
00072: NO											
00073:0 1	0.	780.	1315.	1622.	1748.	1706.	1518.	1181.	735.	304.	209.
00074: SHEAR	0.0	67.3	56.7	40.8	30.9	-38.5	-47.7	-56.1	-63.5	-30.4	1.8
00075:0 2	209.	291.	779.	1152.	1376.	1432.	1360.	1123.	743.	451.	441.
00076: SHEAR	-9.0	28.4	58.5	49.2	39.3	-30.3	-40.4	-50.2	-59.4	-22.7	9.6
00077:0 3	441.	451.	743.	1123.	1360.	1432.	1376.	1153.	779.	291.	209.
00078: SHEAR	-9.6	22.7	59.4	50.2	40.4	30.3	-39.3	-49.2	-58.5	-67.0	9.0
00079:0 4	209.	304.	735.	1181.	1518.	1706.	1748.	1622.	1315.	780.	0.
00080: SHEAR	-1.8	30.4	63.5	56.1	47.7	38.5	-30.9	-40.8	-56.7	-67.3	0.0

00081:1ACCOUNT PAGE 7
00082: -- E N G I N E E R I N G C O M P U T E R C O R P O R A T I O N -- TUE, JUL 17 1984

00083:0LL NO. 1. LIVE LOAD SHEAR ENVELOPES AND ASSOCIATED MOMENTS

	1 LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
00084:0MEMBER											
00085:0POS. V	78.1	67.3	56.7	46.5	36.8	27.8	19.6	12.3	6.2	3.1	1.9
00086: MOM.	0.	780.	1315.	1617.	1707.	1611.	1363.	1002.	574.	160.	139.
00087: NEG. V	-7.3	-9.3	-15.0	-23.9	-34.3	-44.3	-53.8	-62.5	-70.3	-77.2	-87.0
00088: MOM.	0.	404.	777.	1384.	1630.	1659.	1497.	1175.	731.	210.	-1253.
00089: RANGE	85.4	76.6	71.6	70.3	71.1	72.1	73.4	74.8	76.5	80.3	88.9

00090:0LL NO. 1. LIVE LOAD SHEAR ENVELOPES AND ASSOCIATED MOMENTS

	2 LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
00091:0MEMBER											
00092:0POS. V	85.4	74.4	64.5	54.9	44.9	35.1	27.6	21.2	16.1	14.0	13.7
00093: MOM.	-1254.	-428.	779.	1140.	1344.	801.	735.	606.	467.	712.	371.
00094: NEG. V	-9.4	-9.7	-12.8	-18.1	-26.1	-35.9	-46.0	-56.0	-65.5	-74.1	-82.6
00095: MOM.	139.	448.	597.	834.	1262.	1381.	1330.	1111.	740.	252.	-1114.
00096: RANGE	94.8	84.1	77.3	73.0	71.0	71.1	73.6	77.2	81.5	88.1	96.3

00097:0LL NO. 1. LIVE LOAD SHEAR ENVELOPES AND ASSOCIATED MOMENTS

	3 LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
00098:0MEMBER											
00099:0POS. V	82.6	74.1	65.5	56.0	46.0	36.0	26.1	18.1	12.8	9.7	9.4
00100: MOM.	-1114.	252.	740.	1111.	1330.	1381.	1262.	557.	366.	447.	139.
00101: NEG. V	-13.7	-14.0	-16.1	-21.2	-27.6	-35.1	-44.9	-54.9	-64.5	-74.4	-85.4
00102: MOM.	371.	712.	716.	906.	1047.	1094.	1344.	1140.	779.	-369.	-1253.
00103: RANGE	96.3	88.1	81.5	77.2	73.6	71.1	71.0	73.0	77.3	84.1	94.8

00104:0LL NO. 1. LIVE LOAD SHEAR ENVELOPES AND ASSOCIATED MOMENTS

	4 LEFT	.1 PT	.2 PT	.3 PT	.4 PT	.5 PT	.6 PT	.7 PT	.8 PT	.9 PT	RIGHT
00105:0MEMBER											
00106:0POS. V	87.0	77.2	70.3	62.5	53.8	44.3	34.3	23.9	15.0	9.3	7.3
00107: MOM.	-1253.	210.	731.	1175.	1497.	1659.	1630.	1384.	777.	404.	0.
00108: NEG. V	-1.9	-3.1	-6.2	-12.3	-19.6	-27.8	-36.8	-46.5	-56.7	-67.3	-78.1
00109: MOM.	139.	327.	574.	1002.	1363.	1611.	1707.	1617.	1315.	780.	0.
00110: RANGE	88.9	80.3	76.5	74.8	73.4	72.1	71.1	70.3	71.6	76.6	85.4

V·4 of 6

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00092:0POS. V     85.4     74.4     64.5     54.9     44.9     35.1     27.6     21.2     16.1     14.0     13.7
00093: MOM.    -1254.    -428.     779.    1140.    1344.     801.     735.     686.     467.     712.     371.
00094: NEG. V     -9.4     -9.7    -12.8    -18.1    -26.1    -35.9    -46.0    -56.0    -65.5    -74.1    -82.6
00095: MOM.      139.     448.     597.     834.    1262.    1381.    1330.    1111.     740.     252.   -1114.
00096: RANGE     94.8     84.1     77.3     73.0     71.0     71.1     73.6     77.2     81.5     88.1     96.3
00097:0LL NO. 1.                          LIVE LOAD SHEAR ENVELOPES AND ASSOCIATED MOMENTS
00098:0MEMBER  3 LEFT    .1 PT    .2 PT    .3 PT    .4 PT    .5 PT    .6 PT    .7 PT    .8 PT    .9 PT    RIGHT
00099:0POS. V    82.6     74.1     65.5     56.0     46.0     36.0     26.1     18.1     12.8      9.7      9.4
00100: MOM.    -1114.     252.     740.    1111.    1330.    1381.    1262.     557.     366.     447.     139.
00101: NEG. V   -13.7    -14.0    -16.1    -21.2    -27.6    -35.1    -44.9    -54.9    -64.5    -74.4    -85.4
00102: MOM.      371.     712.     716.     906.    1047.    1094.    1344.    1140.     779.    -369.   -1253.
00103: RANGE     96.3     88.1     81.5     77.2     73.6     71.1     71.0     73.0     77.3     84.1     94.8
00104:0LL NO. 1.                          LIVE LOAD SHEAR ENVELOPES AND ASSOCIATED MOMENTS
00105:0MEMBER  4 LEFT    .1 PT    .2 PT    .3 PT    .4 PT    .5 PT    .6 PT    .7 PT    .8 PT    .9 PT    RIGHT
00106:0POS. V    87.0     77.2     70.3     62.5     53.8     44.3     34.3     23.9     15.0      9.3      7.3
00107: MOM.    -1253.     210.     731.    1175.    1497.    1659.    1630.    1384.     777.     404.       0.
00108: NEG. V    -1.9     -3.1     -6.2    -12.3    -19.6    -27.8    -36.8    -46.5    -56.7    -67.3    -78.1
00109: MOM.      139.     327.     574.    1002.    1363.    1611.    1707.    1617.    1315.     780.       0.
00110: RANGE     88.9     80.3     76.5     74.8     73.4     72.1     71.1     70.3     71.6     76.6     85.4
00111:1ACCOUNT                                                                                   PAGE   8
00112:  --E N G I N E E R I N G   C O M P U T E R   C O R P O R A T I O N--  TUE, JUL 17 1984
00113:0LL NO. 1.                          LIVE LOAD SUPPORT RESULTS
00114:0                          MAX.  AXIAL  LOAD                  MAX. LONGITUDINAL MOMENT
00115:                         AXIAL    ------MOMENT------          AXIAL    ------MOMENT------
00116:                         LOAD      TOP        BOT.            LOAD      TOP        BOT.
00117:0SUPPORT JT.  1
00118:            POSITIVE      78.1       0.         0.             0.0       0.         0.
00119:            NEGATIVE      -7.3       0.         0.             0.0       0.         0.
00120:0SUPPORT JT.  2
00121:            POSITIVE     141.2       0.         0.             0.0       0.         0.
00122:            NEGATIVE     -11.2       0.         0.             0.0       0.         0.
00123:0SUPPORT JT.  3
00124:            POSITIVE     133.8       0.         0.             0.0       0.         0.
00125:            NEGATIVE     -24.4       0.         0.             0.0       0.         0.
00126:0SUPPORT JT.  4
00127:            POSITIVE     141.2       0.         0.             0.0       0.         0.
00128:            NEGATIVE     -11.2       0.         0.             0.0       0.         0.
00129:0SUPPORT JT.  5
00130:            POSITIVE      78.1       0.         0.             0.0       0.         0.
00131:            NEGATIVE      -7.3       0.         0.             0.0       0.         0.
00132:0    THE RATIO OF SUBSTRUCTURE / SUPERSTRUCTURE LOADING IS 1.000
00133:1ACCOUNT                                                                                   PAGE   9
00134:  --E N G I N E E R I N G   C O M P U T E R   C O R P O R A T I O N--  TUE, JUL 17 1984
00135:0                                   "APPROXIMATE QUANTITY"
00136:                                    ======================
00137:0                          *****  CONCRETE SUPER              0 C.Y.  *****
00138:
00139:                           *****  CONCRETE SUB                0 C.Y.  *****
00140:0                     THE SUPERSTRUCTURE CONCRETE QUANTITY IS BASED ON THE UNIT
00141:                      WEIGTH OF CONCRETE SUPPLIED ON THE FRAME DESCRIPTION CARD.
00142:                      IT ASSUMES THAT ALL THE DEAD LOAD IS GIVEN IN TRIAL 0.
00143:
00144:                      THE CONCRETE SUBSTRUCTURE QUANTITY IS BASED ON TRIAL 0 ONLY.
00145:
00146:                      THE P/S QUANTITIES FOR STRAND ONLY ARE FOR EACH TRIAL THAT
00147:                      WAS ENTERED AND IN THAT ORDER.  STRAND USE IS BASED ON THE
00148:                      LENGTH FROM ANCHOR TO ANCHOR.
00149:1END OF JOB
BOTTOM
$ QUIT
OK, L

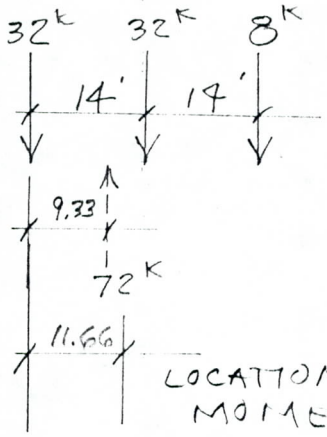
USER=(CCC2)CROY   3  OWNER
```

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040 SHT. V-5 OF 6
DESC. 75TH AVE MCFCD BY AGD
DATE JULY 20, 1987
TSR

VERIFICATION OF COMPUTER PRINT-OUT FOR SHEAR AND MOMENT

AASHTO TRUCK



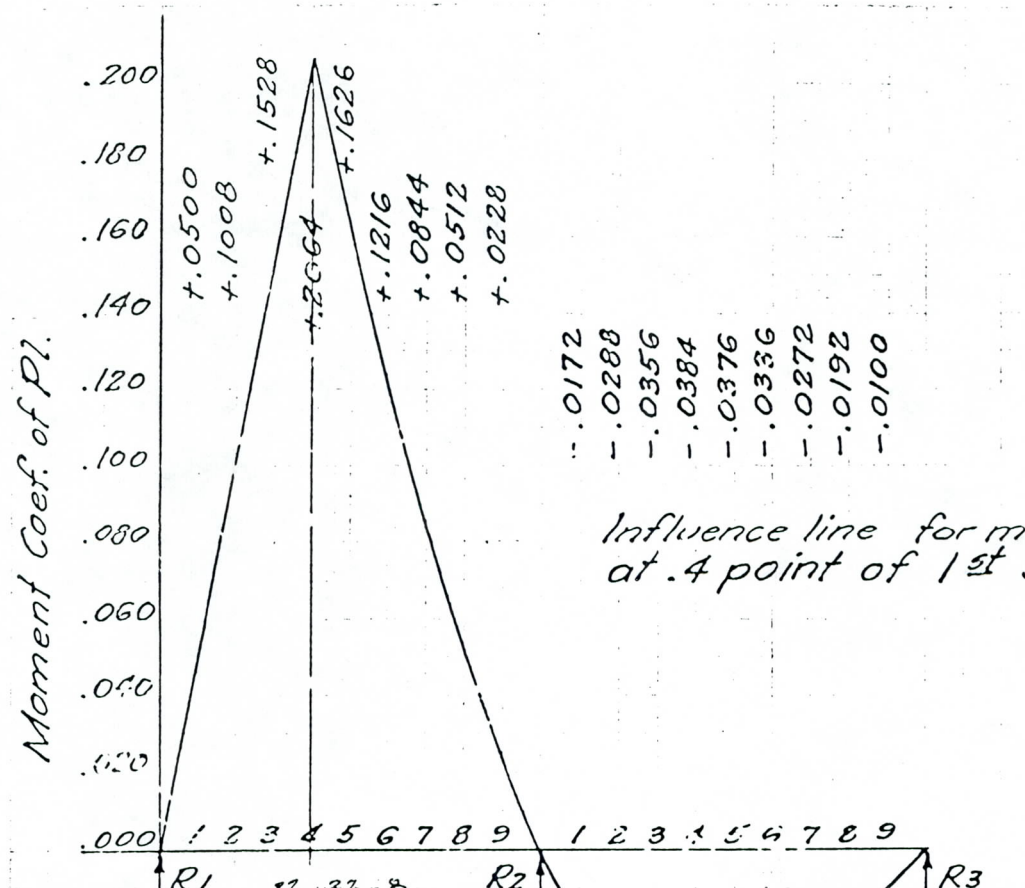
LOCATION FOR POINT OF MAX. MOMENT ON INFLUENCE LINE

$(32)(116)(.1607) = 596.5$ K-FT
$(32)(116)(.1977) = 733.9$
$(8)(116)(.1422) = 132.0$

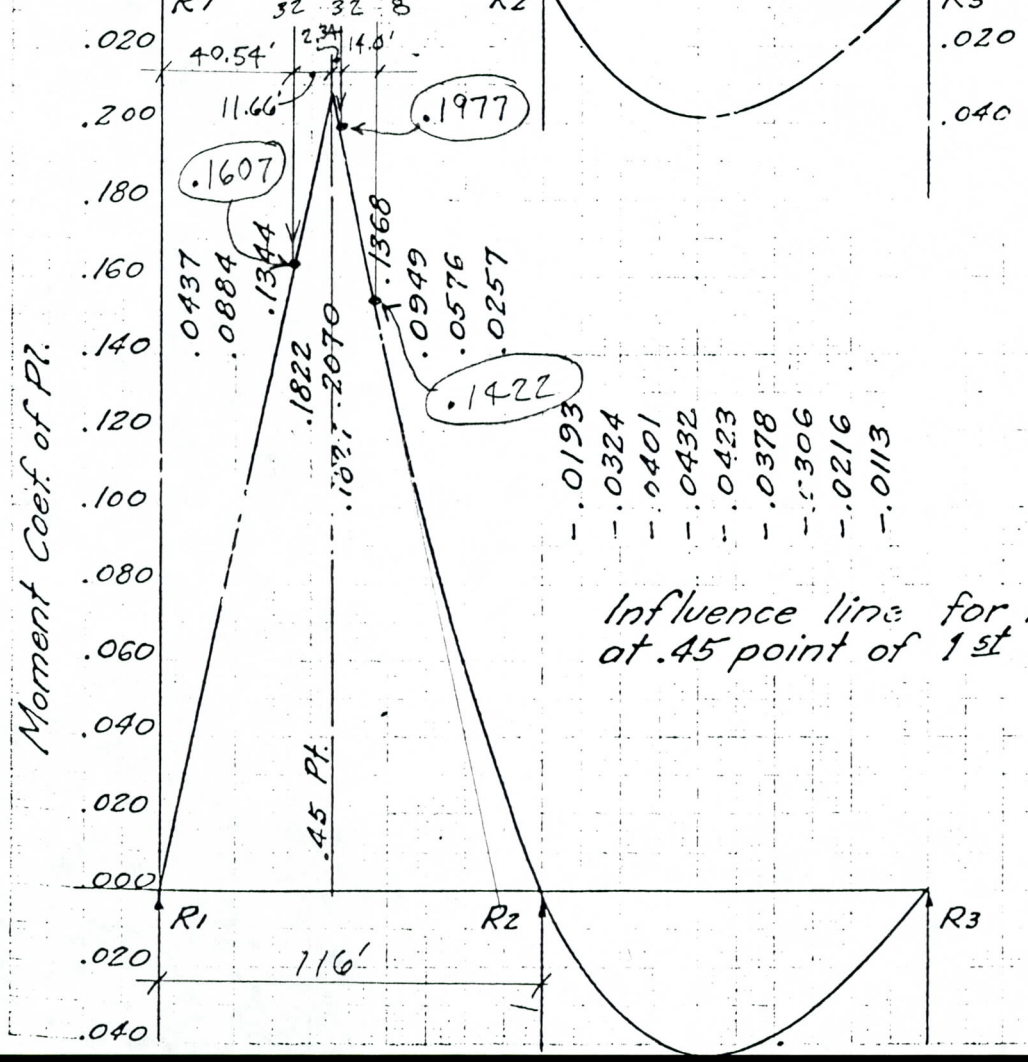
1462.4 K-FT
× 1.207 FOR IMPACT
1765.1 K-FT

THIS MAX POSITIVE MOMENT WOULD BE FOR TWO CONT. SPANS.

THEREFORE THE MAXIMUM MOMENT AT .4PT OF THE END SPAN FOR FOUR CONTINUOUS SPANS OF 1748.1 KIP FEET IS ACCEPTABLE.



Influence line for moment at .4 point of 1st span.



Influence line for moment at .45 point of 1st span.

DECK DESIGN

SPAN LENGTH 1.5.23 (F)

L = CLEAR SPAN PLUS DEPTH OF MEMBER

TYPE VI GIRDERS @ 10'-4" CENTER
TOP FLANGE WIDTH IS 3'-6"
8" CONC. DECK

$$L = (10'-4") - (3'-6") + (0'-8") = 7'-6"$$

$$= 90" \text{ SPAN}$$

IMPACT FACTOR 1.2.12 (C)

$$I = \frac{50}{L+125} = \frac{50}{7.50+125} = 0.377$$

USE .30

TIRE CONTACT AREA 1.3.8

$$P = (1.3)(16000\#) = 20800\#$$

$$\text{AREA} = 0.01P = 208 \text{ IN}^2$$

RATIO OF TIRE LENGTH TO WIDTH = 1/2.5

$$\text{AREA} = (2.5W)W = 208$$

$$W = 9.12 \text{ IN}$$

$$\therefore L = 2.5W = 22.8"$$

MINIMUM DECK THICKNESS 1.5.40

$$\frac{S+10}{30} = \frac{7.58+10}{30} = 0.586 \text{ FT}$$

$$= 7.032"$$

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040 SHT. D-2 OF 5
DESC. MCFCD 75TH AVE BY AGD
DATE JULY 25, 1987
TSR

DECK DESIGN

BENDING MOMENT 1.3.2 (C)

CASE "A"

LIVE LOAD MOMENT

$$\left(\frac{S+2}{32}\right) P_{20} = \left(\frac{7.50+2}{32}\right) 16000 \underbrace{(1.3)}_{\text{IMPACT}} \underbrace{(.8)}_{\text{CONTINUITY}}$$

M_{LL} = 4,940.0 FOOT POUNDS PER FOOT-WIDTH OF SLAB

DEAD LOAD MOMENT PER FOOT WIDTH

8" CONC. DECK	=	100.0 PSF
2" FUTURE A.C.	=	25.0
		125.0 PSF

$$M_{DL} = \frac{1}{8}(125.0)(7.50)^2 \underbrace{(.8)}_{\text{CONTINUITY}} = 703.13 \text{ F-\#}$$

DESIGN MOMENT

$$M_{LL} + M_{DL} = 4{,}940 + 703.13$$
$$= 5643.13 \text{ F-\#}$$

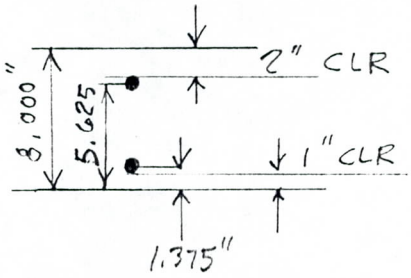
DECK DESIGN

CONCRETE DESIGN (SERVICE LOAD DESIGN)

ALLOWABLE STRESSES

$$f_c = 0.40 f'_c = .4(4000) = 1600 \text{ PSI}$$

$$f_s = 24000 \text{ PSI FOR GRADE 60 REBAR}$$



$$E_c = 57000\sqrt{f'_c} = 3{,}605{,}000 \text{ PSI}$$

$$E_s = 29{,}000{,}000 \text{ PSI}$$

#6 T&B @ 9"

$$n = \frac{E_s}{E_c} = 8.04 \quad \text{USE } 8$$

$$\rho = \frac{A_s}{bd} = \frac{.44}{(9)\,5.625} = 0.00869$$

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n \; ; \; j = 1 - \frac{k}{3}$$

$$k = .297 \qquad j = .901$$

CAPACITY $= A_s f_s j d$

$$(.44)\left(\tfrac{12}{9}\right)(24000^{\#})(.901)(5.625'') = 71{,}359 \text{ IN-\#}$$

$$= 5946 \text{ F-\# PER FOOT}$$

THIS IS GREATER THAN 5643 F-#

USE #6 @ 9" T&B

DECK DESIGN

DEVELOPEMENT LENGTH OF #6 AND #5 BARS

1.5.14

#6 TOP BARS $l_d = \frac{0.04(.44)60{,}000}{\sqrt{4000}} = 16.7''$

$16.7(1.4)(.8) = 18.7''$

TOP REIN. LATERAL SPACING MORE THAN 6"

#5 TOP BARS $l_d = \frac{0.04(.31)60{,}000}{\sqrt{4000}} = 11.76''$

$11.76(1.4)(.8) = 13.17''$

TENSION SPLICES 1.5.22

USE CLASS C SPLICES TO ALLOW FOR EASIEST CONSTRUCTION

#6 $18.7''(1.7) = 31.79''$
LAP 2'-6"

#5 $13.17(1.7) = 22.39''$
LAP 2'-0"

DECK DESIGN

DISTRIBUTION REINFORCEMENT 1.3.2 (D)

SPECIFIED DIST. FOR MIDDLE HALF OF SPAN

$$\text{PERCENTAGE} = \frac{220}{\sqrt{S}} = \frac{220}{\sqrt{7.50}} = 80.3$$

USE MAX. OF 67%

FOR OUTSIDE QUARTERS USE

50% OF 67% = 33.5%

AREA OF STEEL PER FOOT IN CENTER HALF

$$\left(\frac{5643}{5946}\right)\left(\frac{12}{9}\right)(.44) \times .67 = .37 \text{ IN}^2$$

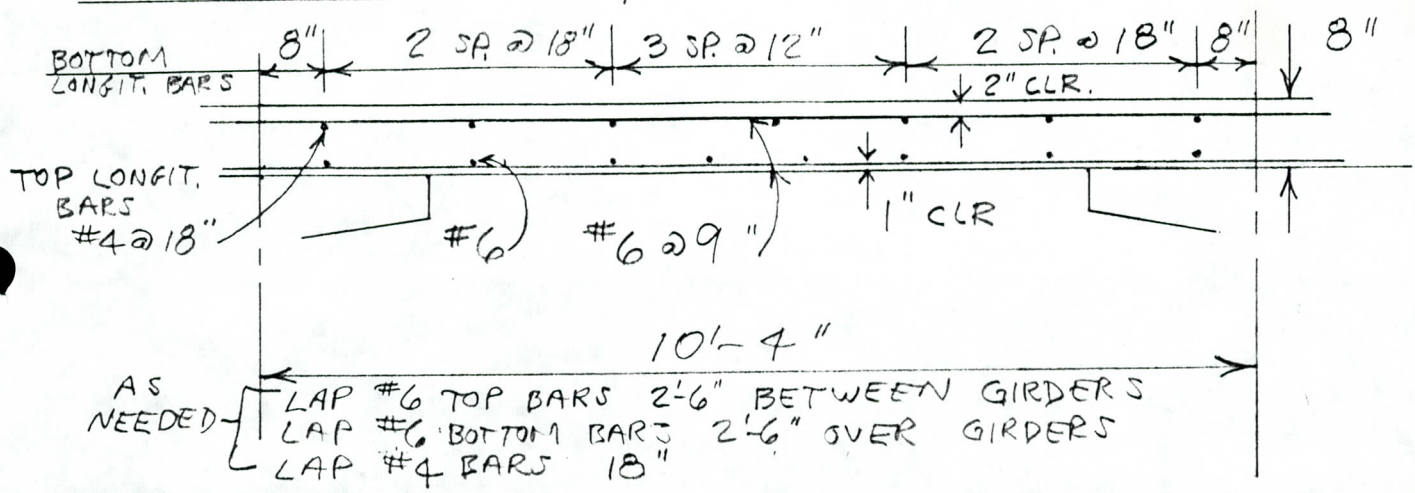
USE #6 @ 12"

AREA OF STEEL PER FOOT IN OUTER QUARTERS

$$\left(\frac{12}{9}\right) \cdot .44 \times .335 = .197 \text{ IN}^2$$

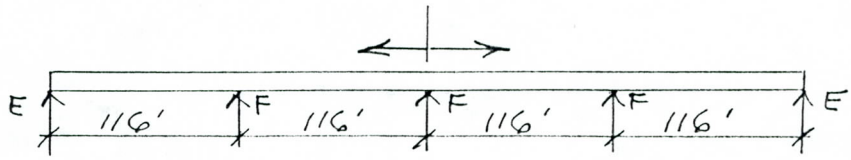
USE #6 @ 18"

DESIGN SUMMARY



EXPANSION JOINT DESIGN

THERMAL EXPANSION: AASHTO 1.2.15



Δ_T = CHANGE DUE TO TEMPERATURE

$= 2 \text{ SPANS} \times 116' \times 100\,F^\circ \times .000006 = 0.1392 \text{ FEET}$

$= 1.67 \text{ INCHES}$

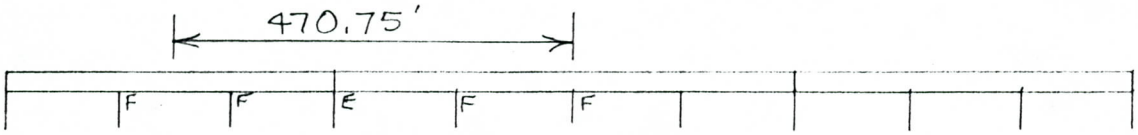
LENGTH CHANGE DUE TO CREEP:

(A.) REFERENCE TO A SIMILAR TYPE BRIDGE

COUNTRY CLUB ROAD IN MESA OVER THE SALT RIVER

8" DECK f'_c 4500 PSI
7'-4" GIRDER SPACING
GIRDER SPANS OF 134'-6"

EXPANSION JOINT DATA:
12/82 - CONSTRUCTION COMPLETION
3" DESIGN OPENNING @ 50°F
12/83 - 4½" MEASURED OPENNING @ 50°F



IN ONE YEAR 1½" OF CREEP OCCURRED IN 470.75' OF SUPERSTRUCTURE

ESTIMATE OF TOTAL CREEP IN COUNTRY CLUB RD. BRIDGE

$$\frac{1.5''}{.82} = 1.829''$$

EXPANSION JOINT DESIGN

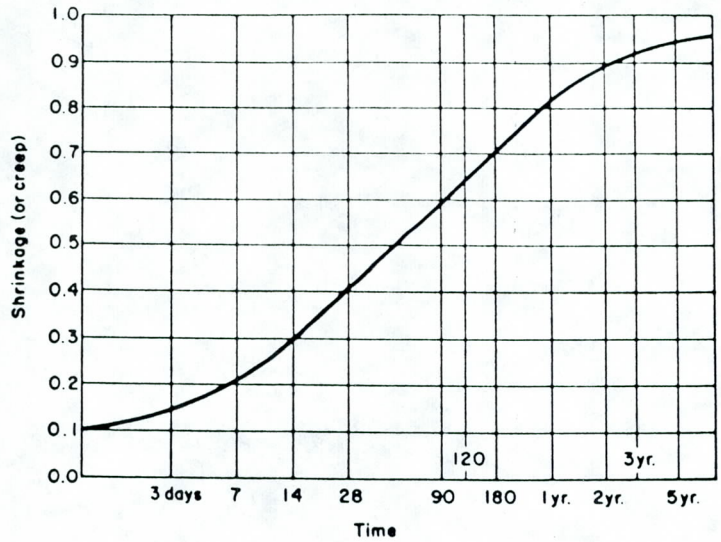


Fig. 8. Proportion of final shrinkage or creep vs. time

CHART FROM PCA ENG. BULLETIN "DESIGN OF CONT. HIGHWAY BRIDGES WITH PRECAST, PRESTRESSED CONC. GIRDERS"

Δ_c = CHANGE IN LENGTH DUE TO CREEP

$$= \frac{1.829''}{470.75} \times (2 \times 116) = 0.90 \text{ INCHES}$$

75 TH AVE MCFCD

(B.) REFERENCE TO CALTRANS MEMO

$$\frac{.12 \times 2}{100} \times (2 \times 116) = 0.5568''$$

USE 0.90 INCHES

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75TH AVE MCFCD

BY AGD
DATE AUG 21, 1984

75K

EXPANSION JOINT DESIGN

USE A MODIFIED ADOT RJ-4 JOINT
" WD-600 COMPRESSION SEAL

MIN. JOINT OPENNING = 2.250 "
100°F TEMP DIFF. = 1.670 "
EST. OF P/S CREEP = .900 "
TOTAL OPENNING = 4.82 "

THIS IS LESS THAN THE MAXIMUM ALLOWED OPENNING OF 5.10 INCHES

TEMP.	INSTALLATION OPENNING
120	2.470
110	2.637
100	2.804
90	2.971
80	3.138
70	3.305
60	3.472
50	3.639
40	3.806
30	3.973
20	4.140

2. Movement Rating (M.R.) Calculations:

The Joint Movements Calculations sheet is to be used to calculate the M.R. for all bridge expansion joints. This sheet is to be placed in the preliminary report for forwarding to Construction. The example illustrates the necessary information to be completed by Design.

The M.R. is equal to the total anticipated movement from widest to narrowest opening of a joint. This is equal to the total thermal movement plus any anticipated shortening. The factors used to calculate the thermal movement are those published in Research and Development's Annual Movement Study of Bridge Deck Expansion Joints and are commonly accepted values. The temperature range is the extreme ambient temperature range of the area which is given in the preliminary report. Research and Development has found that the structures temperature range is very close to the ambient range, therefore, no adjustment will be made to the ambient range. The factors used to calculate the anticipated shortening are about one-half the total expected creep and shrinkage. The reason is that approximately one-half of total anticipated shortening should be out of the structure at the time the joint groove widths are determined.

TYPE OF STRUCTURE	Coefficient of Expansion: Movement/Unit Length/Degree Fahrenheit	Anticipated Shortening: inches/100 ft Contributory Length
Steel	.0000065	0.00
Concrete (conventional)	.0000053	0.06
Concrete (pretensioned)	.0000053	0.12
Concrete (post tensioned)	.0000053	0.50

Note that post tensioned concrete structures are expected to initially shorten about 0.50 in./100 ft. due to stressing. The total long-term shortening is anticipated to be 1.00 in./100 ft. The difference between the long-term shortening (1.00 inch) and the initial shortening is equal to 0.50 in./100 ft. This is the value shown on the "Joint Movements Calculations" form as "Anticipated Shortening for Post Tensioned Concrete Structures."

COMPRESSION SEALS

Dimension Table

Bold numbers indicate inches
Light numbers indicate millimeters

MODEL NO.	MOVEMENT	SEAL SIZE WIDTH	SEAL SIZE HEIGHT	GROOVE WIDTH MIN. A	GROOVE WIDTH MAX. A	GROOVE DEPTH B	INSTALL. WIDTH*
WA-175	.800 20.32	1¾ 44.45	2 50.80	.687 17.45	1.487 37.78	2⅝ 66.68	1 25.40
WA-200	.825 20.96	2 50.80	2 50.80	.875 22.23	1.70 43.18	2⅝ 66.68	1¼ 31.75
WA-250	1.125 28.58	2½ 63.50	2¾ 69.85	1.000 25.40	2.125 53.98	3 9/16 90.49	1½ 38.10
WA-300	1.425 36.20	3 76.20	3⅜ 85.73	1.125 28.58	2.55 64.77	4 5/16 109.54	1¾ 44.45
WA-350	1.600 40.614	3½ 88.90	3½ 88.90	1.375 34.93	2.975 75.57	4 7/16 112.71	2¼ 57.15
WA-400	1.775 45.09	4 101.60	4⅜ 111.13	1.625 41.28	3.40 86.36	5 127.00	2½ 63.50
WA-500	2.563 65.10	5 127.00	5 127.00	1.687 42.85	4.25 107.95	5 15/16 150.81	3 76.20
WA-600	3.100 78.74	6 152.40	6 152.40	2.00 50.80	5.10 129.54	7¾ 196.85	3½ 88.90
WB-437	.207 5.26	7/16 11.11	½ 12.7	.153 3.89	.360 9.14	1 25.40	¼ 6.35
WB-562	.253 6.43	9/16 14.29	⅝ 15.88	.197 5.00	.450 11.43	1⅛ 28.58	5/16 7.94
WB-687	.310 7.87	11/16 17.46	11/16 17.46	.240 6.10	.550 13.97	1¼ 31.75	⅜ 9.53
WB-812	.366 9.30	13/16 20.64	13/16 20.64	.284 7.21	.650 16.51	1½ 38.10	7/16 11.11
WB-1000	.450 11.43	1 25.40	1 25.40	.350 8.90	.800 20.32	1¾ 44.45	½ 12.70
WB-1250	.563 14.30	1¼ 31.75	1⅛ 28.58	.437 11.10	1.00 25.40	2 50.80	⅝ 15.875
WC-1250	.563 14.30	1¼ 31.75	1 25.40	.437 11.10	1.00 25.40	2 50.80	⅝ 15.88
WC-1625	.732 18.59	1⅝ 41.28	1 25.40	.568 14.43	1.30 33.02	2 50.80	¾ 19.05
WC-1750	.919 23.34	1¾ 44.45	1⅛ 28.58	.568 14.43	1.487 37.77	2 50.80	⅞ 22.23
WC-2000	.975 24.77	2 50.80	1½ 38.10	.625 15.88	1.60 40.64	2 50.80	1 25.40
WD-125	.540 13.72	1¼ 31.75	1 5/16 33.34	.520 13.21	1.062 26.97	1¾ 44.45	¾ 19.05
WD-1625	.819 20.80	1⅝ 41.28	1⅝ 41.28	.562 14.27	1.381 35.08	2⅛ 53.98	⅞ 22.23
WD-175	.662 16.82	1¾ 44.45	2 50.80	.825 20.96	1.487 37.77	2½ 63.50	1 25.40
WD-200	.875 22.23	2 50.80	2 50.80	.825 20.96	1.70 43.18	2½ 63.50	1¼ 31.75
WD-200L	1.075 27.31	2 50.80	2 50.80	.625 15.88	1.70 43.18	2½ 63.50	1 25.40
WD-225	1.095 27.81	2¼ 57.15	2¾ 69.85	.825 20.96	1.92 48.77	3½ 88.90	1⅜ 34.93
WD-225L	1.084 27.53	2¼ 57.15	2 5/16 52.39	.828 21.03	1.913 48.59	2⅝ 66.68	1⅜ 34.93
WD-250	1.095 27.81	2½ 63.50	2¾ 69.85	1.030 26.16	2.125 53.98	3½ 88.90	1½ 38.10
WD-300	1.382 35.10	3 76.20	3⅜ 85.73	1.168 29.67	2.55 64.77	4¼ 107.95	2 50.80
WD-350	1.812 46.01	3½ 88.90	3½ 88.90	1.168 29.67	2.98 75.69	4¼ 107.95	2¼ 57.15
WD-400	1.750 44.45	4 101.60	4⅜ 111.13	1.650 41.91	3.40 86.36	5 127.00	2½ 63.50
WD-450	2.165 54.99	4½ 114.30	4½ 114.30	1.66 42.16	3.825 97.16	5⅜ 136.53	2½ 63.50
WD-500	2.600 66.04	5 127.00	5¼ 133.35	1.650 41.91	4.25 107.95	6¼ 158.75	3 76.20
WD-600	2.850 72.39	6 152.40	6 152.40	2.250 57.15	5.10 129.54	7¾ 196.85	3½ 88.90
WE-200	.811 20.60	2 50.80	1½ 38.10	.889 22.58	1.70 43.18	1⅞ 47.63	1 25.40
WE-225	1.20 30.48	2¼ 57.15	1½ 38.10	.712 18.08	1.912 48.56	1⅞ 47.63	1½ 38.10
WE-250	1.065 27.05	2½ 63.50	2 50.80	1.060 26.92	2.125 53.98	2⅜ 60.33	1½ 38.10
WE-300	1.125 28.56	3 76.20	2 5/32 54.77	1.425 36.20	2.55 64.77	2 17/32 64.29	2 50.80
WE-400	2.158 54.81	4 101.60	2 13/16 71.44	1.242 31.55	3.40 86.36	3 3/16 80.96	2⅜ 60.33
WE-500	2.825 71.76	5 127.00	3¼ 82.55	1.425 36.20	4.25 107.95	3⅝ 92.08	2⅞ 73.03
WE-600	3.263 82.88	6 152.40	4 101.60	1.837 46.66	5.10 129.54	4⅜ 111.13	3½ 88.90

PP/product presentation

Serving to produce a leakproof seal through the action of the configuration's compressive forces on the structure, the compression seal line encompasses seven series of web profiles.

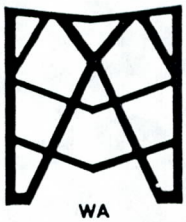
This large number of configurations and movement ratings define a product versatility, meeting the needs of many different project details and specifications.

For heavy traffic and high load applications, the WA, WD, and WG offer durability with movement capabilities to 3⅛".

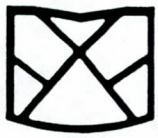
The WB and WC are low-stress seals for small movements to 1"; the WB having a typical use in a construction-contraction joint.

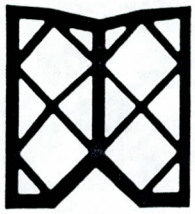
The WE and WF feature low stress concourse seals which present a smooth, even surface to foot or vehicular traffic.

CP/components, parts


WA

WB


WC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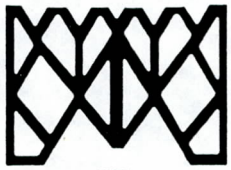
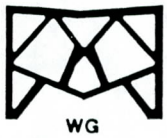
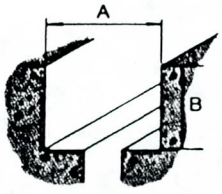
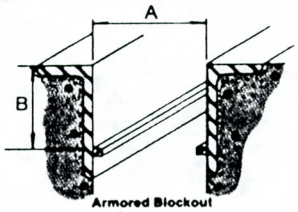

WE

WF


WG



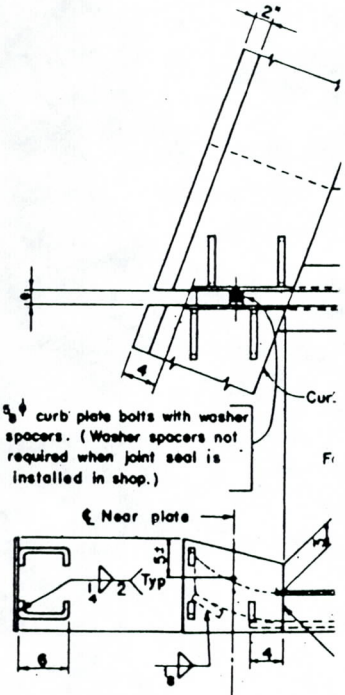
Cast Blockout



Armored Blockout

MODEL NO.	MOVEMENT	SEAL SIZE WIDTH	SEAL SIZE HEIGHT	GROOVE WIDTH MIN. A	GROOVE WIDTH MAX. A	GROOVE DEPTH B	INSTALL. WIDTH*
WF-250	1.000 25.40	2½ 63.50	1 5/16 33.34	1.125 28.58	2.125 53.98	1⅞ 47.63	1½ 38.10
WF-325	1.500 38.10	3¼ 82.55	1¾ 44.45	1.250 31.75	2.750 69.85	2 9/16 58.74	2 50.80
WF-400	2.40 60.96	4 101.60	2 50.80	1.000 25.40	3.400 86.36	2¾ 69.85	2 50.80
WG-200	.950 24.13	2 50.80	1¼ 31.75	.75 19.05	1.70 43.18	1½ 38.10	1¼ 31.75
WG-300	1.380 35.05	3 76.20	2½ 63.50	1.17 29.72	2.55 64.77	3⅛ 79.38	1⅞ 47.63
WG-325	1.575 40.01	3¼ 82.55	3⅛ 79.38	1.185 30.10	2.76 70.10	3¾ 95.25	2 50.80
WG-400	1.750 44.45	4 101.60	2½ 63.50	1.65 41.91	3.40 86.36	3⅛ 79.38	2 9/16 65.08
WG-500	2.370 60.20	5 127.00	3¼ 82.55	1.88 47.75	4.25 10.95	4 101.60	3 1/16 77.79

*Indicates min. width at which seal is easily installed.



$\frac{5}{8}\phi$ curb plate bolts with washer spacers. (Washer spacers not required when joint seal is installed in shop.)

℄ Near plate

$\frac{3}{4}\phi$ holes for $\frac{5}{8}\phi$ curb plate bolts. (Centered on near plate this end, far plate other end.) Remove bolts before pouring curb or barrier concrete.

Erection angles d @ 2'-0
to guard angles for $\frac{5}{8}\phi$
For removal see notes.

$\frac{5}{8}\phi$ x0'-10 bars. Blacksmith bend
Alternate on opposite legs @
9" ctrs., unless noted otherwise.

2x2 or
Polychlorop

REVISIONS						
DRAWN	J.F.B.	8-71		DESCRIPTION OF REVISIONS	DATE	
DESIGN	W.H.B.	8-71		Joint Plate Bolted		
CHECKED	R.R.	8-71		Per Spec. Note	11-5-72	

ELASTOMERIC BEARINGS 1.12.1

EXPANSION BEARINGS:

MAX. DEAD LOAD:

$$2.942 \text{ KLF} \times \frac{116}{2} \text{ FT} = 170.64 \text{ KIPS}$$

MAX. LIVE LOAD WITHOUT IMPACT:

$$78.1^{K} \left(\frac{1}{1.207}\right) \left(\frac{10.33}{11.00}\right) = 60.77 \text{ KIPS}$$

REMOVE IMPACT

LANES PER GIRDER

LL + I FOR ONE LANE FROM MAX. LL ENVELOPE

REQUIRED BEARING AREA FOR D.L. PLUS L.L.:

$$\frac{170.64 + 60.77}{.8} = 289.26 \text{ IN}^2$$

FOR A WIDTH OF 26 INCHES

$$L = \frac{289.26}{26} = 11.125 \text{ IN}$$

REQUIRED BEARING AREA FOR D.L. ONLY

$$\frac{170.64 \text{ KIPS}}{.5} = 341.23 \text{ IN}^2$$

FOR A WIDTH OF 26 INCHES

$$L = \frac{341.23}{26} = 13.126 \text{ IN}^2$$

USE 26" x 16" PADS

ELASTOMERIC BEARINGS

TOTAL ANTICIPATED MOVEMENT IS 2.57"

TOTAL EFFECTIVE ELASTOMER THICKNESS (T).

$$T = \frac{2.57}{.5} = 5.11 \text{ INCHES}$$

DEFLECTION OF 5¼" THICK PAD:

DUROMETER HARDNESS OF 60

$$\text{SHAPE FACTOR} = \frac{LW}{2t(L+W)}$$

$$= \frac{(16)(26)}{2(0.50)(16+26)} = 9.90$$

COMPRESSIVE STRESS UNDER D.L. + L.L.

$$\frac{170.64 + 60.77}{26 \times 16} = 0.556 \text{ KSI}$$

AT ABUTMENTS USE:

26" x 16" x 5¼" THK
FIBERGLASS REINFORCED IMBEDS
ON ½" CENTERS

FIXED BEARINGS

NO HORIZ. MOVEMENT.

AT PIERS USE:

26" x 16" x 1" THK
FIBERGLASS REINFORCED IMBEDS
ON ½" CENTERS

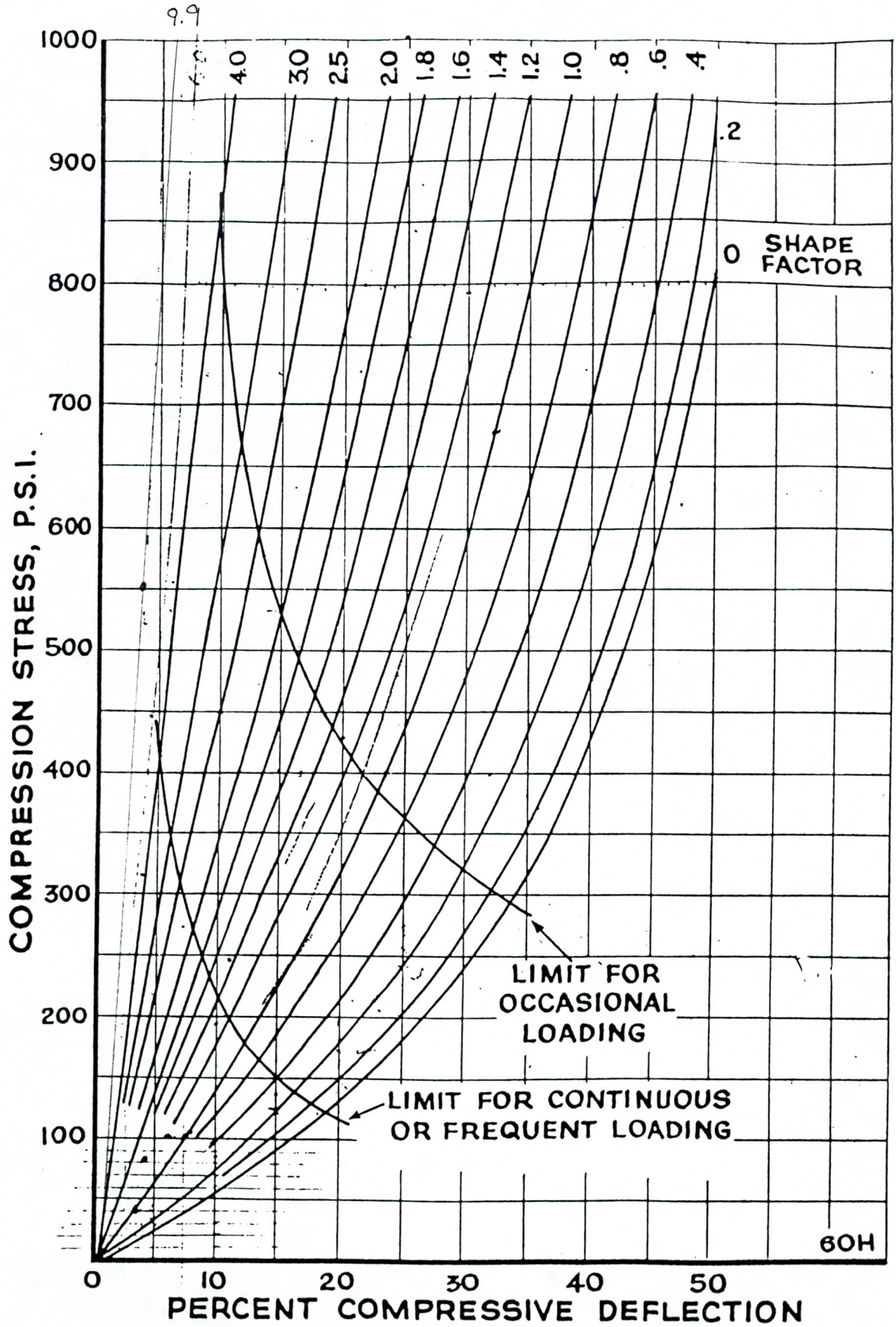


FIG. 5-13 Compression Stress-Strain Curves for 60 Durometer Rubber

ROYDEN
ENGINEERING
COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040 SHT. SS-1 OF 2
DESC.
BY AGD
DATE OCT. 4, 1984
TSR

DUCTILE IRON PIPE SUPPORT SPACING

12" DIA. WATER PIPE

ASSUME $f_b = 20$ KSI

FOR PIPE THICKNESS OF .33 INCHES
OUTSIDE DIA 13.20 INCHES

PIPE WT. PER LIN. FOOT:

CIRCUMFERENCE $= \pi(13.20 - .33) = 40.43''$
VOL. OF IRON PER LIN. FT.:

$$40.33 \times .33 \times 12 = 159.71 \text{ IN}^3$$

$$\text{WT.} = \frac{159.71}{1728} \times 500 \text{ PCF} = 46.21 \text{ PLF}$$

SECTION MODULUS:

$$S = .098\left[\frac{d^4 - d_1^4}{d}\right]$$

$$= [.098]\,\frac{13.20^4 - 12.54^4}{13.20} = 41.81 \text{ IN}^3$$

TOTAL LOAD PER FOOT:

PIPE $= 46.21$ PLF
WATER $\frac{62.4}{1728} \times \pi \times \frac{12.54^2}{4} \times 12 = 53.52$
99.73 PLF

$$f_{b\,MAX} = 20{,}000 = \frac{M}{S} + \frac{P}{A}$$

P = TOTAL FORCE IN PIPE DUE
TO INTERNAL PRESSURE

$$P = \pi\frac{(12.54)^2}{4} \times 350 \text{ PSI} = 43{,}226^{\#}$$

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC.

SS-2 SHT. 2 OF 2
BY AGD
DATE OCT. 4, 1984
TSR

PIPE SUPPORT SPACING

A = AREA OF PIPE
$40.43 \times .33 = 13.34 \text{ IN}^2$

$$20{,}000 = \frac{M_{MAX}}{41.31} + \frac{43{,}226}{13.34}$$

$$M_{MAX} = \frac{wl^2}{8} = 700{,}721 \text{ IN-\#}$$

$$w = 99.73 \text{ PLF} = 8.31 \text{ PLI}$$

$$l_{MAX} = \sqrt{\frac{8(700{,}721)}{8.31}} = 821.3 \text{ IN}$$
$$= 68.4 \text{ FEET}$$

GIRDER DESIGN

AASHTO TYPE VI GIRDER TO WORK IN COMPOSITE WITH CAST-IN-PLACE DECK

EFFECTIVE WIDTH OF DECK 1.5.23(2)(a.)

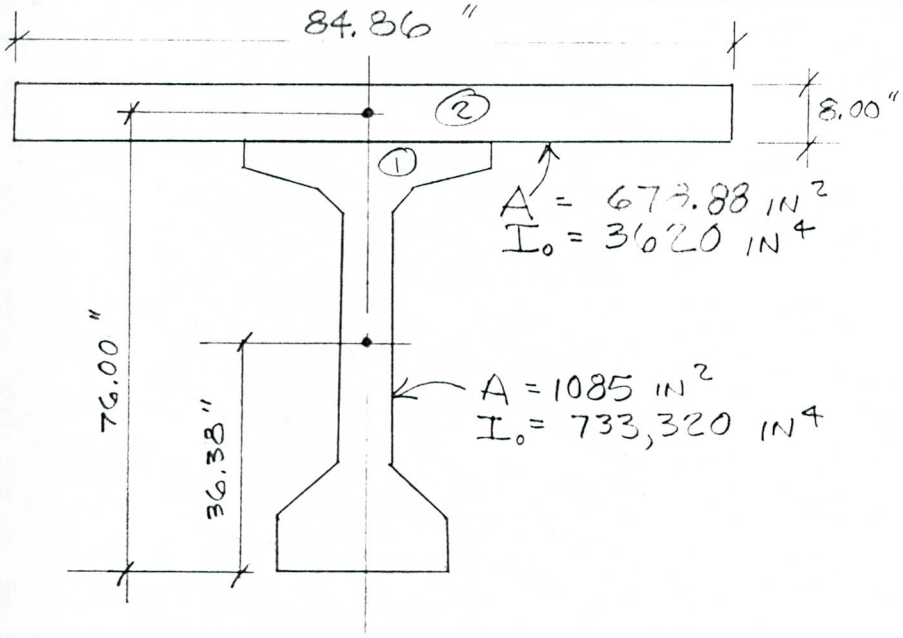
WEB THICKNESS PLUS TWELVE DECK THICKNESS

$8'' + 12(8'') = 104''$

$$\frac{E_{\text{CAST-IN-PLACE}}}{E_{\text{PRESTRESS}}} = \frac{\sqrt{4000}}{\sqrt{6000}} = 0.816$$

$(.816)\,104 = 84.864''$

SECTION PROPERTIES OF COMPOSITE SECTION



GIRDER DESIGN

LOCATE NEUTRAL AXIS:

PART	AREA	Y	AY
1	1085	36.38	39472
2	678	76.00	51528
	1763		91,000

$$\bar{Y} = \frac{91000}{1763} = 51.62 \text{ IN}$$

COMPOSITE MOMENT OF INERTIA

PART	I_o	AREA	$(\bar{Y}-Y)^2$	$A(\bar{Y}-Y)^2$
1	733320	1085	232.25	251991
2	3620	678	594.38	402989
	736,940			654,980

$$I = 736,940 + 654,980 = \underline{1,391,920 \text{ IN}^4}$$

COMPOSITE SECTION MODULUS

$$S_T = \frac{1,391,920}{(80 - 51.62)} = \underline{49,045 \text{ IN}^3}$$

$$S_{bc} = \frac{1,391,920}{(51.62)} = \underline{26,964 \text{ IN}^3}$$

NON COMPOSITE SECTION MODULUS

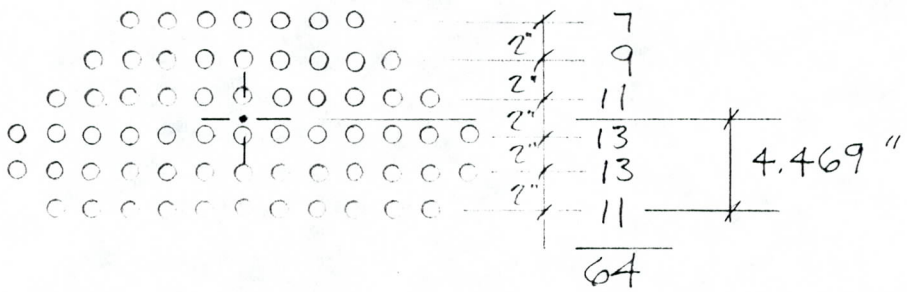
$$S_b = \frac{733,320}{36.38} = \underline{20,157 \text{ IN}^3}$$

GIRDER DESIGN

USE 64 - 1/2" Ø P/S STRAND

AREA OF 1/2" Ø STRAND = 0.153 IN²

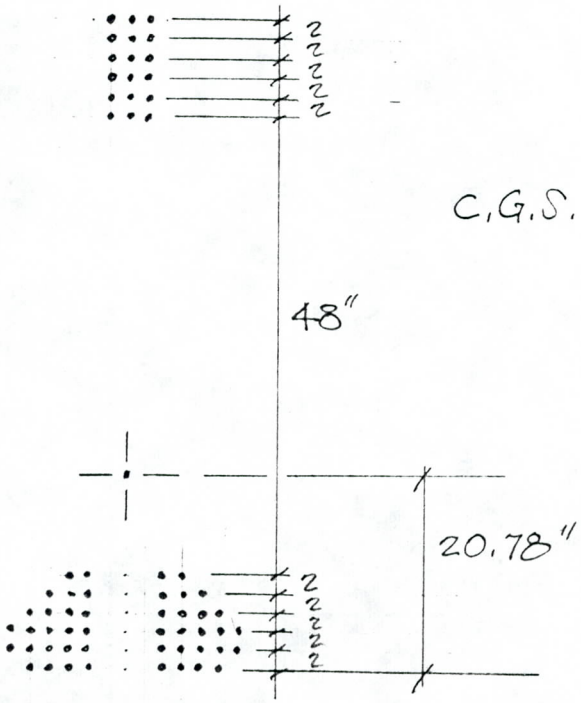
STRAND PATTERN AT MID-SPAN



$$C.G.S. = \frac{7(10)+9(8)+11(6)+13(4)+13(2)}{64}$$

$$= 4.469''$$

STRAND PATTERN AT END OF GIRDER



$$C.G.S. = \frac{10(2)+10(4)+8(6)+6(8)+4(10)+18(63)}{64}$$

GIRDER DESIGN

DEAD LOAD PER GIRDER

CHAIN LINK FENCE & CURB
.160 KLF ÷ 4 GIRDERS = .040 KLF

JERSEY BARRIER
.700 KLF ÷ 4 GIRDERS = .175

DECK
83.17 × .67 × .15 ÷ 8 GIRDERS = 1.045

GIRDER
(1085 ÷ 144) .15 = 1.130

DIAPHRAGMS
21 DIA. × 1 × 6 × 10 × .15 / 8 GIRDERS × 116 FT LONG = 0.204

SUB TOTAL 2.594 KLF

BUILD-UP & MISC. .090

FUTURE WEARING SURFACE
10.33 × .025 = .258

TOTAL D.L. 2.942 KLF

T3R

DEAD LOAD MOMENT PER GIRDER

$$M_{DL} = \frac{1}{8}(2.942)(116)^2 = 4{,}948.44 \text{ KIP-FT}$$
$$= 59{,}382 \text{ KIP-INCHES}$$

LIVE LOAD MOMENT AT .4 PT - SPAN ONE

$$M_{LL} \; (1748.0 \text{ K-FT}) \; \tfrac{1}{2}\left(\frac{10.33}{5.5}\right) = 1{,}641.53 \text{ K-F}$$
$$= 19{,}698.37 \text{ K-IN}$$

LANE LOAD INCLUDES IMPACT

AASHTO TABLE 1.3.1 (B)

$$I = \frac{50}{116+125} = 0.207$$

GIRDER DESIGN

ALLOWABLE STRESSES 1.6.6

(A) TEMPORARY STRESS BEFORE LOSS DUE TO CREEP AND SHRINKAGE

$$.70 f'_s = .7(270) = 189 \text{ KSI}$$

(B) CONCRETE

(1) TEMP. STRESS BEFORE CREEP & SHRINKAGE LOSSES

COMPRESSION
PRESTRESSED .6 (TRANSFER STRENGTH)
POST-TEN. .55 (" ")

(2) STRESSES AT SERVICE LOAD AFTER LOSSES

COMPRESSION $0.40 f'_c$
$0.40(6 \text{ KSI}) = 2.4 \text{ KSI}$

TENSION IN PRECOMPRESSED TENSILE ZONE

$6\sqrt{f'_c}$
$6\sqrt{6000} = 0.464 \text{ KSI}$

(3) CRACKING STRESS

$7.5\sqrt{f'_c}$

$7.5\sqrt{6000} = .580 \text{ KSI}$

GIRDER DESIGN

LOSS OF PRESTRESS 1.6.7 (B)

SHRINKAGE:

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

$$RH = 40\ \%$$

$$17{,}000 - 150(.4) = 16{,}940\ PSI$$

ELASTIC SHORTENING:

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

$$= \frac{(28 \times 10^6)\ f_{cir}}{33(150)^{3/2}\sqrt{4300}} = 7.043\, f_{cir}$$

EST.

f_{cir} = CONC. STRESS AT THE C.G.S. DUE TO PRESTRESSING AND GIRDER D.L.

$$P/S\ FORCE = .63(270)(.153)(64) = 1666\ K$$

$$M_{DL} = \frac{1}{8}\left(\frac{1085}{144} \times .15\right)(116)^2 = 1901.0\ K\text{-}FT = 22{,}812\ K\text{-}IN$$

$$e = C.G.C. - C.G.S. = 36.38 - 6.469 = 29.91\ INCHES$$

$$f_{cir} = \frac{1666}{1085} + \frac{1666(29.91)29.91}{733{,}320} - \frac{(22812)29.91}{733{,}320}$$

$$= 2.637\ KSI\ COMPRESSION$$

$$\therefore ES = 7.043(2637) = 18{,}572\ PSI$$

GIRDER DESIGN

CREEP OF CONCRETE

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

f_{cds} = CONC. STRESS AT C.G.S. DUE TO D.L. EXCEPT D.L. PRESENT AT TIME OF P/S.

M_{DL} COMPOSITE $- M_{DL}$ GIRDER ONLY

$$59,382 - 22,812 = 36,570 \text{ K-IN}$$

$$f_{cds} = \frac{1666}{1085} + \frac{1666(29.91)29.91}{733,320} - \frac{(36,570)29.91}{733,320}$$

$$= 2.076 \text{ KSI}$$

$$CR_c = 12(2637) - 7(2076) = 17,112 \text{ PSI}$$

RELAXATION OF PRESTRESSING STEEL

$$CR_s = 20,000 - .4ES - .2(SH + CR_c)$$

$$CR_s = 20,000 - .4(18,572) - .2(16940 + 17112)$$

$$= 5,761 \text{ PSI}$$

TOTAL LOSS = SH + ES + CR_s + CR_s

16,940
18,572
17,112
5,761

58,385 PSI

USE AASHTO ALLOWABLE OF 45000
1.6.7 (B) 2

GIRDER DESIGN

LOSS OF PRESTRESS 1.6.7

PRETENSIONING — 45 KSI

POST TENSIONING — TO BE CALCULATED

FLEXURE STRESSES AT SERVICE LOAD

$f_s = 189 - 45 = 144$ KSI

TOTAL PRESTRESS FORCE AFTER LOSSES

$P = (64 \text{ STRAND})(.153 \text{ IN}^2/\text{STRAND})(144 \text{ KSI})$

$= 1410.048$ KIPS

FOR FLEXURE STRESS AT .4 PT - SPAN ONE

e = ECCENTRICITY OF C.G.S. FROM C.G.C.
(NON-COMPOSITE)

C.G.S. = 6.469" FROM BOTTOM
C.G.C. = 36.38" " "

$36.38 - 6.469 = \underline{29.911}$ INCHES

e_c = ECCENTRICITY OF C.G.S. FROM C.G.C.
(COMPOSITE)

C.G.S. = 6.469
C.G.C. = 51.62

$51.62 - 6.469 = \underline{45.151}$ INCHES

GIRDER DESIGN

FLEXURE STRESSES AT SERVICE LOAD

$$-\frac{M_{DL}}{S_b} - \frac{M_{LL}}{S_{bc}} + \frac{P}{A} + \frac{Pe}{S_b} = \text{FLEXURE STRESS}$$

$$\text{FLEXURE STRESS} = \frac{-M_{DL}}{20,157} - \frac{M_{LL}}{26,964} + \frac{1410}{1085} + \frac{1410(29.91)}{20,157}$$

$$= \frac{-M_{DL}}{20,157} - \frac{M_{LL}}{26,964} + 3.391 \text{ KSI}$$

AT MID-SPAN OF SPAN ONE

$M_{DL} = 59,382$ K-IN

$M_{LL} = 1706(1/2)\left(\frac{10.33}{5.5}\right)12 = 19,225$ K-IN

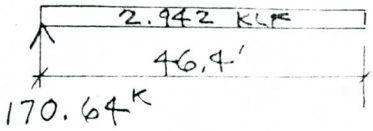
FLEX. STRESS $= -3.659 + 3.391$

$= -0.268$ KSI

THIS IS LESS THAN $6\sqrt{f'_c} = 0.464$ KSI

AT 0.4 PT OF SPAN ONE

$M_{LL} = 1748(1/2)\frac{10.33}{5.5}(12) = 19,699$ K-IN



$M_{DL} = 170.64(46.4) - \frac{2.942(46.4)^2}{2}$

$= 4751$ K-FT

$= 57,008$ K-IN

FLEX. STRESS $= -3.559 + 3.391$

$= -0.168$ KSI

GIRDER DESIGN

ULTIMATE FLEXURAL STRENGTH 1.6.9.

f'_s = ULTIMATE STRENGTH OF P/S STEEL = 270 KSI

$P^* = A^*_s/bd = \frac{(64)(.153)}{(104)(80-6.469)} = 0.00128$

$f'_c = 4$ KSI;

$$f^*_{su} = f'_s\left(1 - 0.5\,\frac{P^* f'_s}{f'_c}\right)$$

$$f^*_{su} = 270\left(1 - \frac{0.5 \times .00128 \times 270}{4}\right) = 258.336 \text{ KSI}$$

$$A_{sf} = 0.85 f'_c (b - b')\,t / f^*_{su}$$

$$= 0.85(4)(104 - 8)\,8 / 258.336$$

$$= 10.107 \text{ IN}^2$$

$$A_{sr} = A^*_s - A_{sf}$$

$$= (64)(.153) - 10.107 < 0$$

THE AREA OF THE FLANGED SECTION IS GREATER THAN WHAT IS NEEDED DEVELOPE THE ULTIMATE STEEL STRENGTH.

THEREFORE CALCULATE ULTIMATE STRENGTH AS A RECTANGULAR SECTION

$$M_u = A^*_s f^*_{su}\, d\left(1 - \frac{0.6\, P^* f^*_{su}}{f'_c}\right)$$

$$= (64)(.153)(258.336)(73.531)\left[1 - \frac{0.6(.00128)\,258.336}{4}\right]$$

$$= 186{,}005\ [1 - 0.0496]$$

$$M_u = 176{,}779 \text{ KIP-INCHES}$$

LOADING IA → SIMPLE SPAN AASHTO MOMENT PLUS IMPACT + D.L.

$$1.3\,[\underbrace{59{,}382}_{D.L.} + 2.2(\underbrace{1883.3 \times 12 \times 1.207}_{L.L.})] = 155{,}210 \text{ K-IN}$$

GIRDER DESIGN

CRACKING LOAD 1.6.10 (B)

$$-\frac{M_{DL}}{S_b} + \frac{P}{A} + \frac{Pe}{S_b} - \frac{M_A}{S_{bc}} = -.580$$

$$-\frac{59,382}{20,157} + \frac{1410}{1085} + \frac{1410(29.91)}{20,157} - \frac{M_A}{27,900} = -.580$$

$$M_{cr} = M_{DL} + M_A$$

$M_A = 28,618$ K-IN

$M_{cr} = 59,382 + 28,618$ K-IN (2384.8 K-FT)

$= 88,000$ K-IN

MINIMUM STEEL RATIO 1.6.10(B)

$$\frac{M_u}{M_{cr}} = \frac{176,779}{88,000} = 2.009$$

THIS EXCEEDS THE MINIMUM REQUIREMENT OF 1.2

GIRDER DESIGN

SHEAR 1.6.13

FACTORED SHEAR = $V_u \leq \phi (V_c + V_s)$

$\phi = 0.85$ 1.5.30(B)

V_c = SHEAR CARRIED BY CONCRETE
LESSER OF V_{ci} OR V_{cw}

V_s = SHEAR CARRIED BY WEB REINFORCEMENT

$V_u = 1.3 [1.0 DL + 2.2 LL]$ TABLE 1.2.22 LOAD FACTOR DESIGN IA

DL = NOMINAL DEAD LOAD
USE 2.942 KLF

LL = NOMINAL LIVE LOAD FOR HS-20-44
USE SHEAR ENVELOPE

LOCATION	DL	LL	V_u	
END	170.7	87.0	470.8	LEFT END SPAN 4
.1 PT & .9 PT	136.5	77.2	398.3	.1 PT SPAN 4
.2 PT & .8 PT	102.4	70.3	334.2	.2 PT SPAN 4
.3 PT & .7 PT	68.3	62.5	267.6	.3 PT SPAN 4
.4 PT & .6 PT	34.1	53.8	198.2	.4 PT SPAN 4
.5 PT	0	47.3	126.7	.5 PT SPAN 4

GIRDER DESIGN

SHEAR 1.6.13

$$V_{ci} = 0.6\sqrt{f'_c}\, b'd + V_d + \frac{V_i M_{cr}}{M_{MAX}}$$

$$V_{ci} \geq 1.7\sqrt{f'_c}\, b'd = 1.7\sqrt{6000}(8)(.8)(72) = 60.6^k$$

$$V_{cw} = \left(3.5\sqrt{f'_c} + .3 f_{pc}\right) b'd + V_p$$

f_{pc} = COMPRESSIVE STRESS IN CONCRETE (AFTER ALLOWANCE FOR ALL P/S LOSSES) AT THE CENTROID OF CROSS SECTION RESISTING EXTERNALLY APPLIED LOADS.

V_p = VERTICAL COMPONENT OF EFFECTIVE P/S FORCE AT SECTION.

V_d = SHEAR FORCE DUE TO UNFACTORED DEAD LOAD

V_i = FACTORED SHEAR LIVE LOAD

M_{MAX} = MAX. FACTORED LIVE LOAD MOMENT OCCURING W/ V_i

$$M_{cr} = \frac{I}{Y_t}\left[6\sqrt{f'_c} + f_{pe} - f_d\right]$$

Y_t = DIST. FROM CENTROIDAL AXIS OF GROSS SECTION, NEG. REIN., TO EXTREME FIBER IN TENSION

f_d = STRESS DUE TO UNFACTORED D.L., AT EXTREME FIBER OF SECTION WHERE TENSILE STRESS IS CAUSED BY EXTERNALLY APPLIED LOADS.

f_{pe} = COMPRESSIVE STRESS IN CONCRETE DUE TO EFFECTIVE PRESTRESS FORCES ONLY (AFTER P/S LOSSES) AT EXTREME FIBER OF SECTION WHERE TENSILE STRESS IS CAUSED BY EXTERNALLY APPLIED LOADS.

GIRDER DESIGN

SHEAR 1.6.13

POINT	UNFACTORED D.L. MOM. (K-IN)	$f_{pe} - f_d$ (KSI)	M_{cr} (K-IN)
0	0	0	0
.1	21,387	+1.568	54,790
.2	38,022	+1.175	42,845
.3	49,907	+.916	37,210
.4	57,042	+.562	27,665
.5	59,382	+.446	24,537

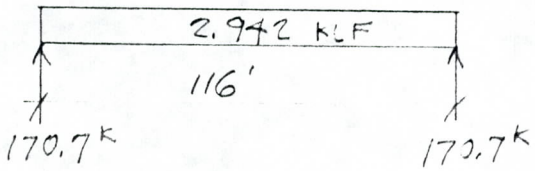
$$M_{cr} = \frac{I}{Y_t}\left[6\sqrt{f'_c} + (f_{pe} - f_d)\right]$$

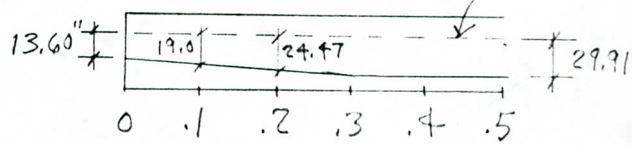
$$\frac{I}{Y_t} = 26,964 \text{ IN}^3$$

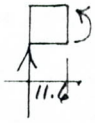
$$6\sqrt{f'_c} = .464 \text{ KSI}$$

EFFECTIVE P/S FORCE
$64(.153)144 = 1410$ K

C.G.C. NON COMPOSITE

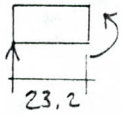




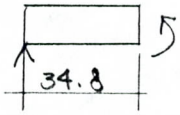


$$M_{.1} = 11.6(170.7) - 2.942\frac{(11.6)^2}{2} = 1782.2 \text{ K-FT}$$
$$21,387 \text{ K-IN}$$

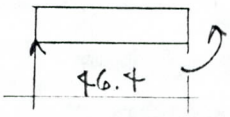
$$(f_{pe} - f_d) = -\frac{21,387}{20,157} + \frac{1410(19)}{20,157} + \frac{1410}{1085}$$



$$M_{.2} = 23.2(170.7) - 2.942\frac{(23.2)^2}{2} = 3,168.5 \text{ K-FT}$$
$$38,022 \text{ K-IN}$$



$$M_{.3} = 34.8(170.7) - 2.942\frac{(34.8)^2}{2} = 4,158.9 \text{ K-FT}$$
$$= 49,907 \text{ K-IN}$$



$$M_{.4} = 46.4(170.7) - 2.942\frac{(46.4)^2}{2} = 4,753.5 \text{ K-FT}$$
$$57,042 \text{ K-IN}$$

GIRDER DESIGN

SHEAR 1.6.13

$$V_{ci} = 0.6\sqrt{f'_c}\, b'd + V_d + V_i \frac{M_{cr}}{M_{MAX}}$$

POINT	V_d (KIPS)	V_i (KIPS)	M_{MAX} (KIP-IN)	M_{cr} (K-IN)	V_{ci} (KIPS)
0		(*)	(*)		—
.1	136.5	72.5	2,367	54,790	1836
.2	102.4	66.1	8,238	42,845	467
.3	68.3	58.7	13,242	37,210	254
.4	34.1	50.6	16,870	27,665	138
.5	0	41.6	18,696	24,537	76

(*) $\frac{10.33}{5} \times \frac{1}{2} \times$ VALUES ON SHEAR & MOMENT ENVELOPE

$$.6\sqrt{6000}(8)(.8)(72) = 21.416 \text{ KIPS}$$

$$V_{cw} = (3.5\sqrt{f'_c} + .3 f_{pc})\, b'd + V_p$$

POINT	$.3 f_{pc}$ (KSI)	V_p (KIPS)	V_{cw}
0	.382	55.0	355
.1	.511	55.0	415
.2	.612	55.0	461
.3	.682	55.0	493
.4	.727	0	459
.5	.741	0	465

CENTROID OF COMPOSITE SECTION IS 51.62 INCHES ABOVE BOTTOM.
CENTROID OF NON-COMPOSITE SECTION IS 36.38 INCHES ABOVE BOTTOM.

$\therefore C = 51.62 - 36.38$
$= 15.24$ INCHES

$$V_p = \frac{(29.91-13.60)}{34.8(12)}(64)(.153)(144)$$
$$= 55.0 \text{ KIPS}$$

$$f_{pc} = \frac{M_{DL}(15.24)}{733{,}320} + \frac{1410}{1085} - \frac{1410\,e}{733{,}320}$$

$$f_{pc} = .0000207\, M_{DL} - .00192\, e + 1.30$$

PT	M_{DL}	e
0	0	13.60
.1	21,387	19.00
.2	38,022	24.47
.3	49,907	29.91
.4	57,042	29.91
.5	59,382	29.91

GIRDER DESIGN

SHEAR 1.6.13

MIN. REQ. FOR V_s

$$V_u \leq \phi (V_c + V_s) \quad ; \phi = .85$$

V_c = LESSER OF V_{ci} OR V_{cw}

$$V_s \geq \frac{V_u}{\phi} - V_c$$

LOCATION	V_u	V_{ci}	V_{cw}	V_s
END	470.8	—	355	199 KIPS
.1 PT	398.3	1836	415	54
.2 PT	334.2	467	461	- 67
.3 PT	267.6	254	493	61
.4 PT	198.2	138	459	96
.5 PT	126.7	76	465	73

WEB REINFORCEMENT 1.6.13 (c)

$$\text{SPACING} = \frac{A_v f_{sy} d}{V_s}$$

$A_v = 2(.60) = 1.20 \text{ IN}^2$ FOR 2 - #7

$2(.31) = .62 \text{ IN}^2$ FOR 2 - #5

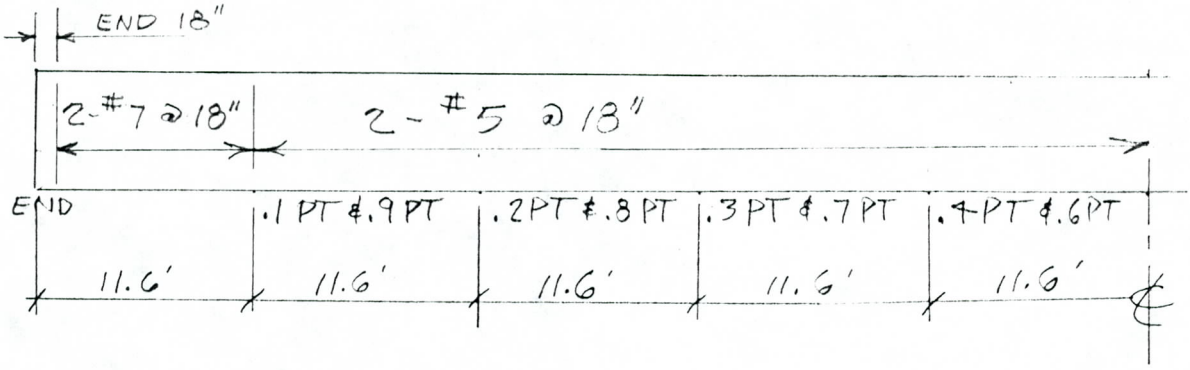
$f_{sy} = 60$ KSI

$d = .8h$ AS PER 1.6.13 (B) = 57.6

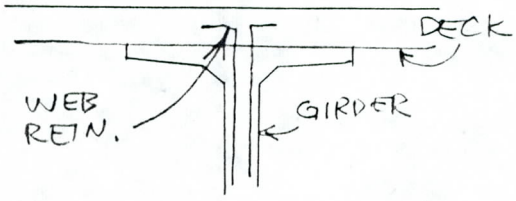
GIRDER DESIGN

WEB REINFORCEMENT

LOCATION	V_s	A_v	SPACING
END	199	1.20	20.84
		.62	10.76
.1 PT	54	1.20	76.8
		.62	39.7
.2 PT	—	1.20	—
		.62	—
.3 PT	61	.62	35.2
.4 PT	96	.62	22.3
.5 PT	73	.62	29.3



NOTE: HORIZONTAL SHEAR IS TRANSFERED BETWEEN DECK AND GIRDER AS PER 1.6.14 (B) AND (D).



GIRDER DESIGN

ANCHORAGE ZONE 1.6.15 (PRETENSIONED)

4% OF 64(.153).7(270) = 74.03 KIPS

AREA OF STEEL TO BE PLACED WITHIN d/4 OF THE END OF THE GIRDER:

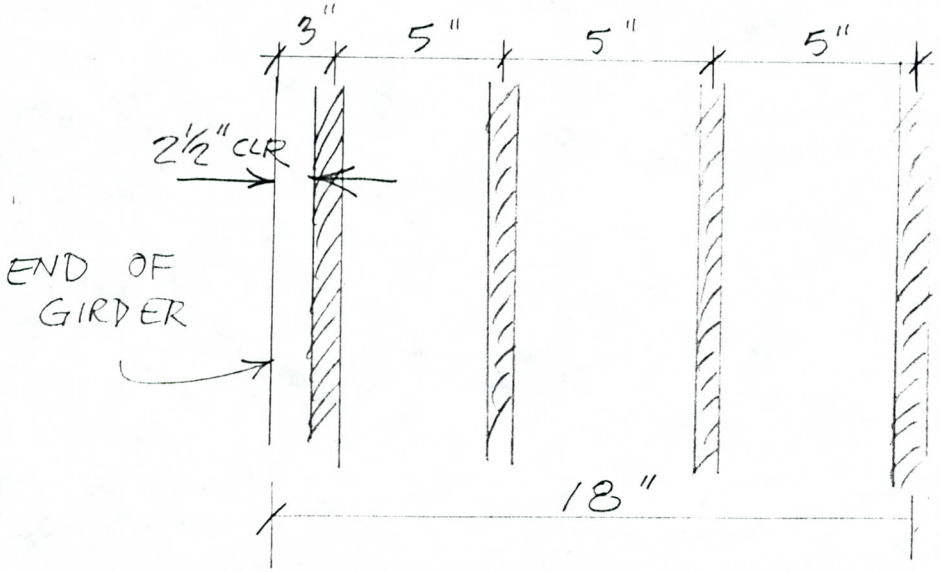
$$\frac{74.03\ K}{20\ KSI} = 3.701\ IN^2$$

USE #7 BARS

$$\frac{3.701}{.60} = 6.17$$

USE 8-#7 BARS
4 SETS OF 2

$$d/4 = 72/4 = 18''$$



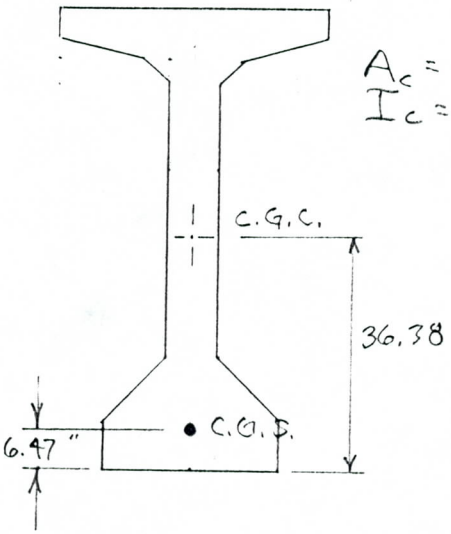
GIRDER DESIGN

TRANSFORMED SECTION PROPERTIES OF NON-COMPOSITE GIRDER

$E_s = 28 \times 10^6$ PSI

$E_{ci} = 57000\sqrt{4500} = 3.824 \times 10^6$ PSI

$$n = \frac{E_s}{E_{ci}} = 7.32$$



$A_c = 1085 \text{ IN}^2$
$I_c = 733{,}320 \text{ IN}^4$

$A_s = 64(.153) = 9.792 \text{ IN}^2$

PART	AREA	Y	AY
CONC.	1085 IN²	36.38	39,472.3
STEEL	9.792(7.32 − 1.0) = 61.89 IN²	6.47	400.4
	1146.89		39,872.7

$$\bar{Y} = \frac{39\,872.7}{1146.89} = 34.77 \text{ IN.}$$

PART	I_o	AY_n^2
CONC.	733320	2,812
STEEL	—	49,567
	733,320	52,379

$$I_T = 733{,}320 + 52{,}378 = 785{,}698 \text{ IN}^4$$

$$S_{T_T} = \frac{785{,}698}{37.23} = 21{,}104 \text{ IN}^3$$

$$S_{b_T} = \frac{785{,}698}{34.77} = 22{,}597 \text{ IN}^3$$

GIRDER DESIGN

CONCRETE STRENGTH AT TRANSFER

1.6.19

TRANSFER LOAD = (64).63(270)(.153) = 1666 KIPS

INCLUDES ELASTIC SHORTENING OF CON AND TENDON RELAXATION 1.6.7 (B)(b)

AT END OF GIRDER:

$M_{DL} = 0$

$e = 36.38 - 22.78 = 13.60''$

$S_t = \frac{733,320 \text{ IN}^4}{(72-36.38)} = 20,587 \text{ IN}^3$

$S_b = \frac{733,320}{36.38} = 20,157 \text{ IN}^3$

STRESS AT TOP = $\frac{1666}{1085} - \frac{1666(13.60)}{20,587} = 0.435$ KSI COMP.

STRESS AT BOTTOM = $\frac{1666}{1085} + \frac{1666(13.60)}{20,157} = 2.660$ KSI COMP

NON TRANFORMED PROPERTIES

AT MID SPAN OF GIRDER:

$M_{DL} = \frac{1}{8}(1.13)(116)^2 = 1900.66$ K-FT

$e = 34.77 - 6.47 = 28.3$ INCHES $= 22,803$ K-IN

[TRANSFORMED PROPERTIES USED]

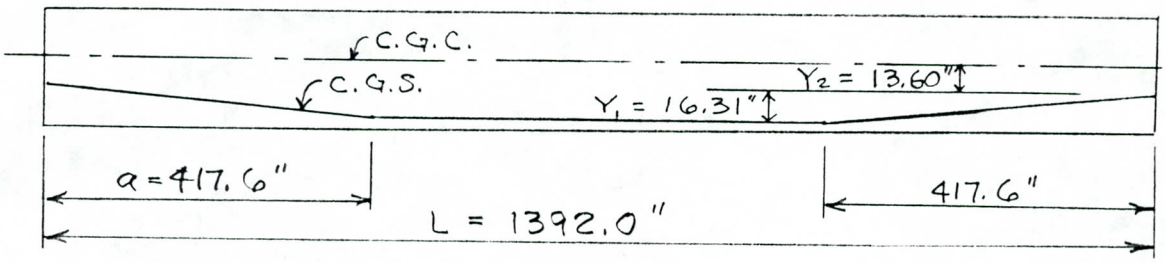
@ TOP = $\frac{1666}{1147} - \frac{1666(28.3)}{21,104} + \frac{22803}{21,104} = 0.299$ KSI COMP.

@ BOTTOM = $\frac{1666}{1147} + \frac{1666(28.3)}{22,597} - \frac{22803}{22,597} = 2.530$ KSI COMP.

MIN. REQ. CONC. STRENGTH = $\frac{2.660}{.6} = 4.433$ KSI

GIRDER DESIGN
CAMBER ESTIMATES

REF: "DESIGN OF PRESTRESSED CONCRETE STRUCTURES" BY T. Y. LIN & N. H. BURNS
3 RD EDITION p. 302



F = PRESTRESS FORCE.
USE P/S FORCE AT TRANSFER = $0.63 f'_s$
AASHTO 1.6.7 (B)(b)

$= (.63)(.153)(270)(64) = 1665.6$ KIPS

$M_1 = FY_1 = (1665.6\ k)(16.31\ in) = 27,165.9$ K-IN

$M_2 = FY_2 = (1665.6\ k)(13.60\ in) = 22,652.2$ K-IN

E = MODULUS OF ELASTICITY

$= W^{1.5}\, 33\sqrt{f'_c} = (150)^{1.5}\, 33\sqrt{4500}$ ← TRANSFER STRENGTH

$= 4.067 \times 10^6$ PSI $= 4.067 \times 10^3$ KSI

I = MOMENT OF INERTIA OF GIRDER ONLY WITH AREA OF P/S STRAND TRANSFORMED INTO CONCRETE

$= 785,698\ IN^4$

TSR

GIRDER DESIGN

CAMBER ESTIMATES

AT TRANSFER:

$$\Delta_{P/S} = \frac{L^2}{8EI}\left[M_2 + M_1 - \frac{M_1}{3}\left(\frac{2a}{L}\right)^2\right]$$

$$= \frac{(1392.0)^2}{(8)(4.067\times10^3)(785,698)}\left[22,652 + 27,166 - \frac{27,166}{3}\left(\frac{2\times417.6}{1392}\right)^2\right]$$

$$= (7.580\times10^{-5})(49,818 - 3,260) = 3.529 \text{ IN.} \uparrow$$

$$\Delta_{W_G} = \frac{5 W_G L^4}{384 EI} = \frac{5(1130/12)(116\times12)^4}{384(4.067\times10^6)(785,698)} = 1.441 \text{ IN} \downarrow$$

EST. CAMBER AT TRANSFER
$3.529 - 1.441 = 2.088'' \uparrow$

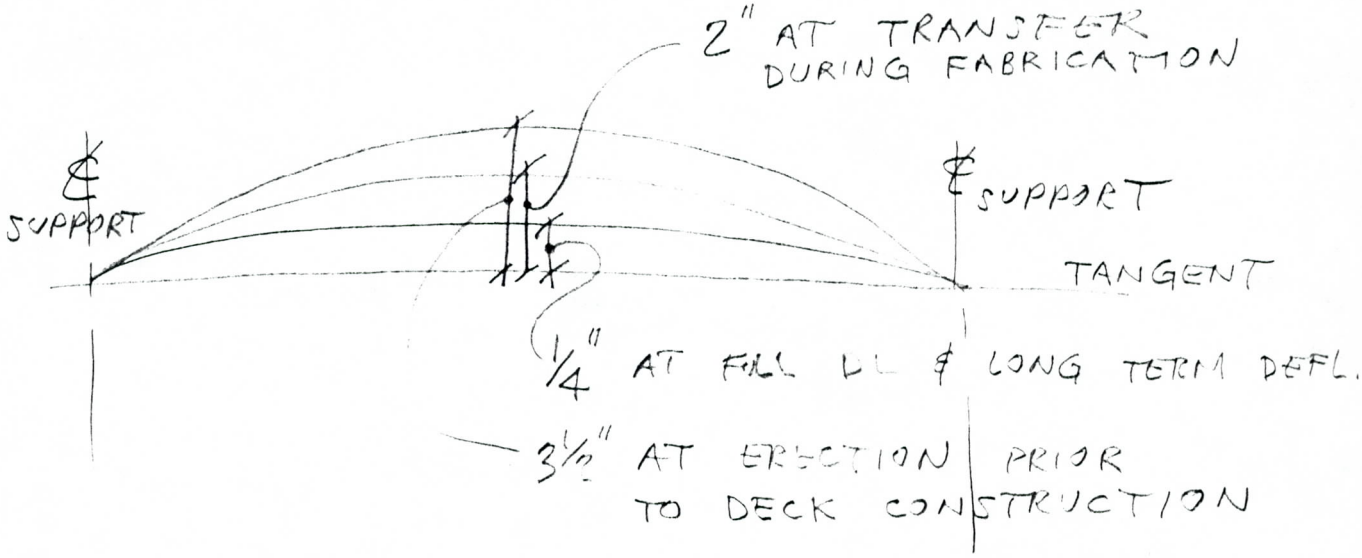
AT ERECTION (30 DAYS LATER):

$\Delta_{P/S} = 3.529(1.80) = 6.352 \uparrow$
$\Delta_{W_G} = 1.441(1.85) = 2.666 \downarrow$

$3.686'' \uparrow$

TSR

CAMBER ESTIMATES



CAMBER ESTIMATES

3½" CAMBER AT ERECTION
¼" CAMBER AT FULL LOADING (LONG TERM)

CURVE FITTING FOR ELEV. ESTIMATES

$$A x (l^3 - 2 l x^2 + x^3) = \text{CAMBER}$$

FOR $x = 58.00$ FT
$l = 116.00$ FT
CAMBER = 3½" = 0.2917 FT

$$A(58)\left[(116)^3 - 2(116)(58)^2 + (58)^3\right] = 0.2917$$

$$A(58)\left[1560896 - 780448 + 195112\right] = 0.2917$$

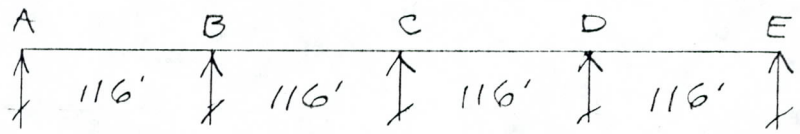
$$A = 5.1553 \times 10^{-9}$$

GIRDER DESIGN

CONTINUITY 1.6.12 (C)

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

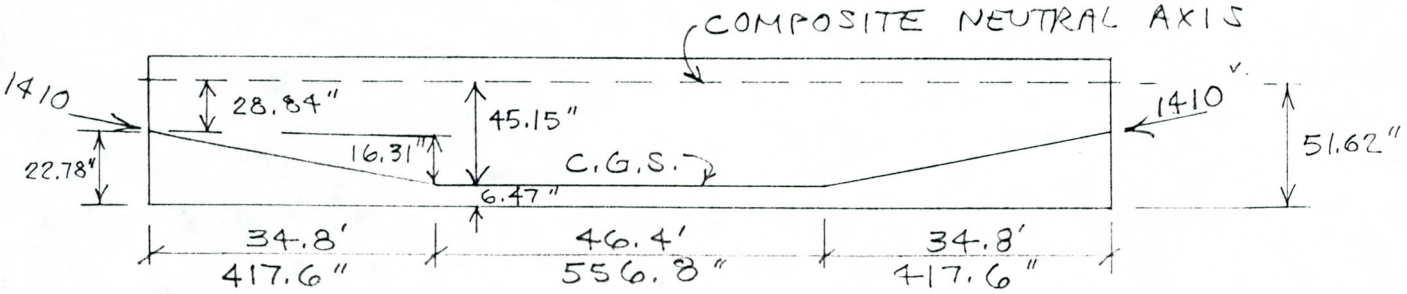
BY PORTLAND CEMENT ASSOCIATION
AUGUST 1969



DESIGN FOR POSITIVE MOMENTS AT PIERS

PRESTRESS CREEP RESTRAINT MOMENTS
(NOT INCLUDING CREEP EFFECT FACTOR)

USE CONJUGATE BEAM THEORY TO CALCULATE THE SLOPE IN THE ENDS OF THE GIRDER DUE TO PRESTRESS ALONE:



GIRDER DESIGN

$$[P/S \text{ SLOPE}] = \frac{1410}{EI}\left[(28.84)(417.6) + \tfrac{1}{2}(16.31)(417.6) + \tfrac{1}{2}(45.15)(556.8)\right]$$

$$[P/S \text{ SLOPE}] = \frac{1410}{EI}\left[12{,}044 + 3{,}406 + 12{,}570\right]$$

$$= \frac{39{,}508{,}200.0}{EI} \text{ IN-KIPS}$$

CREEP RESTRAIN MOMENT DUE TO P/S: (NOT INCLUDING CREEP EFFECT FACTORS)

$$\text{AT } B = \frac{2.57\,EI}{116 \times 12} \times \frac{39{,}508{,}200.0}{EI} = 72{,}943 \text{ IN-K}$$

$$= 6{,}079 \text{ K-FT}$$

NOTE: COEFFICIENT TAKEN FROM PCA PUBLICATION DESIGN EXAMPLE BECAUSE FOUR EQUAL CONTINUOUS SPANS WERE USED IN THE EXAMPLE.

$$\text{AT } C = \frac{1.72}{2.57} \times 6{,}079 = 4069 \text{ K-FT}$$

CREEP RESTRAIN MOMENT DUE TO DEAD LOAD:

(NOT INCLUDING CREEP EFFECT FACTORS)

$$M_D = \tfrac{1}{8}(2.942)(116)^2 = 4{,}948.4 \text{ K-FT}$$

$$\text{AT } B = -0.86 \times 4948.4 = 4{,}255.6 \text{ K-FT}$$

$$\text{AT } C = -0.572 \times 4948.4 = 2{,}830.5 \text{ K-FT}$$

GIRDER DESIGN

SHRINKAGE RESTRAIN MOMENT

NOT INCLUDING CREEP EFFECT FACTORS

1) ASSUME CONTINUITY CONNECTION MADE 28 DAYS AFTER P/S RELEASE

2) 40% RELATIVE HUMIDITY FOR PHOENIX AREA (AASHTO FIGURE 1.6.7)

3) ULTIMATE SHRINKAGE OF 0.0006 IN/IN. (PCA RECOMMENDATION WHICH IS TO BE CORRECTED FOR HUMIDITY)

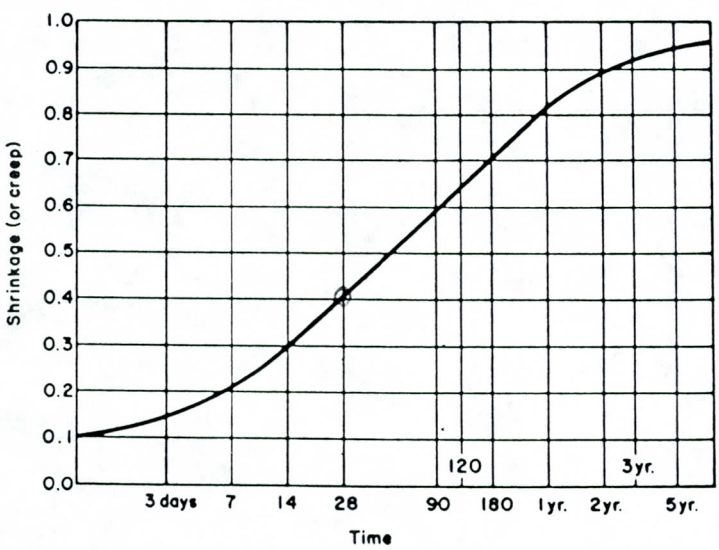


Fig. 8. Proportion of final shrinkage or creep vs. time

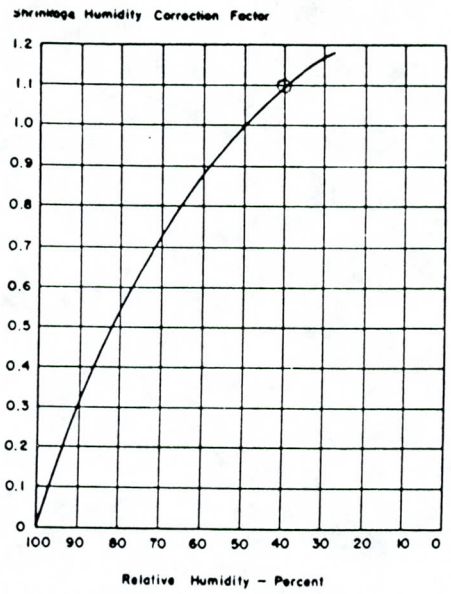


Fig. 10. Shrinkage humidity correction factor

$\epsilon_s = 0.0006 \times 0.40 \times 1.1 = 0.000264$ IN/IN

E_b = ELASTIC MODULUS FOR DECK CONCRETE
$= 57{,}000\sqrt{4000} = 3.605 \times 10^3$ KSI

A_b = CROSS SECTION AREA OF DECK $= 124'' \times 8'' = 992.0 \text{ IN}^2$

$\left(e'_2 + \frac{t}{2}\right)$ = DIST. BETWEEN MID DEPTH OF SLAB AND CENTROID OF COMPOSITE SECTION.
$= 76 - 51.62 = 24.38$

GIRDER DESIGN

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

$$= .00026 \times 3.605 \times 10^3 \times 992.0 \times 24.38$$

$$= 23,017.3 \text{ K-IN} = 1,918.1 \text{ K-FT}$$

THE ELASTIC SHRINKAGE RESTRAINT MOMENTS ARE:

AT B = −1.286 × 1918.1 = −2,466.7

AT C = −0.857 × 1918.1 = −1,643.8

NOTE: SEE PCA DESIGN EXAMPLE

EVALUATION OF CREEP FACTORS

FROM FIG. 5 (SEE BELOW) FOR AN ELASTIC MODULUS AT STRAND RELEASE = $57,000\sqrt{4793} = 3.946 \times 10^6$ PSI

THE SPECIFIC CREEP AT 20 YEARS IS 0.36×10^{-6} IN/IN/PSI

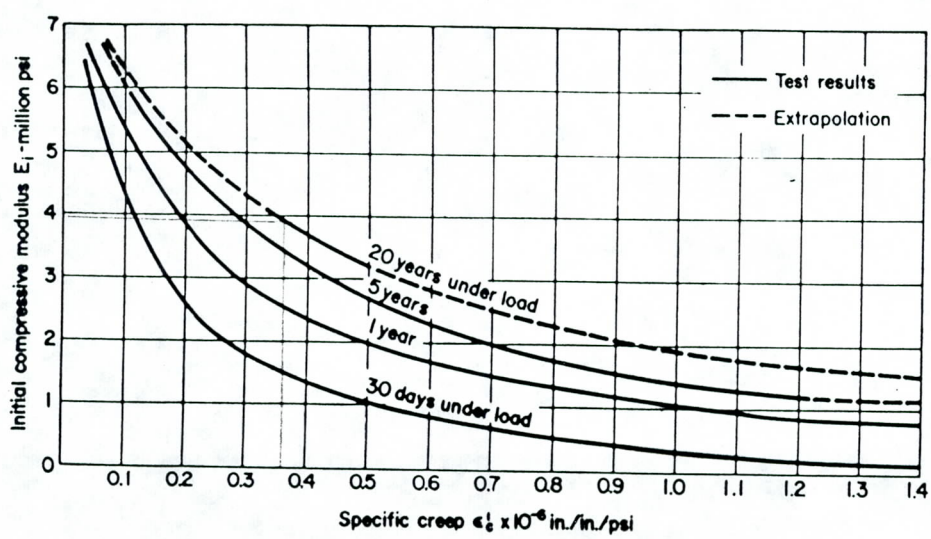


Fig. 5. Prediction of basic creep from elastic modulus

GIRDER DESIGN

FOR RELEASE OF THE PRESTRESS FORCE AT 2 DAYS, THE VALUE OF SPECIFIC CREEP IS ADJUSTED BY FIGURE 6:

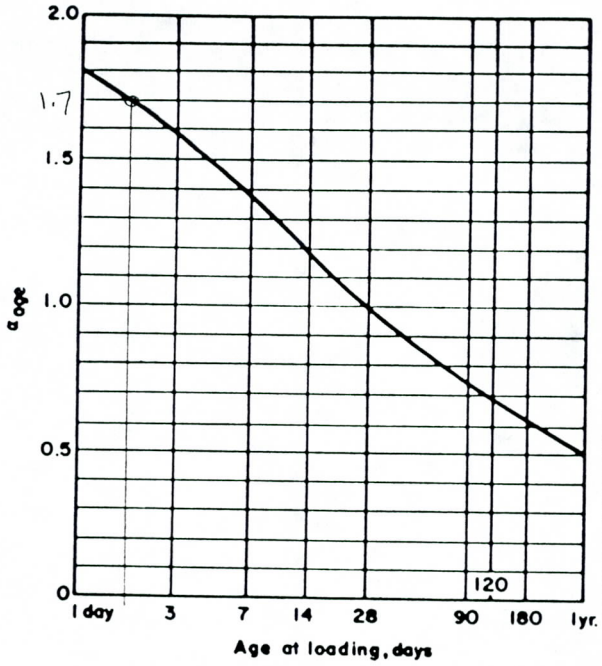


Fig. 6. Creep vs. age at loading

0.36×10^{-6} IN/IN $\times$ 1.7 = 0.612×10^{-6} IN/IN/PSI

ADJUST FOR VOLUME TO SURFACE RATIO OF TYPE VI GIRDER:

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

$0.612 \times 10^{-6} \times 1.18 = 0.722 \times 10^{-6}$ IN/IN/PSI

GIRDER DESIGN

FROM FIGURE 8, AT 28 DAYS 0.4 OF THE CREEP HAS OCCURED LEAVING 0.6 TO OCCUR:

$$0.6 \times 0.722 \times 10^{-6} \text{ IN/IN} = 0.433 \times 10^{-6} \text{ IN/IN/KSI}$$

$$\phi = \epsilon_c \times E = 0.433 \times 10^{-6} \times 57000\sqrt{6000} = 1.912$$

AT 28 DAYS

CREEP FACTOR FOR P/S AND D.L. CREEP:

$$\left(1 - e^{-\phi}\right) = 0.852$$

CREEP FACTOR FOR SHRINKAGE:

$$\left(\frac{1 - e^{-\phi}}{\phi}\right) = 0.446$$

SUMMATION OF POSITIVE RESTRAINT MOMENT AT PIERS

AT B = (6,079 − 4,256) .852 − (2,467) .446 = 452.9 K-FT

AT C = (4,069 − 2,831) .852 − (1,644) .446 = 321.6

P/S D.L. SHRINKAGE

ADD LL + I

AT B = 452.9 + $\left(\frac{10.33}{11.0} \times 209\right)$ = 649.17 K-FT

AT C = 321.6 + $\left(\frac{10.33}{11.0} \times 441\right)$ = 735.74 K-FT

GIRDER DESIGN

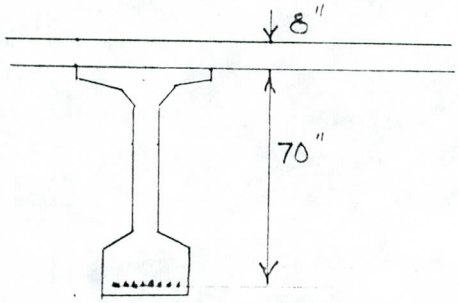
DESIGN FOR POSITIVE MOMENT CONNECTION

DESIGN FOR THE 736 KIP FEET MOMENT.

EXTEND P/S STRAND TO TIE INTO DIAPHRAGM.

ALLOWABLE STRESS $= 0.6 f_y = 0.6 \times 270 = 162$ KSI

ESTIMATE AREA OF STEEL REQ.



$$A_s = M / f_s jd$$

$$\text{EST. } jd = \left(\tfrac{2}{3} \times 8 + 70\right) = 75.33''$$

$$A_s = \frac{736 \times 12}{162 \times 75.33} = 0.724 \text{ in}^2$$

TRY 5 P/S STRAND, $A_s = 5(.153) = .765 \text{ in}^2$

ASSUME COMPRESSION AREA IS IN DECK ONLY

$$\frac{E_s}{E_c} = \frac{28 \times 10^6}{57000\sqrt{4000}} = 7.77$$

SECTION	A	Y	AY	$A\bar{Y}^2$	I_o
SLAB 104 × 8	832	-4	-3328	225	4437
STEEL .765 × 7.77	5.94	-78	-463	32072	—
	837.94		-3791	32297	4437

$$Y_n = -\frac{3791}{837.94} = -4.52''$$

$$I_c = 32{,}297 + 4{,}437 = 36{,}734 \text{ in}^4$$

GIRDER DESIGN

DISTANCE FROM NEUTRAL AXIS TO STEEL

$$78'' - 4.52'' = 73.48''$$

$$\text{STEEL STRESS} = \frac{736 \times 12}{36{,}734} \times 73.48 \times 7.77$$

$$= 137.27 \text{ KSI}$$ WHICH IS LESS THAN $.6 f_y$

MAX STRESS IN CONCRETE:

$$\frac{736 \times 12}{36{,}734} \times 4.52 = 1.087 \text{ KSI}$$

WHICH IS LESS THAN $.4 f'_c = 1.6$ KSI

EMBEDMENT LENGTH

UNIT BOND STRESS IS $.10 f'_c$ NOT TO EXCEED 350 PSI.

PCA PUBLICATION

$$.153 \text{ IN}^2 = \pi r^2$$

$$r \approx 0.22 \text{ IN}$$

$$C = 2\pi r = 1.386 \text{ IN}$$

BOND CAPACITY PER INCH $= 1.386(350) = 485 \#/\text{IN}$

LENGTH TO DEVELOPE $137.27(.153) = 21.0$ KIPS

$$\frac{21.0^k}{.485} = 43.3 \text{ INCHES}$$

EXTEND 5 P/S STRAND 48" BEYOND END OF GIRDER

GIRDER DESIGN

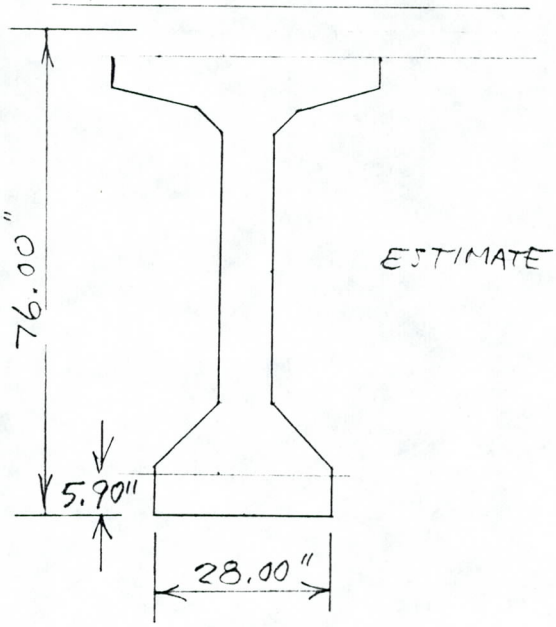
DESIGN NEGATIVE MOMENT REIN. OVER PIERS

LIVE LOAD PLUS IMPACT AT B = 1711.0 K-F
" " AT C = 1546.0 K-F

USE MOMENT AT B

ULTIMATE LIVE LOAD NEGATIVE MOMENT $= \frac{10.33}{11.0} \times 1.3 \left[\overset{0}{M_{DL}} + 2.2 M_{LL} \right]$

$= 2.686 \left[1711.0 \right] = 4{,}595.8$ K-FT



$$\frac{M_u}{bd^2} = \frac{4595.8 \times 12}{28 \times 74.0^2} = .360 \text{ KSI}$$

ESTIMATE $\rho = \frac{A_s}{bd} = .006$

$A_s = .006(76)(28) = 12.77 \text{ IN}^2$

TRY 9 - #11 BARS

$A_s = 9(1.56) = 14.04 \text{ IN}^2$

$f_y = 60$ KSI

CHECK ULTIMATE MOMENT PROVIDED:

$a = \frac{A_s f_y}{.85 f'_c b}$ = DEPTH OF COMPRESSION BLOCK

$= \frac{14.04(60)}{.85(6)28} = 5.90$ INCHES

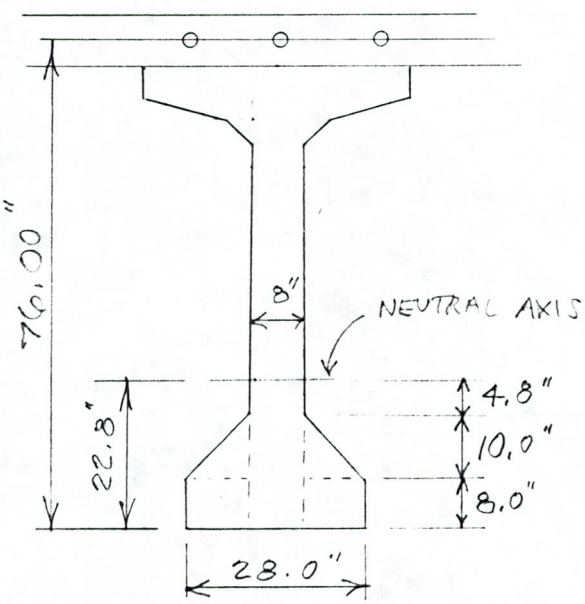
GIRDER DESIGN

$$M_u = A_s f_y \left(d - \frac{a}{2}\right)$$
$$= 14.04(60)\left(76 - \frac{5.90}{2}\right)$$
$$= 61{,}537 \text{ K-IN} = 5{,}128 \text{ K-FT}$$

$M_u > 4{,}595.8$ K-FT REQUIRED

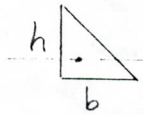
COMPRESSIVE STRESS IN GIRDERS AT PIERS

ASSUME $kd = 22.8''$



$$n = \frac{E_s}{E_c} = \frac{29000}{57\sqrt{6000}} = 6.57$$

$$Y_n = \frac{10{,}856}{534.6} = 20.307''$$



$$I_o = \frac{bh^3}{36}$$

CHECK kd ASSUMTION

SECTION		A	Y	AY	$A(Y-Y_n)^2$	I_o
STEEL	(6.57)(14.04)	92.2	76.0	7007	285,980	
STEM	(8)(22.8)	182.4	11.4	2079	14,470	7902
FLANGE	(10)(10)	100.0	11.3	1130	8,112	555
FLANGE	(8)(20)	160.0	4.0	640	42,545	853
		534.6		10,856	351,107	9,310

GIRDER DESIGN

$$I_c = 351,107 + 9,310 = 360,417 \text{ IN}^4$$

UNFACTORED — WORKING STRESS

$$f_c(LL+I) = \frac{1711.0 \times 12 \times 20.307}{360,417} = 1.157 \text{ KSI}$$

$$f_c(\text{PRESTRESS}) = \frac{1410}{1085} + \frac{1410(36.38-22.76)\,20.307}{733,320} = \frac{1.831}{2.988} \text{ KSI}$$

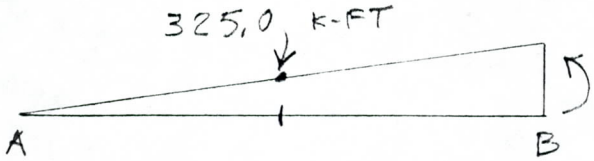
$$2.988 \text{ KSI} < f_c'(.6) = 3.6 \text{ KSI}$$

AASHTO 1.6.12(C)(4)

USE 9-#11 @ 6" OVER EACH GIRDER

TENSILE STRESSES IN GIRDER AT MIDSPAN

IN SPAN AB THE POSITIVE CREEP RESTRAINT MOMENT WOULD BE ½ x 650 K-FT.



TENSILE STRESS DUE TO THIS MOMENT:

$$\frac{325.0 \times 12}{26,964} = -.144 \text{ KSI TENSION}$$

COMPOSITE SECTION

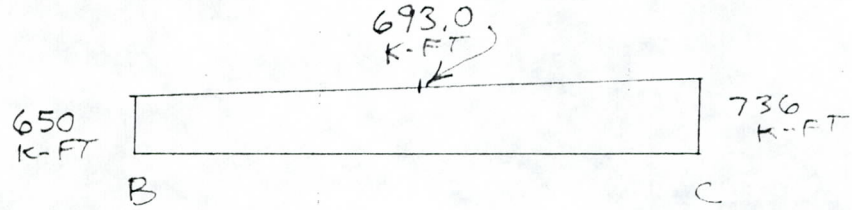
TENSILE STRESS UNDER SERVICE LOAD = -.268 KSI

$$.144 + .268 = 0.412 \text{ KSI}$$

WHICH IS LESS THAN THE MAX. ALLOWABLE OF $6\sqrt{f_c'} = 0.464$ KSI

GIRDER DESIGN

IN SPANS BC AND CD THE POSITIVE CREEP RESISTRAINT MOMENT IS:



(LL + I) AT MID SPAN WOULD BE $1432\left(\frac{10.33}{11}\right)$

$= 1344.8$ K-FT

DEAD LOAD MOMENT IS 59,382 K-IN

$$\text{STRESS} = \frac{1410}{1085} + \frac{1410(29.91)}{20,157} - \frac{59382}{20,157} - \frac{693\times12}{26,964} - \frac{1344.8\times12}{26,964}$$

$$= 1.300 + 2.092 - 2.946 - 0.308 - 0.598$$

$= -0.460$ KSI $\approx$ MAX. ALLOWABLE TENSION OF $6\sqrt{f'_c} = -.464$ KSI

DEVELOPEMENT LENGTH FOR CONTINUITY STEEL

USE #11 BARS $f_y = 60$ KSI AASHTO 1.5.14

$$l_d = \frac{0.04\,A_b f_y}{\sqrt{f'_c}} = \frac{0.04(1.56)\,60,000}{\sqrt{4000}} = 59.2''$$

$$59.2'' \times 1.4 \times .8 = 66.3''$$

TOP REIN. SPACING = 6"

USE 6'-0"

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75TH AVE MCFCD
SHT. G-33 OF 36
BY AGD
DATE JULY 25, 1984
73R

GIRDER DESIGN

CUT-OFF POINTS FOR CONTINUITY STEEL

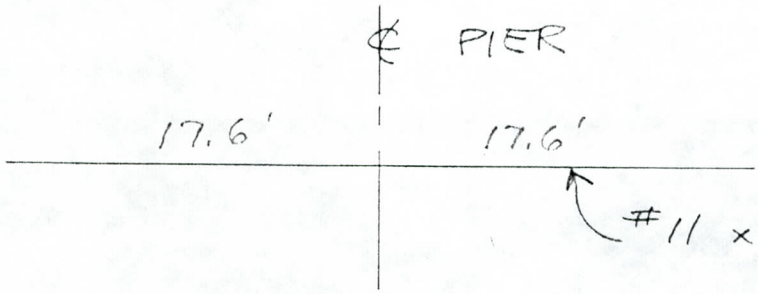
LOCATION	D.L. MOMENT (KIP-FT)	NEGATIVE LL + I MOMENT	D.L. + (LL+I)
SPAN 1			
.1 PT	1782.3	- 72.4	
.2	3168.5	- 144.8	
.3	4158.9	- 217.2	
.4	4753.5	- 289.5	
.5	4948.5	- 361.9	
.6	4753.5	- 434.3	
.7	4158.9	- 506.7	
.8	3168.5	- 579.1	
.9	1782.3	- 938.1	
END	0	-1608.4	-1608.4
SPAN 2			
.1	1782.3	- 976.7	
.2	3168.5	- 707.8	
.3	4158.9	- 647.7	
.4	4753.5	- 625.1	
.5	4948.5	- 601.6	
.6	4753.5	- 579.1	
.7	4158.9	- 555.6	
.8	3168.5	- 590.4	
.9	1782.3	- 845.1	
END	0	-1453.3	-1453.3

NOTE:

CUT-OFF POINTS FOR NEG MOMENT OCCUR WITHIN 1/10 SPAN LENGTH ON BOTH SIDES OF EACH PIER

11.6' + 6.0' = 17.6'

(LL+I) 10.33 / 11.00



POST-TENSION GIRDER

ANCHORAGE ZONES

AASHTO 1.6.15

AN EFFECTIVE P/S FORCE AFTER ALL LOSSES HAVE OCCURRED IS 1410 KIPS.

A LARGE VARIETY OF POST TENSIONING SYSTEMS ARE AVAILABLE TO PROVIDE THE REQUIRED EFFECTIVE P/S FORCE.

A CONSERVATE ESTIMATE FOR THE NUMBER OF POST TENSION STRAND IS THE SAME NUMBER AS IN A STRESS-RELEAVED P/S GIRDER --- I.E. 64 STRAND.

$$P_{JACK} = (64 \text{ STRAND})(.7\, f_{pu} A_{ps}) = (64)(28.9) = 1849.6^{K}$$

REF. "POST-TENSIONED BOX GIRDER BRIDGE MANUAL BY POST-TENSIONING INSTITUTE PAGE 48.

The magnitude of the tensile splitting force can be determined from the following equation[2.6]:

$Z = 0.3\,P(1-a/d)$

Where:
Z = total splitting or bursting force
P = tendon force
a = depth of the anchor plate (ASSUME = 24")
d = depth of the concrete section (ASSUME = 72")

Sufficient vertical reinforcement acting as a unit stress of 20,000 psi to resist the computed value of Z should be distributed within the distance of d/2 of the anchorage location. Normal shear reinforcement may be included in the total area required.

$$Z = 0.3(1849.6)(1 - 24/72) = 369.92 \text{ KIPS}$$

VERTICAL REINFORCEMENT:

$$\frac{369.92}{20.0} = 18.50 \text{ IN}^2$$

$$\frac{18.50}{(.60)4} = 7.71$$

USE 8 - #7 [U]
WITHIN d/2 = 36"

POST-TENSION GIRDER

POSITIVE MOMENT REIN. STEEL CONNECTION

DESIGN FOR THE 736 KIP FEET MOMENT.

ALLOWABLE STEEL STRESS = 0.6 fy

USE GRADE 60 $\longrightarrow$ $f_s = 0.6(60) = 36$ KSI

REFERENCE GIRDER DESIGN SECTION OF THIS PROJECT.

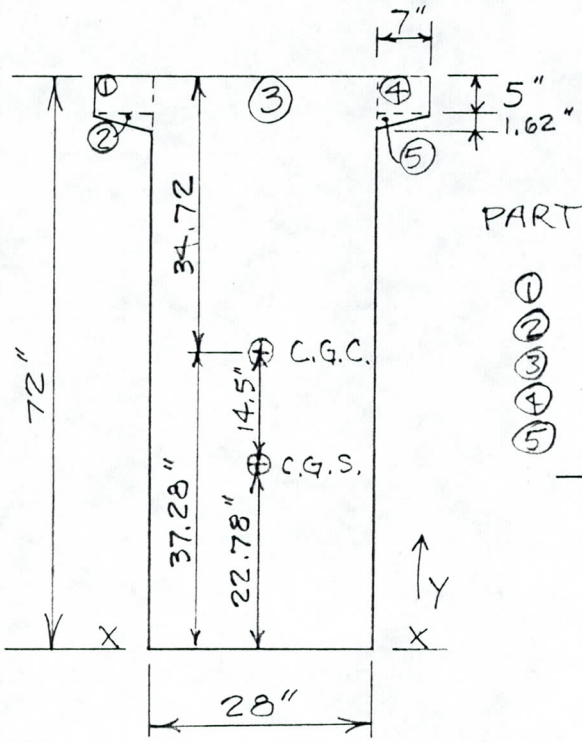
$$A_s = \frac{736 \times 12}{36 \times 75.33} = 3.257 \text{ IN}^2$$

USE 8 - #6

POST-TENSION GIRDER

FLEXURE STRESSES AT END OF GIRDER

$P_{JACK} = 1849.6$ KIPS



NEUTRAL AXIS

PART	AREA	Y	AX
①	35.0	69.5	2,432
②	5.7	66.5	379
③	2016.0	36.0	72,576
④	35.0	69.5	2,432
⑤	5.7	66.5	379
	2097.4		78,198

$$\bar{Y} = \frac{78,198}{2097.4} = 37.28''$$

MOMENT OF INERTIA

PART	I_o	$A(\bar{Y}-Y)^2$
①	72.9	36,334.5
②	0.8	4,866.7
③	870,912.0	2,580.5
④	72.9	36,334.5
⑤	0.8	4,866.7
	871,059.4	84,982.9

$I_{TOTAL} = 956,042.3 \text{ IN}^4$

POST-TENSION GIRDER

FLEXURE STRESSES AT END OF GIRDER

MAX. STRESS AT TOP FIBER

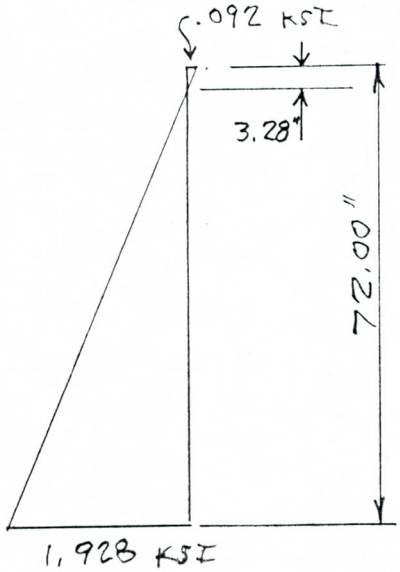
$$\frac{P}{A} - \frac{Pec}{I} = \frac{(1849.6)}{(2097.4)} - \frac{(1849.6)(14.5)(34.72)}{956,042.3}$$

$$= -0.092 \text{ KSI}$$

MAX. STRESS AT BOTTOM FIBER

$$\frac{P}{A} + \frac{Pec}{I} = \frac{(1849.6)}{(2097.4)} + \frac{(1849.6)(14.5)(37.28)}{956,042.3}$$

$$= +1.928 \text{ KSI}$$



TOTAL TENSILE FORCE:

$$(1/2)(42)(3.28)(.092) = 6.337 \text{ K}$$

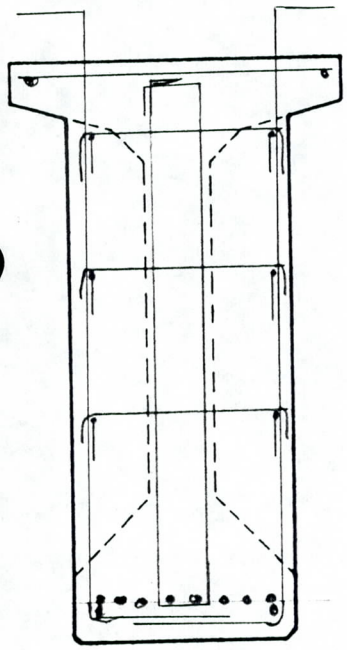
AREA OF STEEL REIN. TO CARRY TENSILE FORCE:

$$\frac{6.337}{20.0 \text{ KSI}} = 0.317 \text{ IN}^2$$

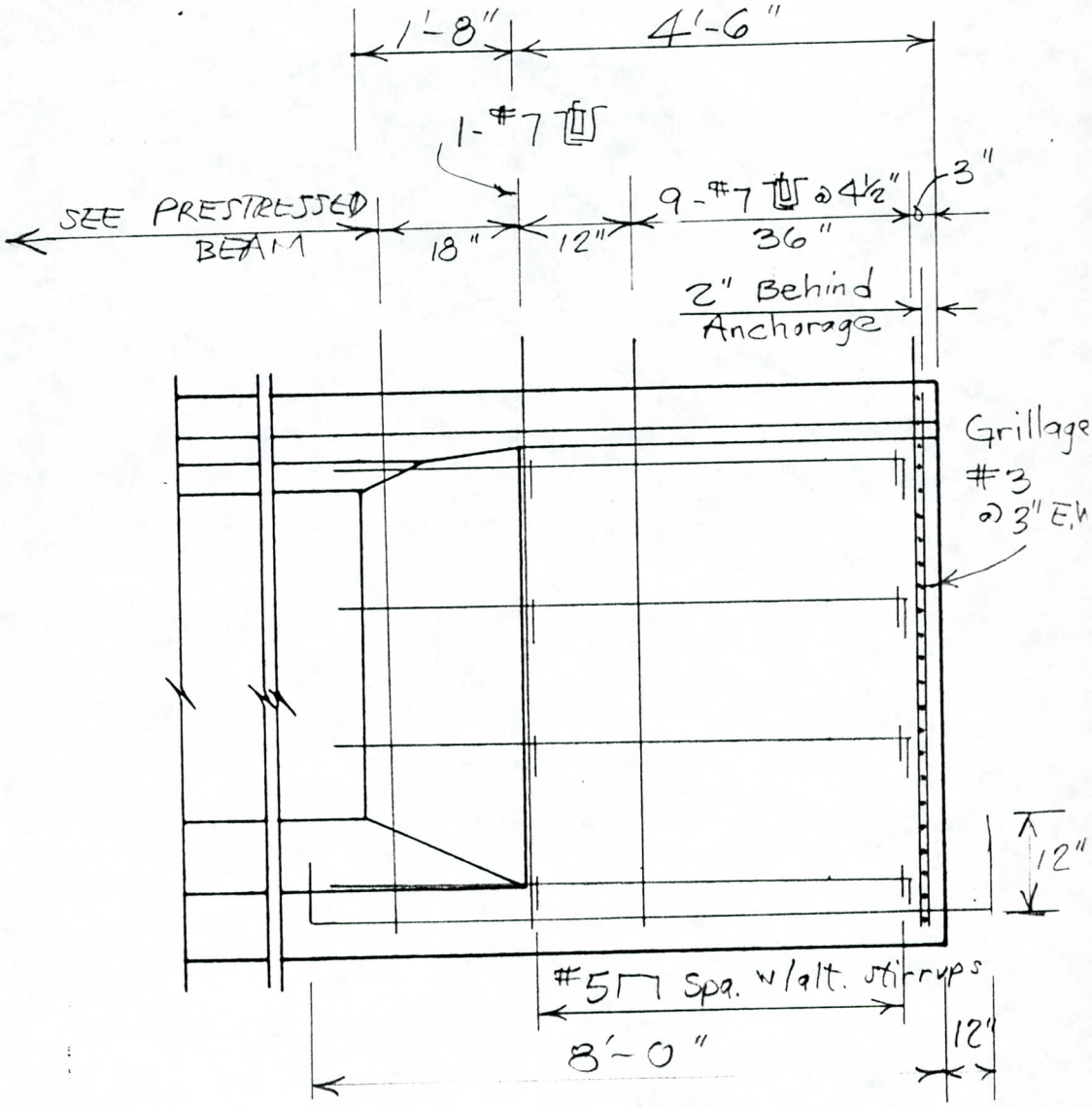
USE 2-#4 IN TOP OF END

G·36 of 36

TSR



Section B-B



Elevation

POST TENSIONED GIRDER END BLOCK DETAIL

½" = 1'-0"

PRELIM

SUBJECT

DATE AU

TSB

PIER LOADS

LONGITUDINAL FORCES (LF) 1.2.13

CONSIDER TWO LANES OF TRAFFIC GOING IN THE SAME DIRECTION.

HS 20-44 LOADING PER LANE:
a.) .640 KLF
b.) 18.0 KIP CONC. LOAD

LOAD PER LANE PER SPAN:

$$.05\left[(116.0)(.640) + 18.0\right] = 4.612^{K}$$

LOAD PER PIER:

$$4 \text{ LANES} \times \underset{\text{REDUCTION}}{.75} \times 4.612 = \underline{13.836^{K}}$$

NOTE:
THE CENTER OF GRAVITY OF THE LONGITUDINAL FORCE SHALL BE LOCATED 6 FEET ABOVE THE DECK SLAB.

PIER LOADS

WIND LOADS (W) 1.2.14

(A) SUPERSTRUCTURE DESIGN

UNIFORMLY DISTRIBUTED WIND LOAD FOR GIRDERS IS 50 PSF

LOADING PER LINEAR FOOT OF DECK:

JERSEY BARRIER	3'-0"
CONC. DECK	0'-8"
P/S GIRDER	6'-0"
	9'-8" DEPTH

$(9.67')(50) = 483.5$ PLF

ASSUME CHAINLINK FENCE EXTENDS 2' ABOVE JERSEY BARRIER AND DOES NOT COMPLETELY STOP OR DEFLECT WIND:

USE 35 PSF FOR FENCING

$(483.5) + 2(35) = 553.5$ PLF

TOTAL WIND LOAD PER SPAN:

$(.5535 \text{ KLF})(116) = \underline{64.2 \text{ KIPS}}$

WIND LOAD ON A MOVING LOAD (WL)

$(100 \text{ PLF})(116' \text{ LONG SPAN}) = \underline{11.6^{K} \text{ PER SPAN}}$

NOTE: APPLY AT RIGHT ANGLES TO LONGITUDINAL AXIS OF BRIDGE. APPLY 6' ABOVE DECK.

TSR

PIER LOADS

WIND LOADS (W) 1.2.14

(B) SUBSTRUCTURE DESIGN

(1) FORCES FROM SUPERSTRUCTURE

USE SAME LOADING AS CALCULATED IN PART (A)

(2) FORCES APPLIED DIRECTLY TO THE SUBSTRUCTURE

WIND LOADING IS ASSUMED TO BE 40 PSF ACTING ON HORIZONTAL LINES AT THE CENTERS OF GRAVITY OF THE EXPOSED AREAS.

FOR LOAD GROUP II AND V

USE 40 PSF

FOR LOAD GROUP III AND VI

USE 40 PSF (1.00 - .70) = 12 PSF

(3) OVERTURNING FORCES

AN UPWARD FORCE APPLIED AT THE WINDWARD QUARTER POINT OF THE TRANSVERSE SUPERSTRUCTURE WIDTH

FOR LOAD GROUP II AND V

20 PSF (83)(116) = 192.56 K/SPAN

FOR LOAD GROUP III AND VI

6 PSF (83)(116) = 57.77 K/SPAN

PIER LOADS

THERMAL FORCES (T) 1.2.15

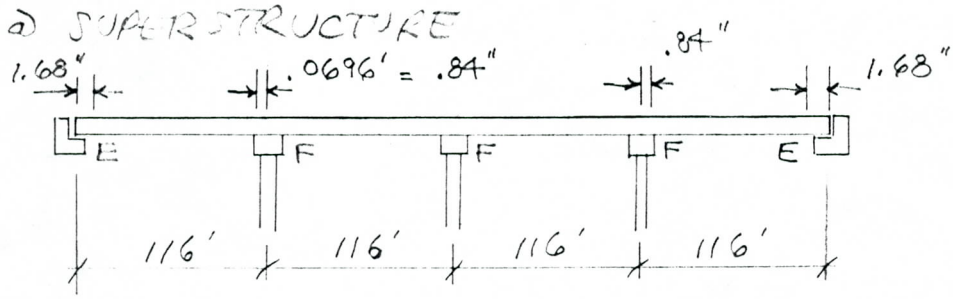
PHOENIX AREA TEMPERATURES RANGE FROM 20°F TO 120°F.

THE BRIDGE MAY BE CONSTRUCTED UNDER EITHER EXTREME. THEREFORE CONSIDERATION OF 100°F SHOULD BE CONSIDERED IN PIER AND SUPERSTRUCTURE DESIGN

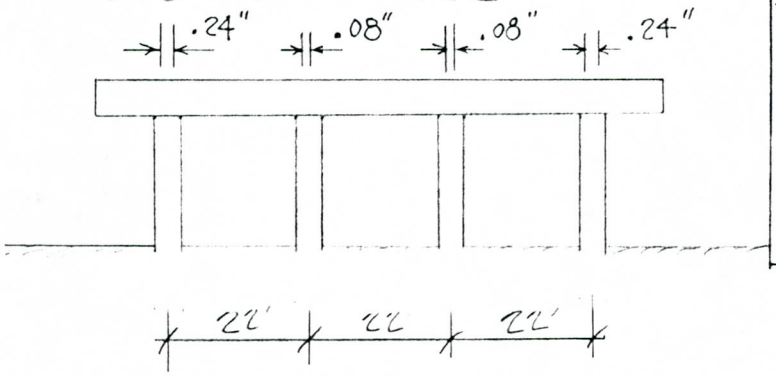
THERMAL COEFFICIENT FOR CONCRETE:
0.000006 PER °F
AASHTO 1.5.23 (E)

FOR 100°F USE 0.0006

a) SUPERSTRUCTURE



a) SUBSTRUCTURE



.0006	.0008	
.08	.107	PIER
.24	.320	
.84	1.12	ABUT
1.68	2.24	

SHRINKAGE COEFFICIENT FOR CONCRETE IS 0.0002. AASHTO 1.5.23(E)

A STRUCTURE CAST IN JULY MAY EXPERIENCE A TOTAL CHANGE OF .0006 + .0002 = .0008 BY THE FOLLOWING DECEMBER.

75R

PIER
LOADINGS

FORCE OF STREAM CURRENT ON PIERS

1.2.17

P = PRESSURE IN PSF

V = VELOCITY OF WATER IN FT/SEC

K = 2/3 FOR CIRCULAR PIERS

$$P = (2/3)(5)^2 = 16.67 \text{ PSF}$$

FLOW DEPTH IS 18'

EFFECTIVE PIER WIDTH = 5'Φ + 4' FOR DEBRIS

FORCE PER COLUMN:

$$18 \times 9 \times 16.67 \text{ PSF} = 2700^{\#}$$

FORCE PER PIER:

$$2700^{\#}/\text{COL} \times 4 \text{ COL/PIER} = 10{,}800^{\#}$$

PIER LOADS

EARTHQUAKE FORCES AASHTO 1.2.20

EQUIVALENT STATIC FORCE METHOD

TOTAL WEIGHT OF STRUCTURE: (W)

SUPERSTRUCTURE: $8 \times 2.942 \times 4 \times 116 = 10921^{K}$
PIER CAPS: $3 \times 88 \times 6 \times 6 \times .15 = 1426$
$12,347^{K}$

TOTAL FORCE TO CAUSE ONE INCH LATERAL DEFLECTION: (P)

(a.) PERPENDICULAR TO AXIS OF PIER CAP

$4 \text{ COL/PIER} \times 3 \text{ PIERS} \times 35.58 \, {}^{K/IN}/_{COL} = 426.96 \, {}^{K}/_{IN}$

(b.) PARALLEL TO AXIS OF PIER CAP

$100^{K}/_{.007 \, FT} = 14,286 \, {}^{K}/_{FT} = 1191 \, {}^{K}/_{IN}$

PERIOD OF VIBRATION OF STRUCTURE (T)

(a.) PERPENDICULAR TO AXIS OF PIER CAP

$$T = 0.32 \sqrt{\frac{12,347}{426.96}} = 1.72 \text{ SEC}$$

(b.) PARALLEL TO AXIS OF PIER CAP

WEIGHT CARRIED PER PIER
SUPERSTR. $3 \times 2.942 \times 116 = 2731^{K}$
CAP $.15 \times 6 \times 6 \times 88 = 476$
3207^{K}

$$T = 0.32 \sqrt{\frac{3,207}{1,191}} = 0.53 \text{ SEC}$$

TSR

PIER LOADS

THE COMBINE RESPONSE COEFFICIENT (C) FOR BOTH DIRECTIONS IS AS FOLLOWS:
THE FRAME FACTOR (F) = 1.0

EQ = C.F.W

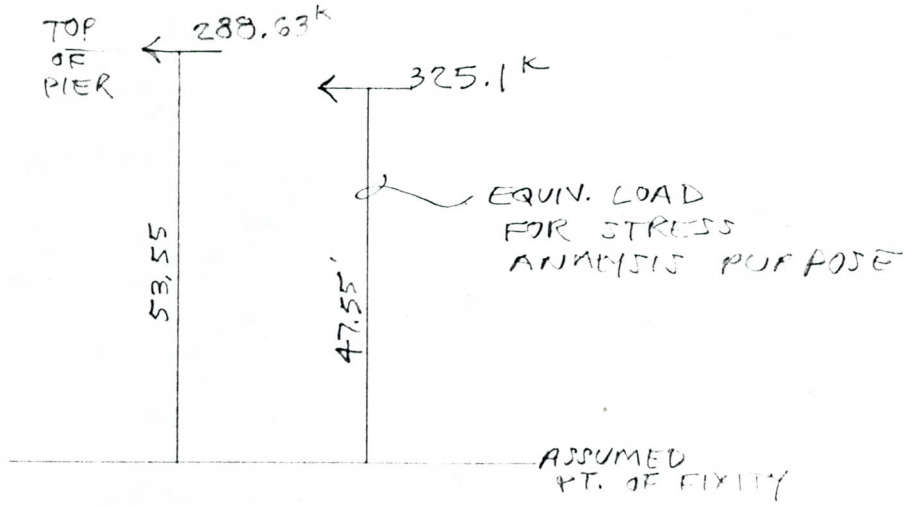
(a.) PERPENDICULAR TO AXIS OF PIER CAP

$$EQ = (.06)(1.0)\frac{12,347}{3 \text{ PIERS}} = 246.94^{K}/\text{PIER}$$

$$\frac{246.94}{4 \text{ COLUMNS}} = \underline{61.74^{K}/\text{COL.}}$$

(b.) PARALLEL TO AXIS OF PIER CAP

$$EQ = (.09)(1.0)\,3207 = \underline{288.63^{K}/\text{PIER}}$$



75K

ABUTMENT FOUNDATION LOADS

DEAD LOAD:

SUPERSTRUCTURE
8 x 116 x 2.942 KLF x 1/2 = 1366 K
BEAM SEAT & BACK WALL
88 x 6 x 6 x .15 KCF = 476

SUB TOTAL 1842 K

COLUMNS TO CHANNEL ℄.
4 x π(2.5)² x 20 x .15 KCF = 236

DEAD LOAD TOTAL 2,078 K

LIVE LOAD:

4 LANES x .75 PROBABILITY x 78.1 K = 235 K

LOADING AS PER AASHTO GROUP IA:
2078 + (2) 235 = 2,548 K

PIER FOUNDATION LOADS

DEAD LOAD:

SUPERSTRUCTURE
3 x 116 x 2.942 KLF = 2731 K
CAP BEAM
88 x 6 x 6 x .15 KCF = 476
COLUMNS = 236

3,443 K

LIVE LOAD: 4 x .75 x 141.2 = 424 K

GROUP IA 3443 + 2(424) = 4,291 K

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040 SHT. P.9 OF 86
DESC. 75 TH AVE MCFCD BY AGD
DATE SEPT 6, 1984

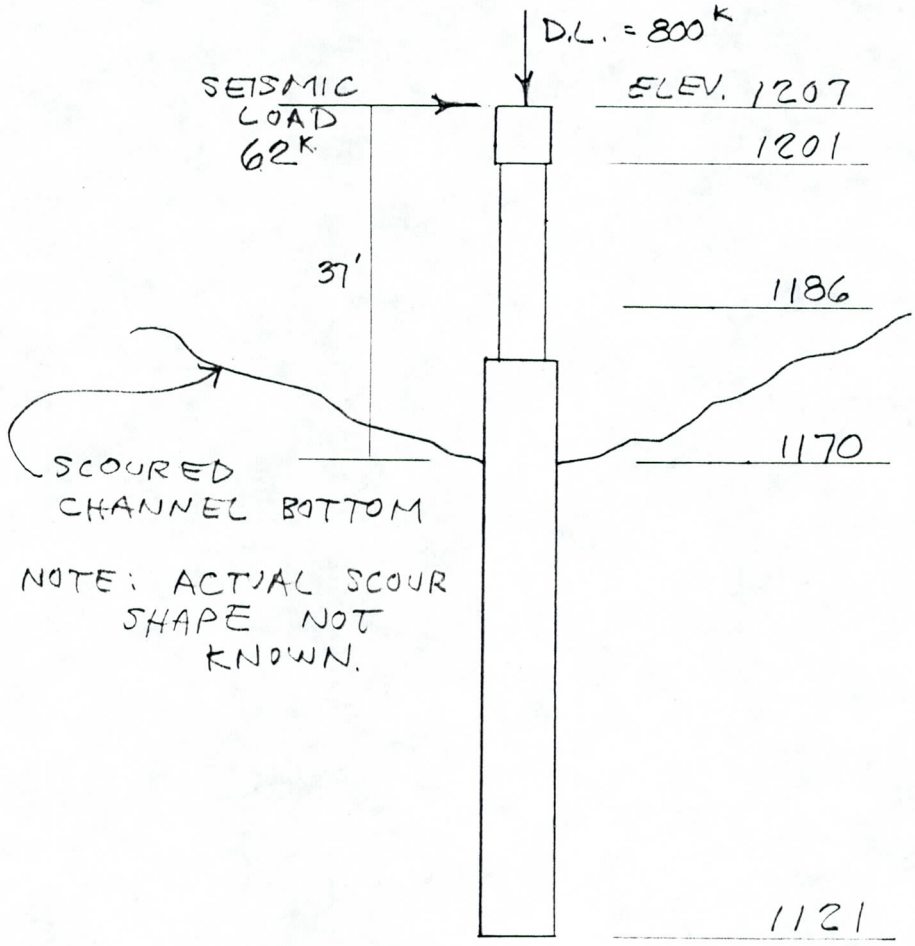
TSR

LATERAL LOAD CAPACITY OF 7' Ø CAISSON

TOP OF PIER CAP ELEV. 1207

CHANNEL INVERT ELEV. 1186

SCOUR DEPTH IS 16 FEET

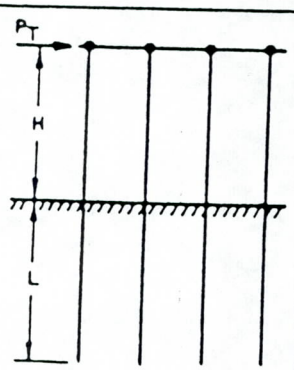
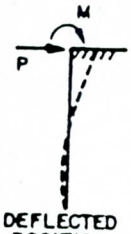


NOTE: ACTUAL SCOUR SHAPE NOT KNOWN.

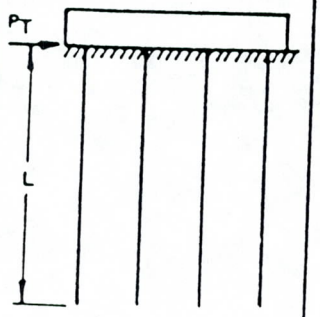
REFERENCE: "FOUNDATIONS AND EARTH STRUCTURES DESIGN MANUAL 7.2"

BY THE NAVAL FACILITIES ENGINEERING COMMAND MAY 1982

CASE I. FLEXIBLE CAP, ELEVATED POSITION

CONDITION	LOAD AT GROUND LINE	DESIGN PROCEDURE
 n = NUMBER OF PILES	FOR EACH PILE: $P = \frac{P_T}{n}$ $M = PH$  DEFLECTED POSITION	FOR DEFINITION OF PARAMETERS SEE FIGURE 12 1. COMPUTE RELATIVE STIFFNESS FACTOR. $T = \left(\frac{EI}{f}\right)^{1/5}$ 2. SELECT CURVE FOR PROPER $\frac{L}{T}$ IN FIGURE 11. 3. OBTAIN COEFFICIENTS F_δ, F_M, F_V AT DEPTHS DESIRED. 4. COMPUTE DEFLECTION, MOMENT AND SHEAR AT DESIRED DEPTHS USING FORMULAS OF FIGURE 11. NOTE: "f" VALUES FROM FIGURE 9 AND CONVERT TO LB/IN^3.

CASE II. PILES WITH RIGID CAP AT GROUND SURFACE

		1. PROCEED AS IN STEP 1, CASE I. 2. COMPUTE DEFLECTION AND MOMENT AT DESIRED DEPTHS USING COEFFICIENTS F_δ, F_M AND FORMULAS OF FIGURE 12. 3. MAXIMUM SHEAR OCCURS AT TOP OF PILE AND EQUALS $P = \frac{P_T}{n}$ IN EACH PILE.

CASE III. RIGID CAP, ELEVATED POSITION

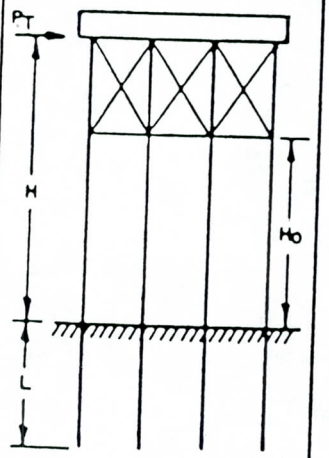
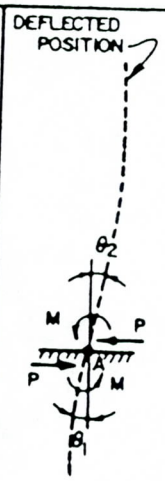
		1. ASSUME A HINGE AT POINT A WITH A BALANCING MOMENT M APPLIED AT POINT A. 2. COMPUTE SLOPE θ_2 ABOVE GROUND AS A FUNCTION OF M FROM CHARACTERISTICS OF SUPERSTRUCTURE. 3. COMPUTE SLOPE θ_1 FROM SLOPE COEFFICIENTS OF FIGURE 13 AS FOLLOWS: $\theta_1 = F_\theta\left(\frac{P_T^2}{EI}\right) + F_\theta\left(\frac{MT}{EI}\right)$ 4. EQUATE $\theta_1 = \theta_2$ AND SOLVE FOR VALUE OF M. 5. KNOWING VALUES OF P AND M, SOLVE FOR DEFLECTION, SHEAR, AND MOMENT AS IN CASE I. NOTE: IF GROUND SURFACE AT PILE LOCATION IS INCLINED, LOAD P TAKEN BY EACH PILE IS PROPORTIONAL TO I/H_o^3.

FIGURE 10
Design Procedure for Laterally Loaded Piles

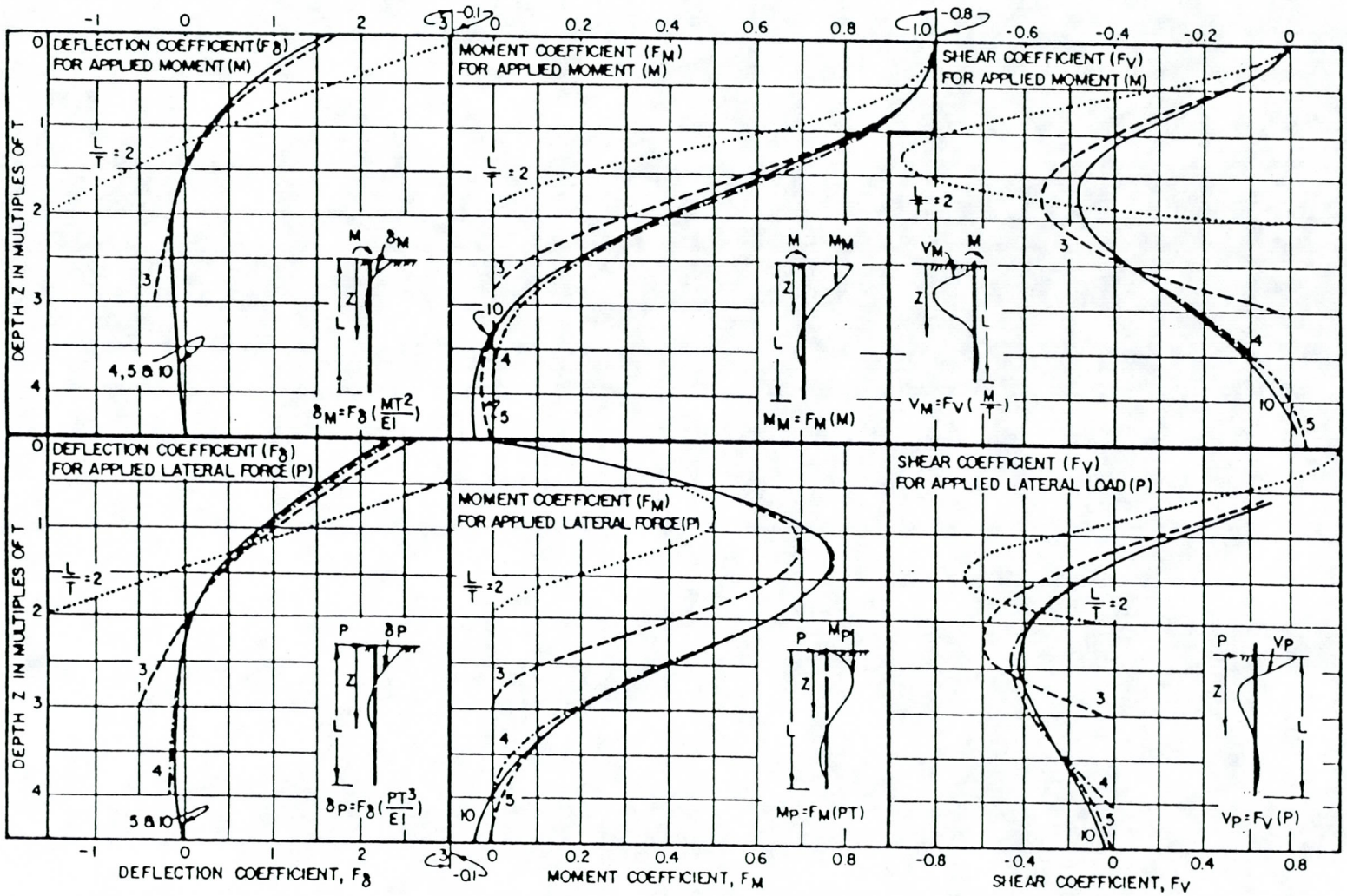


FIGURE 11
Influence Values for Pile with Applied Lateral Load and Moment
(Case I. Flexible Cap or Hinged End Condition)

RELATIVE STIFFNESS FACTOR (T)

E = MODULUS OF ELASTICITY OF CAISSON
$= 57{,}000\sqrt{4000} = 3.6 \times 10^6$ PSI

I = MOMENT OF INERTIA OF CAISSON
$= .785\,R^4 = .785\,(42)^4$
$= 2.44 \times 10^6$ IN^4

f = COEFFICIENT OF SUBGRADE REACTION
= 50 PCI AS PER SUBSURFACE INVEST.

USE 25 PCI TO COMPENSATE FOR CYCLIC LOADING

$$T = \left(\frac{EI}{f}\right)^{1/5} = \left[\frac{(3.6\times10^6)(2.44\times10^6)}{25}\right]^{1/5} = 203.7 \text{ INCHES}$$

$$\frac{L}{T} = \frac{(1170-1121)\,12}{203.7} = 2.88$$

DEFLECTION AT SURFACE DUE TO LATERAL LOAD ONLY

$M_{LATERAL} = (62^K)(37') = 2294.0$ K-FT
$P_{LATERAL} = 62^K$

$$\delta_M = F_\delta\left(\frac{MT^2}{EI}\right) = 2.7\left[\frac{(2294)(12000)(203.7)^2}{3.6\times10^6 \times 2.44\times10^6}\right] = 0.351''$$

$$\delta_P = F_\delta\left(\frac{PT^3}{EI}\right) = 3.0\left[\frac{(62000)(203.7)^3}{3.6\times10^6 \times 2.44\times10^6}\right] = 0.179''$$

$$\delta_M + \delta_P = 0.351 + 0.179 = \underline{\underline{0.530''}}$$

TSR

LOCATION OF POINT OF ZERO DEFLECTION

ASSUME ZERO DEFL. OCCURS 2 MULTIPLES OF T BELOW SURFACE.

$2T = 2(203.7) = 407.4$ INCHES
$= 33.95$ FEET

BOTTOM OF SCOUR $= 1170.00$
$- \ 33.95$

LOCATION OF ZERO DEFL. $= 1136.05$

DEFLECTION AT TOP OF PIER DUE TO LATERAL LOAD ONLY

FOR 5'Ø SECTION

$I = .785(30)^4 = .635 \times 10^6 \ IN^4$

$$\Delta_{5'\phi} = \frac{PL^3}{3EI}$$

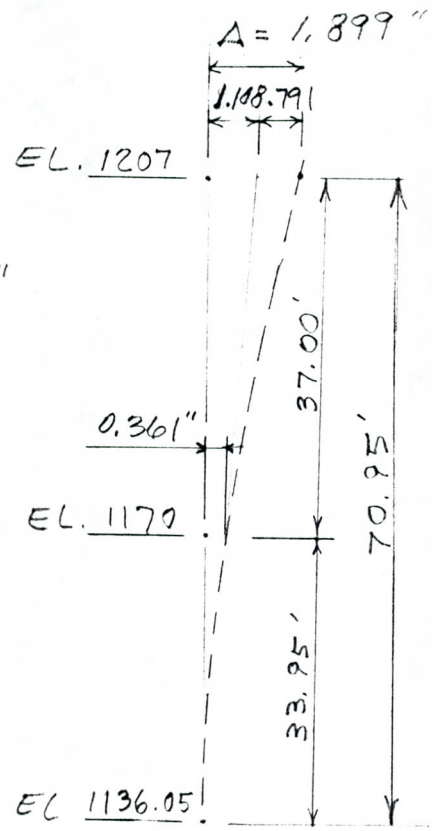
$$\frac{(62000)(37.0 \times 12)^3}{3(.635 \times 10^6)(3.6 \times 10^6)} = 0.791''$$

TO ACCOUNT FOR ROTATION AT SURFACE

$$\frac{.530}{33.95} = \frac{X}{70.95}$$

$X = 1.108$

$\Delta = .791 + 1.108 = 1.899$



TSR

DEFLECTION AT GROUND SURFACE DUE TO DEAD LOAD × LATERAL DEFL.

MOMENT = $800^K \times 1.899'' = 1519.2$ K-IN

$$\delta_{M_{DL}} = 2.7\left[\frac{(1519.2)(1000)(203.7)^2}{3.6\times10^6 \times 2.44\times10^6}\right] = 0.019''$$

NEGLIGIBLE

DESIGN MOMENTS

DEPTH Z IN MULTIPLES OF T ; T = 16.98 FEET

Z = 0 ; ELEV. 1170

MAGNIFICATION FACTOR

$$M = (62)(37) + (800)\left(\frac{1.899}{12}\right) = 2420.6 \text{ K-FT}$$

$P = 62^K$

$M_M = F_M(M) = 1.0(2420.6)$ K-FT

$M_P = F_M(PT) =$ 0

TOTAL MOMENT 2,420.6 K-FT

Z = 0.5 ; ELEV. 1161.5

$M_M = .95(2420.6) =$ 2299.57 K-FT

$M_P = .45(62 \times 16.98) =$ 473.74

TOTAL MOMENT 2,773.31 K-FT

Z = 0.75 ; ELEV 1157.3

$M_M = .90(2420.6) =$ 2178.54 K-FT

$M_P = .65(62 \times 16.98) =$ 684.29

TOTAL MOMENT 2862.83 K-FT ✓

TSR

CHECK ELASTIC STABILITY

SUPERSTRUCTURE AXIAL LOAD = 800 K

COLUMN / CAISSON LOAD

7'φ $\pi(3.5)^2 \times .15 \times 27.55 =$ 159
5'φ $\pi(2.5)^2 \times .15 \times 20.0 =$ 59

1018 K

AREA OF 7φ SECTION : $\pi(3.5)^2 = 38.48\ FT^2$

SECTION MODULUS = $.785(3.5)^3 = 33.66\ FT^3$

MAX. TENSILE STRESS:

$$\frac{P}{A} - \frac{M}{S} = \frac{1018}{38.48} - \frac{2862.83}{33.66}$$

$$= 26.455 - 85.05 = -58.596 \text{ KSF}$$

$$= 0.407 \text{ KSI}$$

MOD. OF RUPTURE $= 7.5\sqrt{4000}$

$= 0.474$ KSI

CAISSON DOES NOT CRACK DURING BENDING AT MAX. MOMENT

TSR

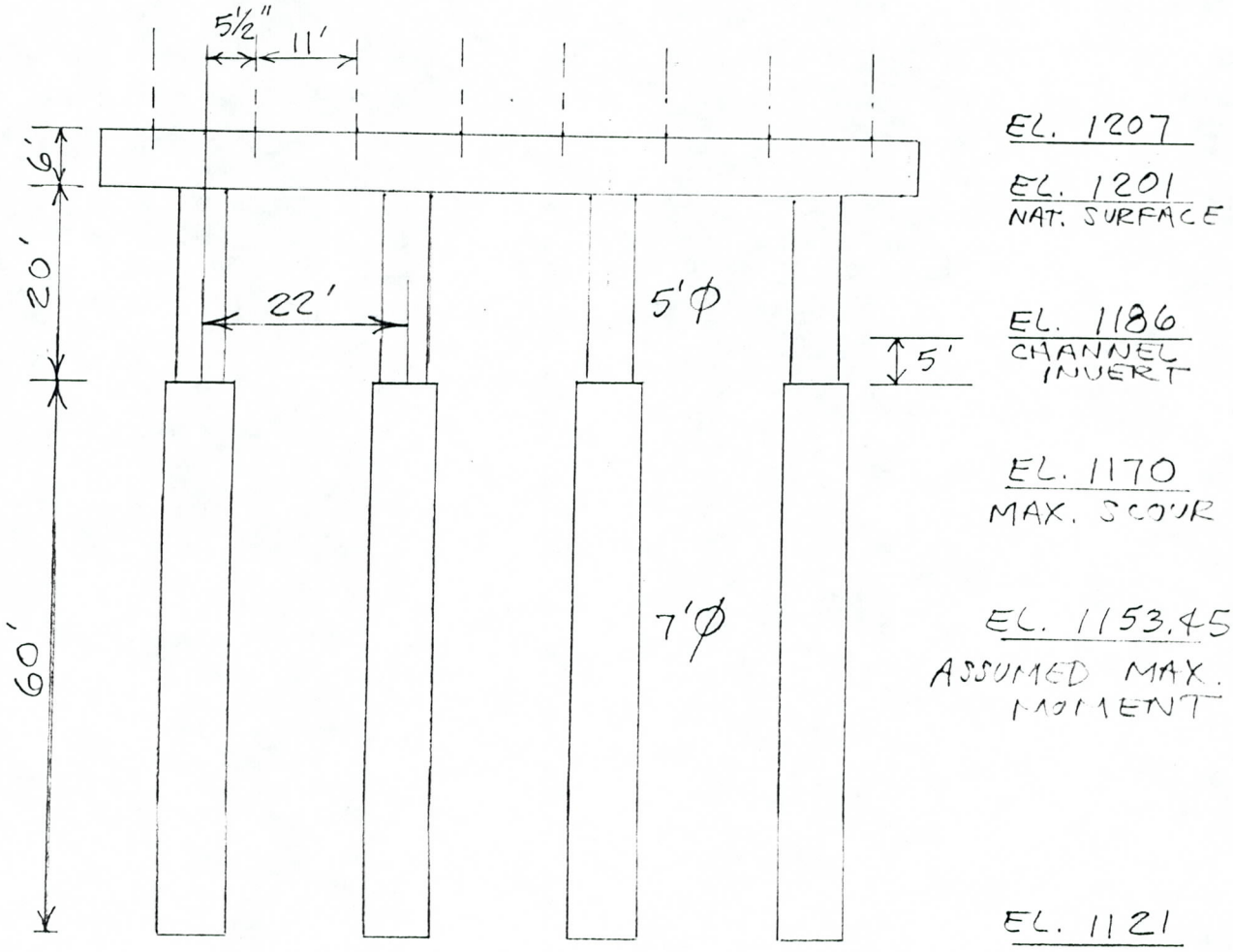
ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75 TH AVE MCFCD

SHT. P.16 OF 86
BY AGD
DATE AUG. 23, 1984

TSR

PIER DESIGN



DEAD LOAD:

SUPERSTRUCTURE	8 x 116 x 2.942 KLF =	2731 K
CAP BEAM	88 x 6 x 6 x .15 KCF =	476
COLUMNS	4 x π(2.5)² x 30 x .15 =	354
		3561 K

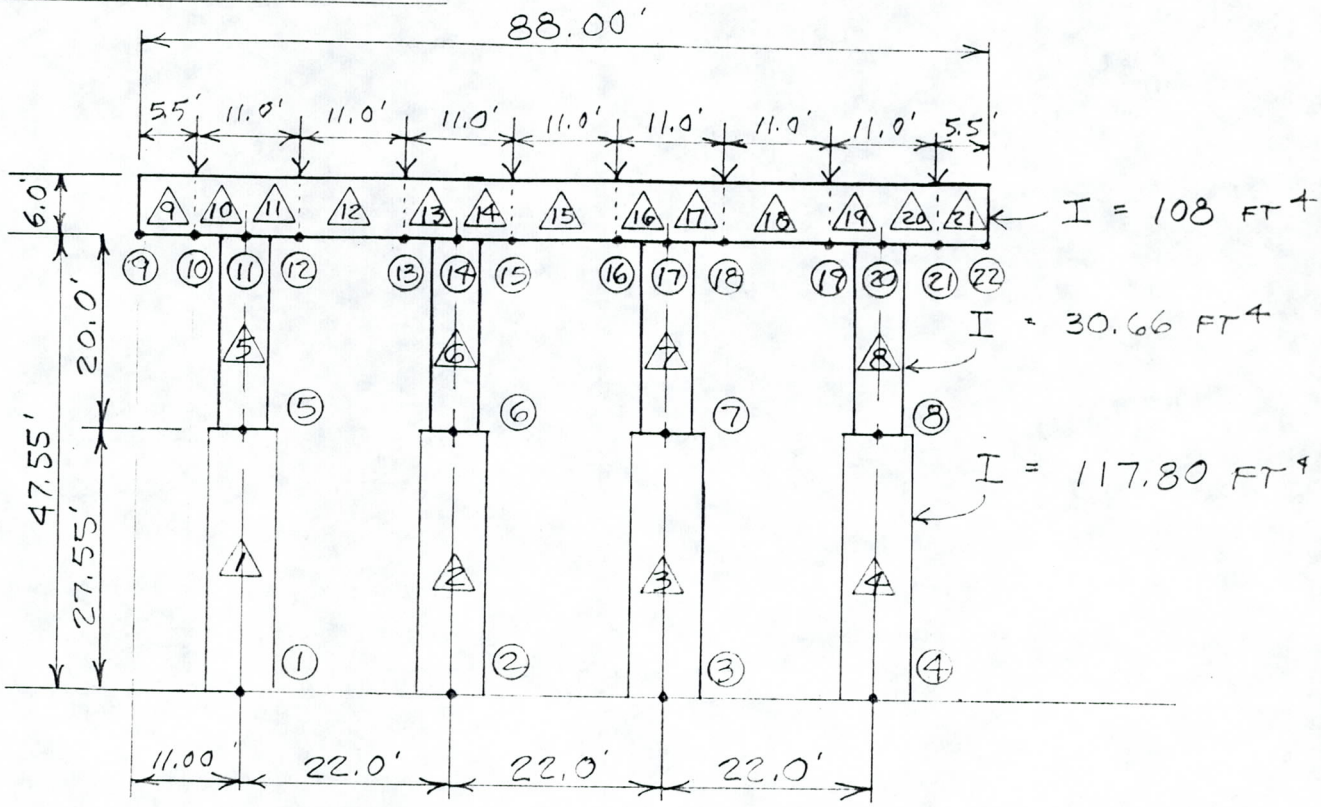
LIVE LOAD: 4 LANE x .75 PROBABILITY x 141.2 K/LANE = 424
3985 K

REQ. BEARING AREA = $\frac{3985^K}{30 \text{ KSF}}$ = 132.8 FT²

BEARING AREA OF 4 - 7.0'Ø CAISSONS:
4 x π(3.5)² = 153.9 FT²

TSR

PIER DESIGN



$E = 57000\sqrt{4000} = 3.605 \times 10^6$ PSI
$= 3.605 \times 10^3$ KSI
$= 519{,}120$ KSF

③ JOINT COORDINATE

△5 MEMBER

TSR

PIER DESIGN

STRUCTURE MCFCD-PIER
TYPE PLANE FRAME
NUMBER OF JOINTS 22
NUMBER OF MEMBERS 21
NUMBER OF SUPPORTS 4
NUMBER OF LOADINGS 4

JOINT COORDINATES

1	11.0	0.	S
2	33.0	0.	S
3	55.0	0.	S
4	77.0	0.	S
5	11.0	27.55	
6	33.0	27.55	
7	55.0	27.55	
8	77.0	27.55	
9	0.0	47.55	
10	5.5	47.55	
11	11.0	47.55	
12	16.5	47.55	
13	27.5	47.55	
14	33.0	47.55	
15	38.5	47.55	
16	49.5	47.55	
17	55.0	47.55	
18	60.5	47.55	

PIER DESIGN

TSR

19	71.5	47.55
20	77.0	47.55
21	82.5	47.55
22	88.0	47.55

MEMBER INCIDENCES

1	1	5
2	2	6
3	3	7
4	4	8
5	5	11
6	6	14
7	7	17
8	8	20
9	9	10
10	10	11
11	11	12
12	12	13
13	13	14
14	14	15
15	15	16
16	16	17
17	17	18

75R

PIER DESIGN

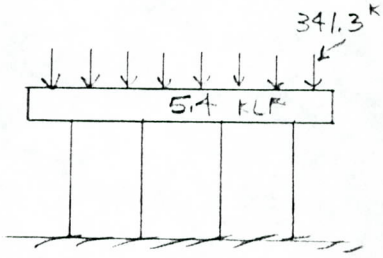
18 18 19
19 19 20
20 20 21
21 21 22

MEMBER PROPERTIES PRIZMATIC
1 THRU 4 AX 38.49 IZ 117.80
5 THRU 8 AX 19.63 IZ 30.66
9 THRU 21 AX 36.00 IZ 108.0

CONSTANTS E 518400 ALL (KSF)

LOADING 1 DEAD LOAD OF CAP AND GIRDERS

MEMBER LOADS
9 FORCE Y UNIFORM - 5.4
10
11
12
13
14
15
16
17
18
19
20
21



JOINT LOADS

10 FORCE Y - 341.3
12
13
15
16
18
19
21

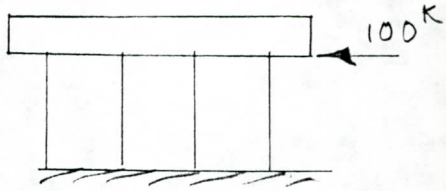
TSR

PIER DESIGN

LOADING 2 LATERAL LOAD

JOINT LOAD

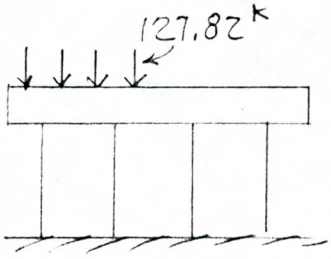
22 FORCE X -100.0



LOADING 3 TRAFFIC LOAD

JOINT LOAD

10 FORCE Y -127.82
12
13
15



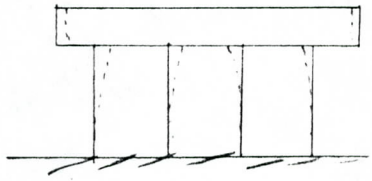
LOADING 4 TEMPERATURE & SHRINKAGE

MEMBER LENGTH COEFFICIENT

9 THRU 21 -.0008

SOLVE

STOP



TEMP .000006 (100) = .0006
SHRINKAGE = .0002
.0008

```
.NULL.
00001: PRIME VERSION OF STRESS(3.2)          12/09/1978
00002: ==================================================
00003:
00004: INPUT LISTING OF STRESS PROBLEM
00005: ==============================
00006:
00007:
00008: STRUCTURE MCFCD-PIER
00009: TYPE PLANE FRAME
00010: NUMBER OF JOINTS 22
00011: NUMBER OF MEMBERS 21
00012: NUMBER OF SUPPORTS 4
00013: NUMBER OF LOADINGS 4
00014: TABULATE ALL
00015: JOINT COORDINATES
00016: 1   11.00   0. S
00017: 2   33.00   0. S
00018: 3   55.00   0. S
00019: 4   77.00   0. S
00020: 5   11.0   27.55
00021: 6   33.0   27.55
00022: 7   55.0   27.55
00023: 8   77.0   27.55
00024: 9    0.0   47.55
00025: 10   5.5   47.55
00026: 11  11.0   47.55
00027: 12  16.5   47.55
00028: 13  27.5   47.55
00029: 14  33.0   47.55
00030: 15  38.5   47.55
00031: 16  49.5   47.55
00032: 17  55.0   47.55
00033: 18  60.5   47.55
00034: 19  71.5   47.55
00035: 20  77.0   47.55
00036: 21  82.5   47.55
00037: 22  88.0   47.55
00038: MEMBER INCIDENCE
00039: 1  1  5
00040: 2  2  6
00041: 3  3  7
00042: 4  4  8
00043: 5  5  11
00044: 6  6  14
00045: 7  7  17
00046: 8  8  20
00047: 9  9  10
00048: 10  10  11
00049: 11  11  12
00050: 12  12  13
00051: 13  13  14
00052: 14  14  15
00053: 15  15  16
00054: 16  16  17
00055: 17  17  18
00056: 18  18  19
00057: 19  19  20
00058: 20  20  21
00059: 21  21  22
```

```
00060: MEMBER PROPERTIES PRISMATIC
00061: 1 THRU 4  AX 38.49   IZ 117.80
00062: 5 THRU 8  AX 19.63   IZ  30.66
00063: 9 THRU 21 AX 36.00   IZ 108.00
00064: CONSTANTS E 519120.0 ALL
00065: LOADING 1  DEAD LOAD OF CAP PLUS GIRDERS
00066: MEMBER LOADS
00067: 9 FORCE Y UNIFORM -5.4
00068: 10 FORCE Y UNIFORM -5.4
00069: 11 FORCE Y UNIFORM -5.4
00070: 12 FORCE Y UNIFORM -5.4
00071: 13 FORCE Y UNIFORM -5.4
00072: 14 FORCE Y UNIFORM -5.4
00073: 15 FORCE Y UNIFORM -5.4
00074: 16 FORCE Y UNIFORM -5.4
00075: 17 FORCE Y UNIFORM -5.4
00076: 18 FORCE Y UNIFORM -5.4
00077: 19 FORCE Y UNIFORM -5.4
00078: 20 FORCE Y UNIFORM -5.4
00079: 21 FORCE Y UNIFORM -5.4
00080: JOINT LOADS
00081: 10 FORCE Y -341.3
00082: 12 FORCE Y -341.3
00083: 13 FORCE Y -341.3
00084: 15 FORCE Y -341.3
00085: 16 FORCE Y -341.3
00086: 18 FORCE Y -341.3
00087: 19 FORCE Y -341.3
00088: 21 FORCE Y -341.3
00089: LOADING 2  LATERAL LOAD
00090: JOINT LOAD
00091: 22 FORCE X -100.0
00092: LOADING 3  TRAFFIC LOADING
00093: JOINT LOADS
00094: 10 FORCE Y -127.82
00095: 12 FORCE Y -127.82
00096: 13 FORCE Y -127.82
00097: 15 FORCE Y -127.82
00098: LOADING 4  TEMPERATURE PLUS SHRINKAGE
00099: MEMBER LENGTH COEFFICIENT
00100: 9 THRU 21 -.0008
00101: SOLVE
00102:1
00103:1
00104:
00105:
00106:
```

STRUCTURE MCFCD-PIER
==========

LOADING 1 DEAD LOAD OF CAP PLUS GIRDERS
==========

MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	826.857	4.038	88.57
1	5	-826.857	-4.038	22.67
2	2	776.046	-0.792	-17.49
2	6	-776.046	0.792	-4.33
3	3	776.045	0.791	17.47
3	7	-776.045	-0.791	4.33
4	4	826.814	-4.037	-88.56
4	8	-826.814	4.037	-22.66
5	5	826.856	4.038	-22.67
5	11	-826.856	-4.038	103.43
6	6	776.046	-0.792	4.33
6	14	-776.046	0.792	-20.17
7	7	776.045	0.791	-4.33
7	17	-776.045	-0.791	20.16
8	8	826.814	-4.037	22.66
8	20	-826.814	4.037	-103.41
9	9	0.000	-0.032	-0.07
9	10	-0.000	29.731	-81.78
10	10	-0.000	-371.049	81.70
10	11	0.000	400.748	-2204.14
11	11	-4.038	426.085	2100.67
11	12	4.038	-396.385	161.12
12	12	-4.038	55.084	-161.16
12	13	4.038	4.316	440.39
13	13	-4.038	-345.626	-440.41
13	14	4.038	375.326	-1542.21
14	14	-3.246	400.703	1562.34
14	15	3.246	-371.003	559.84
15	15	-3.246	29.700	-559.85
15	16	3.246	29.700	559.85
16	16	-3.246	-371.012	-559.88
16	17	3.246	400.712	-1562.37
17	17	-4.037	375.320	1542.19
17	18	4.037	-345.621	440.39
18	18	-4.037	4.321	-440.41
18	19	4.037	55.079	161.25
19	19	-4.037	-396.392	-161.27
19	20	4.037	426.092	-2100.57
20	20	0.000	400.713	2203.94
20	21	-0.000	-371.013	-81.70
21	21	0.000	29.696	81.67
21	22	-0.000	0.004	-0.02

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
5	-0.000	-0.000	0.00
6	0.000	-0.000	-0.00
7	-0.000	-0.000	-0.00
8	0.000	-0.000	0.00
9	0.000	-0.032	-0.07
10	-0.000	-341.317	-0.08
11	-0.000	-0.023	-0.04
12	0.000	-341.301	-0.04
13	-0.000	-341.310	-0.03
14	-0.000	-0.017	-0.03
15	0.000	-341.303	-0.01
16	0.000	-341.312	-0.03
17	-0.000	-0.012	-0.02
18	0.000	-341.299	-0.02
19	-0.000	-341.313	-0.03
20	-0.000	-0.009	-0.04
21	-0.000	-341.317	-0.03
22	-0.000	0.004	-0.02

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	-4.038	826.857	88.57
2	0.792	776.046	-17.49
3	-0.791	776.045	17.47
4	4.037	826.814	-88.56

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
5	0.0003	-0.0011	-0.0000
6	-0.0001	-0.0011	0.0000
7	0.0001	-0.0011	-0.0000
8	-0.0003	-0.0011	0.0000
9	-0.0000	-0.0045	0.0002
10	-0.0000	-0.0035	0.0002
11	-0.0000	-0.0028	0.0001
12	-0.0000	-0.0028	-0.0000
13	-0.0000	-0.0028	0.0000
14	-0.0000	-0.0026	-0.0000
15	-0.0000	-0.0029	-0.0001
16	0.0000	-0.0029	0.0001
17	0.0000	-0.0026	0.0000
18	0.0000	-0.0028	-0.0000
19	0.0000	-0.0028	0.0000
20	0.0000	-0.0028	-0.0001
21	0.0000	-0.0035	-0.0002
22	0.0000	-0.0045	-0.0002

1

00225:
00226:
00227: STRUCTURE MCFCD-PIER
00228: ==========
00229:
00230:
00231:
00232:
00233: LOADING 2 LATERAL LOAD
00234: ==========
00235:
00236:
00237:
00238: MEMBER FORCES
00239:

00240: MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
00241: 1	1	26.510	-23.826	-767.93
00242: 1	5	-26.510	23.826	111.52
00243: 2	2	-8.754	-25.755	-811.35
00244: 2	6	8.754	25.755	101.82
00245: 3	3	9.331	-25.979	-818.39
00246: 3	7	-9.331	25.979	102.67
00247: 4	4	-27.083	-24.434	-787.60
00248: 4	8	27.083	24.434	114.46
00249: 5	5	26.510	-23.826	-111.52
00250: 5	11	-26.510	23.826	-365.00
00251: 6	6	-8.754	-25.755	-101.82
00252: 6	14	8.754	25.755	-413.28
00253: 7	7	9.331	-25.979	-102.67
00254: 7	17	-9.331	25.979	-416.92
00255: 8	8	-27.083	-24.434	-114.46
00256: 8	20	27.083	24.434	-374.22
00257: 9	9	0.000	-0.002	-0.00
00258: 9	10	0.000	0.002	-0.01
00259: 10	10	0.000	-0.005	0.00
00260: 10	11	0.000	0.005	-0.03
00261: 11	11	23.819	26.505	365.03
00262: 11	12	-23.819	-26.505	-219.25
00263: 12	12	23.821	26.505	219.25
00264: 12	13	-23.821	-26.505	72.31
00265: 13	13	23.822	26.505	-72.31
00266: 13	14	-23.822	-26.505	218.09
00267: 14	14	49.575	17.751	195.19
00268: 14	15	-49.575	-17.751	-97.56
00269: 15	15	49.582	17.752	97.56
00270: 15	16	-49.582	-17.752	97.71
00271: 16	16	49.582	17.751	-97.71
00272: 16	17	-49.582	-17.751	195.34
00273: 17	17	75.559	27.082	221.57
00274: 17	18	-75.559	-27.082	-72.62
00275: 18	18	75.561	27.082	72.62
00276: 18	19	-75.561	-27.082	225.28
00277: 19	19	75.556	27.082	-225.28
00278: 19	20	-75.556	-27.082	374.23
00279: 20	20	99.986	-0.001	-0.01
00280: 20	21	-99.986	0.001	-0.00
00281: 21	21	99.995	0.000	0.00
00282: 21	22	-99.995	-0.000	0.00

00283:
00284:

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
5	0.000	-0.000	0.00
6	0.000	0.000	0.00
7	0.000	-0.000	0.00
8	0.000	0.000	0.00
9	0.000	-0.002	-0.00
10	0.000	-0.003	-0.01
11	-0.007	-0.000	-0.00
12	0.002	-0.000	-0.00
13	0.002	0.000	-0.00
14	-0.002	0.000	0.00
15	0.006	0.000	0.00
16	-0.000	-0.000	-0.00
17	-0.002	-0.000	-0.00
18	0.002	-0.000	-0.00
19	-0.005	-0.000	-0.00
20	-0.003	0.000	-0.00
21	0.009	0.001	0.00
22	-99.995	-0.000	0.00

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	23.826	26.510	-767.93
2	25.755	-8.754	-811.35
3	25.979	9.331	-818.39
4	24.434	-27.083	-787.60

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
5	-0.0034	-0.0000	0.0002
6	-0.0036	0.0000	0.0002
7	-0.0036	-0.0000	0.0002
8	-0.0035	0.0000	0.0002
9	-0.0068	-0.0005	0.0000
10	-0.0068	-0.0003	0.0000
11	-0.0068	-0.0001	0.0000
12	-0.0068	0.0000	0.0000
13	-0.0068	0.0000	-0.0000
14	-0.0068	0.0000	0.0000
15	-0.0068	0.0000	-0.0000
16	-0.0068	-0.0000	-0.0000
17	-0.0069	-0.0000	0.0000
18	-0.0069	-0.0000	-0.0000
19	-0.0069	-0.0000	0.0000
20	-0.0070	0.0001	0.0000
21	-0.0070	0.0003	0.0000
22	-0.0070	0.0005	0.0000

1

00344:
00345:
00346: STRUCTURE MCFCD-PIER
00347: ==========
00348:
00349:
00350:
00351:
00352: LOADING 3 TRAFFIC LOADING
00353: ==========
00354:
00355:
00356:
00357: MEMBER FORCES
00358:

	MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
00359:					
00360:	1	1	271.354	0.129	-6.53
00361:	1	5	-271.354	-0.129	10.09
00362:	2	2	214.596	0.364	-1.36
00363:	2	6	-214.596	-0.364	11.39
00364:	3	3	33.338	0.605	3.95
00365:	3	7	-33.338	-0.605	12.71
00366:	4	4	-7.975	-1.098	-33.46
00367:	4	8	7.975	1.098	3.20
00368:	5	5	271.354	0.129	-10.09
00369:	5	11	-271.354	-0.129	12.68
00370:	6	6	214.596	0.364	-11.39
00371:	6	14	-214.596	-0.364	18.68
00372:	7	7	33.338	0.605	-12.71
00373:	7	17	-33.338	-0.605	24.81
00374:	8	8	-7.975	-1.098	-3.20
00375:	8	20	7.975	1.098	-18.76
00376:	9	9	0.000	-0.007	-0.01
00377:	9	10	0.000	0.007	-0.02
00378:	10	10	0.000	-127.836	0.01
00379:	10	11	0.000	127.836	-703.11
00380:	11	11	-0.130	143.513	690.41
00381:	11	12	0.130	-143.513	98.91
00382:	12	12	-0.130	15.691	-98.91
00383:	12	13	0.130	-15.691	271.51
00384:	13	13	-0.130	-112.133	-271.52
00385:	13	14	0.130	112.133	-345.22
00386:	14	14	-0.494	102.458	326.53
00387:	14	15	0.494	-102.458	236.99
00388:	15	15	-0.494	-25.362	-236.99
00389:	15	16	0.494	25.362	-41.99
00390:	16	16	-0.494	-25.363	41.99
00391:	16	17	0.494	25.363	-181.48
00392:	17	17	-1.098	7.974	156.68
00393:	17	18	1.098	-7.974	-112.82
00394:	18	18	-1.098	7.974	112.82
00395:	18	19	1.098	-7.974	-25.10
00396:	19	19	-1.099	7.975	25.10
00397:	19	20	1.099	-7.975	18.76
00398:	20	20	-0.001	-0.000	-0.00
00399:	20	21	0.001	0.000	0.00
00400:	21	21	-0.000	-0.000	-0.00
00401:	21	22	0.000	0.000	0.00

00402:
00403:

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
5	0.000	0.000	0.00
6	0.000	-0.000	0.00
7	0.000	-0.000	0.00
8	0.000	0.000	0.00
9	0.000	-0.007	-0.01
10	0.000	-127.829	-0.02
11	-0.001	-0.005	-0.01
12	0.000	-127.822	-0.01
13	0.000	-127.824	-0.01
14	0.000	-0.004	-0.01
15	0.000	-127.820	-0.00
16	0.000	-0.001	-0.00
17	0.000	-0.000	-0.00
18	0.000	0.000	0.00
19	-0.000	0.000	-0.00
20	-0.000	0.000	-0.00
21	0.001	0.000	0.00
22	0.000	0.000	0.00

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	-0.129	271.354	-6.53
2	-0.364	214.596	-1.36
3	-0.605	33.338	3.95
4	1.098	-7.975	-33.46

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
5	-0.0000	-0.0004	0.0000
6	-0.0000	-0.0003	0.0000
7	-0.0000	-0.0000	0.0000
8	-0.0001	0.0000	0.0000
9	-0.0003	-0.0014	0.0001
10	-0.0003	-0.0011	0.0001
11	-0.0003	-0.0009	0.0000
12	-0.0003	-0.0009	-0.0000
13	-0.0003	-0.0009	0.0000
14	-0.0003	-0.0007	0.0000
15	-0.0003	-0.0006	0.0000
16	-0.0003	-0.0003	0.0000
17	-0.0003	-0.0001	0.0000
18	-0.0003	-0.0000	0.0000
19	-0.0003	0.0000	-0.0000
20	-0.0003	0.0000	-0.0000
21	-0.0003	0.0000	-0.0000
22	-0.0003	0.0000	-0.0000

STRUCTURE MCFCD-PIER
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LOADING 4 TEMPERATURE PLUS SHRINKAGE
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MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	-86.349	90.839	2941.97
1	5	86.349	-90.839	-439.36
2	2	86.319	33.591	1053.13
2	6	-86.319	-33.591	-127.71
3	3	86.302	-33.552	-1051.91
3	7	-86.302	33.552	127.56
4	4	-86.291	-90.804	-2940.83
4	8	86.291	90.804	439.19
5	5	-86.349	90.839	439.36
5	11	86.349	-90.839	1377.43
6	6	86.319	33.591	127.71
6	14	-86.319	-33.591	544.11
7	7	86.302	-33.552	-127.56
7	17	-86.302	33.552	-543.49
8	8	-86.291	-90.804	-439.19
8	20	86.291	90.804	-1376.90
9	9	-0.033	0.006	0.01
9	10	0.033	-0.006	0.03
10	10	0.004	0.014	-0.02
10	11	-0.004	-0.014	0.10
11	11	-90.811	-86.330	-1377.54
11	12	90.811	86.330	902.73
12	12	-90.818	-86.329	-902.73
12	13	90.818	86.329	-46.90
13	13	-90.824	-86.331	46.89
13	14	90.824	86.331	-521.71
14	14	-124.412	-0.013	-22.40
14	15	124.412	0.013	22.33
15	15	-124.404	-0.013	-22.33
15	16	124.404	0.013	22.18
16	16	-124.396	-0.015	-22.19
16	17	124.396	0.015	22.11
17	17	-90.850	86.287	521.38
17	18	90.850	-86.287	-46.80
18	18	-90.838	86.286	46.80
18	19	90.838	-86.286	902.35
19	19	-90.859	86.286	-902.35
19	20	90.859	-86.286	1376.92
20	20	-0.082	-0.005	-0.03
20	21	0.082	0.005	-0.00
21	21	-0.039	0.000	0.00
21	22	0.039	0.000	0.00

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
5	-0.001	0.000	-0.00
6	-0.000	0.000	-0.00
7	0.000	-0.000	0.00
8	0.001	0.000	0.00
9	-0.033	0.006	0.01
10	0.037	0.008	0.00
11	0.025	0.006	-0.01
12	-0.008	0.000	-0.00
13	-0.006	-0.001	-0.00
14	0.003	-0.001	-0.01
15	0.008	-0.000	-0.00
16	0.008	-0.001	-0.00
17	-0.005	-0.001	-0.00
18	0.012	-0.000	-0.00
19	-0.021	-0.001	-0.00
20	-0.027	0.001	-0.00
21	0.043	0.005	-0.00
22	0.039	0.000	0.00

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	-90.839	-86.349	2941.97
2	-33.591	86.319	1053.13
3	33.552	86.302	-1051.91
4	90.804	-86.291	-2940.83

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
5	0.0131	0.0001	-0.0008
6	0.0046	-0.0001	-0.0003
7	-0.0046	-0.0001	0.0003
8	-0.0131	0.0001	0.0008
9	0.0350	0.0022	-0.0002
10	0.0306	0.0012	-0.0002
11	0.0262	0.0003	-0.0002
12	0.0219	-0.0003	-0.0001
13	0.0131	-0.0004	0.0000
14	0.0087	-0.0003	-0.0000
15	0.0044	-0.0003	-0.0000
16	-0.0044	-0.0003	0.0000
17	-0.0087	-0.0003	0.0000
18	-0.0131	-0.0004	-0.0000
19	-0.0218	-0.0003	0.0001
20	-0.0262	0.0003	0.0002
21	-0.0306	0.0012	0.0002
22	-0.0350	0.0022	0.0002

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STRUCTURE MCFCD-PIER

MEMBER FORCES FOR MEMBER 1

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	826.857	4.038	88.57
1	5	-826.857	-4.038	22.67
2	1	26.510	-23.826	-767.93
2	5	-26.510	23.826	111.52
3	1	271.354	0.129	-6.53
3	5	-271.354	-0.129	10.89
4	1	-86.349	90.839	2941.97
4	5	86.349	-90.839	-439.36

MEMBER FORCES FOR MEMBER 2

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	2	776.046	-0.792	-17.49
1	6	-776.046	0.792	-4.33
2	2	-8.754	-25.755	-811.35
2	6	8.754	25.755	101.82
3	2	214.596	0.364	-1.36
3	6	-214.596	-0.364	11.39
4	2	86.319	33.591	1053.13
4	6	-86.319	-33.591	-127.71

MEMBER FORCES FOR MEMBER 3

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	3	776.045	0.791	17.47
1	7	-776.045	-0.791	4.33
2	3	9.331	-25.979	-818.39
2	7	-9.331	25.979	102.67
3	3	33.338	0.605	3.95
3	7	-33.338	-0.605	12.71
4	3	86.302	-33.552	-1051.91
4	7	-86.302	33.552	127.56

MEMBER FORCES FOR MEMBER 4

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	4	826.814	-4.037	-88.56
1	8	-826.814	4.037	-22.66
2	4	-27.083	-24.434	-787.60
2	8	27.083	24.434	114.46
3	4	-7.975	-1.098	-33.46
3	8	7.975	1.098	3.20
4	4	-86.291	-90.804	-2940.83
4	8	86.291	90.804	439.19

MEMBER FORCES FOR MEMBER 5

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	5	826.856	4.038	-22.67
1	11	-826.856	-4.038	103.43
2	5	26.510	-23.826	-111.52
2	11	-26.510	23.826	-365.00
3	5	271.354	0.129	-10.09
3	11	-271.354	-0.129	12.68
4	5	-86.349	90.839	439.36
4	11	86.349	-90.839	1377.43

MEMBER FORCES FOR MEMBER 6

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	6	776.046	-0.792	4.33
1	14	-776.046	0.792	-20.17
2	6	-8.754	-25.755	-101.82
2	14	8.754	25.755	-413.28
3	6	214.596	0.364	-11.39
3	14	-214.596	-0.364	18.68
4	6	86.319	33.591	127.71
4	14	-86.319	-33.591	544.11

MEMBER FORCES FOR MEMBER 7

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	7	776.045	0.791	-4.33
1	17	-776.045	-0.791	20.16
2	7	9.331	-25.979	-102.67
2	17	-9.331	25.979	-416.92
3	7	33.338	0.605	-12.71
3	17	-33.338	-0.605	24.81
4	7	86.302	-33.552	-127.56
4	17	-86.302	33.552	-543.49

MEMBER FORCES FOR MEMBER 8

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	8	826.814	-4.037	22.66
1	20	-826.814	4.037	-103.41
2	8	-27.083	-24.434	-114.46
2	20	27.083	24.434	-374.22
3	8	-7.975	-1.098	-3.20
3	20	7.975	1.098	-18.76
4	8	-86.291	-90.804	-439.19
4	20	86.291	90.804	-1376.90

MEMBER FORCES FOR MEMBER 9

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	9	0.000	-0.032	-0.07
1	10	-0.000	29.731	-81.78
2	9	0.000	-0.002	-0.00
2	10	0.000	0.002	-0.01
3	9	0.000	-0.007	-0.01
3	10	0.000	0.007	-0.02
4	9	-0.033	0.006	0.01
4	10	0.033	-0.006	0.03

MEMBER FORCES FOR MEMBER 10

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	10	-0.000	-371.049	81.70
1	11	0.000	400.748	-2204.14
2	10	0.000	-0.005	0.00
2	11	0.000	0.005	-0.03
3	10	0.000	-127.836	0.01
3	11	0.000	127.836	-703.11
4	10	0.004	0.014	-0.02
4	11	-0.004	-0.014	0.10

MEMBER FORCES FOR MEMBER 11

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	11	-4.038	426.085	2100.67
1	12	4.038	-396.385	161.12
2	11	23.819	26.505	365.03
2	12	-23.819	-26.505	-219.25
3	11	-0.130	143.513	698.41
3	12	0.130	-143.513	98.91
4	11	-90.811	-86.330	-1377.54
4	12	90.811	86.330	902.73

MEMBER FORCES FOR MEMBER 12

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	12	-4.038	55.084	-161.16
1	13	4.038	4.316	440.39
2	12	23.821	26.505	219.25
2	13	-23.821	-26.505	72.31
3	12	-0.130	15.691	-98.91
3	13	0.130	-15.691	271.51
4	12	-90.818	-86.329	-902.73
4	13	90.818	86.329	-46.90

MEMBER FORCES FOR MEMBER 13

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	13	-4.838	-345.626	-440.41
1	14	4.838	375.326	-1542.21
2	13	23.822	26.505	-72.31
2	14	-23.822	-26.505	218.09
3	13	-0.130	-112.133	-271.52
3	14	0.130	112.133	-345.22
4	13	-90.824	-86.331	46.89
4	14	90.824	86.331	-521.71

MEMBER FORCES FOR MEMBER 14

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	14	-3.246	400.703	1562.34
1	15	3.246	-371.003	559.84
2	14	49.575	17.751	195.19
2	15	-49.575	-17.751	-97.56
3	14	-0.494	102.458	326.53
3	15	0.494	-102.458	236.99
4	14	-124.412	-0.013	-22.40
4	15	124.412	0.013	22.33

MEMBER FORCES FOR MEMBER 15

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	15	-3.246	29.700	-559.85
1	16	3.246	29.700	559.85
2	15	49.582	17.752	97.56
2	16	-49.582	-17.752	97.71
3	15	-0.494	-25.362	-236.99
3	16	0.494	25.362	-41.99
4	15	-124.404	-0.013	-22.33
4	16	124.404	0.013	22.18

MEMBER FORCES FOR MEMBER 16

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	16	-3.246	-371.012	-559.88
1	17	3.246	400.712	-1562.37
2	16	49.582	17.751	-97.71
2	17	-49.582	-17.751	195.34
3	16	-0.494	-25.363	41.99
3	17	0.494	25.363	-181.48
4	16	-124.396	-0.015	-22.19
4	17	124.396	0.015	22.11

MEMBER FORCES FOR MEMBER 17

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	17	-4.037	375.320	1542.19
1	18	4.037	-345.621	440.39
2	17	75.559	27.082	221.57
2	18	-75.559	-27.082	-72.62
3	17	-1.098	7.974	156.68
3	18	1.098	-7.974	-112.82
4	17	-90.850	86.287	521.38
4	18	90.850	-86.287	-46.80

MEMBER FORCES FOR MEMBER 18

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	18	-4.037	4.321	-440.41
1	19	4.037	55.079	161.25
2	18	75.561	27.082	72.62
2	19	-75.561	-27.082	225.28
3	18	-1.098	7.974	112.82
3	19	1.098	-7.974	-25.10
4	18	-90.838	86.286	46.80
4	19	90.838	-86.286	902.35

MEMBER FORCES FOR MEMBER 19

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	19	-4.037	-396.392	-161.27
1	20	4.037	426.092	-2100.57
2	19	75.556	27.082	-225.28
2	20	-75.556	-27.082	374.23
3	19	-1.099	7.975	25.10
3	20	1.099	-7.975	18.76
4	19	-90.859	86.286	-902.35
4	20	90.859	-86.286	1376.92

MEMBER FORCES FOR MEMBER 20

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	20	0.000	400.713	2203.94
1	21	-0.000	-371.013	-81.70
2	20	99.986	-0.001	-0.01
2	21	-99.986	0.001	-0.00
3	20	-0.001	-0.000	-0.00
3	21	0.001	0.000	0.00
4	20	-0.082	-0.005	-0.03
4	21	0.082	0.005	-0.00

MEMBER FORCES FOR MEMBER 21

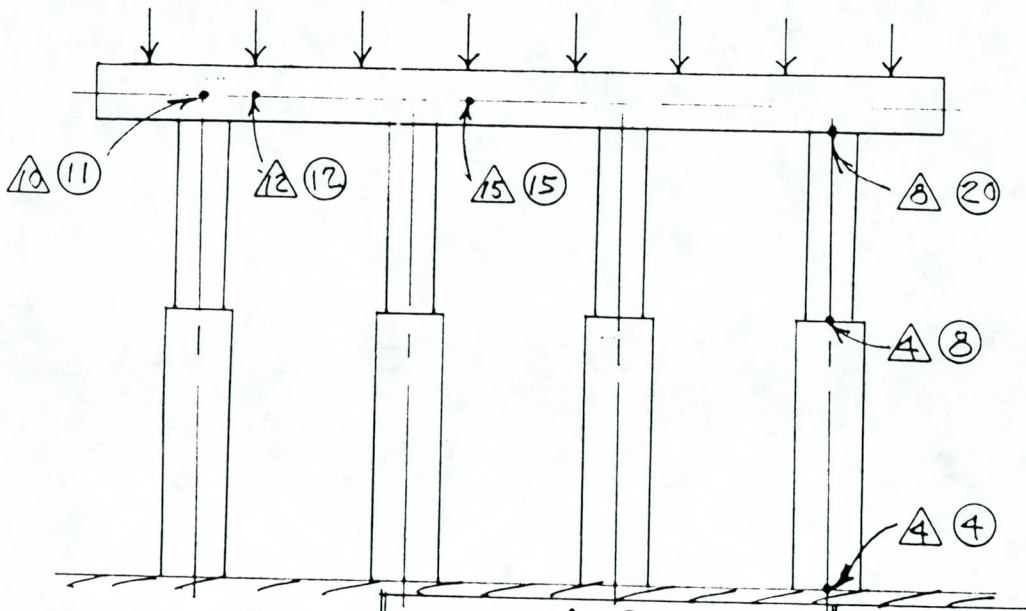
LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	21	0.000	29.696	81.67
1	22	-0.000	0.004	-0.02
2	21	99.995	0.000	0.00
2	22	-99.995	-0.000	0.00
3	21	-0.000	-0.000	-0.00
3	22	0.000	0.000	0.00
4	21	-0.039	0.000	0.00
4	22	0.039	0.000	0.00

PROBLEM SOLUTION COMPLETED.

BOTTOM

$ QUIT

HOYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541



AASHTO SERVICE LOAD GROUP I

ALLOWABLE STRESS INCREASE NONE %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△10 ⑪ M	△10 ⑪ β×M	△12 ⑫ M	△12 ⑫ β×M	△15 ⑮ M	△15 ⑮ β×M
D	1	-2204.1	-2204.1	-161.2	-161.2	-559.9	-559.9
L+I	1	-703.1	-703.1	-93.9	-98.9	-237.0	-237.0
SF	1	0	0	+23.7	+23.7	+10.6	+10.6
W	0	0	0	∓149.8	0	±62.7	0
WL	0	0	0	∓25.4	0	±11.3	0
S+T	0	0	0	∓902.7	0	∓22.3	0
EQ	0	0	0	∓712.6	0	∓317.3	0
TOTAL MOMENT (K-FT)			-2907.2		-236.4		-786.3

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

JOB NO. 1040
DESC. 75TH AVE MCFCD
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DATE SEPT 20, 1984

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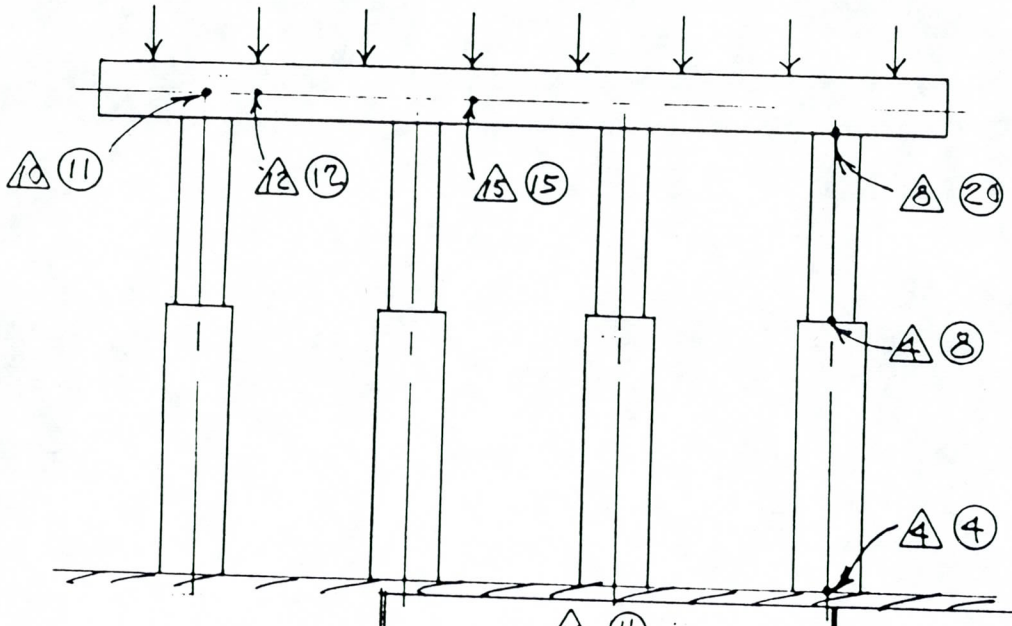
JOB NO. ____
DESC. 75TH AVE MCFCD
SHT. 1 OF 4
BY AGD
DATE SEPT 20, 1984



AASHTO SERVICE LOAD GROUP IA

ALLOWABLE STRESS INCREASE 50 %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△10 ⑪ M	△10 ⑪ β×M	△12 ⑫ M	△12 ⑫ β×M	△15 ⑮ M	△15 ⑮ β×M
D	1	-2204.1	-2204.1	-161.2	-161.2	-559.9	-559.9
L+I	2	-703.1	-1406.9	-98.9	-197.8	-237.0	-474.0
SF	0	0	0	+23.7	0	+10.6	0
W	0	0	0	±140.8	0	±62.7	0
WL	0	0	0	±25.4	0	±11.3	0
S+T	0	0	0	±902.7	0	±22.3	0
EQ	0	0	0	±712.6	0	±317.3	0
TOTAL MOMENT (K-FT)			-3610.3		-359.0		-1033.9

SF = .108 x 100K LATERAL LOADING
W = .642 " "
-2406.7

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x 579.3 " " -639.2

ENGINEERING COMPANY
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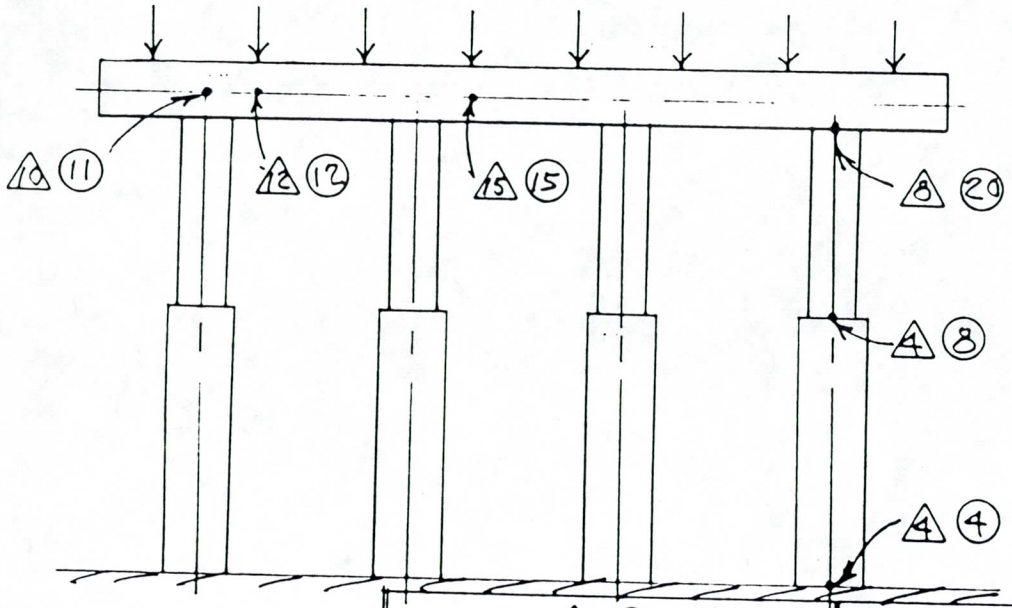
DESC. 75TH AVE MCFCD
BY AGD
DATE SEPT 20, 1984



AASHTO SERVICE LOAD GROUP II

ALLOWABLE STRESS INCREASE 25 %

△ MEMBER

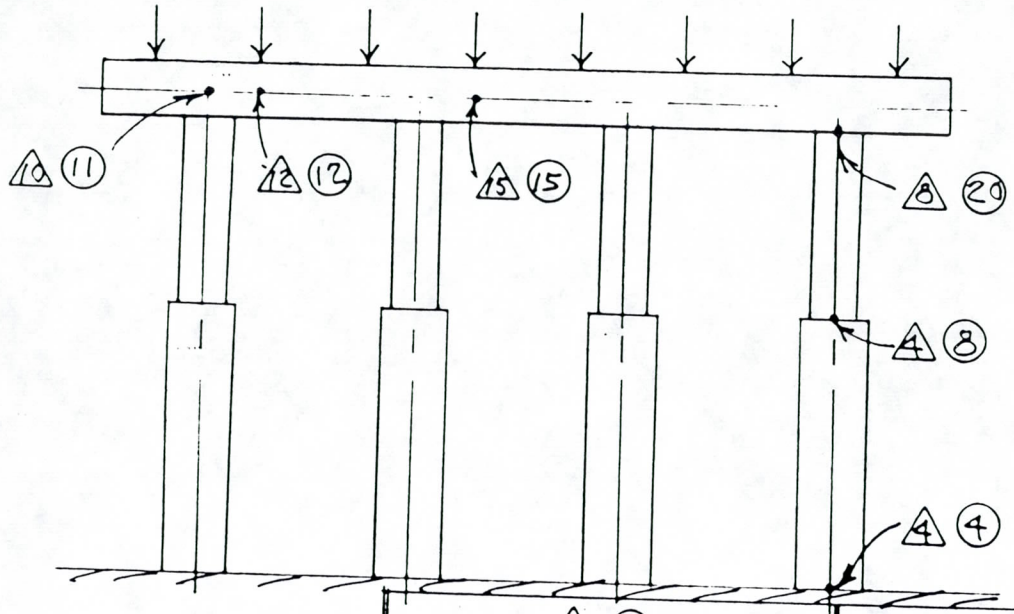
○ JOINT

LOADING	β FACTOR	△10 ⑪ M	△10 ⑪ β×M	△12 ⑫ M	△12 ⑫ β×M	△15 ⑮ M	△15 ⑮ β×M
D	1	-2204.1	-2204.1	-161.2	-161.2	-559.9	-559.9
L+I	0	-703.1	0	-98.9	0	-237.0	0
SF	1	0	0	+23.7	+23.7	+10.6	+10.6
W	1	0	0	±140.8	±140.3	±62.7	±62.7
WL	0	0	0	±25.4	0	±11.3	0
S+T	0	0	0	±902.7	0	±22.3	0
EQ	0	0	0	±712.6	0	±317.3	0
TOTAL MOMENT (K-FT)		-1763.3	-2204.1	+2.64 -222.6	+3.3 -278.3	-389.3 -489.6	-486.6 -612.0

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541



AASHTO SERVICE LOAD GROUP III

ALLOWABLE STRESS INCREASE 25 %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△10 ⑪ M	△10 ⑪ β×M	△12 ⑫ M	△12 ⑫ β×M	△15 ⑮ M	△15 ⑮ β×M
D	1	-2204.1	-2204.1	-161.2	-161.2	-559.9	-559.9
L+I	1	-703.1	-703.1	-98.9	-98.9	-237.0	-237.0
SF	1	0	0	+23.7	+23.7	+10.6	+10.6
W	.3	0	0	±140.8	±43.2	±62.7	±18.8
WL	1	0	0	±25.4	±25.4	±11.3	±11.3
S+T	0	0	0	±902.7	0	±22.3	0
EQ	0	0	0	+712.6	0	±317.3	0
TOTAL MOMENT (K-FT)		-2325.8 ←	-2907.2	-134.3 -244.0 ←	-167.8 -305.0	+24.1 -653.12 ←	+30.1 -816.4

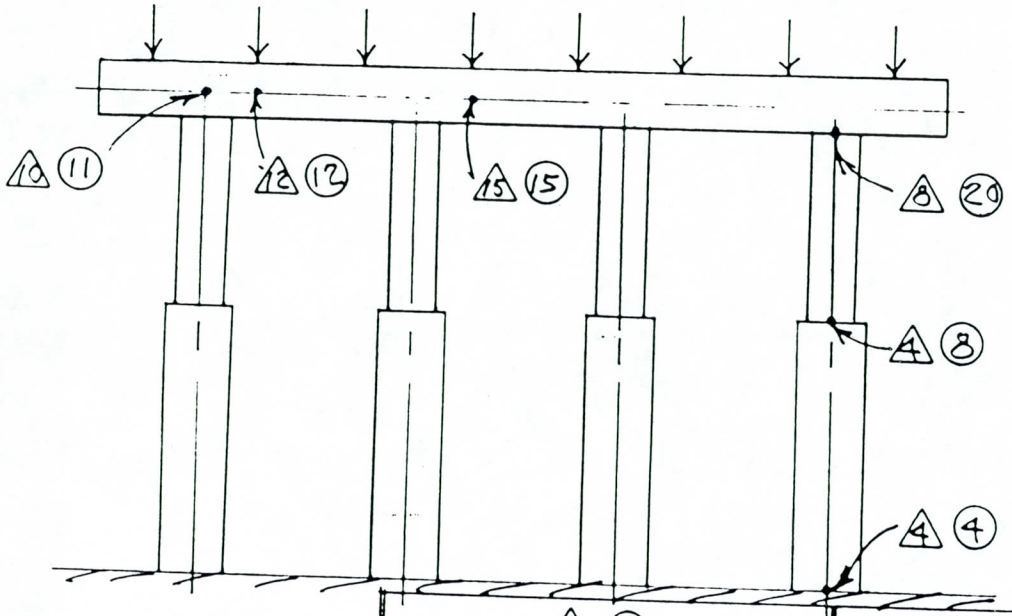
SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

DESC. 75TH AVE MCFCD
BY AGD
DATE SEPT 20, 1984

ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO.
DESC. 75TH AVE MCFCD
SHT. FIG 2 OF 06
BY AGD
DATE SEPT 20, 1984



AASHTO SERVICE LOAD GROUP IV

ALLOWABLE STRESS INCREASE 25 %

△ MEMBER

○ JOINT

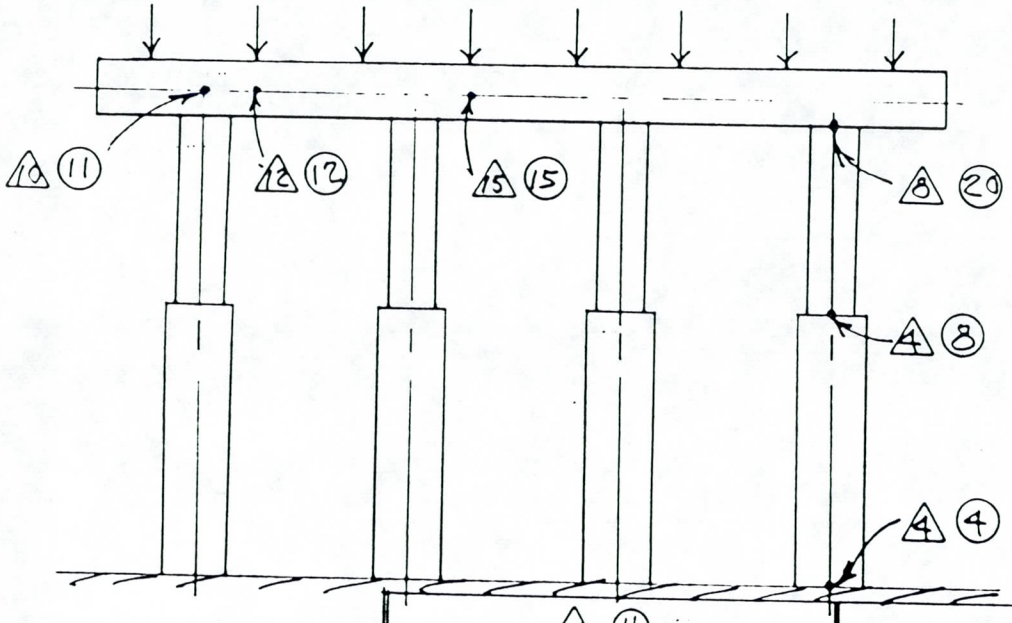
LOADING	β FACTOR	△10 ⑪ M	△10 ⑪ β×M	△12 ⑫ M	△12 ⑫ β×M	△15 ⑮ M	△15 ⑮ β×M
D	1	-2204.1	-2204.1	-161.2	-161.2	-559.9	-559.9
L+I	1	-703.1	-703.1	-98.9	-93.9	-237.0	-237.0
SF	1	0	0	+23.7	+23.7	+10.6	+10.6
W	0	0	0	±140.8	0	±62.7	0
WL	0	0	0	±25.4	0	±11.3	0
S+T	1	0		±902.7	±902.7	±22.3	±22.3
EQ	0	0	0	±712.6	0	±317.3	0
TOTAL MOMENT (K-FT)		-2325.7 ←	-2907.1	+533.1 -911.28 ←	+666.3 -1139.1	-611.2 -846.9 ←	-764.0 -303.6

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

DESC. 75TH AVE MCFCD
BY AGD
DATE SEPT 20, 1984



AASHTO SERVICE LOAD GROUP V

ALLOWABLE STRESS INCREASE 40 %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△10 ⑪ M	△10 ⑪ β×M	△12 ⑫ M	△12 ⑫ β×M	△15 ⑮ M	△15 ⑮ β×M
D	1	-2204.1	-2204.1	-161.2	-161.2	-559.9	-559.9
L+I	0	-703.1	0	-98.9	0	-237.0	0
SF	1	0	0	+23.7	+23.7	+10.6	+10.6
W	1	0	0	±140.8	±140.8	±62.7	±62.7
WL	0	0	0	±25.4	0	±11.3	0
S+T	1	0	0	±902.7	±902.7	±22.3	±22.3
EQ	0	0	0	±712.6	0	±317.3	0
TOTAL MOMENT (K-FT)			-2204.0	+647.2 -943.6	+906.0 -1181.0	-331.6 -453.1	-464.3 -634.3

SF = .108 x 100K LATERAL LOADING
W = .642 " " "
-1574.3

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

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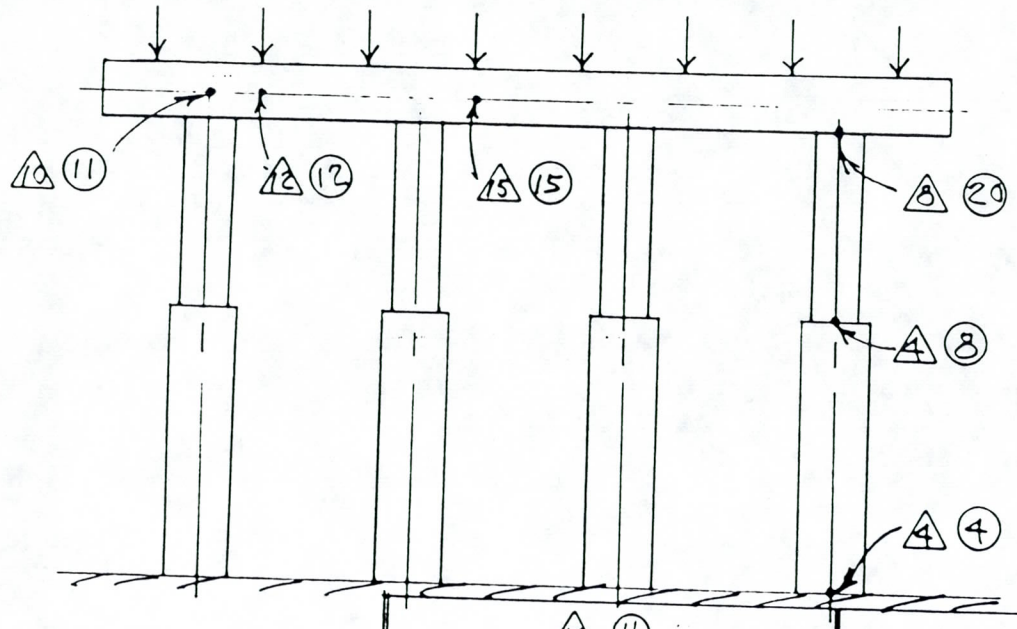

AASHTO SERVICE LOAD GROUP VI

ALLOWABLE STRESS INCREASE 40 %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△10 ⑪ M	△10 ⑪ β×M	△12 ⑫ M	△12 ⑫ β×M	△15 ⑮ M	△15 ⑮ β×M
D	1	-2204.1	-2204.1	-161.2	-161.2	-559.9	-559.9
L+I	1	-703.1	-703.1	-98.9	-98.9	-237.0	-237.0
SF	1	0	0	+23.7	+23.7	+10.6	+10.6
W	.3	0	0	±140.8	±42.2	±62.7	±18.8
WL	1	0	0	±25.4	±25.4	±11.3	±11.3
S+T	1	0	0	±902.7	±902.7	±22.3	±22.3
EQ	0	0	0	±712.6	0	±317.3	0
TOTAL MOMENT (K-FT)			-2907.1	+524.2 -361.9	+733.9 -1206.7	-524.2 -599.1	-733.9 -838.7

SF = .108 x 100K LATERAL LOADING
W = .642 " " "
-2076.5

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

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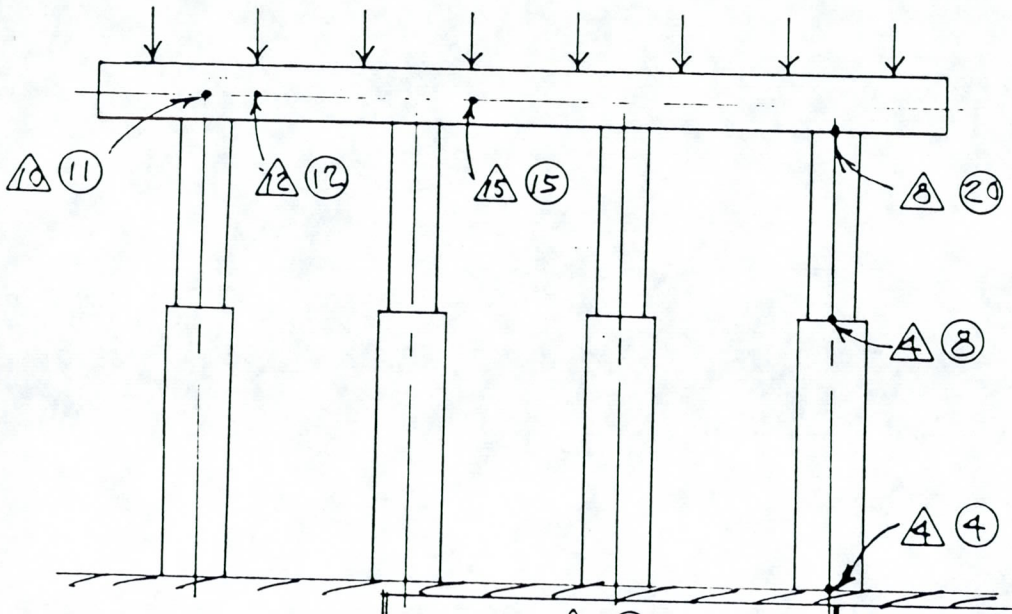
PHOENIX, ARIZONA PH. 279-3541



AASHTO SERVICE LOAD
GROUP VII

ALLOWABLE STRESS INCREASE 33 %

△ MEMBER
○ JOINT

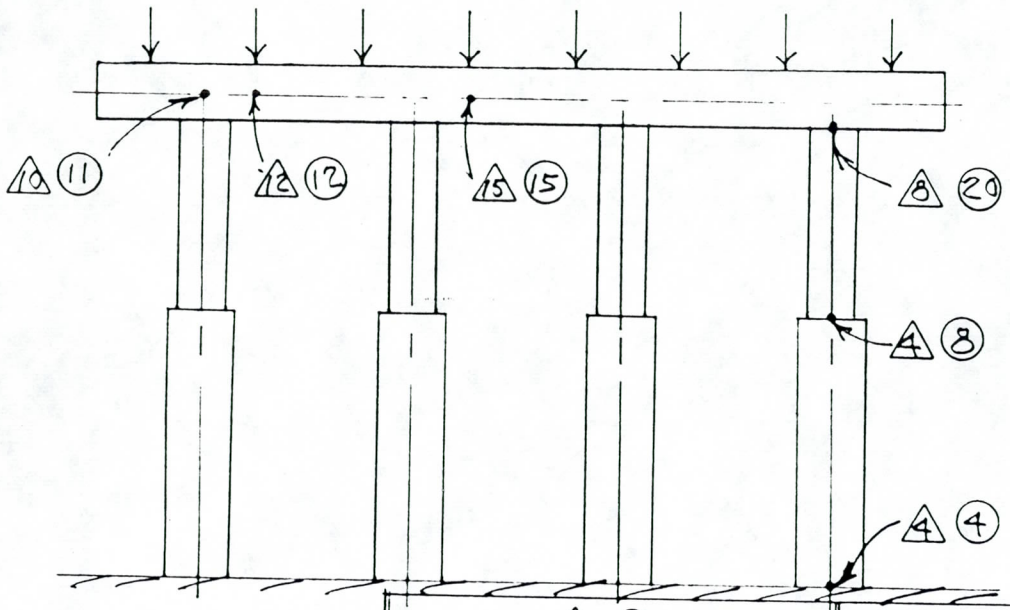
LOADING	β FACTOR	△10 ⑪ M	△10 ⑪ β×M	△12 ⑫ M	△12 ⑫ β×M	△15 ⑮ M	△15 ⑮ β×M
D	1	-2204.1	-2204.1	-161.2	-161.2	-559.9	-559.9
L+I	0	-703.1	0	-98.9	0	-237.0	0
SF	1	0	0	+23.7	+23.7	+10.6	+10.6
W	0	0	0	+140.8	0	+62.7	0
WL	0	0	0	+25.4	0	+11.3	0
S+T	0	0	0	-902.7	0	-22.3	0
EQ	1	0	0	±712.6	±712.6	±317.3	±317.3
TOTAL MOMENT (K-FT)			-2204.1	+432.4 -639.2	+575.1 -850.1	-174.3 -651.6	-232.0 -866.6

SF = .108 x 100K LATERAL LOADING
W = .642 " " "
-1657.2

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

BY MCECD
DATE SEPT 20, 1984

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541



AASHTO SERVICE LOAD GROUP I

ALLOWABLE STRESS INCREASE NONE %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△ 8 (2) M	△ 8 (2) β×M	△ 4 (8) M	△ 4 (8) β×M	△ 4 (4) M	△ 4 (4) β×M
D	1	-103.4	-103.4	-22.7	-22.7	-88.6	-88.6
L+I	1	-18.8	-18.8	+3.2	+3.2	-33.5	-33.5
SF	1	-40.4	-40.4	+12.4	+12.4	-85.1	-85.1
W	0	±240.2	0	±73.5	0	±505.6	0
WL	0	±43.4	0	±13.3	0	±91.4	0
S+T	0	±1377.0	0	±439.2	0	±2940.8	0
EQ	0	±1216.2	0	±372.0	0	±2559.8	0
TOTAL MOMENT (K-FT)			-162.6		-7.1		-207.2

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

HOYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75 TH AVE MCFCD

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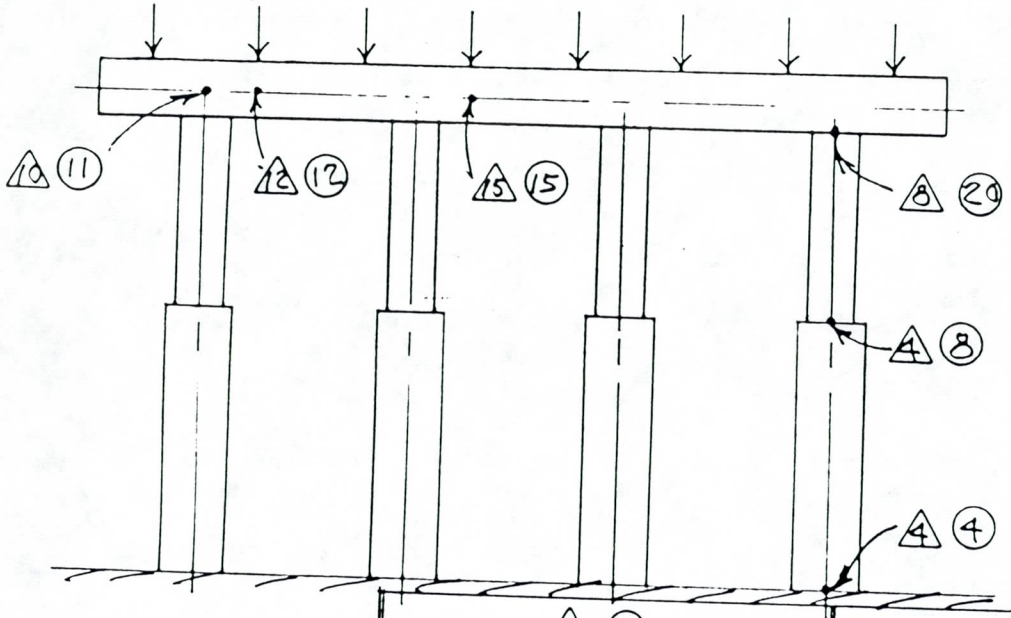


AASHTO SERVICE LOAD GROUP IA

ALLOWABLE STRESS INCREASE 50 %

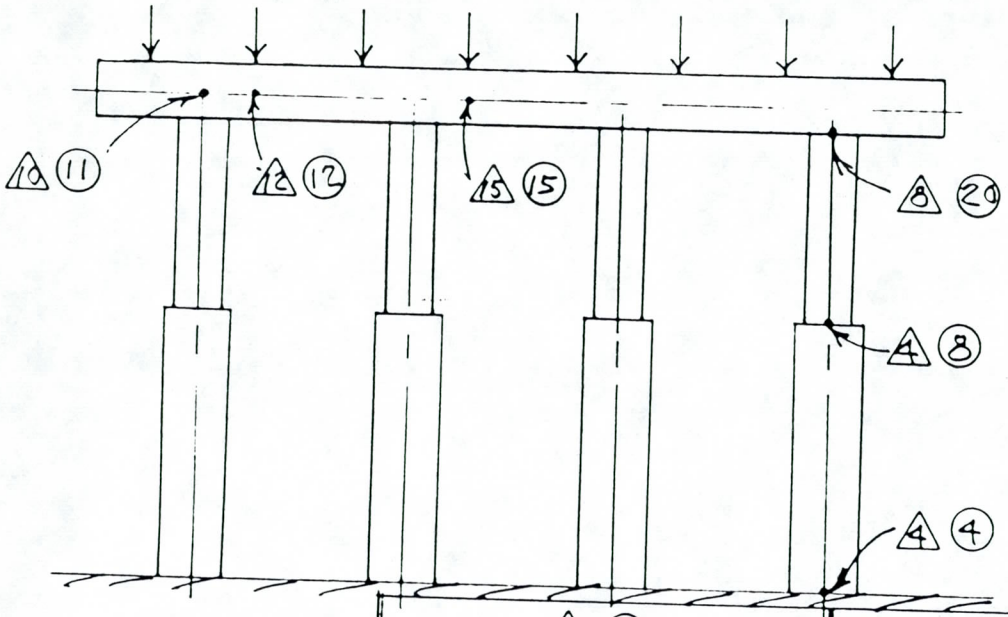
△ MEMBER

○ JOINT

LOADING	β FACTOR	△ 8 ⑳ M	△ 8 ⑳ β×M	△ 4 ⑧ M	△ 4 ⑧ β×M	△ 4 ④ M	△ 4 ④ β×M
D	1	-103.4	-103.4	-22.7	-22.7	-88.6	-88.6
L+I	2	-18.8	-37.6	+3.2	+6.4	-33.5	-67.0
SF	0	-40.4	0	+12.4	0	-85.1	0
W	0	±240.2	0	±73.5	0	±505.6	0
WL	0	±43.4	0	±13.3	0	±91.4	0
S+T	0	±1377.0	0	±439.2	0	±2940.8	0
EQ	0	±1216.2	0	±372.0	0	±2559.8	0
TOTAL MOMENT (K-FT)		-94.0 ←	-141.0	-10.87 ←	-16.3	-103.7 ←	-155.6

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "



AASHTO SERVICE LOAD GROUP II

ALLOWABLE STRESS INCREASE 25 %

△ MEMBER

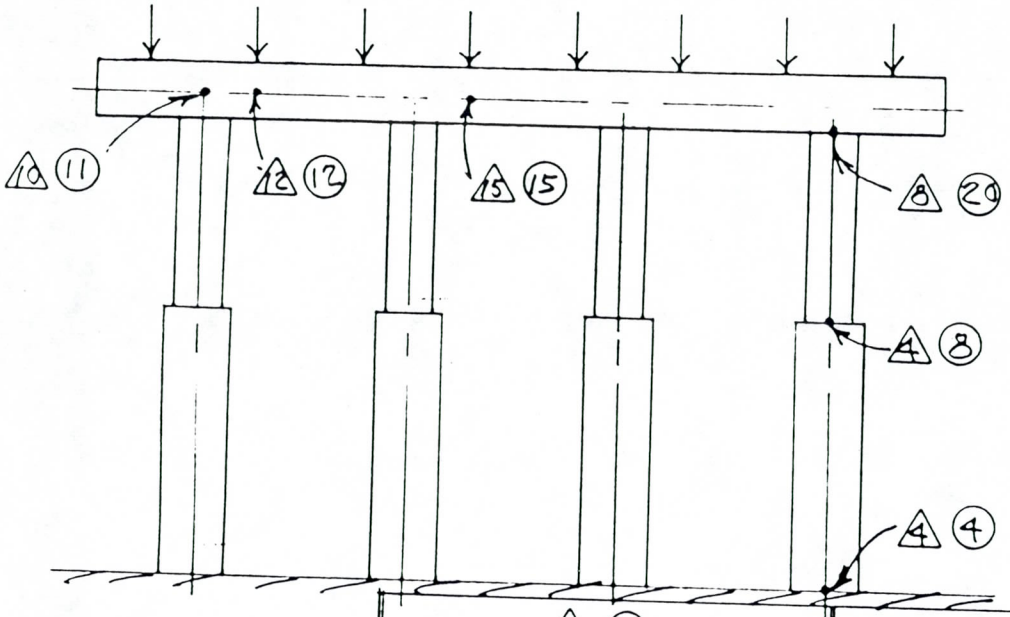
○ JOINT

LOADING	β FACTOR	△ 8 ⑳ M	△ 8 ⑳ β×M	△ 4 ⑧ M	△ 4 ⑧ β×M	△ 4 ④ M	△ 4 ④ β×M
D	1	-103.4	-103.4	-22.7	-22.7	-88.6	-88.6
L+I	0	-18.8	0	+3.2	0	-33.5	0
SF	1	-40.4	-40.4	+12.4	+12.4	-85.1	-85.1
W	1	±240.2	±240.2	±73.5	±73.5	±505.6	±505.6
WL	0	±43.4	0	±13.3	0	±91.4	0
S+T	0	±1377.0	0	±439.2	0	±2940.8	0
EQ	0	±1216.2	0	±372.0	0	±2559.8	0
TOTAL MOMENT (K-FT)		+77.2 -307.2	+96.4 -334.4	+50.6 -67.1	+63.2 -83.8	+265.5 -543.4	+331.9 -679.3

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

HOYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541



AASHTO SERVICE LOAD GROUP III

ALLOWABLE STRESS INCREASE 25 %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△ 8 (20) M	△ 8 (20) β×M	△ 4 (8) M	△ 4 (8) β×M	△ 4 (4) M	△ 4 (4) β×M
D	1	-103.4	-103.4	-22.7	-22.7	-83.6	-83.6
L+I	1	-18.8	-18.8	+3.2	+3.2	-33.5	-33.5
SF	1	-40.4	-40.4	+12.4	+12.4	-85.1	-85.1
W	.3	±240.2	±72.1	±73.5	±22.1	±505.6	±151.7
WL	1	±43.4	±43.4	±13.3	±13.3	±91.4	±91.4
S+T	0	±1377.0	0	±439.2	0	±2940.8	0
EQ	0	±1216.2	0	±372.0	0	±2559.8	0
TOTAL MOMENT (K-FT)		-37.7 -222.5	-47.1 -273.1	+22.6 -34.0	+28.3 -42.5	+28.7 -360.2	+35.9 -450.3

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

JOB NO. 1040
DESC. 75TH AVE MCFCD
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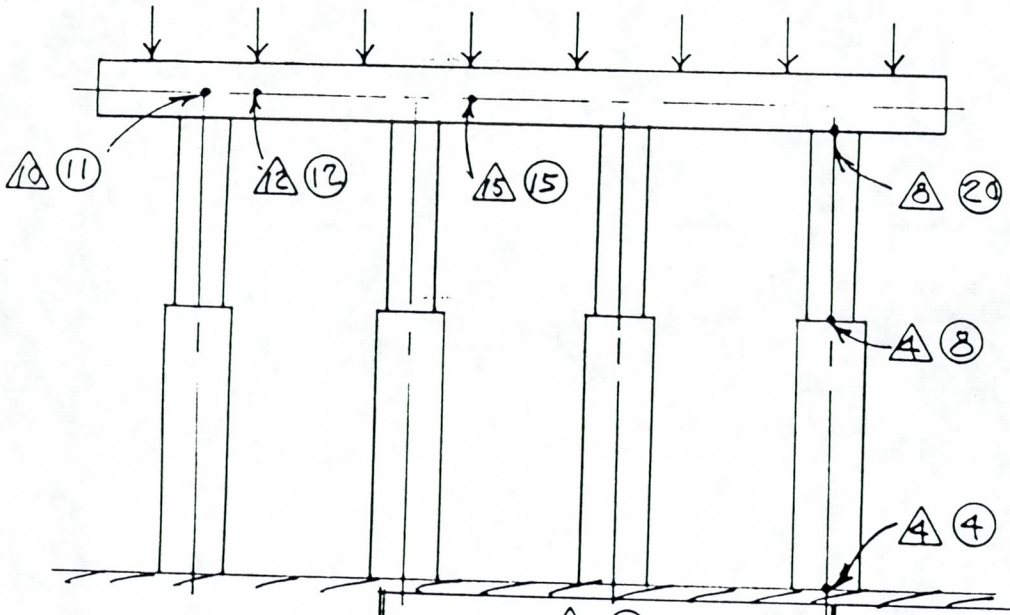

AASHTO SERVICE LOAD GROUP IV

ALLOWABLE STRESS INCREASE 25 %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△ 8 20 M	△ 8 20 β×M	△ 4 8 M	△ 4 8 β×M	△ 4 4 M	△ 4 4 β×M
D	1	-103.4	-103.4	-22.7	-22.7	-88.6	-88.6
L+I	1	-18.8	-18.8	+3.2	+3.2	-33.5	-33.5
SF	1	-40.4	-40.4	+12.4	+12.4	-85.1	-85.1
W	0	±240.2	0	±73.5	0	±505.6	0
WL	0	±43.4	0	±13.3	0	±91.4	0
S+T	1	±1377.0	±1377.0	±439.2	±439.2	±2940.8	±2940.8
EQ	0	±1216.2	0	±372.0	0	±2559.8	0
TOTAL MOMENT (K-FT)		+971.5 -1231.7	+1214.4 -1539.6	+345.7 -357.1	+432.1 -446.3	+2186.9 -2518.4	+2733.6 -3148.0

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

JOB NO. 1040
DESC. 75 TH AVE MCFCD
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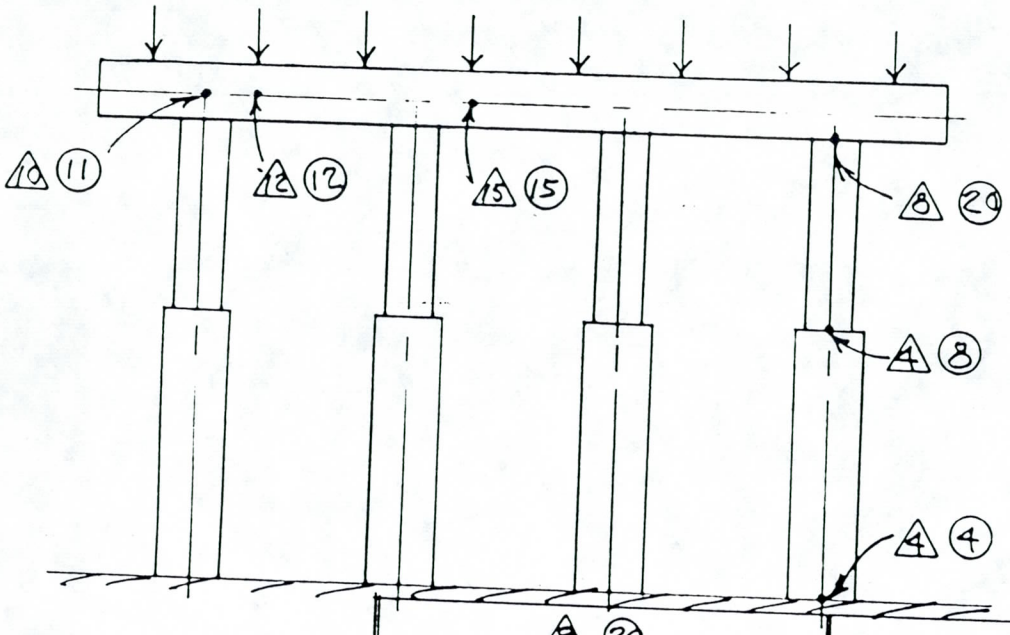
BY AGP
DATE SEPT. 20, 1984

HOYDEN
ENGINEERING
COMPANY
PHOENIX, ARIZONA PH. 279-3541



AASHTO SERVICE LOAD
GROUP V

ALLOWABLE STRESS INCREASE 40 %

△ MEMBER
○ JOINT

LOADING	β FACTOR	△ 8 (20) M	△ 8 (20) β×M	△ 4 (8) M	△ 4 (8) β×M	△ 4 (4) M	△ 4 (4) β×M
D	1	-103.4	-103.4	-22.7	-22.7	-88.6	-88.6
L+I	0	-18.8	0	+3.2	0	-33.5	0
SF	1	-40.4	-40.4	+12.4	+12.4	-85.1	-85.1
W	1	±240.2	±240.2	±73.5	±73.5	±505.6	±505.6
WL	0	±43.4	0	±13.3	0	±91.4	0
S+T	1	±1377.0	±1377.0	±439.2	±439.2	±2940.8	±2940.8
EQ	0	±1216.2	0	±372.0	0	±2559.8	0
TOTAL MOMENT (K-FT)		+1052.4 -1257.9	+1473.4 -1761.0	+358.9 -373.6	+502.4 -523.0	+2337.6 -2585.8	+3272.7 -3620.1

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

JOB NO. 1040
DESC. 75 TH AVE MCFCD
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DATE SEPT. 20, 1984

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75 TH AVE MCFCD

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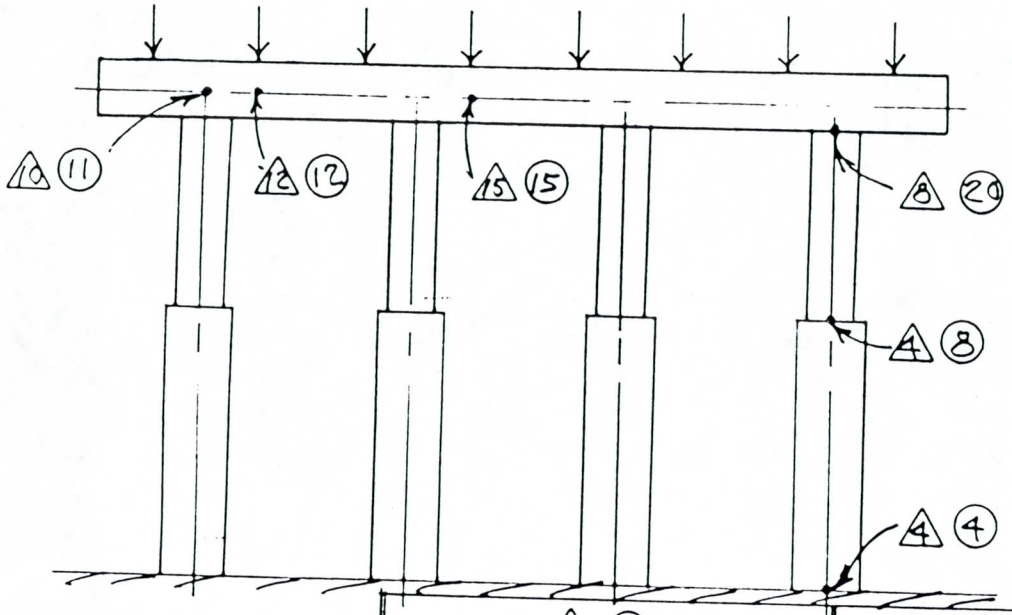
BY AGP
DATE SEPT. 20, 1984



AASHTO SERVICE LOAD GROUP VI

ALLOWABLE STRESS INCREASE 40 %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△ 8 ⑳ M	△ 8 ⑳ β×M	△ 4 ⑧ M	△ 4 ⑧ β×M	△ 4 ④ M	△ 4 ④ β×M
D	1	-103.4	-103.4	-22.7	-22.7	-88.6	-88.6
L+I	1	-18.8	-18.8	+3.2	+3.2	-33.5	-33.5
SF	1	-40.4	-40.4	+12.4	+12.4	-85.1	-85.1
W	.3	±240.2	±72.1	±73.5	±22.1	±505.6	±151.7
WL	1	±43.4	±43.4	±13.3	±13.3	±91.4	±91.4
S+T	1	±1377.0	±1377.0	±439.2	±439.2	±2940.8	±2940.8
EQ	0	±1216.2	0	±372.0	0	±2559.8	0
TOTAL MOMENT (K-FT)		+949.9 -1192.2	+1329.9 -1655.1	+333.9 -344.1	+467.5 -481.7	+2126.2 -2422.2	+2970.7 -3391.1

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

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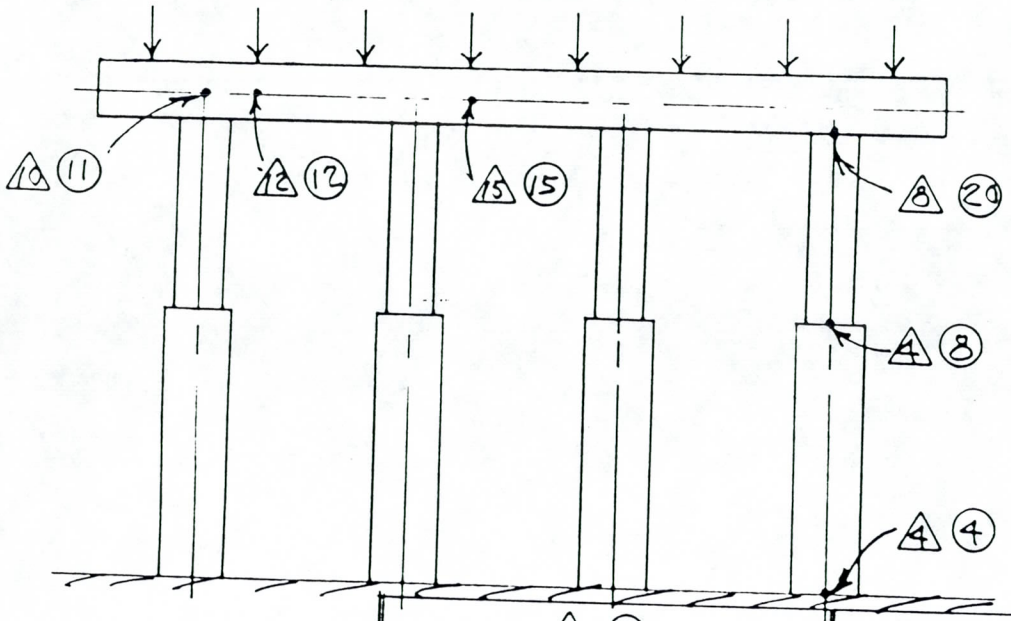


AASHTO SERVICE LOAD GROUP VII

ALLOWABLE STRESS INCREASE 33 %

△ MEMBER

○ JOINT

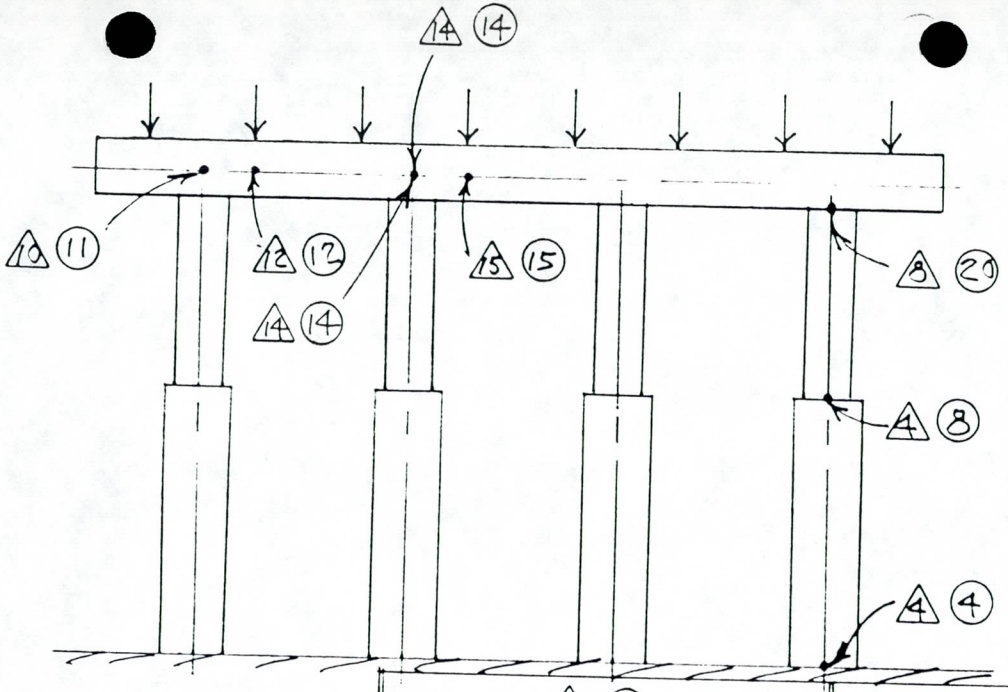
LOADING	β FACTOR	△ 29 M	△ 29 β×M	△ 8 M	△ 8 β×M	△ 4 M	△ 4 β×M
D	1	-103.4	-103.4	-22.7	-22.7	-88.6	-88.6
L+I	0	-18.8	0	+3.2	0	-33.5	0
SF	1	-40.4	-40.4	+12.4	+12.4	-85.1	-85.1
W	0	±240.2	0	±73.5	0	±505.6	0
WL	0	±43.4	0	±13.3	0	±91.4	0
S+T	0	±1377.0	0	±439.2	0	±2940.8	0
EQ	1	±1216.2	±1216.2	±372.0	±372.0	±2559.8	±2559.8
TOTAL MOMENT (K-FT)		+806.3 -1022.6	+1072.4 -1360.0	+272.0 -233.4	+361.7 -332.3	+1794.1 -2055.3	+2386.1 -2733.5

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

JOB NO. 1040
DESC. 75 TH AVE MCFCD

SHT. P.54 OF 86
BY AGD
DATE SEPT. 20, 1984



AASHTO SERVICE LOAD GROUP I

ALLOWABLE STRESS INCREASE NONE %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△14 (14) M	△14 (14) β×M	△ ○ M	△ ○ β×M	△ ○ M	△ ○ β×M
D	1	-1562.3	-1562.3				
L+I	1	-326.5	-326.5				
SF	1	-21.1	-21.1				
W	0	±125.3	0				
WL	0	±22.6	0				
S+T	0	±22.4	0				
EQ	0	±634.4	0				
TOTAL MOMENT (K-FT)			-1909.9				

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

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JOB NO. 1040
DESC. 75 TH AVE MCFCD
SHT. P.55 OF 86
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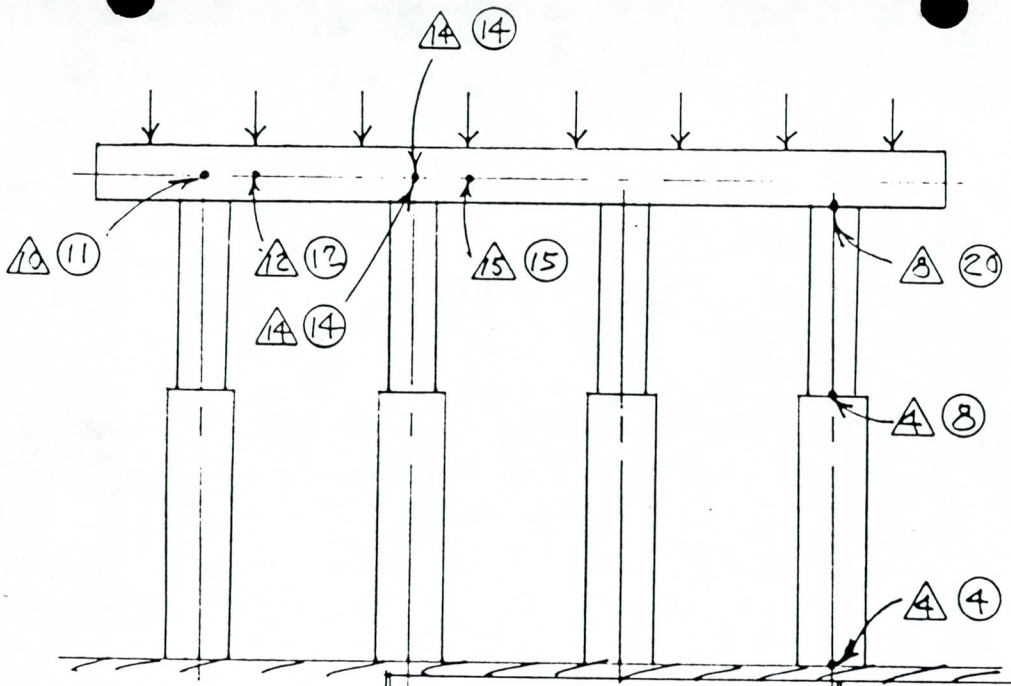
PHOENIX, ARIZONA PH. 279-3541



AASHTO SERVICE LOAD GROUP IA

ALLOWABLE STRESS INCREASE 50 %

△ MEMBER

○ JOINT

		△14 ○14		△ ○		△ ○	
LOADING	β FACTOR	M	β×M	M	β×M	M	β×M
D	1	-1562.3	-1562.3				
L+I	2	-326.5	-653.0				
SF	0	-21.1	0				
W	0	±125.3	0				
WL	0	±22.6	0				
S+T	0	±22.4	0				
EQ	0	±634.4	0				
TOTAL MOMENT (K-FT)		-1476.7	-2215.3				

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

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JOB NO. 1040
DESC. 75 TH AVE MCFCD
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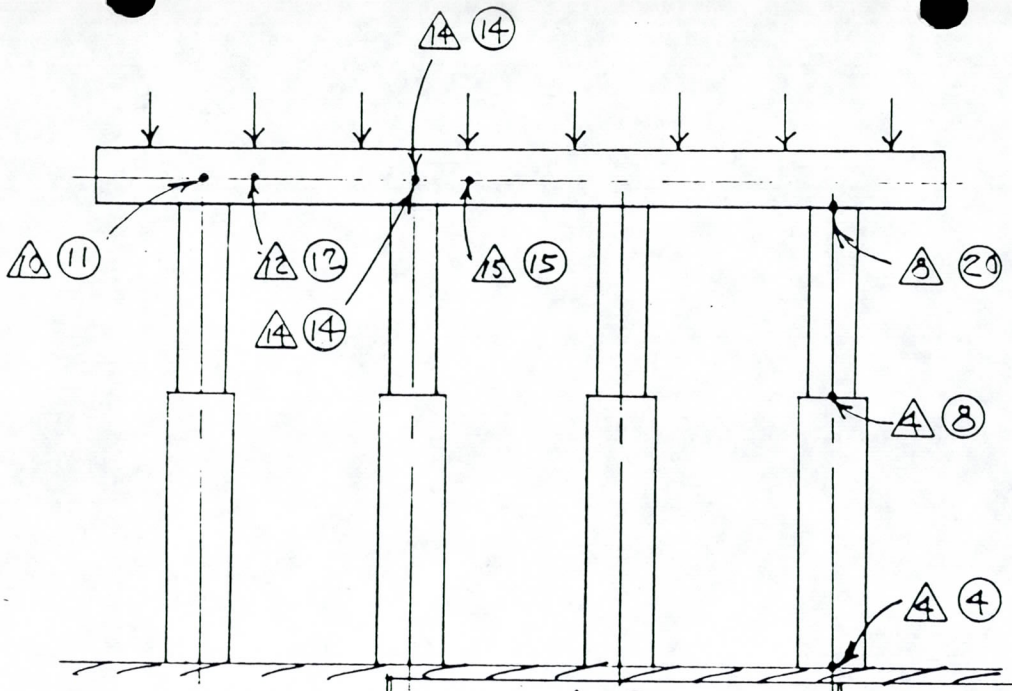
DATE SEPT. 20, 1984



AASHTO SERVICE LOAD GROUP II

ALLOWABLE STRESS INCREASE 25 %

△ MEMBER

○ JOINT

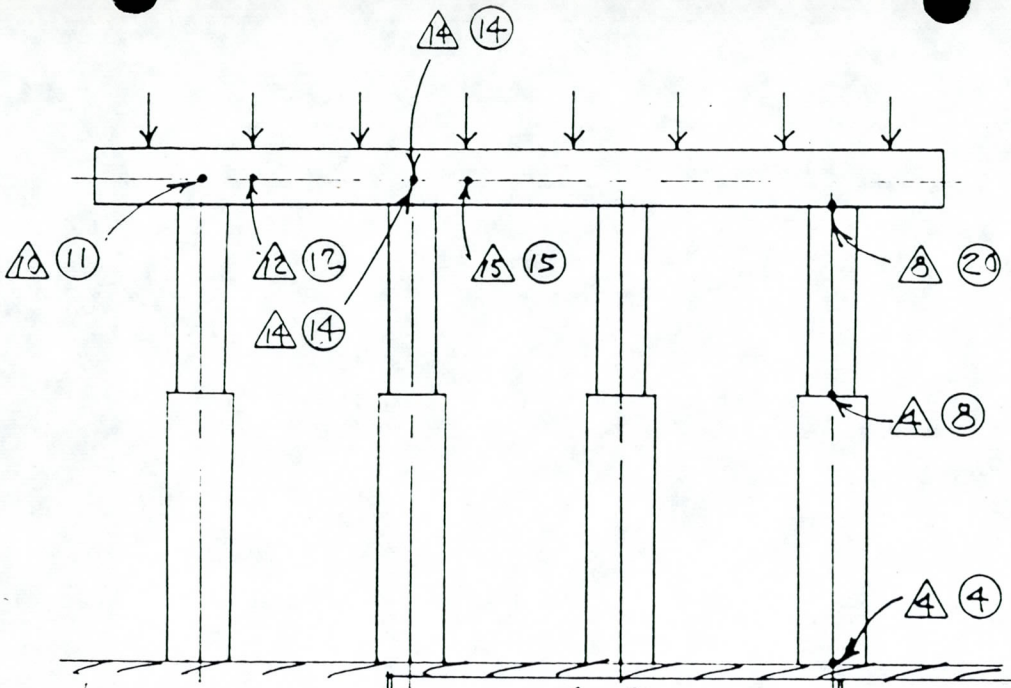
LOADING	β FACTOR	△14 ○14 M	△14 ○14 β×M	△ ○ M	△ ○ β×M	△ ○ M	△ ○ β×M
D	1	-1562.3	-1562.3				
L+I	0	-326.5	0				
SF	1	-21.1	-21.1				
W	1	±125.3	±125.3				
WL	0	±22.6	0				
S+T	0	±22.4	0				
EQ	0	±634.4	0				
TOTAL MOMENT (K-FT)		-1166.5 -1367.0 ←	-1458.1 -1708.7				

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75 TH AVE MCFCD
SHT. P.58 OF 86
BY AGD
DATE SEPT. 20, 1984



AASHTO SERVICE LOAD GROUP III

ALLOWABLE STRESS INCREASE 25 %

△ MEMBER

○ JOINT

		△ 14 ○ 14		△ ○		△ ○	
LOADING	β FACTOR	M	β × M	M	β × M	M	β × M
D	1	-1562.3	-1562.3				
L+I	1	-326.5	-326.5				
SF	1	-21.1	-21.1				
W	.3	±125.3	±37.6				
WL	1	±22.6	±22.6				
S+T	0	±22.4	0				
EQ	0	±634.4	0				
TOTAL MOMENT (K-FT)		-1479.8 -1576.1 ←	-1849.7 -1970.1				

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

ROYDEN ENGINEERING COMPANY
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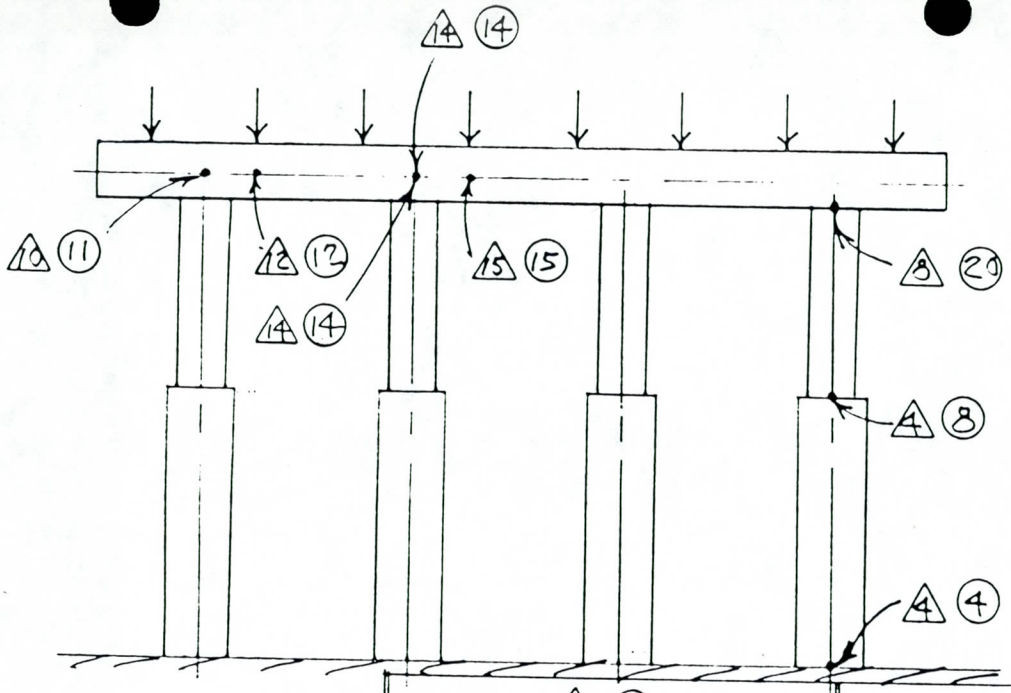


AASHTO SERVICE LOAD GROUP IV

ALLOWABLE STRESS INCREASE 25 %

△ MEMBER

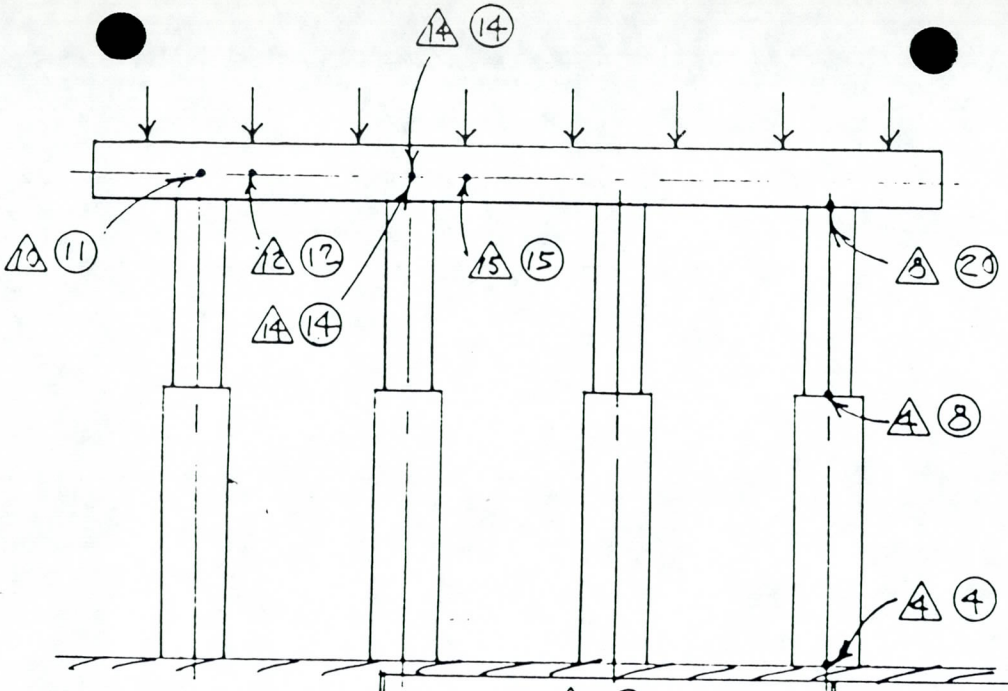
○ JOINT

LOADING	β FACTOR	△14 ○14 M	△14 ○14 β×M	△ ○ M	△ ○ β×M	△ ○ M	△ ○ β×M
D	1	-1562.3	-1562.3				
L+I	1	-326.5	-326.5				
SF	1	-21.1	-21.1				
W	0	±125.3	0				
WL	0	±22.6	0				
S+T	1	±22.4	±22.4				
EQ	0	±634.4	0				
TOTAL MOMENT (K-FT)		-1510.0 -1545.9 ←	-1887.5 -1932.3				

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541



AASHTO SERVICE LOAD GROUP V

ALLOWABLE STRESS INCREASE 40 %

△ MEMBER

○ JOINT

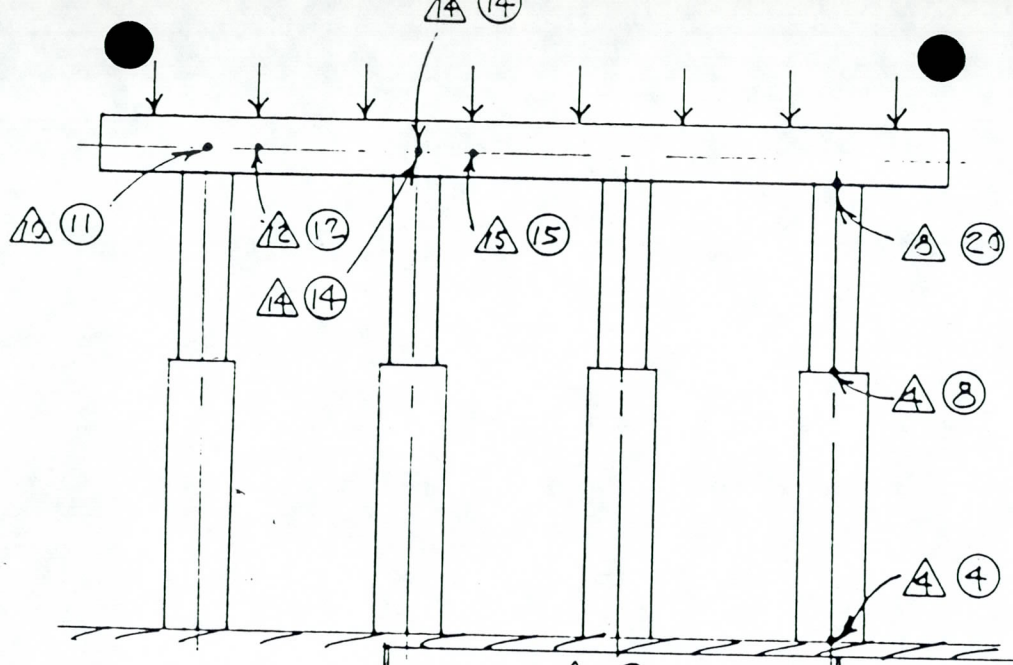
		△14 ○14		△ ○		△ ○	
LOADING	β FACTOR	M	β×M	M	β×M	M	β×M
D	1	-1562.3	-1562.3				
L+I	0	-326.5	0				
SF	1	-21.1	-21.1				
W	1	±125.3	±125.3				
WL	0	±22.6	0				
S+T	1	±22.4	±22.4				
EQ	0	±634.4	0				
TOTAL MOMENT (K-FT)		-1025.5 -1236.5 ←	-1435.7 -1731.1				

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

JOB NO. 1040
DESC. 75 TH AVE MCFCD
SHT. P.60 OF 86
BY AGD
DATE SEPT. 20, 1984

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541



AASHTO SERVICE LOAD GROUP VI

ALLOWABLE STRESS INCREASE 40 %

△ MEMBER

○ JOINT

LOADING	β FACTOR	△14 ○14 M	△14 ○14 β×M	△ ○ M	△ ○ β×M	△ ○ M	△ ○ β×M
D	1	-1562.3	-1562.3				
L+I	1	-326.5	-326.5				
SF	1	-21.1	-21.1				
W	.3	±125.3	±125.3				
WL	1	±22.6	±22.6				
S+T	1	±22.4	±22.4				
EQ	0	±634.4	0				
TOTAL MOMENT (K-FT)		-1242.6 -1485.9	-1739.6 -2080.2				

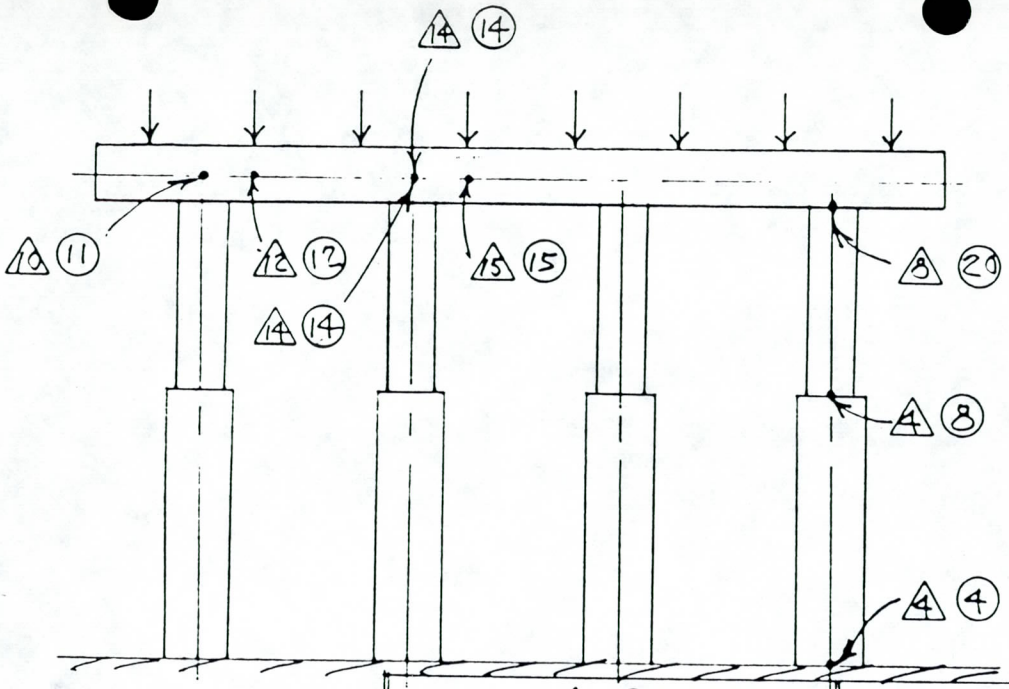
SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

JOB NO. 1040
DESC. 75 TH AVE MCFCD
SHT. P.61 OF 86
BY AGD
DATE SEPT. 20, 1984

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75 TH AVE MCFCD
SHT. P.62 OF 86
BY A GD
DATE SEPT. 20, 1984



AASHTO SERVICE LOAD GROUP VII

ALLOWABLE STRESS INCREASE 33 %

△ MEMBER

○ JOINT

		△ 14 ○ 14		△ ○		△ ○	
LOADING	β FACTOR	M	β×M	M	β×M	M	β×M
D	1	-1562.3	-1562.3				
L+I	0	-326.5	-326.5				
SF	1	-21.1	-21.1				
W	0	±125.3	0				
WL	0	±22.6	0				
S+T	0	±22.4	0				
EQ	1	±634.4	±634.4				
TOTAL MOMENT (K-FT)		-959.1 -1913.0	-1275.5 -2544.3				

SF = .108 x 100K LATERAL LOADING
W = .642 " " "

WL = .116 x 100K LATERAL LOAD
EQ = 3.25 x " " "

TSR

COLUMN FLEXURE & AXIAL CAPACITY

SERVICE LOAD DESIGN METHOD

AASHTO 1.5.28 → USE 35% OF THAT COMPUTED IN ACCORDANCE WITH ARTICLE 1.5.33.
$\phi = 1.0$

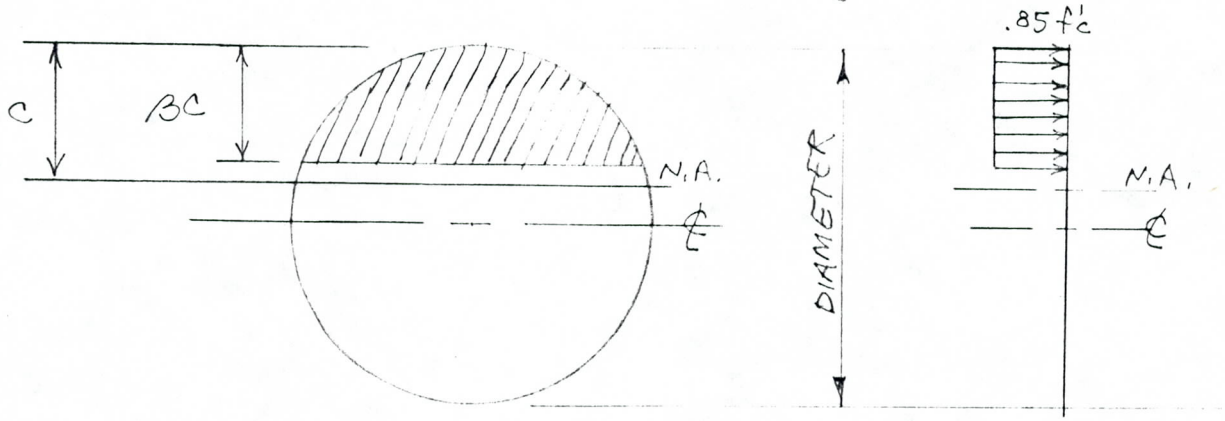
AASHTO 1.5.33 → USE STRESS AND STRAIN COMPATIBILITY GIVEN IN ARTICLE 1.5.31

SLENDERNESS EFFECTS HAVE BEEN ACCOUNTED FOR IN MULTIPLYING D.L. x COLUMN DEFLECT.

AASHTO 1.5.31 → ?

CONCRETE STRAIN = 0.003

RECTANGULAR CONCRETE STRESS DIST.:
STRESS = $0.85 f'_c$



$\beta_1 = .85$ FOR $f'_c = 4$ KSI

C = DIST. FROM EXTREME COMPRESSION FIBER TO NEUTRAL AXIS.

PIER DESIGN

REINFORCING STEEL FOR 7' Ø CAISSON

LATERAL LOADINGS MOMENT FROM IS 2907.2 K-FT

LONGITUDINAL LOADINGS MOMENT FROM IS 2887.7 K-FT

USE 2907.2 K-FT

ADDITIONAL MOMENT DUE TO D.L. x DEFL.

$800^K \times \left(\frac{2}{12}\right)$ FT = 133.3 K-FT

NOTE: THIS ACCOUNTS FOR THE MAGNIFICATION FACTOR

TOTAL SERVICE LOAD MOMENT 47.55' BELOW CAP

$$\begin{array}{r} 2907.2 \\ 133.3 \\ \hline 3040.5 \end{array} \text{ K-FT}$$

REQUIRED ULTIMATE CAPACITY 47.55' BELOW CAP

$$\frac{3040.5}{.35} = 8,687.1 \text{ K-FT}$$

REQUIRED ULTIMATE AXIAL CAPACITY

$$\frac{800}{.35} = 2,285.7 \text{ KIPS}$$

7'Φ FLEX. & AXIAL CAP. (0.3% STEEL)

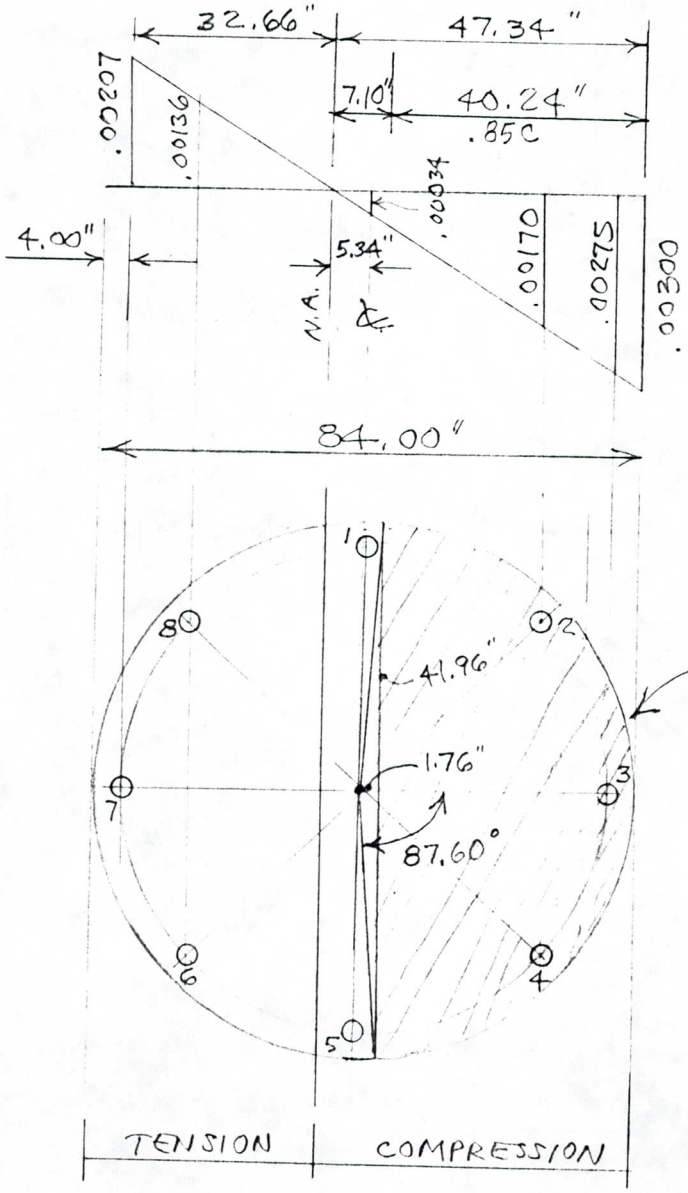
$A_s = .003\ A_G$
$f_c = 4$ KSI
$f_y = 60$ KSI

$$A_G = \pi (42)^2 = 5{,}541.77 \text{ IN}^2$$

$$A_s = 16.63 \text{ IN}^2$$

$$E_s = 29{,}000{,}000 \text{ psi}$$

$$\epsilon_s = \frac{60{,}000}{29{,}000{,}000} = 0.00207$$



CONCRETE COMPRESSION AREA:

$$\frac{2 \times 87.6^\circ}{360.00^\circ} \cdot [5541.77]$$

$$-(41.96)1.76$$

$$-\frac{3}{8}(16.63)$$

$$= 2616.7 \text{ IN}^2$$

7'Ø FLEX. & AXIAL CAP. (0.3% STEEL)

STEEL AREAS:

$$\frac{16.63 \text{ IN}^2}{8 \text{ AREAS}} = 2.08 \text{ IN}^2/\text{AREA}$$

AREA	STRAIN	STRESS (KSI)	FORCE (KIPS)	DIST. TO N.A. (INCHES)	
① & ⑤	.00034	9.86	20.5	5.34	COMPRESSION
② & ④	.00170	49.28	102.5	32.21	COMPRESSION
③	.00275	60.00	124.8	43.34	COMPRESSION
⑥ & ⑧	.00136	39.42	82.0	21.53	TENSION
⑦	.00207	60.00	124.8	32.66	TENSION

CONCRETE AREA:

FORCE = 2616.9 (.85)(4) = 8897.5 KIPS

DIST. FROM CENTROID TO N.A.
EST. (.42)(42) + 5.34 = 22.78

$\Sigma F_V = 0$

TENSION

124.8 = (−) 124.8
(2) 82.0 = (−) 164.0

COMPRESSION

(2) 20.5 = (+) 41.0
(2) 102.5 = (+) 205.0
124.8 = (+) 124.8
8897.5 = (+) 8897.5

BALANCED AXIAL LOAD 8979.5K

7'Ø FLEX & AXIAL CAP (0.3% STEEL)

BALANCED MOMENT

−(2)	82.0(21.53)	=	−	3,530.9 K-IN
−	124.8(32.66)	=	−	4,076.0
+(2)	20.5(5.34)	=	+	218.9
+(2)	102.5(32.21)	=	+	6,603.1
+	124.8(43.34)	=	+	5,408.8
+	8877.5(22.98)	=	+	204,464.3

BALANCED MOMENT = +209,088.2 K-IN

= 17,424.0 K-FT

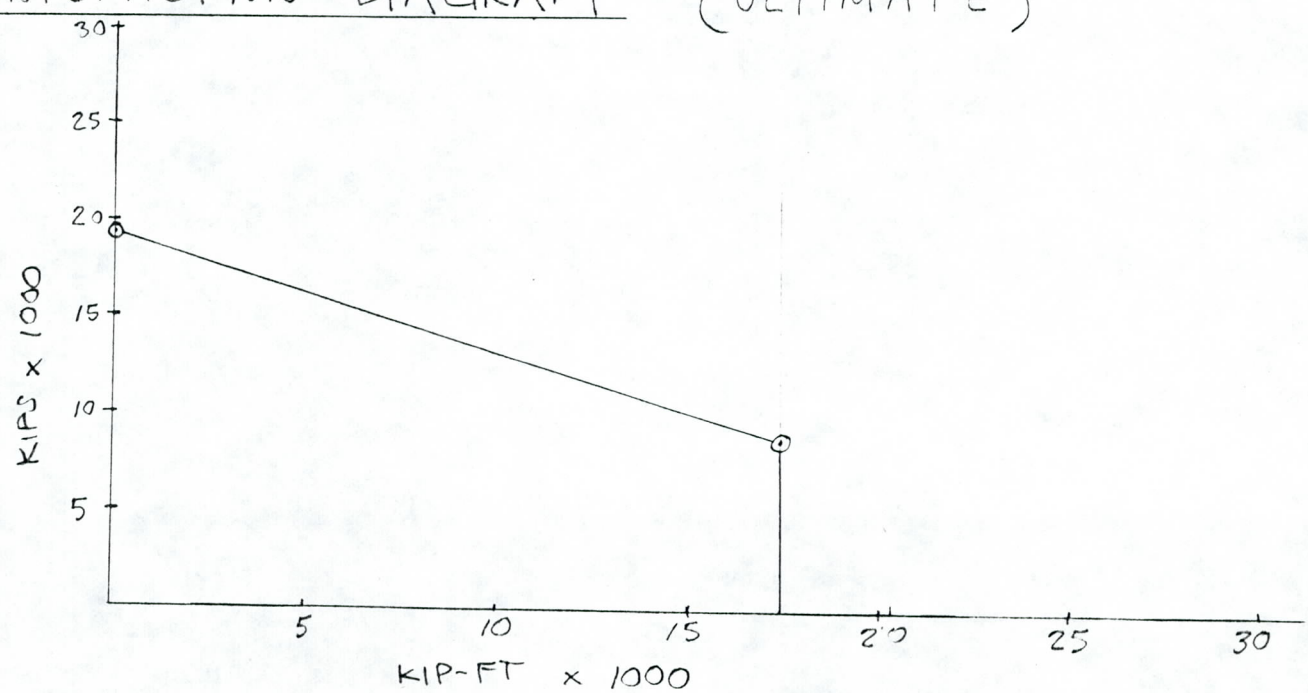
PURE COMPRESSION AASHTO 1.5.33

$$P_o = 0.85 f'_c (A_G - A_{ST}) + A_{ST} f_y$$

$$= 0.85(4)(5541.77 - 16.63) + (16.63)(60)$$

$$= 19,783.3 \text{ KIPS}$$

INTERACTION DIAGRAM (ULTIMATE)



7'Ø FLEX. & AXIAL CAP. (3% STEEL)

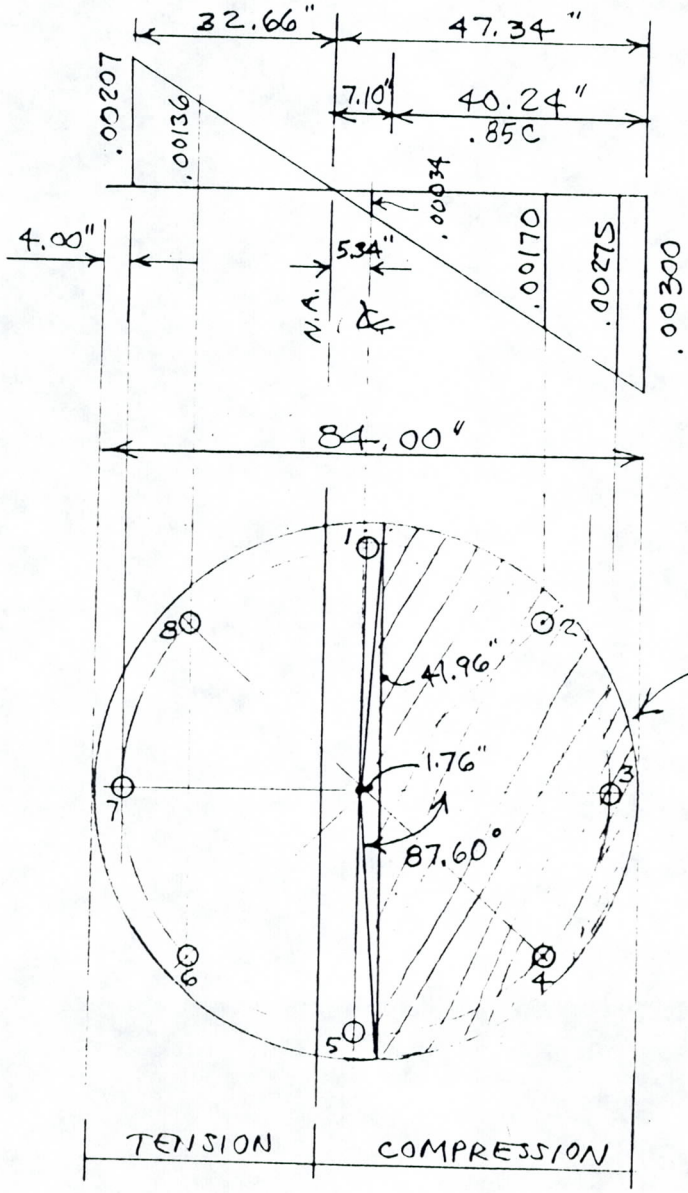
$A_s = .03 \; A_G$
$f'_c = 4$ KSI
$f_y = 60$ KSI

$$A_G = \pi (42)^2 = 5{,}541.77 \text{ IN}^2$$

$$A_s = 166.25 \text{ IN}^2$$

$$E_s = 29{,}000{,}000 \text{ psi}$$

$$\epsilon_s = \frac{60{,}000}{29{,}000{,}000} = 0.00207$$



CONCRETE COMPRESSION AREA:

$$\frac{2 \times 87.6^\circ}{360.00^\circ} \cdot [5541.77] - (41.96)1.76 - \frac{3}{8}(166.25)$$

$$= 2560.8 \text{ IN}^2$$

7'Ø FLEX. & AXIAL CAP. (3% STEEL)

STEEL AREAS:

$$\frac{166.25\ IN^2}{8\ AREAS} = 20.78\ IN^2/\ AREA$$

AREA	STRAIN	STRESS (KSI)	FORCE (KIPS)	DIST. TO N.A. (INCHES)	
① & ⑤	.00034	9.86	204.9	5.34	COMPRES
② & ④	.00170	49.28	1024.0	32.21	
③	.00275	60.00	1246.8	43.34	
⑥ & ⑧	.00136	39.42	819.1	21.53	TENSION
⑦	.00207	60.00	1246.8	32.66	

CONCRETE AREA:

FORCE = 2560.8(.85)(4) = 8,706.7 KIPS

DIST. FROM CENTROID TO N.A.
EST. (.42)(42) + 5.34 = 22.98

$\Sigma F_V = 0$

TENSION
1246.8 = (−) 1246.8
(2) 819.1 = (−) 1638.2

COMPRESSION
(2) 204.9 = (+) 409.8
(2) 1024.0 = (+) 2048.0
1246.8 = (+) 1246.8
8706.7 = (+) 8706.7

BALANCED AXIAL LOAD (+) 9526.3

(73P)

7'Ø FLEX & AXIAL CAP (3% STEEL)

BALANCED MOMENT

−(2)819.1	(21.53)	=	−35,270	K-IN
− 1246.8	(32.66)	=	−40,720	
+(2)204.9	(5.34)	=	+ 2,188	
+(2)1024.0	(32.21)	=	+ 65,966	
+ 1246.8	(43.34)	=	+ 54,036	
+ 8706.7	(22.98)	=	+200,080	

BALANCED MOMENT = +246,280 K-IN

= 20,523 K-FT

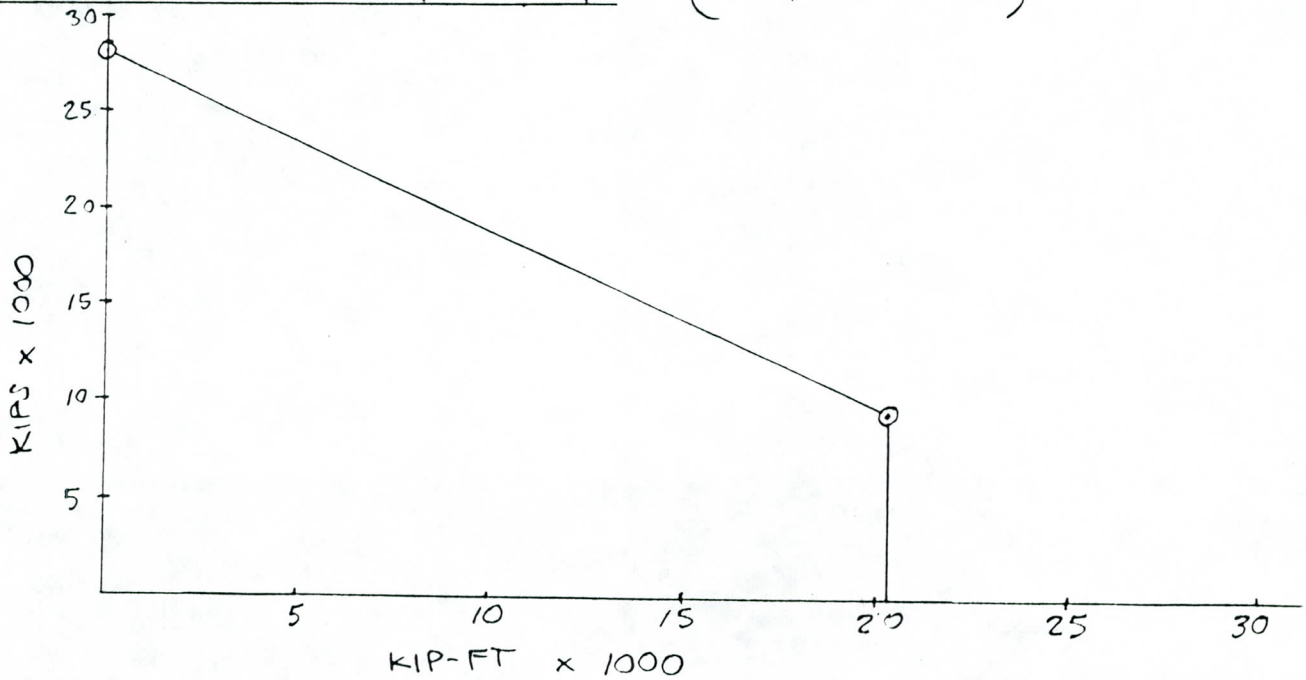
PURE COMPRESSION AASHTO 1.5.33

$$P_o = 0.85 f'_c (A_G - A_{ST}) + A_{ST} f_y$$

$$= 0.85(4)(5541.77 - 166.25) + (166.25)(60)$$

$$= 28,251.7 \text{ KIPS}$$

INTERACTION DIAGRAM (ULTIMATE)



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]RUN
FLEXURAL & AXIAL CAPACITY OF A SEVEN FOOT DIAMETER COLUMN

ULTIMATE STRENGTH ANALYSIS FOR F'C= 4 KSI AND FY= 60 KSI

PERCENTAGE OF STEEL
?.3

BALANCED AXIAL LOAD (KIPS)
8979.47545

BALANCED MOMENT (KIP-FEET)
17423.7679

PURE COMPRESSION - NO MOMENT  (ULTIMATE LOAD IN KIPS)
19782.9919

]RUN
FLEXURAL & AXIAL CAPACITY OF A SEVEN FOOT DIAMETER COLUMN

ULTIMATE STRENGTH ANALYSIS FOR F'C= 4 KSI AND FY= 60 KSI

PERCENTAGE OF STEEL
?.4

BALANCED AXIAL LOAD (KIPS)
8999.7306

BALANCED MOMENT (KIP-FEET)
17538.5806

PURE COMPRESSION - NO MOMENT  (ULTIMATE LOAD IN KIPS)
20096.6558

]RUN
FLEXURAL & AXIAL CAPACITY OF A SEVEN FOOT DIAMETER COLUMN

ULTIMATE STRENGTH ANALYSIS FOR F'C= 3 KSI AND FY= 60 KSI

PERCENTAGE OF STEEL
?1

BALANCED AXIAL LOAD (KIPS)
6909.24838

BALANCED MOMENT (KIP-FEET)
13991.4519

PURE COMPRESSION - NO MOMENT  (ULTIMATE LOAD IN KIPS)
21978.6391
```

```
]PR#0
]RUN
FLEXURAL & AXIAL CAPACITY OF A SEVEN FOOT DIAMETER COLUMN

ULTIMATE STRENGTH ANALYSIS FOR F'C= 4 KSI AND FY= 60 KSI

PERCENTAGE OF STEEL
?1

BALANCED AXIAL LOAD (KIPS)
9121.2615

BALANCED MOMENT (KIP-FEET)
18227.457

PURE COMPRESSION - NO MOMENT  (ULTIMATE LOAD IN KIPS)
21978.6391
```

```
]RUN
FLEXURAL & AXIAL CAPACITY OF A SEVEN FOOT DIAMETER COLUMN

ULTIMATE STRENGTH ANALYSIS FOR F'C= 4 KSI AND FY= 60 KSI

PERCENTAGE OF STEEL
?2

BALANCED AXIAL LOAD (KIPS)
9323.813

BALANCED MOMENT (KIP-FEET)
19375.5844

PURE COMPRESSION - NO MOMENT  (ULTIMATE LOAD IN KIPS)
25115.2779
```

```
]RUN
FLEXURAL & AXIAL CAPACITY OF A SEVEN FOOT DIAMETER COLUMN

ULTIMATE STRENGTH ANALYSIS FOR F'C= 4 KSI AND FY= 60 KSI

PERCENTAGE OF STEEL
?3

BALANCED AXIAL LOAD (KIPS)
9526.3645

BALANCED MOMENT (KIP-FEET)
20523.7117

PURE COMPRESSION - NO MOMENT  (ULTIMATE LOAD IN KIPS)
28251.9168
```

73R

PIER DESIGN

REINFORCING STEEL FOR 5'Ø COLUMN

MOMENT FROM LATERAL LOADING:

TOP OF COLUMN = 1257.9 K-FT
BOTTOM " = 358.9

MOMENT FROM LONGIT. LOADING:

TOP $62^K \times 6$ FT $\div 1.33 = 279.7$ K-FT
BOTTOM $62^K \times 26$ FT $\div 1.33 = 1212.0$ K-FT

DEFLECTION x D.L. @ BOTTOM (LOADING VII)

$800^K \times \frac{1}{12}$ FT $= 66.7$ K-FT

DESIGN MOMENT:

$$\begin{array}{r} 1212.0 \\ + \quad 66.7 \\ \hline 1278.7 \end{array}$$ K-FT

REQUIRED ULTIMATE CAPACITY FOR FLEXURE

$$\frac{1278.7}{.35} = 3{,}653 \text{ K-FT}$$

REQUIRED ULTIMATE CAPACITY FOR AXIAL

$$\frac{800}{.35} = 2{,}285.7 \text{ KIPS}$$

NOTE: CAN USE 0.1 % STEEL TO SATISFY THE ABOVE REQ.

5'Ø FLEX. & AXIAL CAP (1% STEEL)

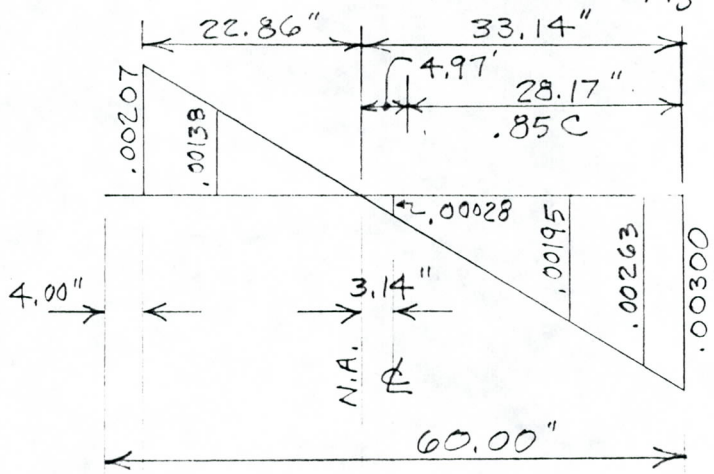
$A_S = 0.01\, A_G$

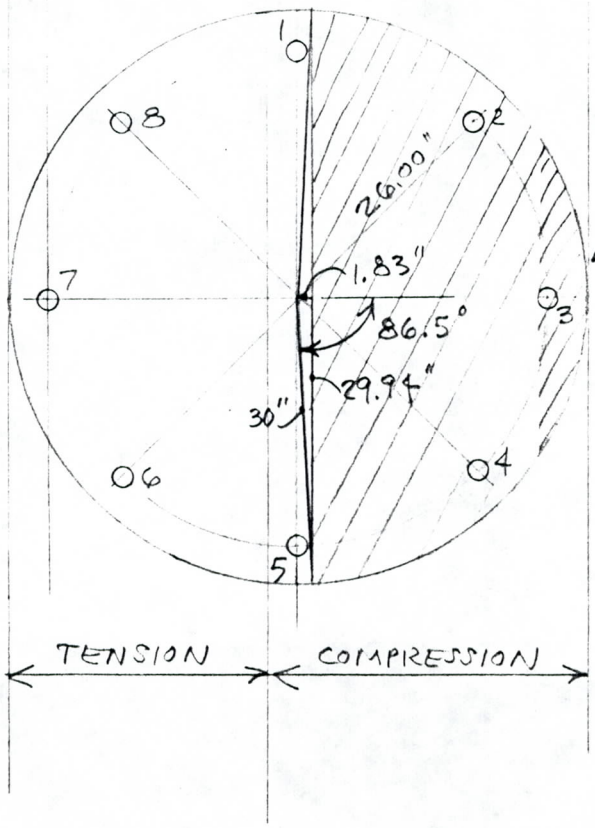
$f'_c = 4$ KSI

$f_y = 60$ KSI

$A_G = \pi (30)^2 = 2{,}827.4 \text{ IN}^2$

$A_S = 28.27 \text{ IN}^2$





CONCRETE COMPRESSION AREA

$$(2)\frac{86.5^\circ}{360.0}\left[2{,}827.4\right] - 1.83(29.94) - \frac{3}{8}(28.27) = 1293.38 \text{ IN}^2$$

5' Ø FLEX. & AXIAL CAP. (1% STEEL)

STEEL AREAS: $\frac{28.27}{8} = 3.53 \text{ IN}^2$

AREA	STRAIN	STRESS (KSI)	FORCE (KIPS)	DIST. TO N.A. (INCHES)	
① & ⑤	.00028	8.11	28.6	3.14	COMPRESSION
② & ④	.00195	56.52	199.5	21.52	COMPRESSION
③	.00263	60.00	211.8	29.14	COMPRESSION
⑥ & ⑧	.00138	40.00	141.2	15.24	TENSION
⑦	.00207	60.00	211.8	22.86	TENSION

CONCRETE AREA:

FORCE = 1293.38(.85) 4 = 4,397.2 KIPS

DIST. FROM CENTROID TO N.A.
(.42)(30) + 3.14 = 15.74

$\Sigma F_v = 0$

TENSION	(2) 141.2 =	−	282.4
	211.8 =	−	211.8
COMPRESSION	(2) 28.6 =	+	57.2
	(2) 199.5 =	+	399.0
	211.8 =	+	211.8
	4397.2 =	+	4397.2

BALANCED AXIAL LOAD (+) 4,571.0 KIPS

BR

5' Ø FLEX. & AXIAL CAP. (1% STEEL)

$\Sigma M = 0$

−	(2) 141.2	(15.24)	=	−	4303.8
−	211.8	(22.86)	=	−	4,841.7
+	(2) 28.6	(3.14)	=	+	179.6
+	(2) 199.5	(21.52)	=	+	8,586.5
+	211.8	(29.14)	=	+	6,171.9
+	4397.2	(15.74)		+	69,211.9

BALANCED MOMENT + 75,004.4 K-IN

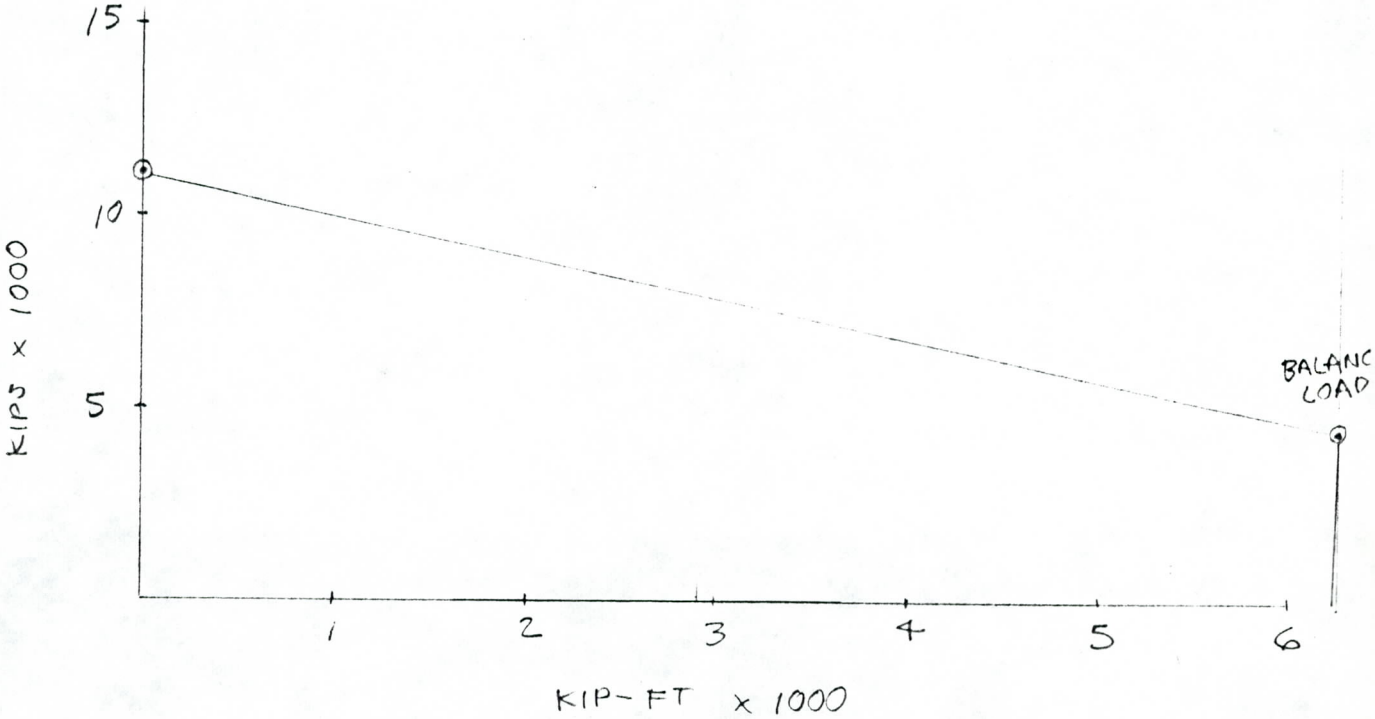
6,250.4 K-FT

PURE COMPRESSION

$$P_o = .85(4)(2827.4 - 28.27) + 28.27(60)$$

$$= 11,213.2 \text{ KIPS}$$

INTERACTION DIAGRAM (ULTIMATE)

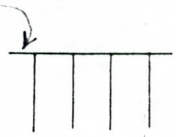


```
]RUN
FLEXURAL & AXIAL CAPACITY OF A FIVE FOOT DIAMETER COLUMN

ULTIMATE STRENGTH ANALYSIS FOR F'C= 4 KSI AND FY= 60 KSI

PERCENTAGE OF STEEL
?.2

BALANCED AXIAL LOAD (KIPS)
4460.97186

BALANCED MOMENT (KIP-FEET)
5902.30783

PURE COMPRESSION - NO MOMENT  (ULTIMATE LOAD IN KIPS)
9933.3306

]RUN
FLEXURAL & AXIAL CAPACITY OF A FIVE FOOT DIAMETER COLUMN

ULTIMATE STRENGTH ANALYSIS FOR F'C= 4 KSI AND FY= 60 KSI

PERCENTAGE OF STEEL
?.1

BALANCED AXIAL LOAD (KIPS)
4447.16693

BALANCED MOMENT (KIP-FEET)
5858.70049

PURE COMPRESSION - NO MOMENT  (ULTIMATE LOAD IN KIPS)
9773.298

]RUN
FLEXURAL & AXIAL CAPACITY OF A FIVE FOOT DIAMETER COLUMN

ULTIMATE STRENGTH ANALYSIS FOR F'C= 4 KSI AND FY= 60 KSI

PERCENTAGE OF STEEL
?1

BALANCED AXIAL LOAD (KIPS)
4571.41132

BALANCED MOMENT (KIP-FEET)
6251.16654

PURE COMPRESSION - NO MOMENT  (ULTIMATE LOAD IN KIPS)
11213.5914
```

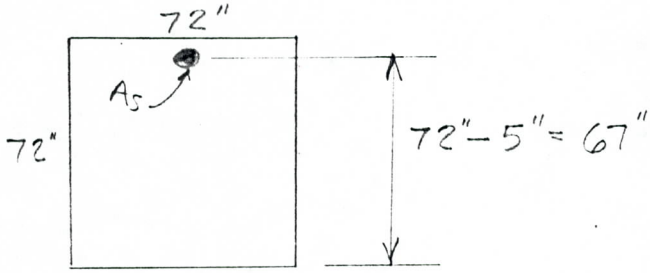
PIER DESIGN

CAP FLEXURE REINFORCING

AT CANTILEVER



TOTAL MOMENT AS PER LOAD GROUP I
IS 2907.2 K-FT




```
]RUN
WORKING STRESS DESIGN:

FOR F'C=4 KSI AND FY=60 KSI
WORKING STRESSES OF FC=1.6 KSI AND FS=24 KSI

BEAM WIDTH (INCHES):
?72

DEPTH TO REINFORCING (INCHES):
?67

REQUIRED MOMENT CAPACITY (KIP-FT):
?2907.2

AREA OF REINFORCING STEEL:
24.5524669        SQUARE INCHES

PERCENT OF BALANCED STEEL RATIO:
43.7402408
```

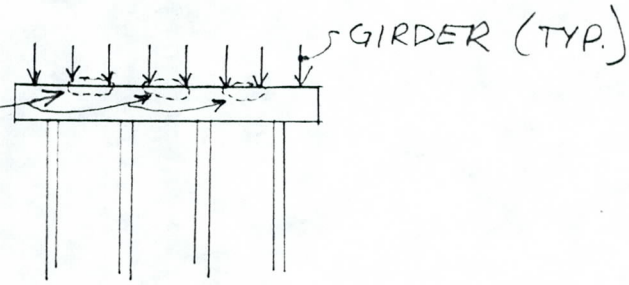
$\frac{24.55}{1.56} = 15.73 \longrightarrow$ USE MINIMUM STEEL AREA AS PER AASHTO 1.5.7 (B)

16-#11 BARS

PIER DESIGN

CAP FLEXURE REINFORCING

TOP OF CAP



NEGATIVE MOMENT OCCURS UNDER EACH GIRDER BETWEEN COLUMNS.

THE MAX. NEGATIVE MOMENT IN THIS AREA IS 911.28 KIP-FT. AS PER LOADING IV WITH THERMAL FORCES CONTRIBUTING THE MOST OF THIS FLEXURE.

WORKING STRESS DESIGN:

FOR F'C=4 KSI AND FY=60 KSI
WORKING STRESSES OF FC=1.6 KSI AND FS=24 KSI

BEAM WIDTH (INCHES):
?72

DEPTH TO REINFORCING (INCHES):
?67

REQUIRED MOMENT CAPACITY (KIP-FT):
?911.28

AREA OF REINFORCING STEEL:
7.6961241 SQUARE INCHES

PERCENT OF BALANCED STEEL RATIO:
13.7106517

MIN. STEEL AS PER 1.5.7(A)

$\rightarrow 1.33 \times 7.7\ in^2 = 10.24\ in^2$

USE 7-#11 BARS

MIN. STEEL AREA AS PER 1.5.7 (B):

1.2 x CRACKING MOMENT:

$$S = \frac{(72)(72)^2}{6} = 62208\ in^3$$

PIER DESIGN

CAP FLEXURE REINFORCING

$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{4000} = 474.3 \text{ PSI}$$

$$M_c = (1.2)\,S\,f_r = (1.2)(62{,}208)(474.3)$$
$$= 35{,}406{,}305 \text{ IN-\#}$$
$$= 2950.5 \text{ K-FT}$$

```
WORKING STRESS DESIGN:

FOR F'C=4 KSI AND FY=60 KSI
WORKING STRESSES OF FC=1.6 KSI AND FS=24 KSI

BEAM WIDTH (INCHES):
?72

DEPTH TO REINFORCING (INCHES):
?67

REQUIRED MOMENT CAPACITY (KIP-FT):
?2950.5

AREA OF REINFORCING STEEL:
24.9181527        SQUARE INCHES

PERCENT OF BALANCED STEEL RATIO:
44.3917103
```

→ 16-#11 BARS

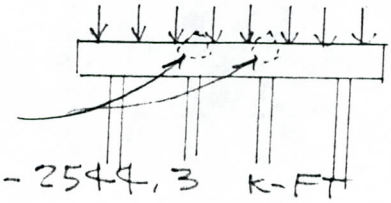
USE

7-#11 BARS
AS PER
1.5.7 (A)

PIER DESIGN

CAP FLEXURE REINFORCING OVER INTERIOR COLUMNS

MAX. DESIGN MOMENT OCCURS IN LOADING VII



TOTAL MOMENT = -2544.3 K-FT

DESIGN MOMENT = 2544.3 ÷ 1.33 = 1913.0 K-FT

ALLOWABLE STRESS INCREASE FOR LOADING VII

```
WORKING STRESS DESIGN:

FOR F'C=4 KSI AND FY=60 KSI
WORKING STRESSES OF FC=1.6 KSI AND FS=24 KSI

BEAM WIDTH (INCHES):
?72

DEPTH TO REINFORCING (INCHES):
?67

REQUIRED MOMENT CAPACITY (KIP-FT):
?1913

AREA OF REINFORCING STEEL:
16.1560502        SQUARE INCHES

PERCENT OF BALANCED STEEL RATIO:
28.7820173
```

THIS MOMENT IS LESS THAN 1.2 x CRACKING MOMENT = 2950.5 K-FT

THERE FOR DESIGN MOMENT MUST BE 1.33 x 1913 = 2544.3 K-FT

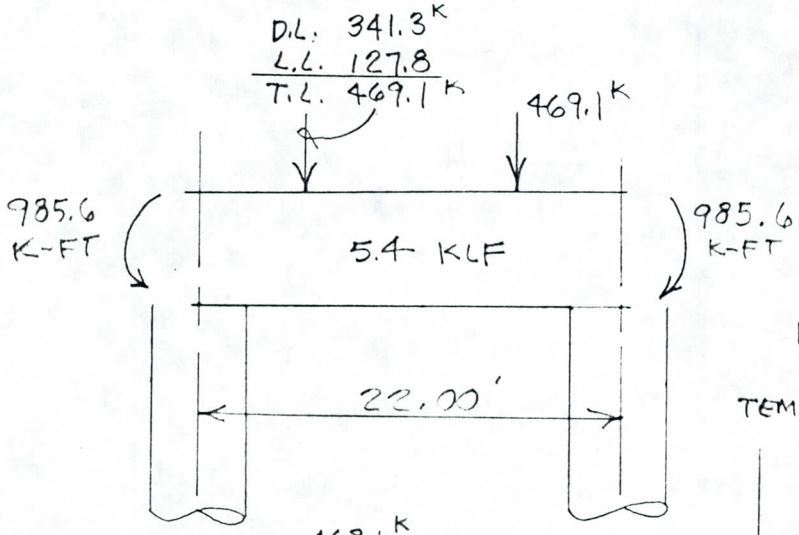
$A_s = 16.156 \times 1.33 = 21.49 \text{ IN}^2$

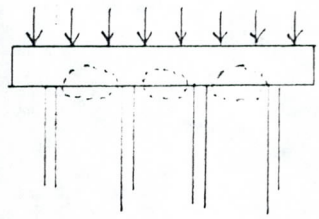
USE 14 - # 11 BARS

PIER DESIGN

CAP FLEXURE REINFORCING

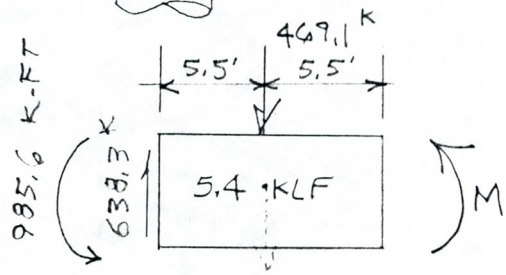
POSITIVE MOMENT BETWEEN COLUMNS





D.L. 1542.2 K-FT
L.L. 156.7
TEMP + SHR -521.4
-191.9
+985.6 K-FT

341.3
59.4
127.8
86.3
23.5
638.3 K



S.F. = .108 x 100K LATERAL LOAD
W = .672 " "
WL = .116 " "
.866

.866 x 221.6 K-FT = 191.9 K-FT

MOMENT DUE TO 100K LATERAL LOAD

.866 x 27.1 = 23.5K
SHEAR DUE TO 100 K LAT. LOAD

$$M = -469.1(5.5) = -2580.1$$
$$+638.3(11.0) = +7021.3$$
$$-985.6 = -985.6$$
$$-\frac{5.4}{2}(11.0)^2 = -326.7$$
$$+3128.9 \text{ K-FT}$$

PIER DESIGN

CAP FLEXURE REINFORCEMENT

POSITIVE MOMENT BETWEEN COLUMNS

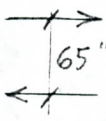
```
]RUN
WORKING STRESS DESIGN:

FOR F'C=4 KSI AND FY=60 KSI
MAXIMUM ALLOWABLE STRESS:
FC=1.6 KSI AND FS=24.0 KSI

BEAM WIDTH (INCHES):
?72

DEPTH TO REINFORCING (INCHES):
?65

REQUIRED MOMENT CAPACITY (KIP-FT):
?3128.9

AREA OF REINFORCING STEEL:
27.2249811          SQUARE INCHES

PERCENT OF BALANCED STEEL RATIO:
50.174244

FC (KSI) =          1.06545769
FS (KSI) =          24
```

USE 16 - #11 ON BOTTOM & 8 - #11 ON TOP

$\frac{27.22}{1.56} = 17.45 \Rightarrow$ 18 - #11 BARS FOR SINGLE REIN. BEAM

DOUBLE REIN:

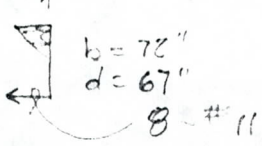
$M_1 \Rightarrow$



$8 \times 1.56 \times 24$ KSI $= 299.5$ K

$(299.5)\left(\frac{65}{12}\right) = 1622.4$ K-FT

$M_2 \Rightarrow$



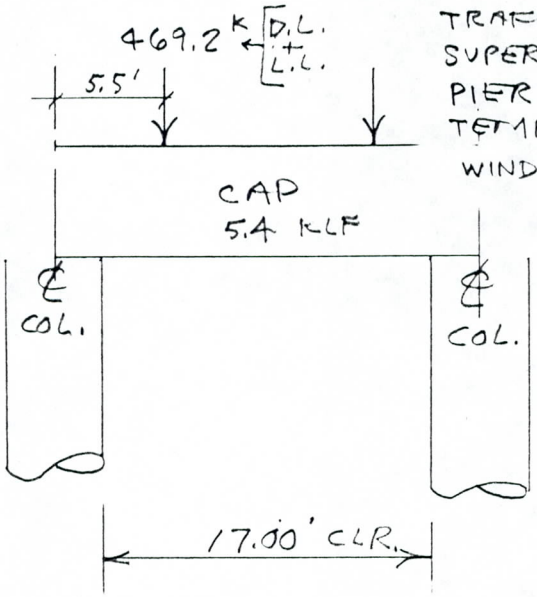
$1913 \times \frac{12.48}{16.16} = 1477.4$ K-FT

$M_1 + M_2 = 3099.8$ K-FT

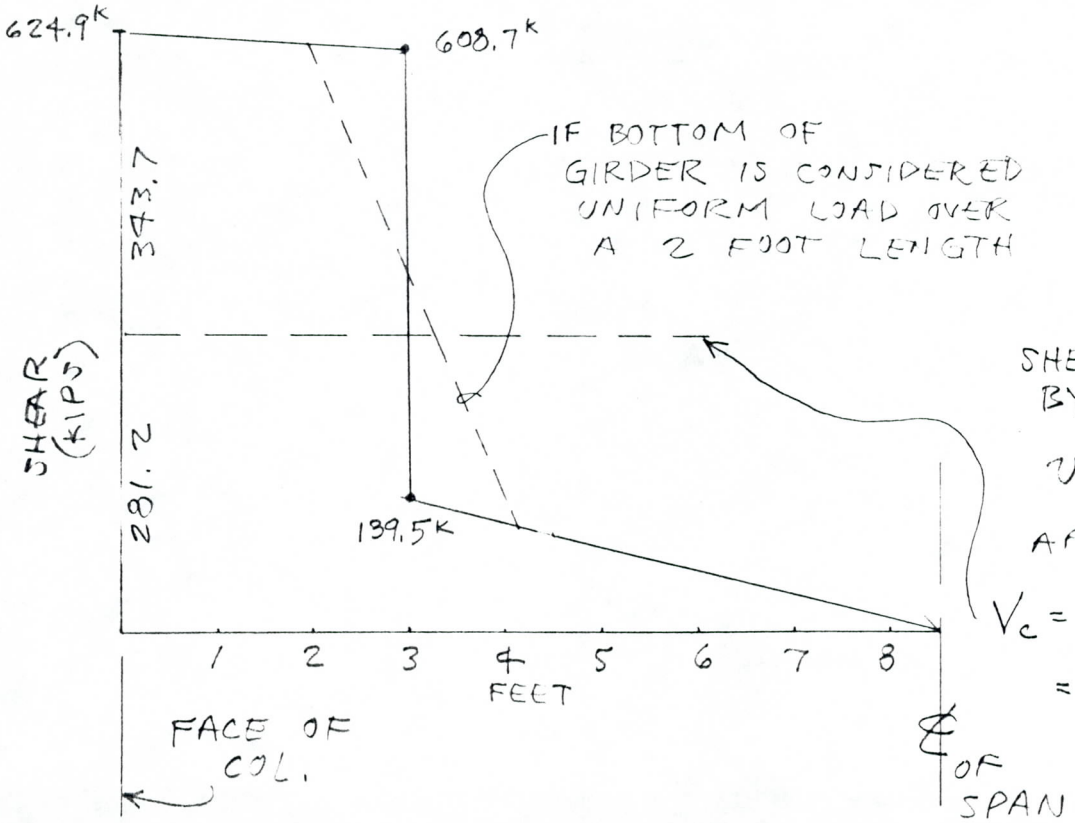
$\frac{3099.8}{3128.9} = .99$

PIER DESIGN

CAP SHEAR REINFORCING



TRAFFIC L.L.	=	127.9
SUPERSTR. + GIRDER D.L.	=	341.3K
PIER CAP D.L.	=	45.9
TEMP. + SHRINKAGE	=	86.3
WIND LOADS + S.F.	=	23.5
TOTAL SHEAR		624.9 K



SHEAR CARRIED BY CONCRETE

$v_c = .95\sqrt{f'}$

AASHTO 1.5.29

$V_c = .95\sqrt{4000} \times 72 \times 65$

$= 281.2$ KIPS

PIER DESIGN

CAP SHEAR REINFORCING

$$v - v_c = \frac{343.7K}{72 \times 65} = 0.073 \text{ KSI}$$

$$A_v = \frac{(v - v_c)\, b_w S}{f_s}$$

S = 24" MAX. AS PER AASHTO 1.5.10

FOR S = 12"

$$A_v = \frac{(0.073)\, 72\, (12)}{24 \text{ KSI}} = 2.623 \text{ IN}^2$$

FOR #6 BARS

$$\frac{2.628}{.44} = 5.97$$

FOR $v - v_c = 0$ AASHTO 1.5.10

$$A_v = \frac{50\, b_w S}{f_y} = \frac{50(72)(18)}{60{,}000} = 1.08 \text{ IN}^2$$

USE 1 SET OF 3 - #6 [ ] @ 12" UP TO 4' FROM FACE OF COLUMNS

1 SET OF 3 - #6 [ ] @ 18" ELSEWHERE

FR

PIER DESIGN

REINFORCING SUMMARY

DEVELOPEMENT LENGTH OF #11 BAR:

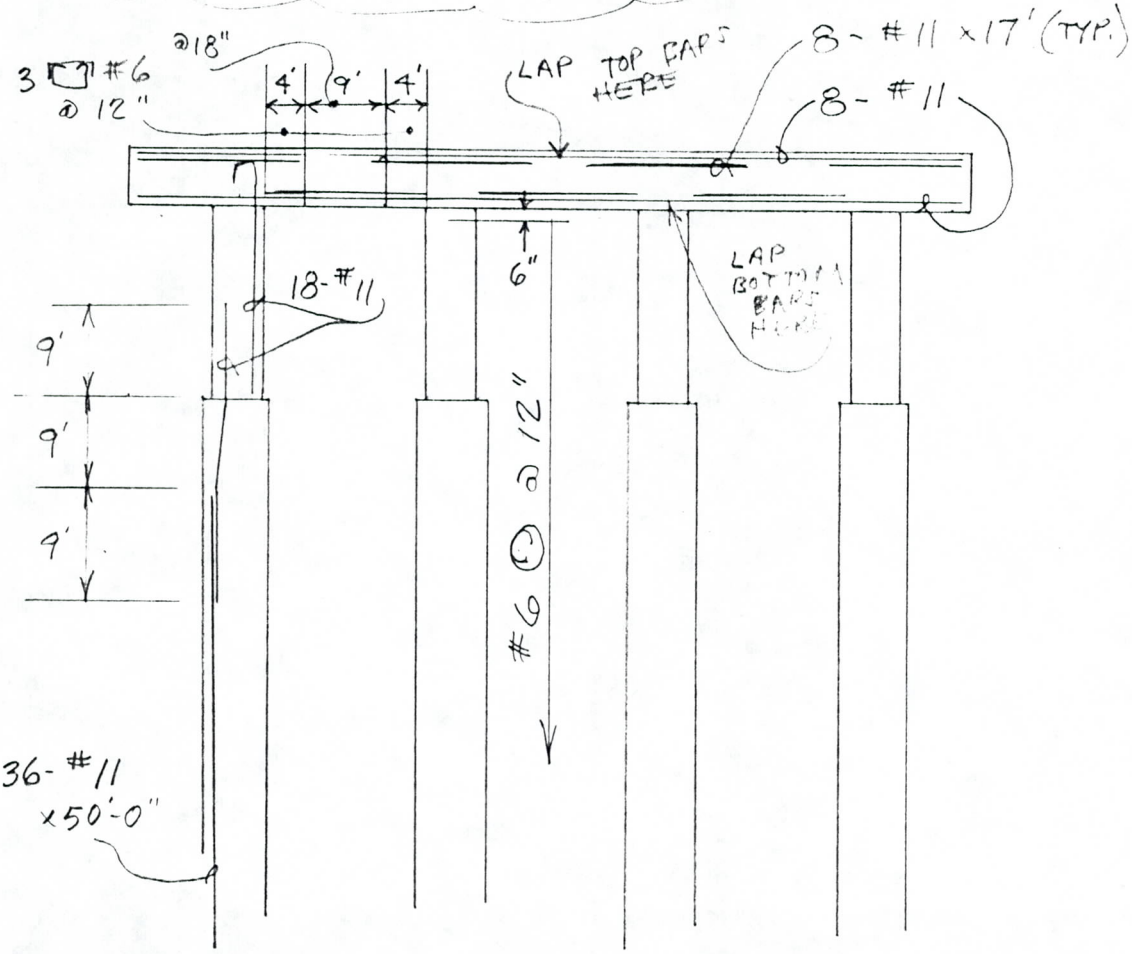
$$l_d = \frac{0.04(1.56)60{,}000}{\sqrt{4000}} = 59.20 \text{ INCHES} = 5'-0''$$

$$\text{DEV. LEN.} = l_d \times 1.4 = 82.88 \text{ IN}$$

TOP BAR

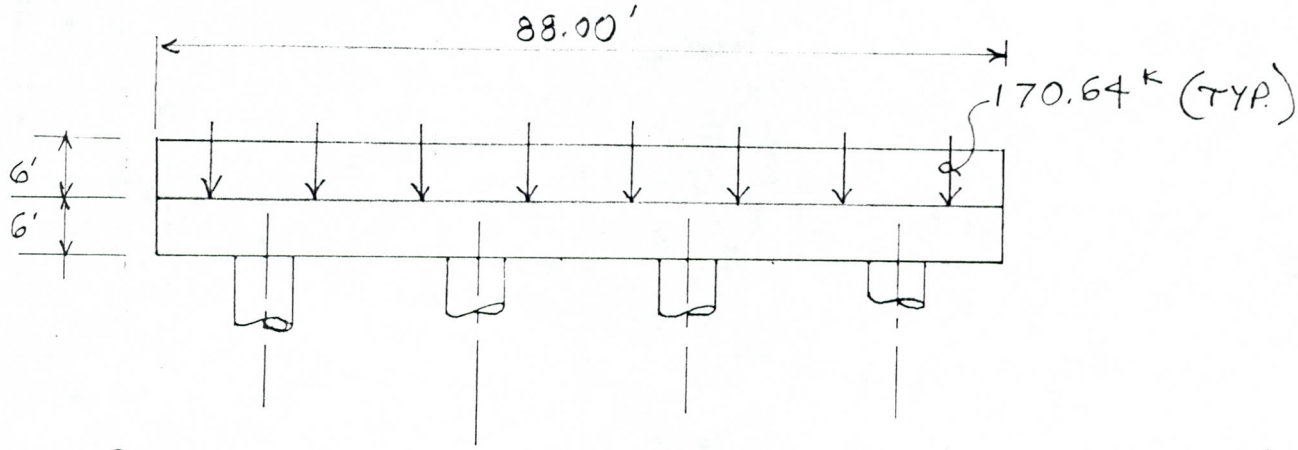
6'-10"

USE CLASS C SPLICES IN COLUMNS $= 1.7\, l_d = 8'-6''$



ABUTMENT LOADS

DEAD LOADS

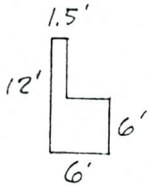


GIRDER + DECK D.L. = 2.942 KLF × $\frac{116}{2}$ = 170.64 K

BEAM SEAT D.L. = 6×6×.15 KCF = 5.40 KLF

BACK WALL D.L. = 6×1.5×.15 = 1.35

6.75 KLF



TRAFFIC LIVE LOAD

IMPACT NEED NOT BE CONSIDERED - 1.2.12

AASHTO LANE LOAD REACTION = 78.1 K

TOTAL TRAFFIC LOAD:

4 LANES × .75 × 78.1 K/LANE = 234.3 K

REDUCTION 1.2.9

CARRY THE LOADING ON 4 GIRDERS:

234.3 ÷ 4 = 58.58 K/GIRDER

TSR

ABUTMENT LOADS

LONGITUDINAL FORCES 1.2.13

THIS LOADING IS PLACED 6' ABOVE TOP OF BACKWALL.

$.05 \times 234.3 =$ 11.715 K

WIND LOADS 1.2.14

USE ½ OF WIND LOADS ON PIERS

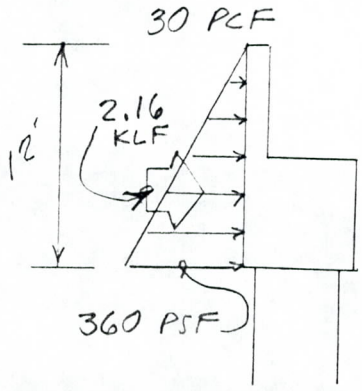
$\frac{64.2}{2} =$ 32.1 KIPS

WIND LOAD ON LIVE LOAD

USE ½ WIND LOAD ON PIERS

$\frac{11.6}{2} =$ 5.8 KIPS

EARTH PRESSURE 1.2.19



$2.16 \text{ KLF} \times 22 =$ 47.52 K/COL.

TSR

ABUTMENT LOADS

THERMAL & SHRINKAGE LOADS

AS THE SUPERSTRUCTURE EXP. & CONTR. IT EXERTS FORCES ON THE ABUT. THROUGH THE ELASTOMERIC BEARING PADS.

THE PAD DIMENSIONS ARE 26" x 16" x 5¼" THK.

THE MAX. SUPERSTRUCTURE MOVEMENT IS 2.57". THE PAD DURO HARDNESS IS 60.

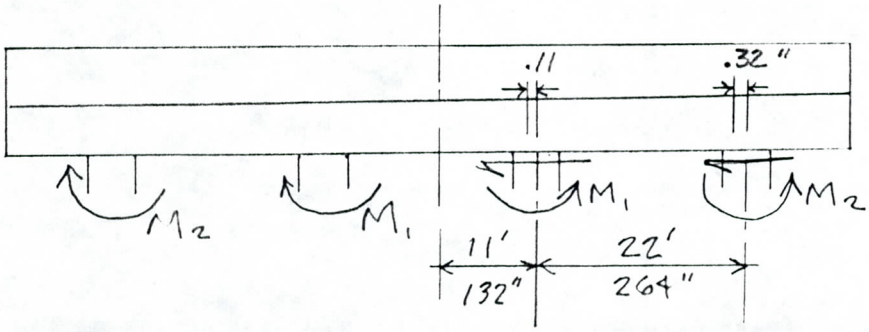
THE SHEAR MODULUS FOR A DURO OF 60 IS 160 PSI

$$F_{THERMAL} = \frac{(MODULUS)(AREA)(\Delta)}{T}$$

$$= \frac{(160)(26 \times 16)(2.57)}{5.25} = 32{,}582^{\#}$$

A FORCE OF 32.6^{K} CAN BE GENERATED PER PAD.

FOR CAP TEMP. & SHRINKAGE
USE .0008 IN/IN



$M_1 = 7211.1$ K-FT/IN $\times$ .11 = 793.2 K-FT

$M_2 = 7211.1$ K-FT/IN $\times$ .32 = 2307.6 K-FT

TSR

ABUTMENT LOADS

EARTHQUAKE LOADS

EQ. LOAD PARALLEL TO THE ABUTMENT CAP ARE ASSUMED TO BE ½ THE LOAD PARALLEL TO THE PIER CAPS.

$$\frac{288.63^{K}}{2} = 144.32^{K}$$

EQ. LOAD PERPENDICULAR TO THE ABUTMENT CAP ARE FROM THE ELASTOMERIC PADS AS THEY DEFORM DUE TO SUPERSTRUCTURE MOVEMENT : 2" ±

$$F_{SEISMIC} = \frac{(160)(26 \times 16)(2.0)}{5.25} = 25{,}357^{\#}$$

SEE THERMAL & SHRINKAGE FOR FORMULA

A FORCE OF 25.4 KIPS CAN BE GENERATED PER PAD.

ABUTMENT DESIGN

CAISSON DESIGN

REQ. DIAMETER

GIRDER + DECK D.L.	$171^K \times 8$	=	1368^K
ABUTMENT	88×6.75 KLF	=	594
	DEAD LOAD		1962^K
TRAFFIC LOAD		=	235
			2197^K

REQ. AREA OF END BEARING CAISSONS

$$\frac{2197^K}{30 \text{ KSF} @ 40'} = 73.23 \text{ FT}^2$$

USE 4 - 5' Ø CAISSONS:

TOTAL AREA $= 4 \times \pi \times (2.5)^2 = 78.54$ FT

MAXIMUM MOMENT

REF. "FOUNDATIONS AND EARTH STRUCTURES DESIGN MANUAL 7.2"

$$I = 0.785\, R^4 = .785(30)^4 = 635850 \text{ IN}^4$$

$$E = 57000\sqrt{f'_c} = 57000\sqrt{4000} = 3.6 \times 10^6 \text{ PSI}$$

$f = 50$ PCI $\longrightarrow$ USE 25 FOR CYCLIC LOADIN

$$T = \left[\frac{EI}{f}\right]^{1/5} = \left[\frac{635850 \times 3.6 \times 10^6}{25}\right]^{.2}$$

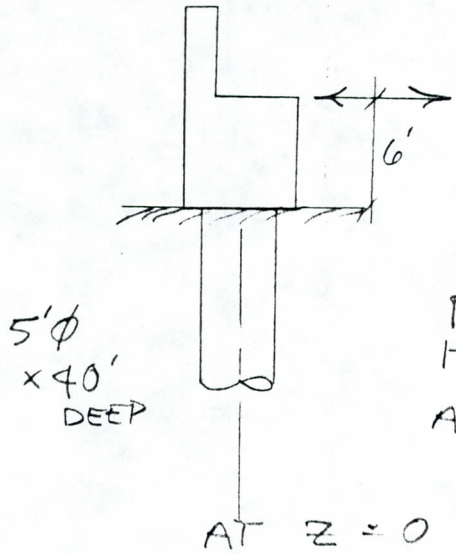
$= 155.72$ INCHES

$= 12.98$ FEET

ABUTMENT DESIGN

CAISSON DESIGN

EARTHQUAKE APPLIED PERPENDICULAR TO CAP



$32.6 \times 2 = 65.2^K$ / CAISSON

$\frac{L}{T} = \frac{40}{18.36} = 2.18$

$P = 65.2^K$
$H = 6$ FT

APPLIED MOMENT $= 65.2 \times 6 = 391.2$ K-FT

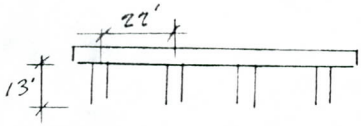
AT $Z = 0$

$M_M = 1.0(391.2) = 391.2$ K-FT
$M_P = 0(65.2 \times 12.98) = 0$
391.2 K-FT

AT $Z = 0.8$

$M_M = 0.9(391.2) = 352.08$ K-FT
$M_P = 0.5(65.2 \times 12.98) = 423.15$
775.23 K-FT

STIFFNESS RATIOS



I CAP $= 108.0$ FT^4
I CAISSON $= .785(2.5)^4 = 30.67$ FT^4

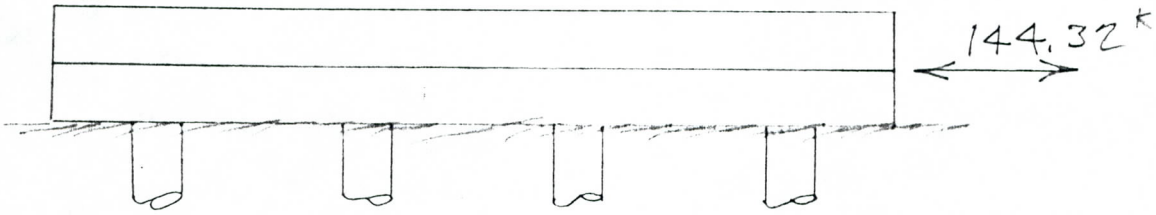
$\frac{I}{L}$ CAP $= \frac{108.0}{22.0} = 4.91 \longrightarrow \frac{4.91}{4.91 + 2.36} = 0.675$

$\frac{I}{L}$ CAISSON $= \frac{30.67}{13} = 2.36 \longrightarrow 0.325$

ABUTMENT DESIGN

CAISSON DESIGN

LOADS APPLIED PARALLEL TO CAP



$$P = \frac{144.32}{4} = 36.08^{K}/\text{CAISSON}$$

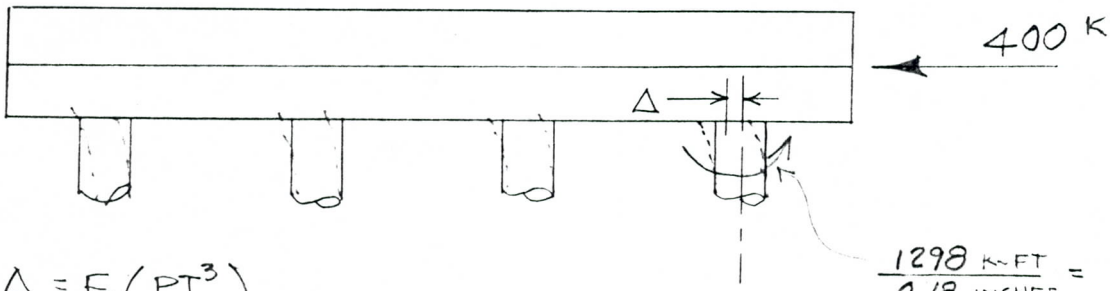
AT $Z = 1.75$

$$M_p = F_M (PT)$$
$$= 0$$

AT $Z = 0$

$$M_p = 1.0(36.08)(12.98) = 468.32 \text{ K-FT}$$

DEFLECTION FOR 100 K PER CAISSON



$$\frac{1298 \text{ K-FT}}{0.18 \text{ INCHES}} = 7211.1 \text{ K-FT/IN}$$

$$\Delta = F_\delta\left(\frac{PT^3}{EI}\right)$$

$$= 1.1\left(\frac{100 \times [12.98 \times 12]^3}{635850 \times 3.6 \times 10^3}\right) = 0.18 \text{ INCHES}$$

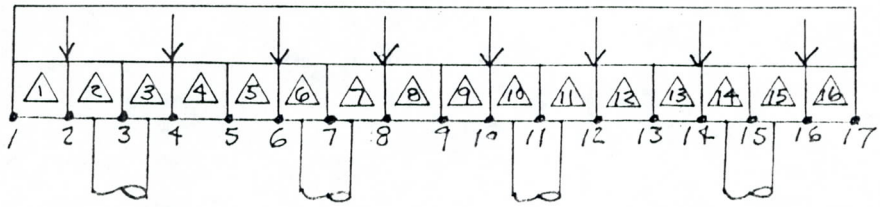
$$M = F_M(PT) = 1.0(100 \times 12.98) = 1298.0 \text{ K-FT}$$

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040 SHT. A-8 OF 39
DESC. 75 TH AVE MCFCD BY AGD
DATE SEPT. 25, 1984
TSB

ABUTMENT DESIGN

STRESS PROGRAM



$E = 519{,}120$ KSF

$I = \frac{(6)(6)^3}{12} = 108.0$ FT^4

STRUCTURE MCFCD - ABUTMENT
TYPE PLANE FRAME
NUMBER OF JOINTS 17
NUMBER OF MEMBERS 16
NUMBER OF SUPPORTS 4
NUMBER OF LOADINGS 4
TABULATE ALL

JOINT COORDINATES

1	0.	0.	
2	5.5	0.	
3	11.0	0.	S
4	16.5	0.	
5	22.0	0.	
6	27.5	0.	
7	33.0	0.	S
8	38.5	0.	
9	44.0	0.	
10	49.5	0.	
11	55.0	0.	S
12	61.5	0.	
13	66.0	0.	
14	71.5	0.	
15	77.0	0.	S
16	82.5	0.	
17	88.0	0.	

ABUTMENT DESIGN

STRESS PROGRAM

```
MEMBER INCIDENCE
1    1    2
2    2    3
3    3    4
4    4    5
5    5    6
6    6    7
7    7    8
8    8    9
9    9    10
10   10   11
11   11   12
12   12   13
13   13   14
14   14   15
15   15   16
16   16   17

MEMBER PROPERTIES PRISMATIC
 1 THRU 16   AX 36.00   IZ 108.00

CONSTANTS E 519120.0 ALL
LOADING 1 DEAD LOAD OF CAP PLUS GIRDERS
MEMBER LOADS
1   FORCE Y UNIFORM - 6.75
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
```

ABUTMENT DESIGN

STRESS PROGRAM

JOINT LOADS
2 FORCE Y - 170.64
4
6
8
10
12
14
16

LOADING 2 LATERAL LOAD (100 K)
JOINT LOADS
3 MOMENT Z 324.5
7
11
15

LOADING 3 TRAFFIC
JOINT LOADS
2 FORCE Y - 58.58
4
6
8

LOADING 4 TEMPERATURE PLUS SHRINKAGE
JOINT LOADS
3 MOMENT Z + 2307.6
7 + 793.2
11 - 793.2
15 - 2307.6

SOLVE

TSR

```
.NULL.
00001: PRIME VERSION OF STRESS(3.2)          25/09/1978
00002: ==========================================================
00003:
00004: INPUT LISTING OF STRESS PROBLEM
00005: ==========================
00006:
00007:
00008: STRUCTURE MCFCD-ABUTMENT
00009: TYPE PLANE FRAME
00010: NUMBER OF JOINTS 17
00011: NUMBER OF MEMBERS 16
00012: NUMBER OF SUPPORTS 4
00013: NUMBER OF LOADINGS 4
00014: TABULATE ALL
00015: JOINT COORDINATES
00016: 1   0.   0.
00017: 2   5.5  0.
00018: 3  11.0  0. S
00019: 4  16.5  0.
00020: 5  22.0  0.
00021: 6  27.5  0.
00022: 7  33.0  0. S
00023: 8  38.5  0.
00024: 9  44.0  0.
00025: 10 49.5  0.
00026: 11 55.0  0. S
00027: 12 61.5  0.
00028: 13 66.0  0.
00029: 14 71.5  0.
00030: 15 77.0  0. S
00031: 16 82.5  0.
00032: 17 88.0  0.
00033: MEMBER INCIDENCE
00034: 1   1   2
00035: 2   2   3
00036: 3   3   4
00037: 4   4   5
00038: 5   5   6
00039: 6   6   7
00040: 7   7   8
00041: 8   8   9
00042: 9   9  10
00043: 10 10  11
00044: 11 11  12
00045: 12 12  13
00046: 13 13  14
00047: 14 14  15
00048: 15 15  16
00049: 16 16  17
00050: JOINT RELEASES
00051: 3 MOMENT Z
00052: 7 MOMENT Z
00053: 11 MOMENT Z
00054: 15 MOMENT Z
00055: MEMBER PROPERTIES PRISMATIC
00056: 1 THRU 16  AX 36.0  IZ 108.00
00057: CONSTANTS  E  519120.0  ALL
```

```
00058: LOADING 1 DEAD LOAD OF CAP PLUS GIRDERS
00059: MEMBER LOADS
00060: 1 FORCE Y  UNIFORM -6.75
00061: 2 FORCE Y  UNIFORM -6.75
00062: 3 FORCE Y  UNIFORM -6.75
00063: 4 FORCE Y  UNIFORM -6.75
00064: 5 FORCE Y  UNIFORM -6.75
00065: 6 FORCE Y  UNIFORM -6.75
00066: 7 FORCE Y  UNIFORM -6.75
00067: 8 FORCE Y  UNIFORM -6.75
00068: 9 FORCE Y  UNIFORM -6.75
00069: 10 FORCE Y UNIFORM -6.75
00070: 11 FORCE Y UNIFORM -6.75
00071: 12 FORCE Y UNIFORM -6.75
00072: 13 FORCE Y UNIFORM -6.75
00073: 14 FORCE Y UNIFORM -6.75
00074: 15 FORCE Y UNIFORM -6.75
00075: 16 FORCE Y UNIFORM -6.75
00076: JOINT LOADS
00077: 2 FORCE Y -170.64
00078: 4 FORCE Y -170.64
00079: 6 FORCE Y -170.64
00080: 8 FORCE Y -170.64
00081: 10 FORCE Y -170.64
00082: 12 FORCE Y -170.64
00083: 14 FORCE Y -170.64
00084: 16 FORCE Y -170.64
00085: LOADING 2  LATERAL LOAD (100 KIPS)
00086: JOINT LOADS
00087: 3 MOMENT Z 324.5
00088: 7 MOMENT Z 324.5
00089: 11 MOMENT Z 324.5
00090: 15 MOMENT Z 324.5
00091: LOADING 3  TRAFFIC LOADING
00092: JOINT LOADS
00093: 2 FORCE Y -58.58
00094: 4 FORCE Y -58.58
00095: 6 FORCE Y -58.58
00096: 8 FORCE Y -58.58
00097: LOADING 4  TEMPERATURE PLUS SHRINKAGE
00098: JOINT LOADS
00099: 3 MOMENT Z   2307.6
00100: 7 MOMENT Z    793.2
00101: 11 MOMENT Z  -793.2
00102: 15 MOMENT Z -2307.6
00103: SOLVE
00104:1
00105:1
```

00106:
00107:
00108:
00109:
00110: STRUCTURE MCFCD-ABUTMENT
00111: ==========
00112:
00113:
00114:
00115:
00116: LOADING 1 DEAD LOAD OF CAP PLUS GIRDERS
00117: ==========
00118:
00119:
00120:
00121: MEMBER FORCES
00122:

	MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
00123:					
00124:	1	1	0.000	-0.005	-0.01
00125:	1	2	0.000	37.130	-102.11
00126:	2	2	0.000	-207.773	102.10
00127:	2	3	0.000	244.898	-1346.95
00128:	3	3	0.000	265.420	1346.94
00129:	3	4	0.000	-228.295	10.77
00130:	4	4	0.000	57.655	-10.77
00131:	4	5	0.000	-20.530	225.78
00132:	5	5	0.000	20.529	-225.78
00133:	5	6	0.000	16.596	236.59
00134:	6	6	0.000	-187.236	-236.59
00135:	6	7	0.000	224.361	-895.30
00136:	7	7	0.000	243.372	895.30
00137:	7	8	0.000	-206.247	341.15
00138:	8	8	0.000	35.606	-341.15
00139:	8	9	0.000	1.519	434.89
00140:	9	9	0.000	-1.520	-434.89
00141:	9	10	0.000	38.645	324.44
00142:	10	10	0.000	-209.285	-324.44
00143:	10	11	0.000	246.410	-928.72
00144:	11	11	0.000	218.127	928.72
00145:	11	12	0.000	-174.252	346.51
00146:	12	12	0.000	3.611	-346.51
00147:	12	13	0.000	26.764	294.42
00148:	13	13	0.000	-26.764	-294.42
00149:	13	14	0.000	63.889	45.12
00150:	14	14	0.000	-234.529	-45.12
00151:	14	15	0.000	271.654	-1346.89
00152:	15	15	0.000	244.890	1346.89
00153:	15	16	0.000	-207.765	-102.09
00154:	16	16	0.000	37.125	102.09
00155:	16	17	0.000	0.000	0.00

00156:

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	0.000	-0.005	-0.01
2	0.000	-170.643	-0.01
4	0.000	-170.640	-0.00
5	0.000	-0.000	-0.00
6	0.000	-170.640	0.00
8	0.000	-170.641	-0.00
9	0.000	-0.001	-0.00
10	0.000	-170.641	-0.00
12	0.000	-170.641	-0.00
13	0.000	-0.001	-0.00
14	0.000	-170.640	0.00
16	0.000	-170.640	-0.00
17	0.000	0.000	0.00

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
3	0.000	510.318	-0.00
7	0.000	467.732	0.00
11	0.000	464.538	-0.00
15	0.000	516.544	0.00

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
1	0.0000	-0.0011	0.0001
2	0.0000	-0.0005	0.0001
4	0.0000	0.0000	-0.0000
5	0.0000	-0.0001	-0.0000
6	0.0000	-0.0001	0.0000
8	0.0000	-0.0002	-0.0000
9	0.0000	-0.0003	0.0000
10	0.0000	-0.0002	0.0000
12	0.0000	-0.0001	-0.0000
13	0.0000	-0.0001	0.0000
14	0.0000	-0.0000	0.0000
16	0.0000	-0.0004	-0.0001
17	0.0000	-0.0010	-0.0001

SUPPORT JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
3	0.0000	0.0000	0.0000
7	0.0000	0.0000	-0.0000
11	0.0000	0.0000	0.0000
15	0.0000	0.0000	-0.0000

1

00216:
00217:
00218:
00219: STRUCTURE MCFCD-ABUTMENT
00220: ==========
00221:
00222:
00223:
00224:
00225: LOADING 2 LATERAL LOAD (100 KIPS)
00226: ==========
00227:
00228:
00229:
00230: MEMBER FORCES
00231:

	MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
00232:					
00233:	1	1	0.000	-0.002	-0.00
00234:	1	2	0.000	0.002	-0.01
00235:	2	2	0.000	-0.003	0.00
00236:	2	3	0.000	0.003	-0.02
00237:	3	3	0.000	24.584	324.51
00238:	3	4	0.000	-24.584	-189.30
00239:	4	4	0.000	24.584	189.30
00240:	4	5	0.000	-24.584	-54.09
00241:	5	5	0.000	24.584	54.09
00242:	5	6	0.000	-24.584	81.12
00243:	6	6	0.000	24.584	-81.12
00244:	6	7	0.000	-24.584	216.34
00245:	7	7	0.000	9.833	108.16
00246:	7	8	0.000	-9.833	-54.08
00247:	8	8	0.000	9.833	54.08
00248:	8	9	0.000	-9.833	0.00
00249:	9	9	0.000	9.833	-0.00
00250:	9	10	0.000	-9.833	54.08
00251:	10	10	0.000	9.833	-54.08
00252:	10	11	0.000	-9.833	108.16
00253:	11	11	0.000	24.584	216.34
00254:	11	12	0.000	-24.584	-56.54
00255:	12	12	0.000	24.583	56.54
00256:	12	13	0.000	-24.583	54.08
00257:	13	13	0.000	24.583	-54.08
00258:	13	14	0.000	-24.583	189.29
00259:	14	14	0.000	24.583	-189.29
00260:	14	15	0.000	-24.583	324.50
00261:	15	15	0.000	0.000	0.00
00262:	15	16	0.000	-0.000	-0.00
00263:	16	16	0.000	0.000	0.00
00264:	16	17	0.000	-0.000	-0.00

00265:

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	0.000	-0.002	-0.00
2	0.000	-0.001	-0.00
4	0.000	0.000	0.00
5	0.000	0.000	-0.00
6	0.000	0.000	-0.00
8	0.000	0.000	-0.00
9	0.000	0.000	0.00
10	0.000	-0.000	-0.00
12	0.000	-0.000	-0.00
13	0.000	-0.000	-0.00
14	0.000	-0.000	-0.00
16	0.000	0.000	0.00
17	0.000	-0.000	-0.00

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
3	0.000	24.587	324.50
7	0.000	-14.751	324.50
11	0.000	14.751	324.50
15	0.000	-24.583	324.50

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
1	0.0000	-0.0003	0.0000
2	0.0000	-0.0002	0.0000
4	0.0000	0.0001	0.0000
5	0.0000	0.0001	-0.0000
6	0.0000	0.0000	-0.0000
8	0.0000	0.0000	-0.0000
9	0.0000	-0.0000	-0.0000
10	0.0000	-0.0000	-0.0000
12	0.0000	-0.0000	-0.0000
13	0.0000	-0.0001	-0.0000
14	0.0000	-0.0001	0.0000
16	0.0000	0.0002	0.0000
17	0.0000	0.0003	0.0000

SUPPORT JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
3	0.0000	0.0000	0.0000
7	0.0000	0.0000	0.0000
11	0.0000	0.0000	0.0000
15	0.0000	0.0000	0.0000

00326:
00327:
00328: STRUCTURE MCFCD-ABUTMENT
00329: ==========
00330:
00331:
00332:
00333:
00334: LOADING 3 TRAFFIC LOADING
00335: ==========
00336:
00337:
00338:
00339: MEMBER FORCES
00340:

	MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
00341:					
00342:	1	1	0.000	-0.001	-0.00
00343:	1	2	0.000	0.001	-0.00
00344:	2	2	0.000	-58.582	0.00
00345:	2	3	0.000	58.582	-322.20
00346:	3	3	0.000	64.134	322.20
00347:	3	4	0.000	-64.134	30.53
00348:	4	4	0.000	5.554	-30.53
00349:	4	5	0.000	-5.554	61.08
00350:	5	5	0.000	5.553	-61.08
00351:	5	6	0.000	-5.553	91.62
00352:	6	6	0.000	-53.027	-91.62
00353:	6	7	0.000	53.027	-200.02
00354:	7	7	0.000	51.868	200.02
00355:	7	8	0.000	-51.868	85.25
00356:	8	8	0.000	-6.712	-85.25
00357:	8	9	0.000	6.712	48.33
00358:	9	9	0.000	-6.712	-48.33
00359:	9	10	0.000	6.712	11.41
00360:	10	10	0.000	-6.713	-11.41
00361:	10	11	0.000	6.713	-25.51
00362:	11	11	0.000	1.159	25.51
00363:	11	12	0.000	-1.159	-17.97
00364:	12	12	0.000	1.159	17.97
00365:	12	13	0.000	-1.159	-12.75
00366:	13	13	0.000	1.159	12.75
00367:	13	14	0.000	-1.159	-6.38
00368:	14	14	0.000	1.159	6.38
00369:	14	15	0.000	-1.159	0.00
00370:	15	15	0.000	-0.000	-0.00
00371:	15	16	0.000	0.000	0.00
00372:	16	16	0.000	-0.000	-0.00
00373:	16	17	0.000	0.000	0.00

00374:
00375:

APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	0.000	-0.001	-0.00
2	0.000	-58.581	-0.00
4	0.000	-58.580	-0.00
5	0.000	-0.000	-0.00
6	0.000	-58.580	-0.00
8	0.000	-58.580	-0.00
9	0.000	-0.000	-0.00
10	0.000	-0.000	-0.00
12	0.000	0.000	0.00
13	0.000	0.000	0.00
14	0.000	0.000	0.00
16	0.000	-0.000	-0.00
17	0.000	0.000	0.00

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
3	0.000	122.715	0.00
7	0.000	104.894	-0.00
11	0.000	7.872	-0.00
15	0.000	-1.159	0.00

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
1	0.0000	-0.0002	0.0000
2	0.0000	-0.0001	0.0000
4	0.0000	-0.0000	-0.0000
5	0.0000	-0.0000	-0.0000
6	0.0000	-0.0000	0.0000
8	0.0000	-0.0000	-0.0000
9	0.0000	-0.0000	0.0000
10	0.0000	-0.0000	0.0000
12	0.0000	0.0000	0.0000
13	0.0000	0.0000	-0.0000
14	0.0000	0.0000	-0.0000
16	0.0000	-0.0000	-0.0000
17	0.0000	-0.0000	-0.0000

SUPPORT JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
3	0.0000	0.0000	0.0000
7	0.0000	0.0000	0.0000
11	0.0000	0.0000	0.0000
15	0.0000	0.0000	-0.0000

STRUCTURE MCFCD-ABUTMENT

LOADING 4 TEMPERATURE PLUS SHRINKAGE

MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	1	0.000	-0.015	-0.03
1	2	0.000	0.015	-0.05
2	2	0.000	-0.022	0.01
2	3	0.000	0.022	-0.13
3	3	0.000	147.507	2307.72
3	4	0.000	-147.507	-1496.43
4	4	0.000	147.508	1496.43
4	5	0.000	-147.508	-685.14
5	5	0.000	147.510	685.14
5	6	0.000	-147.510	126.16
6	6	0.000	147.511	-126.16
6	7	0.000	-147.511	937.48
7	7	0.000	-0.001	-144.28
7	8	0.000	0.001	144.27
8	8	0.000	-0.001	-144.27
8	9	0.000	0.001	144.26
9	9	0.000	-0.002	-144.26
9	10	0.000	0.002	144.25
10	10	0.000	-0.002	-144.25
10	11	0.000	0.002	144.24
11	11	0.000	-147.505	-937.44
11	12	0.000	147.505	-21.34
12	12	0.000	-147.502	21.34
12	13	0.000	147.502	-685.10
13	13	0.000	-147.499	685.11
13	14	0.000	147.499	-1496.35
14	14	0.000	-147.498	1496.36
14	15	0.000	147.498	-2307.60
15	15	0.000	-0.000	-0.01
15	16	0.000	0.000	0.00
16	16	0.000	-0.000	-0.01
16	17	0.000	0.000	0.01

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APPLIED JOINT LOADS, FREE JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
1	0.000	-0.015	-0.03
2	0.000	-0.007	-0.04
4	0.000	0.000	0.00
5	0.000	0.002	-0.00
6	0.000	0.001	-0.00
8	0.000	-0.000	-0.00
9	0.000	-0.000	-0.00
10	0.000	-0.001	-0.00
12	0.000	0.003	0.00
13	0.000	0.003	0.01
14	0.000	0.001	0.00
16	0.000	-0.000	-0.00
17	0.000	0.000	0.01

REACTIONS,APPLIED LOADS SUPPORT JOINTS

JOINT	FORCE X	FORCE Y	MOMENT Z
3	0.000	147.529	2307.59
7	0.000	-147.513	793.20
11	0.000	-147.503	-793.20
15	0.000	147.498	-2307.60

FREE JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
1	0.0000	-0.0026	0.0002
2	0.0000	-0.0013	0.0002
4	0.0000	0.0008	0.0001
5	0.0000	0.0007	-0.0001
6	0.0000	0.0003	-0.0001
8	0.0000	-0.0001	-0.0000
9	0.0000	-0.0002	0.0000
10	0.0000	-0.0001	0.0000
12	0.0000	0.0004	0.0001
13	0.0000	0.0007	0.0001
14	0.0000	0.0008	-0.0001
16	0.0000	-0.0013	-0.0002
17	0.0000	-0.0026	-0.0002

SUPPORT JOINT DISPLACEMENTS

JOINT	X-DISPLACEMENT	Y-DISPLACEMENT	ROTATION
3	0.0000	0.0000	0.0002
7	0.0000	0.0000	-0.0000
11	0.0000	0.0000	0.0000
15	0.0000	0.0000	-0.0002

00544:
00545:
00546: STRUCTURE MCFCD-ABUTMENT
00547: ==========
00548:
00549:
00550:
00551:
00552: MEMBER FORCES FOR MEMBER 1
00553: ==========
00554:

	LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
00555:					
00556:	1	1	0.000	-0.005	-0.01
00557:	1	2	0.000	37.130	-102.11
00558:	2	1	0.000	-0.002	-0.00
00559:	2	2	0.000	0.002	-0.01
00560:	3	1	0.000	-0.001	-0.00
00561:	3	2	0.000	0.001	-0.00
00562:	4	1	0.000	-0.015	-0.03
00563:	4	2	0.000	0.015	-0.05

00564:
00565:
00566:
00567: MEMBER FORCES FOR MEMBER 2
00568: ==========
00569:

	LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
00570:					
00571:	1	2	0.000	-207.773	102.10
00572:	1	3	0.000	244.898	-1346.95
00573:	2	2	0.000	-0.003	0.00
00574:	2	3	0.000	0.003	-0.02
00575:	3	2	0.000	-58.582	0.00
00576:	3	3	0.000	58.582	-322.20
00577:	4	2	0.000	-0.022	0.01
00578:	4	3	0.000	0.022	-0.13

00579:
00580:
00581:
00582: MEMBER FORCES FOR MEMBER 3
00583: ==========
00584:

	LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
00585:					
00586:	1	3	0.000	265.420	1346.94
00587:	1	4	0.000	-228.295	10.77
00588:	2	3	0.000	24.584	324.51
00589:	2	4	0.000	-24.584	-189.30
00590:	3	3	0.000	64.134	322.20
00591:	3	4	0.000	-64.134	30.53
00592:	4	3	0.000	147.507	2307.72
00593:	4	4	0.000	-147.507	-1496.43

00594:

MEMBER FORCES FOR MEMBER 4

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	4	0.000	57.655	-10.77
1	5	0.000	-20.530	225.78
2	4	0.000	24.584	189.30
2	5	0.000	-24.584	-54.09
3	4	0.000	5.554	-30.53
3	5	0.000	-5.554	61.08
4	4	0.000	147.508	1496.43
4	5	0.000	-147.508	-685.14

MEMBER FORCES FOR MEMBER 5

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT	
1	5	0.000	20.529	-225.78	— DL
1	6	0.000	16.596	236.59	
2	5	0.000	24.584	54.09	— CAT
2	6	0.000	-24.584	81.12	
3	5	0.000	5.553	-61.08	DT
3	6	0.000	-5.553	91.62	
4	5	0.000	147.510	685.14	— TNS
4	6	0.000	-147.510	126.16	

MEMBER FORCES FOR MEMBER 6

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	6	0.000	-187.236	-236.59
1	7	0.000	224.361	-895.30
2	6	0.000	24.584	-81.12
2	7	0.000	-24.584	216.34
3	6	0.000	-53.027	-91.62
3	7	0.000	53.027	-200.02
4	6	0.000	147.511	-126.16
4	7	0.000	-147.511	937.48

00641:
00642: MEMBER FORCES FOR MEMBER 7
00643: ==========
00644:

00645: LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT	
00646: 1	7	0.000	243.372	895.30	DL
00647: 1	8	0.000	-206.247	341.15	
00648: 2	7	0.000	9.833	108.16	LAT
00649: 2	8	0.000	-9.833	-54.08	
00650: 3	7	0.000	51.868	200.02	DIT
00651: 3	8	0.000	-51.868	85.25	
00652: 4	7	0.000	-0.001	-144.28	TIC
00653: 4	8	0.000	0.001	144.27	

00654:
00655:
00656:
00657: MEMBER FORCES FOR MEMBER 8
00658: ==========
00659:

00660: LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
00661: 1	8	0.000	35.606	-341.15
00662: 1	9	0.000	1.519	434.89
00663: 2	8	0.000	9.833	54.08
00664: 2	9	0.000	-9.833	0.00
00665: 3	8	0.000	-6.712	-85.25
00666: 3	9	0.000	6.712	48.33
00667: 4	8	0.000	-0.001	-144.27
00668: 4	9	0.000	0.001	144.26

00669:
00670:
00671:
00672: MEMBER FORCES FOR MEMBER 9
00673: ==========
00674:

00675: LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
00676: 1	9	0.000	-1.520	-434.89
00677: 1	10	0.000	38.645	324.44
00678: 2	9	0.000	9.833	-0.00
00679: 2	10	0.000	-9.833	54.08
00680: 3	9	0.000	-6.712	-48.33
00681: 3	10	0.000	6.712	11.41
00682: 4	9	0.000	-0.002	-144.26
00683: 4	10	0.000	0.002	144.25

00684:

MEMBER FORCES FOR MEMBER 10

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	10	0.000	-209.285	-324.44
1	11	0.000	246.410	-928.72
2	10	0.000	9.833	-54.08
2	11	0.000	-9.833	108.16
3	10	0.000	-6.713	-11.41
3	11	0.000	6.713	-25.51
4	10	0.000	-0.002	-144.25
4	11	0.000	0.002	144.24

MEMBER FORCES FOR MEMBER 11

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	11	0.000	218.127	928.72
1	12	0.000	-174.252	346.51
2	11	0.000	24.584	216.34
2	12	0.000	-24.584	-56.54
3	11	0.000	1.159	25.51
3	12	0.000	-1.159	-17.97
4	11	0.000	-147.505	-937.44
4	12	0.000	147.505	-21.34

MEMBER FORCES FOR MEMBER 12

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	12	0.000	3.611	-346.51
1	13	0.000	26.764	294.42
2	12	0.000	24.583	56.54
2	13	0.000	-24.583	54.08
3	12	0.000	1.159	17.97
3	13	0.000	-1.159	-12.75
4	12	0.000	-147.502	21.34
4	13	0.000	147.502	-685.10

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MEMBER FORCES FOR MEMBER 13

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	13	0.000	-26.764	-294.42
1	14	0.000	63.889	45.12
2	13	0.000	24.583	-54.08
2	14	0.000	-24.583	189.29
3	13	0.000	1.159	12.75
3	14	0.000	-1.159	-6.38
4	13	0.000	-147.499	685.11
4	14	0.000	147.499	-1496.35

MEMBER FORCES FOR MEMBER 14

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	14	0.000	-234.529	-45.12
1	15	0.000	271.654	-1346.89
2	14	0.000	24.583	-189.29
2	15	0.000	-24.583	324.50
3	14	0.000	1.159	6.38
3	15	0.000	-1.159	0.00
4	14	0.000	-147.498	1496.36
4	15	0.000	147.498	-2307.60

MEMBER FORCES FOR MEMBER 15

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	15	0.000	244.890	1346.89
1	16	0.000	-207.765	-102.09
2	15	0.000	0.000	0.00
2	16	0.000	-0.000	-0.00
3	15	0.000	-0.000	-0.00
3	16	0.000	0.000	0.00
4	15	0.000	-0.000	-0.01
4	16	0.000	0.000	0.00

MEMBER FORCES FOR MEMBER 16

LOADING	JOINT	AXIAL FORCE	SHEAR FORCE	MOMENT
1	16	0.000	37.125	102.09
1	17	0.000	0.000	0.00
2	16	0.000	0.000	0.00
2	17	0.000	-0.000	-0.00
3	16	0.000	-0.000	-0.00
3	17	0.000	0.000	0.00
4	16	0.000	-0.000	-0.01
4	17	0.000	0.000	0.01

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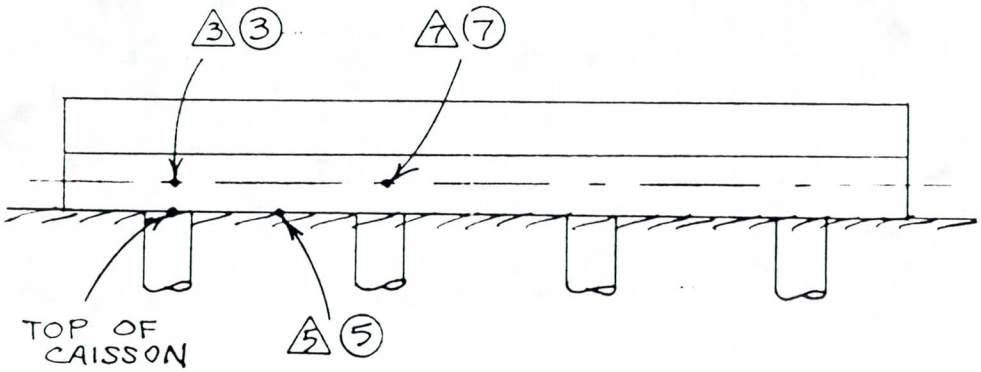

△ MEMBER

○ JOINT

AASHTO SERVICE LOAD GROUP I

ALLOWABLE STRESS INCREASE NONE %

LOADING	β FACTOR	△3 ③ M	△3 ③ β×M	△5 ⑤ M	△5 ⑤ β×M	△7 ⑦ M	△7 ⑦ β×M	TOP OF CAISSON M	TOP OF CAISSON β×M
D	1	-1345.0	-1345.0	+225.8	+225.3	-895.3	-895.3		
L+I	1	-322.2	-322.2	± 61.1	+ 61.1	-200.1	-200.1		
W	0	± 104.2	0	± 17.4	0	± 34.7	0		
WL	0	± 18.9	0	± 3.2	0	± 6.3	0		
S+T	0	± 2307.7	0	± 635.2	0	± 144.3	0		
EQ	0	± 463.3	0	± 78.1	0	± 156.1	0		
TOTAL MOMENT								.325 ×	△3 ③
MOMENT/STRESS INCR.			-1667.2		+286.9		-1095.4		-541.9

W = .321 × 100^k LATERAL LOAD
WL = .058 × " " "
EQ = 1.443 × " " "

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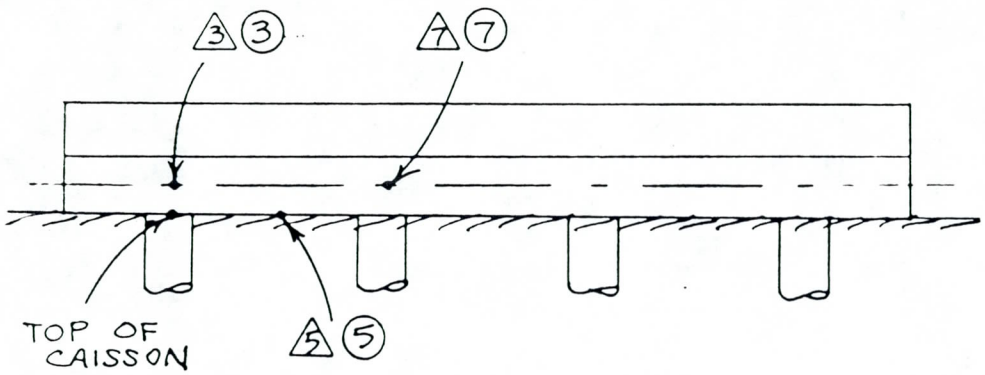


△ MEMBER

○ JOINT

AASHTO SERVICE LOAD GROUP IA

ALLOWABLE STRESS INCREASE 50 %

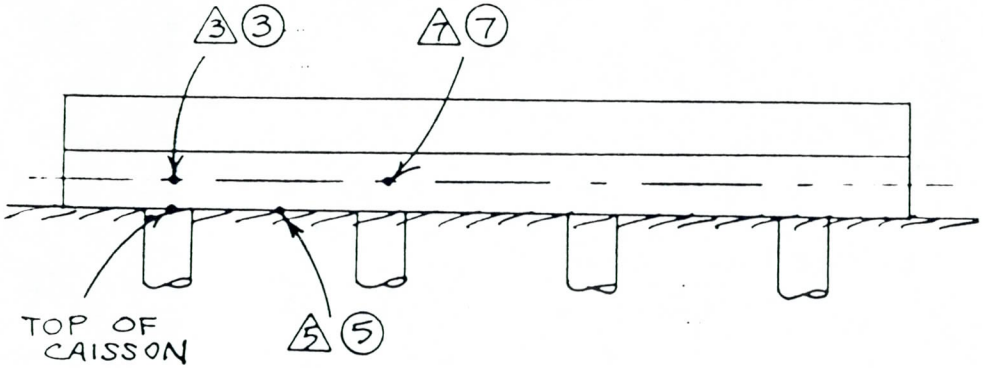
LOADING	β FACTOR	△3 ③ M	△3 ③ β×M	△5 ⑤ M	△5 ⑤ β×M	△7 ⑦ M	△7 ⑦ β×M	TOP OF CAISSON M	TOP OF CAISSON β×M
D	1	-1345.0	-1345.0	+225.8	+225.8	-895.3	-895.3		
L+I	2	-322.2	-644.4	+61.1	+122.2	-200.1	-400.2		
W	0	±104.2	0	±17.4	0	±34.7	0		
WL	0	±18.9	0	±3.2	0	±6.3	0		
S+T	0	±2307.7	0	±685.2	0	±144.3	0		
EQ	0	±468.3	0	±78.1	0	±156.1	0		
TOTAL MOMENT			-1989.4		+348.0		-1295.5	.325 ×	△3 ③
MOMENT/STRESS INCR.			-1326.3		+232.0		-863.7		-431.0

W = .321 × 100^K LATERAL LOAD
WL = .058 × " " "
EQ = 1.442 × " " "

JOB NO. 1040
DESC. 75 TH AVE MCFCD
SHT. A-27 OF 39
BY AGD
DATE SEPT. 26, 1984

HOYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75 TH AVE MCFCD
SHT. A-28 OF 39
BY AGD
DATE SEPT. 26, 1984



△ MEMBER

○ JOINT

AASHTO SERVICE LOAD GROUP II

ALLOWABLE STRESS INCREASE 25 %

LOADING	β FACTOR	△3 ③ M	△3 ③ β×M	△5 ⑤ M	△5 ⑤ β×M	△7 ⑦ M	△7 ⑦ β×M	TOP OF CAISSON M	TOP OF CAISSON β×M
D	1	-1345.0	-1345.0	+225.8	+225.8	-895.3	-895.3		
L+I	0	-322.2	0	+61.1	0	-200.1	0		
W	1	±104.2	±104.2	±17.4	±17.4	±34.7	±34.7		
WL	0	±18.9	0	±3.2	0	±6.3	0		
S+T	0	±2307.7	0	±685.2	0	±144.3	0		
EQ	0	±468.3	0	±78.1	0	±156.1	0		
TOTAL MOMENT			-1240.8 -1449.2		+243.2		-930.0	.325 ×	△3 ③
MOMENT/STRESS INCR.			-1159.4		+194.6		-744.0		-376.8

W = .321 × 100K LATERAL LOAD
WL = .058 × " " "
EQ = 1.443 × " " "

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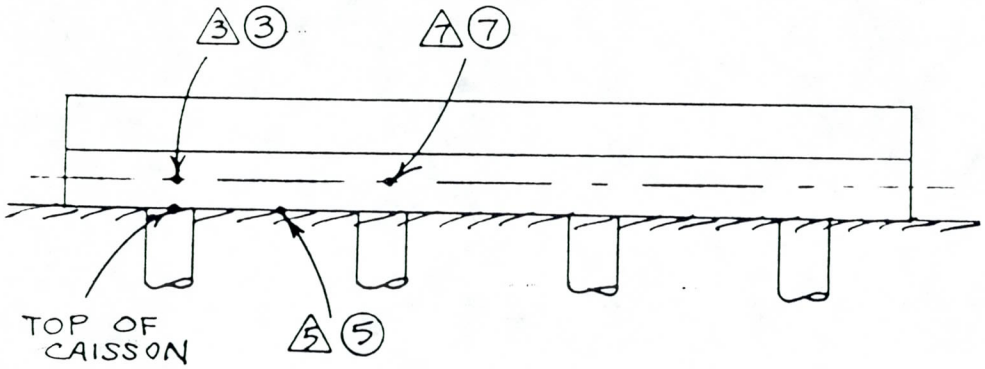

△ MEMBER

○ JOINT

AASHTO SERVICE LOAD GROUP III

ALLOWABLE STRESS INCREASE 25 %

LOADING	β FACTOR	△3 ③ M	△3 ③ β×M	△5 ⑤ M	△5 ⑤ β×M	△7 ⑦ M	△7 ⑦ β×M	TOP OF CAISSON M	TOP OF CAISSON β×M
D	1	-1345.0	-1345.0	+225.8	+225.8	-895.3	-895.3		
L+I	1	-322.2	-322.2	+61.1	+61.1	-200.1	-200.1		
W	.3	±104.2	±31.3	±17.4	±5.22	±34.7	±10.4		
WL	1	±18.9	±18.9	±3.2	±3.2	±6.3	±6.3		
S+T	0	±2307.7	0	±685.2	0	±144.3	0		
EQ	0	±468.3	0	±78.1	0	±156.1	0		
TOTAL MOMENT			-1717.4		+295.3		-1112.1	.325 ×	△3 ③
MOMENT/STRESS INCR.			-1373.9		+236.3		-889.7		-446.5

W = .321 × 100K LATERAL LOAD
WL = .058 × " " "
EQ = 1.443 × " " "

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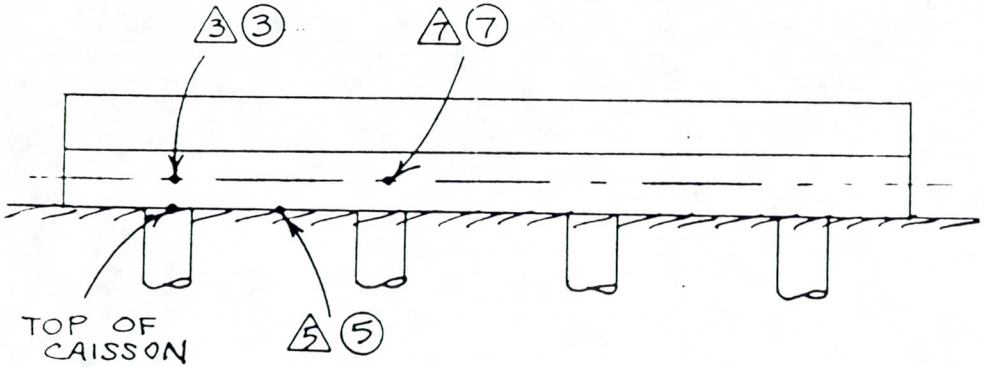

△ MEMBER

○ JOINT

AASHTO SERVICE LOAD GROUP IV

ALLOWABLE STRESS INCREASE 25 %

LOADING	β FACTOR	△3 ③ M	△3 ③ β×M	△5 ⑤ M	△5 ⑤ β×M	△7 ⑦ M	△7 ⑦ β×M	TOP OF CAISSON M	TOP OF CAISSON β×M
D	1	-1345.0	-1345.0	+225.8	+225.8	-895.3	-895.3		
L+I	1	-322.2	-322.2	+61.1	+61.1	-200.1	-200.1		
W	0	±104.2	0	±17.4	0	±34.7	0		
WL	0	±18.9	0	±3.2	0	±6.3	0		
S+T	1	±2307.7	±2307.7	±685.2	±685.2	±144.3	±144.3		
EQ	0	±468.3	0	±78.1	0	±156.1	0		
TOTAL MOMENT			+640.5 -3974.9		+972.1 -398.3		-1239.7 -951.1	.325 ×	△3 ③
MOMENT/STRESS INCR.			+512.4 -3179.9		+777.7 -318.7		-991.8 -760.9		+166.5 -1033.5

W = .321 × 100K LATERAL LOAD
WL = .058 × " " "
EQ = 1.443 × " " "

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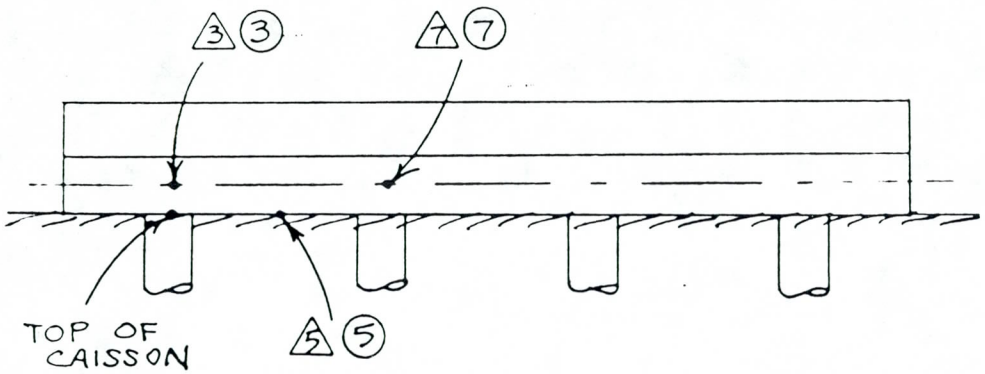

△ MEMBER

○ JOINT

AASHTO SERVICE LOAD GROUP V

ALLOWABLE STRESS INCREASE 40 %

		△3 ③		△5 ⑤		△7 ⑦		TOP OF CAISSON	
LOADING	β FACTOR	M	β×M	M	β×M	M	β×M	M	β×M
D	1	-1345.0	-1345.0	+225.8	+225.8	-895.3	-895.3		
L+I	0	-322.2	0	+61.1	0	-200.1	0		
W	1	±104.2	±104.2	±17.4	±17.4	±34.7	±34.7		
WL	0	±18.9	0	±3.2	0	±6.3	0		
S+T	1	±2307.7	±2307.7	±685.2	±685.2	±144.3	±144.3		
EQ	0	±468.3	0	±78.1	0	±156.1	0		
TOTAL MOMENT			-3756.9 +1066.9		+928.4 -476.8		-1074.3 -716.3	.325 ×	△3 ③
MOMENT/STRESS INCR.			-2683.5 +762.1		+663.2 -340.6		-767.4 -511.7		-872.2 +247.7

W = .321 × 100^k LATERAL LOAD
WL = .058 × " " "
EQ = .1442 × " " "

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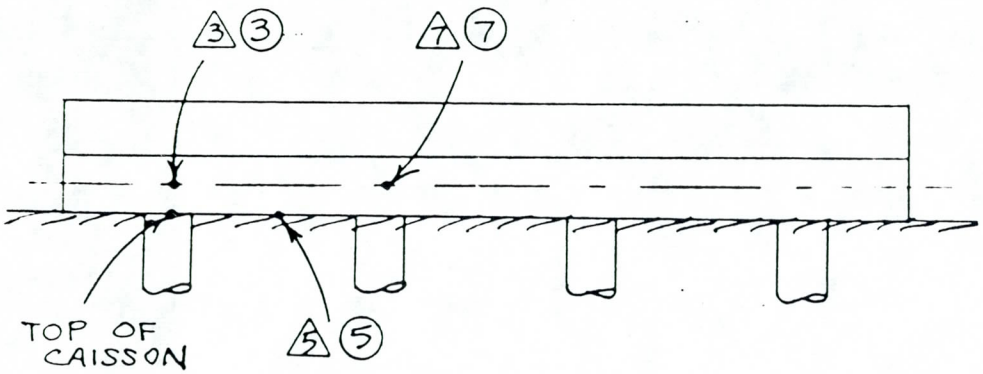


△ MEMBER

○ JOINT

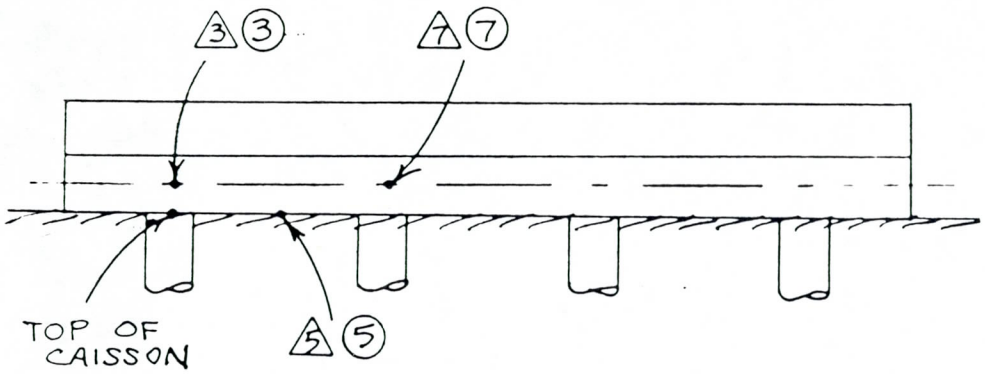
AASHTO SERVICE LOAD GROUP VI

ALLOWABLE STRESS INCREASE 40 %

LOADING	β FACTOR	△3 ③ M	△3 ③ β×M	△5 ⑤ M	△5 ⑤ β×M	△7 ⑦ M	△7 ⑦ β×M	TOP OF CAISSON M	TOP OF CAISSON β×M
D	1	-1345.0	-1345.0	+225.8	+225.8	-895.3	-895.3		
L+I	1	-322.2	-322.2	+61.1	+61.1	-200.1	-200.1		
W	.3	±104.2	±31.1	±17.4	±5.22	±34.7	±10.4		
WL	1	±18.9	±18.9	±3.2	±3.2	±6.3	±6.3		
S+T	1	±2307.7	±2307.7	±685.2	±685.2	±144.3	±144.3		
EQ	0	±468.3	0	±78.1	0	±156.1	0		
TOTAL MOMENT			+690.5 -4024.9		+930.5 -406.7		-1256.4 -934.4	.325 ×	△3 ③
MOMENT/STRESS INCR.			+493.2 -2874.9		+700.4 -290.5		-897.4 -667.4		+160.3 -934.4

W = .321 × 100K LATERAL LOAD
WL = .058 × " " "
EQ = 1.443 × " " "

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541



△ MEMBER

○ JOINT

AASHTO SERVICE LOAD GROUP VII

ALLOWABLE STRESS INCREASE 33 %

LOADING	β FACTOR	△3 ③ M	△3 ③ β×M	△5 ⑤ M	△5 ⑤ β×M	△7 ⑦ M	△7 ⑦ β×M	TOP OF CAISSON M	TOP OF CAISSON β×M
D	1	-1345.0	-1345.0	+225.8	+225.8	-895.3	-895.3		
L+I	0	-322.2	0	+61.1	0	-200.1	0		
W	0	±104.2	0	±17.4	0	±34.7	0		
WL	0	±18.9	0	±3.2	0	±6.3	0		
S+T	0	±2307.7	0	±685.2	0	±144.3	0		
EQ	1	±468.3	±468.3	±78.1	±78.1	±156.1	±156.1		
TOTAL MOMENT			-976.7 -1813.3		+303.9 +147.7		-1051.4	.325 ×	△3 ③
MOMENT/STRESS INCR.			-659.2 -1363.4		+228.5		-790.6		-214.3 -443.1

W = .321 × 100K LATERAL LOAD
= .058 × " " "
EQ = 1.443 × " " "

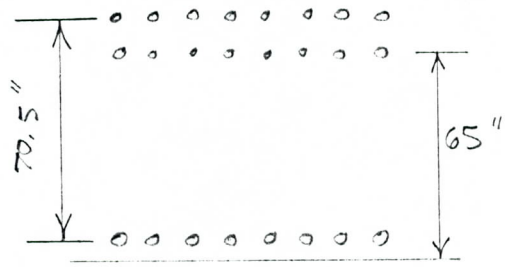
ABUTMENT DESIGN

FLEXURE REIN. @ EXT. CAISSONS

THE MAX. MOMENT OCCURS UNDER LOADING IV WITH TEMP. & SHRINKAGE BEING THE MAJOR CONTRIBUTOR.

TOTAL MOMENT IS 3179.9 K-FT

TRY DOUBLE REIN. BEAM



M_1 CAPACITY:

$\frac{70.5''}{12} \times 8 \times 1.56\ in^2 \times 24\ KSI$

$M_1 = 1759.68$ K-FT

M_2 REQ. = 3179.9 K-FT
− 1759.7
1420.2 K-FT

```
]RUN
WORKING STRESS DESIGN:

FOR F'C=4 KSI AND FY=60 KSI
MAXIMUM ALLOWABLE STRESS:
FC=1.6 KSI AND FS=24.0 KSI

BEAM WIDTH (INCHES):
?72

DEPTH TO REINFORCING (INCHES):
?65

REQUIRED MOMENT CAPACITY (KIP-FT):
?1420.20

AREA OF REINFORCING STEEL:
12.3573518          SQUARE INCHES

PERCENT OF BALANCED STEEL RATIO:
22.7739657

FC (KSI) =        .683243682
FS (KSI) =        24
```

$\frac{12.36}{1.56} = 7.9$

USE 16 # 11 ON TOP
& 8 # 11 BOTTOM

ABUTMENT DESIGN

FLEXURE REIN. BETWEEN CAISSONS

THE MAX. MOMENT OCCURS UNDER LOADING IV WITH TEMP. & SHRINKAGE BEING THE MAJOR CONTRIBUTOR

THE TOTAL MOMENT IS + 777.7 K-FT

THE MAX. NEG. MOMENT IS -340.6 K-FT FROM LOADING V.

THE MOMENTS +777.7 AND -340.6 ARE LESS THAN 1.2 x CRACKING MOMENT OF 2950.5 K-FT (SEE PIER DESIGN)

THEREFORE REIN. FOR BOTH POS. & NEG. MOMENT OF 1.33 x 777.7 K-FT = 1034.34 K-FT

```
]RUN
WORKING STRESS DESIGN:

FOR F'C=4 KSI AND FY=60 KSI
MAXIMUM ALLOWABLE STRESS:
FC=1.6 KSI AND FS=24.0 KSI

BEAM WIDTH (INCHES):
?72

DEPTH TO REINFORCING (INCHES):
?67

REQUIRED MOMENT CAPACITY (KIP-FT):
?1034.34

AREA OF REINFORCING STEEL:
8.73127722          SQUARE INCHES

PERCENT OF BALANCED STEEL RATIO:
15.610959

FC (KSI) =          .5558083
FS (KSI) =          24
```

USE 8-#11 T & B

$$\frac{8.73}{1.56} = 5.60$$

ABUTMENT DESIGN

FLEXURE REIN @ INT. CAISSONS

THE MAXIMUM LOADING OCCURS UNDER LOADING I.

TOTAL MOMENT IS -1095.4 K-FT.

DESIGN MOMENT 1.33 x 1095.4 = 1456.88 K-FT

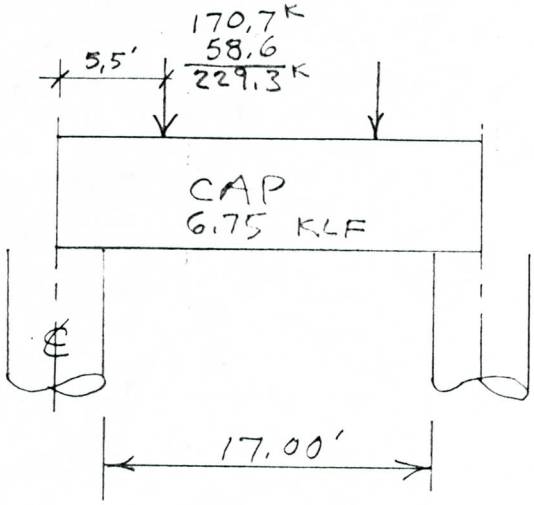
```
]RUN
WORKING STRESS DESIGN:

FOR F'C=4 KSI AND FY=60 KSI
MAXIMUM ALLOWABLE STRESS:
FC=1.6 KSI AND FS=24.0 KSI

BEAM WIDTH (INCHES):
?72

DEPTH TO REINFORCING (INCHES):
?67

REQUIRED MOMENT CAPACITY (KIP-FT):
?1456.88

AREA OF REINFORCING STEEL:
12.2981062          SQUARE INCHES

PERCENT OF BALANCED STEEL RATIO:
21.9882185

FC (KSI) =          .670160144
FS (KSI) =          24
```

$$\frac{12.30}{1.56} = 7.88$$

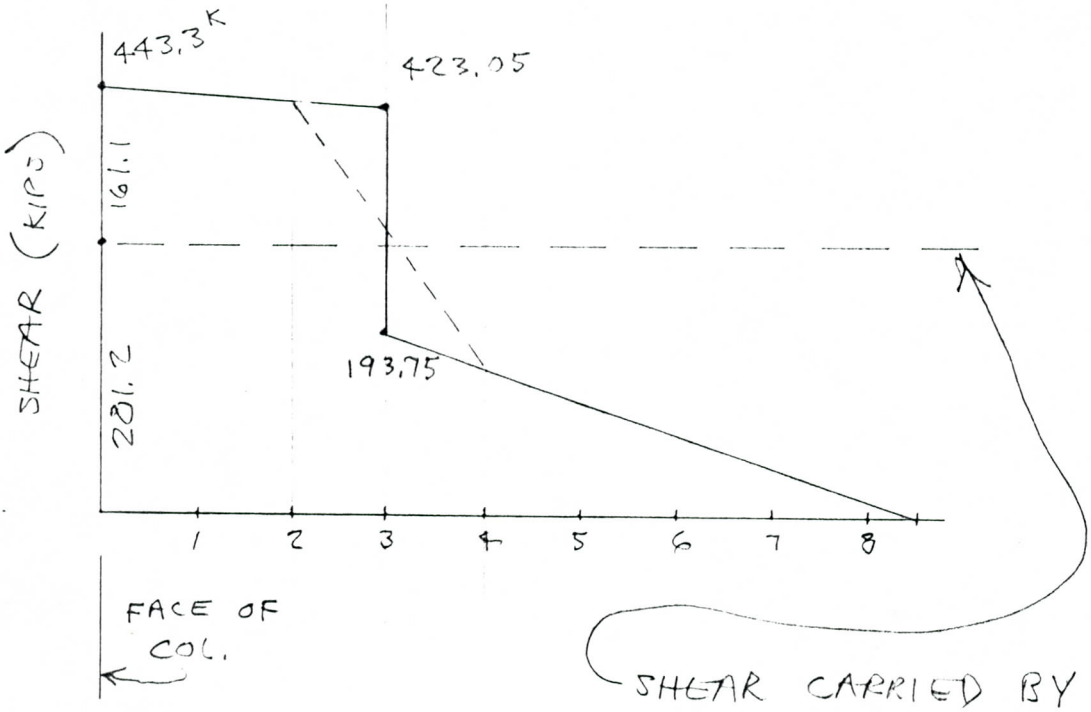
USE 8-#11
T & B

ABUTMENT DESIGN

CAP SHEAR REINFORCING



SUPERSTR. + GIRDER D.L.	=	170.7 K
TRAFFIC L.L.	=	58.6
ABUT. CAP D.L.	=	57.4
TEMP. + SHR.	=	147.5
WIND LOADS	=	9.1
		443.3 K



SHEAR CARRIED BY CONCRETE

$v_c = .95\sqrt{4000}$

$V_c = .95\sqrt{4000} \times 72 \times 65 = 281.2^K$

ABUTMENT DESIGN

CAP SHEAR REINFORCING

$$v - v_c = \frac{161.1^K}{72 \times 65} = 0.034 \text{ KSI}$$

$$A_v = \frac{(v - v_c)\, b_w\, s}{f_s}$$

FOR $s = 18''$

$$A_v = \frac{(0.034)\,72\,(12)}{24} = 1.224 \text{ IN}^2$$

FOR #6 BARS

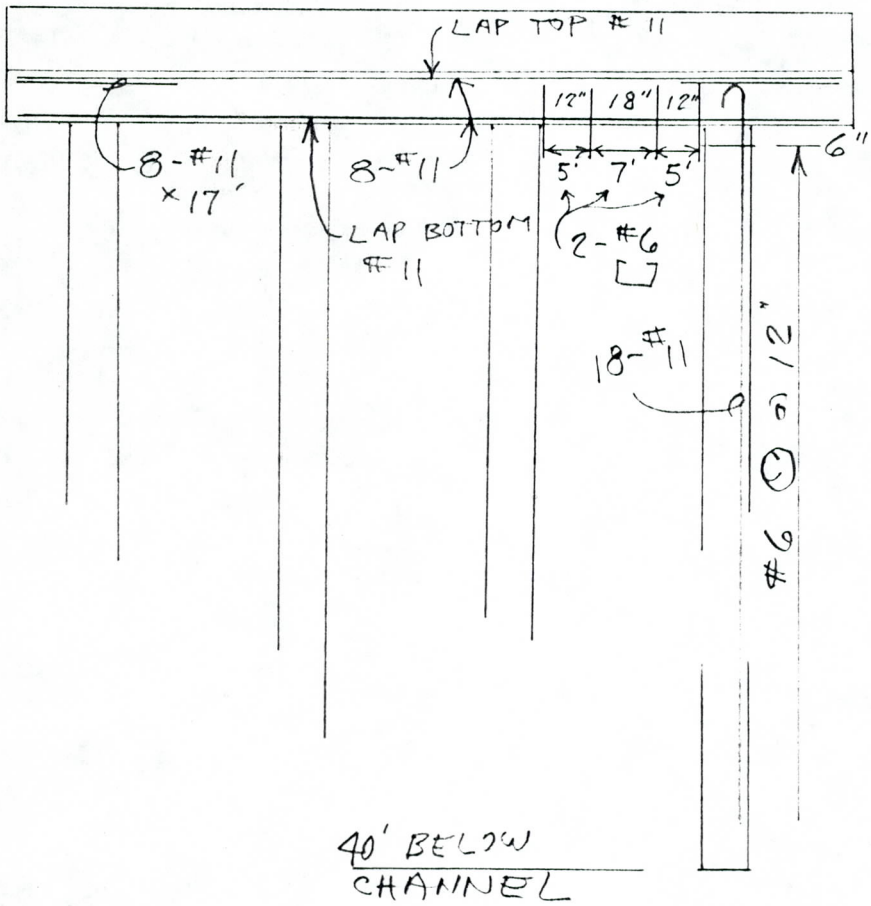
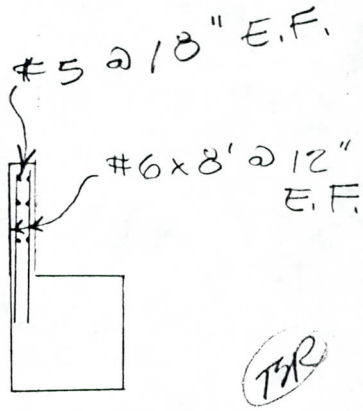
$$\frac{1.224}{.44} = 2.78$$

USE 1 SET OF 2-#6 [ ] @ 12"
UP TO 5' FROM FACE OF COL.
1 SET OF 2-#6 [ ] @ 18" ELSE WHERE

ROYDEN ENGINEERING COMPANY
PHOENIX, ARIZONA PH. 279-3541

JOB NO. 1040
DESC. 75TH AVE MCFCD

SHT. A-39 OF 39
BY AGD
DATE SEPT 27, 1984

ABUTMENT DESIGN

REIN. SUMMARY