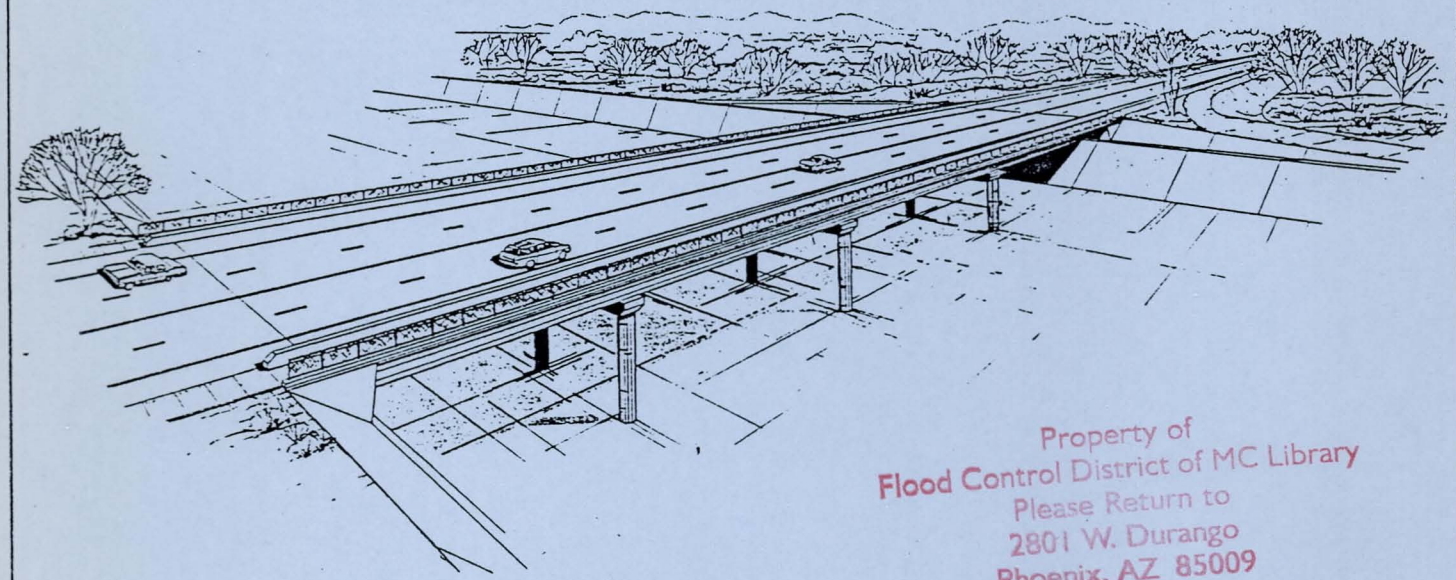
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

STRUCTURAL CALCULATIONS



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NORTHERN AVE. BRIDGE OVER THE NEW RIVER

CONTRACT NO. 89-04
14 FEBRUARY 1990
90% SUBMITTAL

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY 1959

BRW

Northern Avenue Bridge Over
The New Rives
-Structural Calculations-
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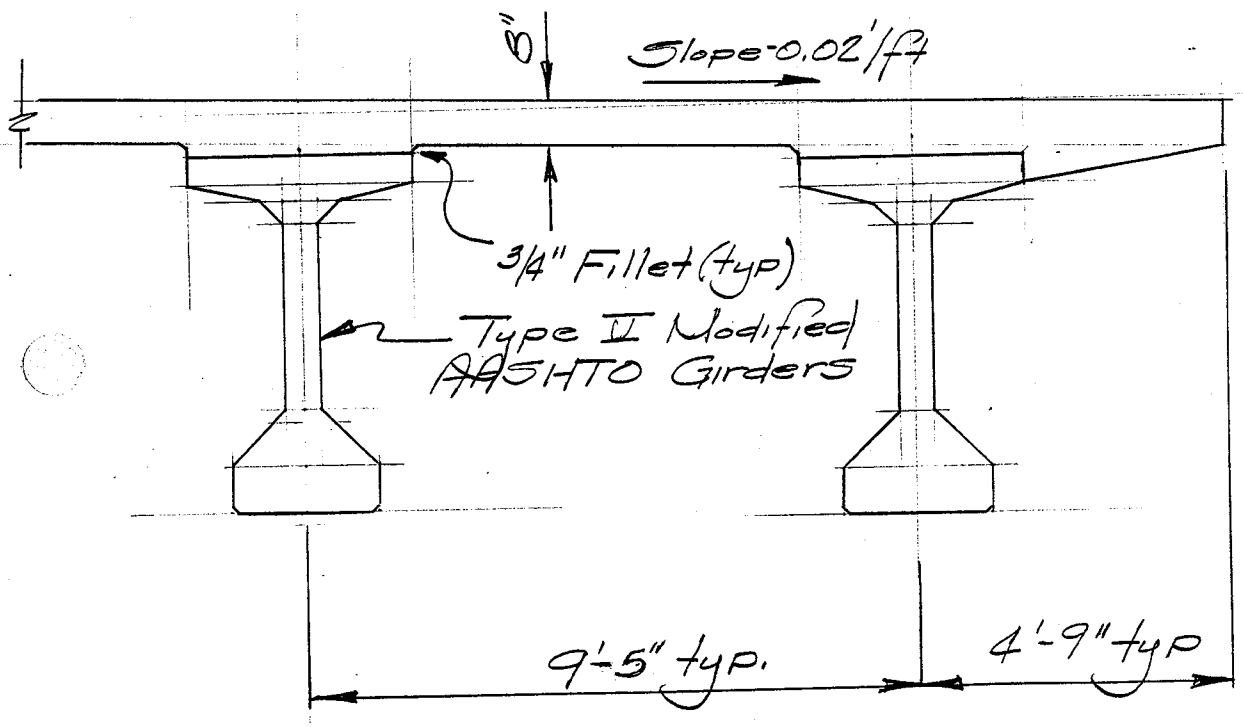
JOB _____ SHEET NO. _____
MADE BY _____ DATE _____ JOB NO. _____
CHECKED BY _____ DATE _____ SEC. SHT. NO. _____
CALCULATIONS FOR _____

Northern Avenue Bridge
Over the New River
Superstructure Design

Deck Slab

The protective system provided will be 2½ inches of cover for top bars of which the top ½ inch will be an integral wearing surface not included in computing section properties of either the slab or the composite effect with the girders.



Effective Slab Span = (9'-5") - (3'-4") + (1'-5") = 7'-6" (= S

(Ref ADOT "Procedures for Development of Structure Plans by Consultant") Section 200.5.4.E

$M = \frac{1}{10} WS^2$ (Ft-Kips)

W includes 25#/ft³ for future Wearing Surface

W = 0.67(150) + 25 = 125#/Lf

JOB Northern Avenue Brdg SHEET NO. 5-1
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 1
CALCULATIONS FOR Deck Slab Design

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Northern Avenue Bridge Over the New River

Deck Slab Design (Cont'd)

$$DM = 0.125\left(\frac{7.50}{10}\right)^2 = 0.703 \frac{^{1K}}{ft}$$

$$LM_{+I} = \frac{P(S+2)}{40} \times 1.30 \quad \text{where } P = 16^{kips}$$

$$LM_{+I} = 16\left(\frac{7.50+2}{40}\right) \times 1.30 = 4.94 \frac{^{1K}}{ft}$$

Assume #5 bars will be used both transverse and as longitudinal distribution steel. It is also assumed that #6 will be used as longitudinal continuity bars for negative moment.

$$DM + LM_{+I} = 0.703 + 4.94 = 5.64 \frac{^{1K}}{ft}$$

Slab Design Allowables $f'_c = 4{,}000$ psi, $f_c = 1600$ psi, $n = 8$

$$K = \frac{f_c}{\frac{f_s}{n} + f_c} = \frac{1600}{\frac{24{,}000}{8} + 1600} = 0.348$$

$$j = 1 - \frac{K}{3} = 1 - \frac{0.348}{3} = 0.884$$

$$\rho = \frac{f_c}{2f_s} \times K = \frac{1600}{2(24{,}000)}(0.348) = 0.0116$$

$$R_{allow} = 1/2\, jKf_c = 1/2(.884)(.348)1600 = 246 \text{ psi}$$

$$\text{Required } bd^2 = \frac{M_{DL+LL}}{R} = \frac{5.64\,(12{,}000)}{246} = 275$$

$$d = \left(\frac{275}{12}\right)^{1/2} = 4.78''$$

$$\text{Minimum Slab Depth} = 4.78 + 2.5 + (.5)(5/8) = 7.60 \therefore 8'' \text{ is OK}$$

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JOB Northern Avenue Bridge SHEET NO. S-2
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T. Stones DATE 2/90 SEC. SHT. NO. 2
CALCULATIONS FOR Deck Slab Design

Northern Avenue Bridge Over the New River

Deck Slab Design (Cont'd)

$d_{top} = 8 - 2\frac{1}{2} - \frac{1}{2}(\frac{5}{8}) = 5.18''$

$d_{bot} = 8 - 1 - \frac{1}{2}(5/8) = 6.68''$

Required Steel Area

$$A_s = \frac{M_{DL+LL+I}}{f_s j d} = \frac{5.64(12)}{24(.884)5.18} = 0.62 \frac{\square''}{ft}$$

for top use #5 @ 12" straight bars with a #5 truss @ 12" in between

$A_{s\,prov} = 0.62 \frac{\square''}{ft} \geq 0.62 \frac{\square''}{ft}$ ∴ OK

Bottom Steel

$$A_s = \frac{5.64(12)}{24(.884)6.68} = 0.48 \frac{\square''}{ft}$$

for bottom use #4 @ 12" straight bars with the #5 truss @ 12" in between

$A_{prov} = 0.51 \frac{\square''}{ft} > 0.48 \frac{\square''}{ft}$ ∴ OK

Distribution Reinforcement

$$\frac{220}{\sqrt{S}} = \frac{220}{(7.50)^{1/2}} = 80\% > 67\%$$

∴ Use 67% max

$0.67(.48) = 0.32 \frac{\square''}{ft}$ Use #5 @ 10" oc in the middle half & #5 @ 1'-4" o.c in the outer quarter

Use #5 @ 12" centers longitudinal in the top of the top ~~slab~~, which is an ADOT minimum

The deck overhang is not critical for design, by inspection

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JOB Northern Avenue Bridge SHEET NO. 5-3
MADE BY Lee Arnold DATE JOB NO. 33-8962
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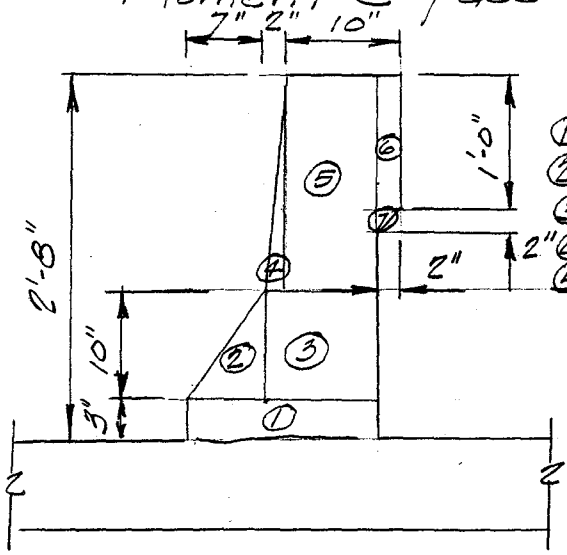
CALCULATIONS FOR Deck Slab Design

Northern Avenue Bridge Over the New River
Slab Design (Cont'd)

✓ Moment @ face of Rail



Rail Weight	about face of Curb	cg X Wt
① 17 x 3 x $\frac{150}{144}$ = 53.12	8.5	451.6
② 5 x 7 x " = 36.46	4.67	170.3
③ 10 x 10 x " = 104.17	12.0	1250.0
④ 1 x 19 x " = 19.00	8.33	158.3
⑤ 19 x 8 x " = 158.33	13.00	2058.3
372		4089

c.g. of Rail from face of Rail $= \frac{4089}{372} = 11''$

$\mathbb{L}M$ @ face of Rail $= 10^k \times \frac{32}{12} \times \left[\frac{1}{5+(0.8)\left(\frac{11}{12}\right)}\right] = 4.65 \frac{k}{ft}$

this is slightly less than the $\mathbb{L}M$ used in design, ∴ reinforcement is OK

Longitudinal Slab Reinforcement

The girders will be designed as simply supported. The superstructure will be effectively pinned at intermediate supports, but continuous for live load and superimposed DL (Barrier, FWS & Utilities). Longitudinal deck reinforcement will be added to resist these loads

Distribution of Wheel loads to longitudinal beams $= \frac{S}{5.5}$ (Table 3.23.1) $= \frac{9.42}{5.5} = 1.712$ Wheel Lines $= 0.856$ lanes

FWS $= 25^{lb}/ft^2 \times 9.42' = 236$ lbs/Lf

Utilities $= 6.25^{lb}/ft^2 \times 9.42 = 59$ lbs/ft

Total Added DL $= 295$ lbs/ft Plus Barrier

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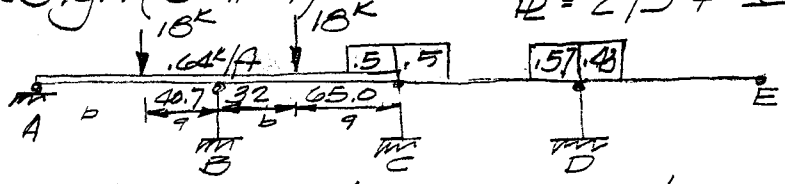
JOB Northern Avenue Bridge SHEET NO. S-4
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 4
CALCULATIONS FOR Deck Slab Design

Northern Avenue Bridge Over the New River

Slab Design (Cont'd)

$$\text{Barriers} \quad \phi = 295 + \frac{2(372)}{6\text{ girs}} = 419 \ \frac{lb}{Lf}$$



Maximum Negative Moment @ B / per lane

Span 1

FEM

$$\text{Uniform} = \frac{Wl^2}{8} = 0.64\frac{(97)^2}{8} = 753^{'K}$$

$$\text{Concentrated} = \frac{P}{L^2}\left[b^2a + \frac{a^2b}{2}\right] = \frac{18}{(97)^2}\left[(56.26)^2(40.74) + \frac{(40.74)^2(56.26)}{2}\right] = 336^{'K}$$

Span 2

FEM

$$\text{Uniform} = \frac{Wl^2}{12} = 0.64\frac{(97)^2}{12} = 502^{'K}$$

$$\text{Concentrated} \quad M_B = \frac{Pa^2b}{L^2} = 18\frac{(65.0)^2(32.0)}{(97)^2} = 259^{'K}$$

$$M_C = \frac{Pb^2a}{L^2} = 18\frac{(32.0)^2(65.0)}{(97)^2} = 127^{'K}$$

.43	.57	.5	.5	.57	.43
-1089	+761	-629			
+141	+187	+314	+315		
	+157	+94		+157	
-67	-90	-47	-47	-90	-67
	-23	-45	-45		
+10	+13	+45	+45		
	+23				
-10	-13				
1015	(*1066)				

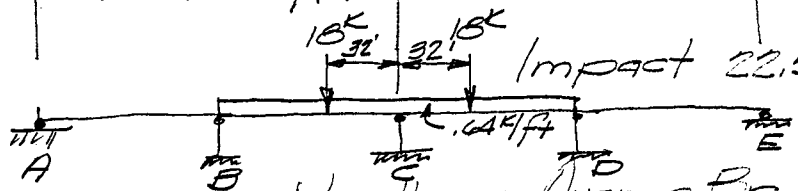
Impact 22.5%

* Multiply all LLM by # of Lanes plus Impact = 0.856 x 1.225 = 1.05

Maximum Negative Moment @ C / per lane

.43	.57	.5	.5	.57	.43
	629	-761	761	-629	
	-359			+359	
		-180	+180		
(*988)		-941	941	(*988)	

Impact 22.5%



JOB Northern Avenue Br SHEET NO. S-5

MADE BY Lee Arnold DATE JOB NO. 93-8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 5

CALCULATIONS FOR Longitudinal Deck Reinforcement

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Northern Avenue Bridge Over the New River

Loading Spans 1 & 2, find reaction at Abut 1

$\leftarrow \Sigma M_B = -1015 \rightarrow$

$18(40.7) = 732.6$

$.64\left(\frac{97}{2}\right)^2 = 3010.9$

$R_A(97)$

$R_A = \frac{3010.9 + 732.6 - 1015}{97} = 28.13^k$

Calculate Moment @ 9/10 Pt of Span 1

$\leftarrow \Sigma M_{9/10\ S\text{-}1} \rightarrow$

$18(31.0) = 558$

$.64\left(\frac{87.3}{2}\right)^2 = 2438.8$

$28.13(87.3) = 2455.6$

$M_{9/10} = 541^{'k}$ (*568)

Find reaction at Pier C (Loading Span 1 & 2)

$\leftarrow \Sigma M_B = +1015 \rightarrow$

$R_C(97)$

$18(32) = 576$

$.64\left(\frac{97}{2}\right)^2 = 3010.9$

$R_C = \frac{3010.9 + 576 - 1015}{97} = 26.51^k$

Calculate Moment @ 0.10 Pt of Span 2

$\leftarrow \Sigma M_{1/10\ S\text{-}2} \rightarrow$

$26.51(87.3) = 2314.3$

$18(22.3) = 401.4$

$0.64\left(\frac{87.3}{2}\right)^2 = 2438.8$

$M_{1/10\ S\text{-}2} = 526^{'k}$ (*552)

Find Reaction at Pier B Loading Spans 2 & 3

$\leftarrow \Sigma M_C = -941 \rightarrow$

$18(32) = 576$

$0.64(97) = 3010.9$

$R_B(97)$

Similar $R_B = 27.28$

Calculate Moment @ 9/10 Pt of Span 2

$\leftarrow \Sigma M_{9/10\ S\text{-}2} \rightarrow$

401.4

2438.8

$27.28(87.3)$

$M_{9/10\ S\text{-}2} = +459$ (*483)

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JOB Northern Avenue Bridge SHEET NO. S-6

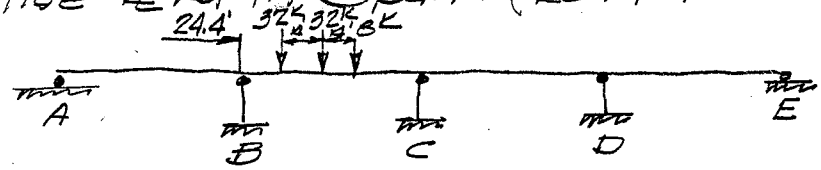
MADE BY Lee Arnold DATE _____ JOB NO. 33-8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 6

CALCULATIONS FOR Longitudinal Deck Reinforcement

Northern Avenue Bridge Over the New River
Slab Longitudinal Reinforcement

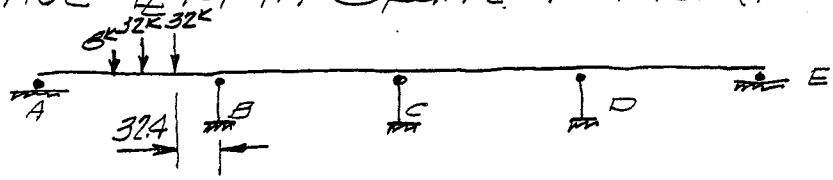
Negative ℄M in Span 1 (Load Truck in Span 2)



$FEM_{BC} = +974^{'K}$ $FEM_{CB} = -544^{'K}$

.43	.57	.5	.5	.57	.43
	+974	-544			
-418	-556	+272	+272		
	+136	-278		+136	
-58	-78	+139	+139	-78	-58
	+70		-39	+70	
-30	-40	+20	+19		
	+10				
-4	-6				
(535) -510					

Negative ℄M in Span 2 1st Pier (Truck in Span 1)



$FEM_{BA} = 1290^{'K}$

.43	.57	.5	.5	.57	.43
-1290					
+553	+737				
		+368			
		-184	-184		
	-92			-92	
+39	+53			+53	
		+26	+26		
		-26	-26		
	-13				
+6	+7				
-692	+692'K (727)				

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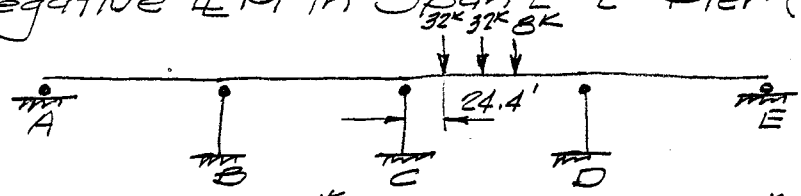
MINNEAPOLIS PHOENIX DENVER

JOB Northern Avenue Bridge SHEET NO. 9-7
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 7
CALCULATIONS FOR Longitudinal Deck Steel

Northern Avenue Bridge Over the New River
Slab Longitudinal Reinforcement

Negative ℄M in Span 2 2nd Pier (Truck in Span 3)



$FEM_{CD} = 974^{'K}$ $FEM_{DC} = -544^{'K}$

.43	.57	.5	.5	.57	.43
			+974	-544	
		-487	-487	+311	
	-243		+156		
	+139	-78	-78		
	-39	+70		-39	
	+22	-35	-35	+22	
		+11	+11		
		-11	-11		
	(556'K)	-530			

Superimposed Φ = FWS + Utilities + Barrier

$= 236^{lb} + 59^{lb} + \frac{2(372)}{6\,Girders} = 419 \frac{lb}{Lf}$ per girder

Dead Load Moments

@ B $= -0.1071 \times WL^2 = -0.1071(.419)(97)^2 = -423^{'K}$

@ C $= -0.0714 \times WL^2 = -0.0714(.419)(97)^2 = -282^{'K}$

@ 38.1' from Abut ℄ in Span 1

$M_{pos} = 0.0772 \times WL^2 = 0.0772(.419)(97)^2 = +304^{'K}$

@ 52.0' from Pier 2 in Span 2

$M_{pos} = 0.0364 \times WL^2 = 0.0364(.419)(97)^2 = +144^{'K}$

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JOB Northern Avenue Bridge SHEET NO. 5-8
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 8
CALCULATIONS FOR Longitudinal Deck Steel

Northern Avenue Bridge Over the New River
Slab Longitudinal Reinforcement
Superimposed D + L_{LI} Moment Diagram
Per Girder

There are 9 Longitudinal #5 in top mat & 8 in bottom mat

$d \approx 63 + 1 + \frac{8-3.5}{2} = 66''$

Resisting Neg Moment

$M = A_s f_s j d$

$= 17(.31)24(.884)\frac{66}{12}$

614^{k}

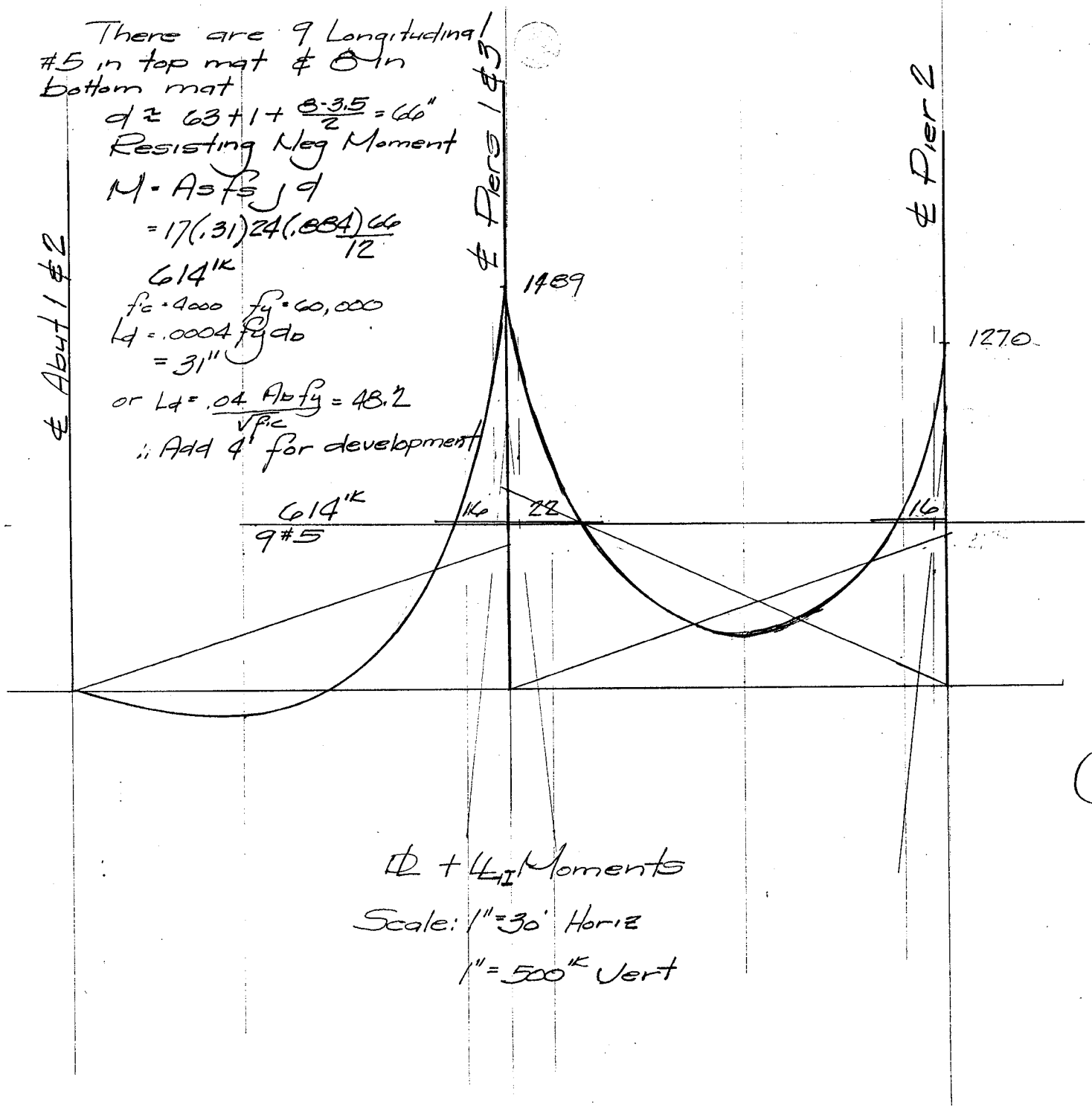
$f'_c = 4000 \quad f_y = 60{,}000$

$L_d = .0004 f_y d_b$

$= 31''$

or $L_d = \frac{.04 A_b f_y}{\sqrt{f'_c}} = 48.2$

∴ Add 4' for development



D + L_{LI} Moments

Scale: 1" = 30' Horiz

1" = 500^{k} Vert

JOB Northern Avenue Bridge SHEET NO. S-9

MADE BY Lee Arnold DATE JOB NO. 33-8962

CHECKED BY Tsbnes DATE 2/90 SEC. SHT. NO. 2

CALCULATIONS FOR Longitudinal Deck Steel

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Northern Avenue Bridge Over the New River
Slab Longitudinal Reinforcement

Settlement is projected to be less than .89" occuring during application of dead load. Differential settlement is estimated to be 1/2 of total settlement.

Consider
1. Abut 1 settling & Pier 2 Settling
2. Pier 1 & Pier 3 Settling

$E = 4{,}000$ Ksi

Calculate I for 1 Line of Girders

$I_{girder} = 479{,}459 \text{ in}^4$ $A_{girder} = 887 \text{ in}^2$ $y_b = 32.02$ in

I of Composite Section

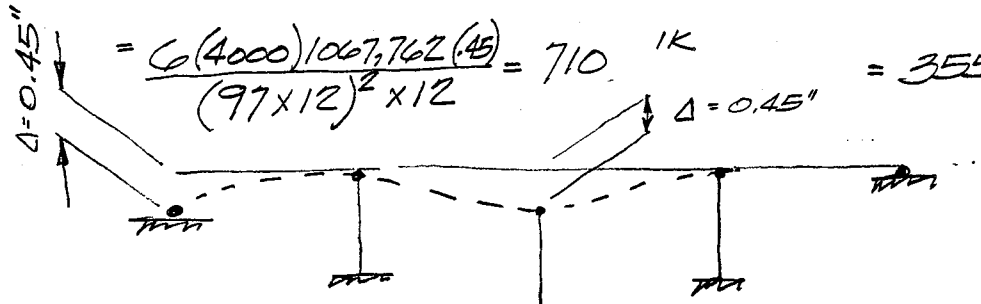
	A	y	Ay	Ay^2	I_{oo}
Slab	864	63.25	58,968	4,074,566	4,608
Filler	50	63.625	3181	202,407	6
Girder	887 / 1801	32.02	28,402 / 90,551	909,423 / 5,136,396	479,459 / 484,073

$y_b = 50.278$

$I_{xx} = 484{,}073 + 5{,}136{,}396 - 1801(50.278)^2$
$= 1{,}067{,}762 \text{ in}^4$

$FEM = \frac{6EI\Delta}{L^2}$ (Ends Fixed) $FEM = \frac{3EI\Delta}{L^2}$ (one end hinged)

$= \frac{6(4000)1067{,}762(.45)}{(97 \times 12)^2 \times 12} = 710$ 'K $= 355$ 'K



Settlement Case 1

JOB Northern Avenue Bridge SHEET NO. 5-10
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 10
CALCULATIONS FOR Longitudinal Deck Steel

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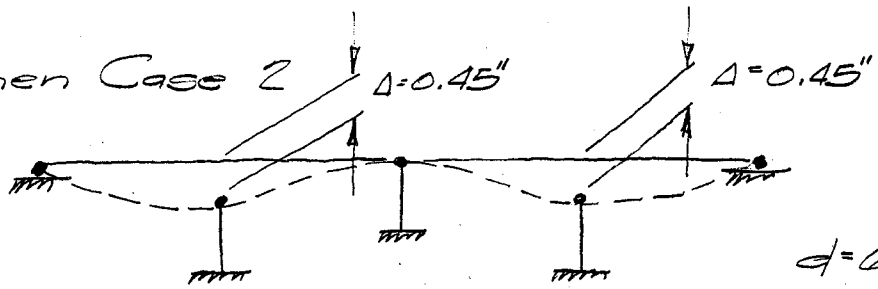
Northern Avenue Bridge Over the New River
Slab Longitudinal Reinforcement
Settlement Case 1 (Cont'd)

.43	.57	.50	.50	.57	.43
-355	+710	+710	-710	-710	
-153	-205			+404	+306
		-101	+202		
		-50	-50		
	-25			-25	
+11	+14			+14	+11
		+7	+7		
		-7	-7		
	-4			-4	
+2	+2			+2	+2
-495	+495	+558			

This case produces the maximum negative moment at B

Settlemen Case 2



excess girder camber

Filler

$d = 63 + 1.25 + 1.19 + 8 - 2.5 - \frac{5}{8} - \frac{1}{2}(1.27)$

$= 69.68''$

.43	.57	.50	.50	.57	.43
+355	-710	-710	+710	+710	-355
+153	+202			-255	-169
		+112	-112		
		-609	+609		

Maximum Moments due to Superimposed DL plus LL+I and settlement at each pier

Pier 1 & 3 $M = -1489 - 495 = -1984^{'k}/Gdr$

Pier 2 $M = -1270 - 609 = -1879^{'k}/Gdr$

This can be reduced to the moment at face of the diaphragm, but the diaphragm is 2'-0" wide and will reduce the moment only slightly. Design for maximum moment

$$A_s = \frac{M}{f_s j d} = \frac{1984(12)}{24(.884)69.68} = 16.10 \frac{\square''}{girder}$$

Use 10 #11 or #11 @ 12" oc

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MINNEAPOLIS PHOENIX DENVER

JOB Northern Ave Bridge SHEET NO. S-11
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 11
CALCULATIONS FOR Longitudinal Deck Steel

Verify Geometrics:

New River Channel: from Coe & Van Loo Plans of New River Channelization.

Channel width between toe of slope (not bottom of soil cement) is 325 ft.

Channel bottom slope -0.002 ft/ft downstream south

Elevations

sta. 167+80	Lt TOB 1077.70	℄ 1060.00	Rt TOB 1077.7
160+25	1076.16	1058.46	1076.16
160+00		1058.41	1076.86
159+75	1076.06	1058.36	
159+45	1075.30	1058.30	
159+00	1075.20	1058.22	1076.7
148+00	1073.03	1056.03	1074.53

Channel Bearing N 0° 11'02" E

Northern Ave. Bearing N 12° 23' 52" W

Ties: ℄ intersection w/ ⅊ Rt (East) 848.15 ft to NE Cnr 5 T2N R1E

Lt (West) 1777.53 ft to N ¼ 5

NE Cnr 5 N 23,058.1166 E 15,385.4924

N ¼ Cnr 5 N 23,008.3339 E 12,760.2844

Intersection ℄ New River @ ⅊ Northern sta 159+85.42

Elev.'s. Lt TOB 1076.081 ℄ 1058.381 Rt TOB 1076.837

BRW

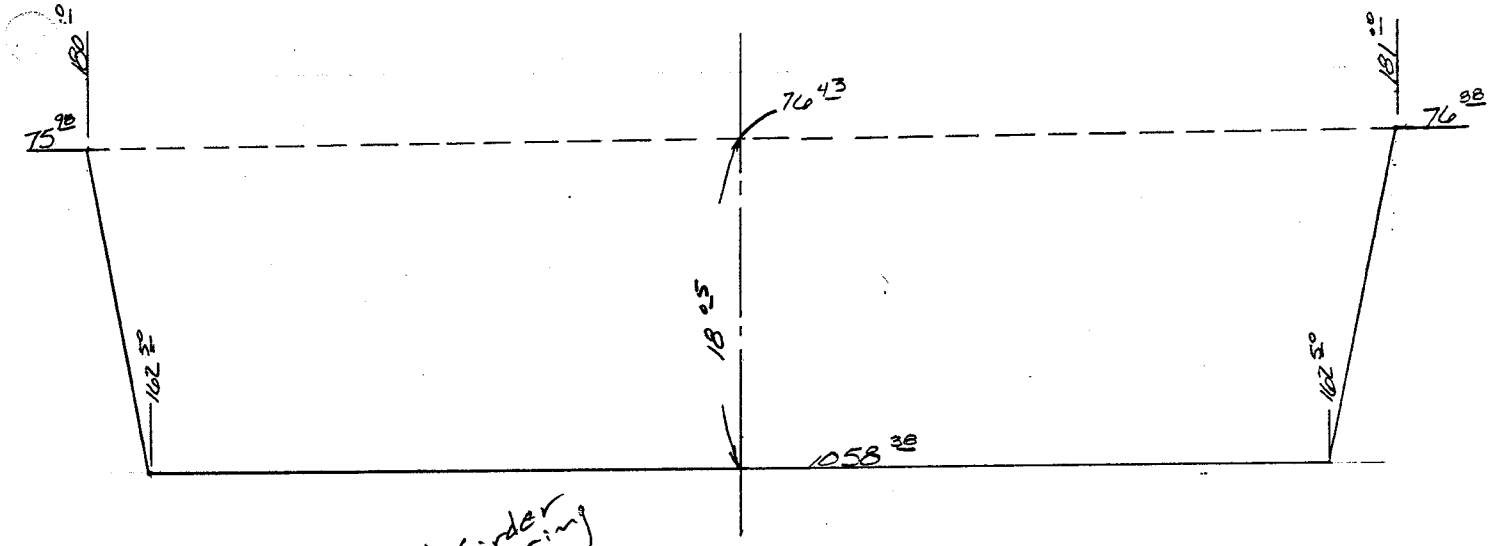
JOB Northern Ave. @ New River SHEET NO. G 1

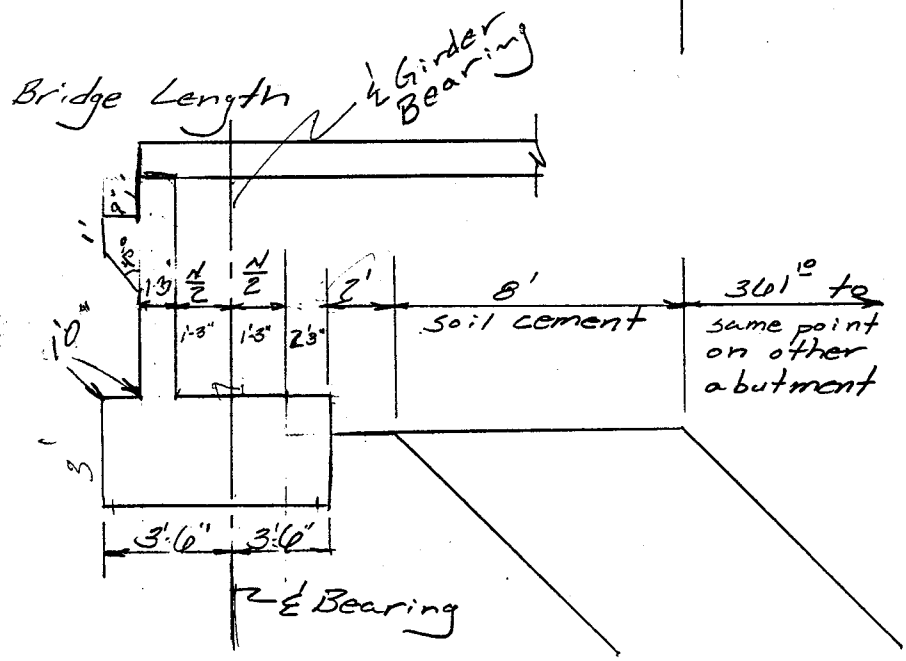
MADE BY T. Stones DATE 9-5-89 JOB NO. 33-8962

CHECKED BY Lee Arnold DATE 10/89 SEC. SHT. NO. 12

CALCULATIONS FOR Layout

From Typical X-sections the following ℄ X-section is interpolated





From California DOT Memo to Designers 5-1 January 1988

$$N = (12 + 0.03L + 0.12H)\left(1 + \frac{S^2}{8000}\right)$$

N = support width in inches; $N_{min} = 2'6"$
L = Length of bridge deck from abutment to adjacent expansion jt.
H = Average Height of columns
S = Skew of Abutment in degrees

BRW

JOB Northern Ave. @ New River SHEET NO. G 2
MADE BY T. Stones DATE 9-5-89 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 10/89 SEC. SHT. NO. 13
CALCULATIONS FOR Layout

Bridge Length (cont.)

Assume $N = 2'\text{-}6''$ $\quad L = 361^{10} + 2\left(8 + 2 + \frac{N}{2} + 3^{25}\right) = 390^{10}$ ft

$S = 0°$ $\quad H \cong 18$ ft

$N = \left[12 + 0.03(390^{10}) + 0.12(18)\right] = 25^{88} < 2'\text{-}6''$ use $N_{min} = 2'\text{-}6''$

℄ Brg Abut to ℄ Brg Abut $= 361^{10} + 2(8 + 2 + 3^{25}) = 387^{60}$ ft

make an even 388 ft.

Hinge @ Piers

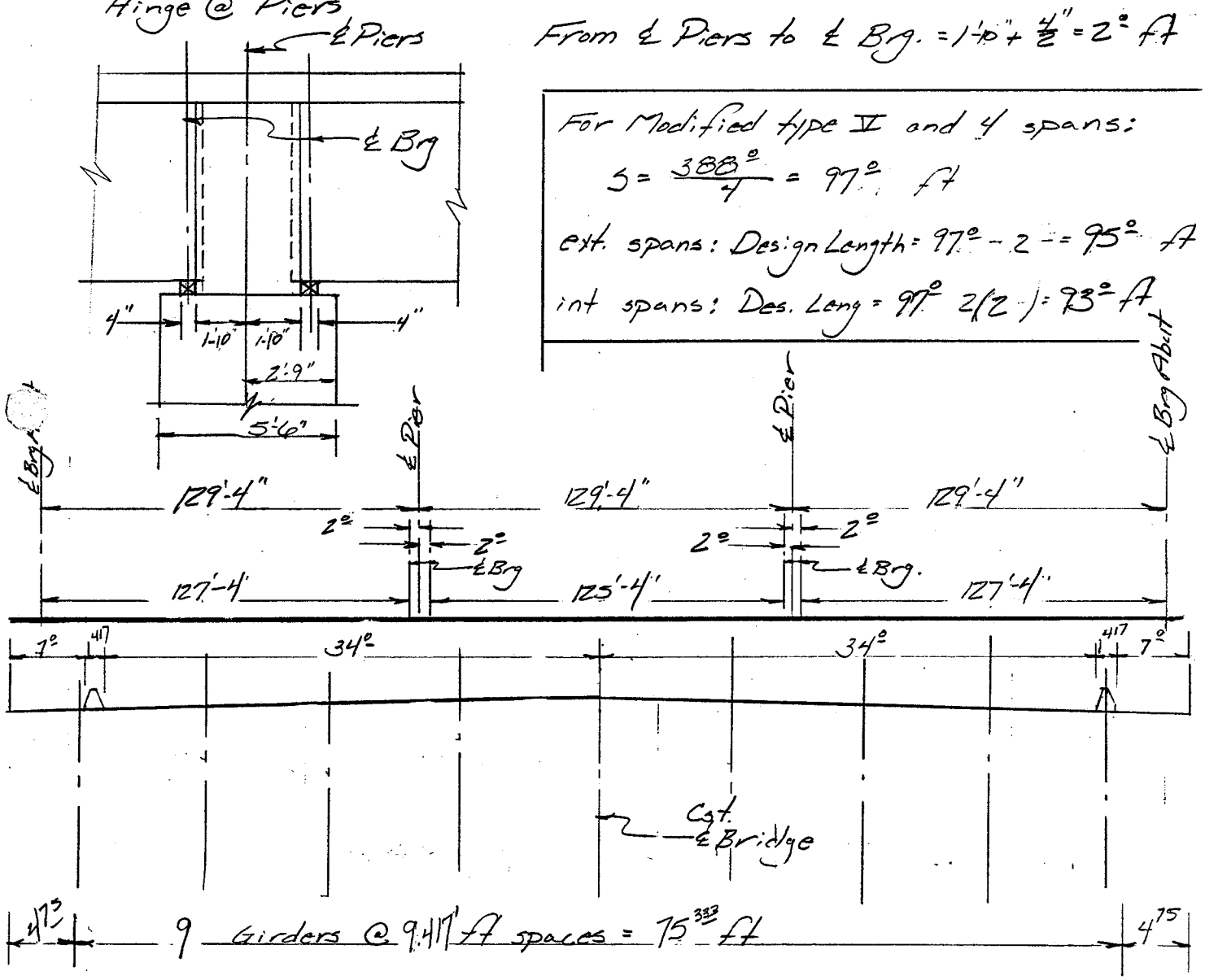
From ℄ Piers to ℄ Brg. $= 1'\text{-}0'' + \frac{4''}{2} = 2^{0}$ ft

For Modified type V and 4 spans:

$S = \frac{388^{0}}{4} = 97^{0}$ ft

ext. spans: Design Length $= 97^{0} - 2 = 95^{0}$ ft

int spans: Des. Leng $= 97^{0} - 2(2) = 93^{0}$ ft



BRW

JOB Northern Ave. @ New River SHEET NO. G3

MADE BY T. Stones DATE 9-6-89 JOB NO. 33-8962

CHECKED BY Lee Arnold DATE 10/89 SEC. SHT. NO. 14

CALCULATIONS FOR Layout

T-PAC
PRESTRESS DIVISION

RANCHO CUCAMONGA, CALIF.

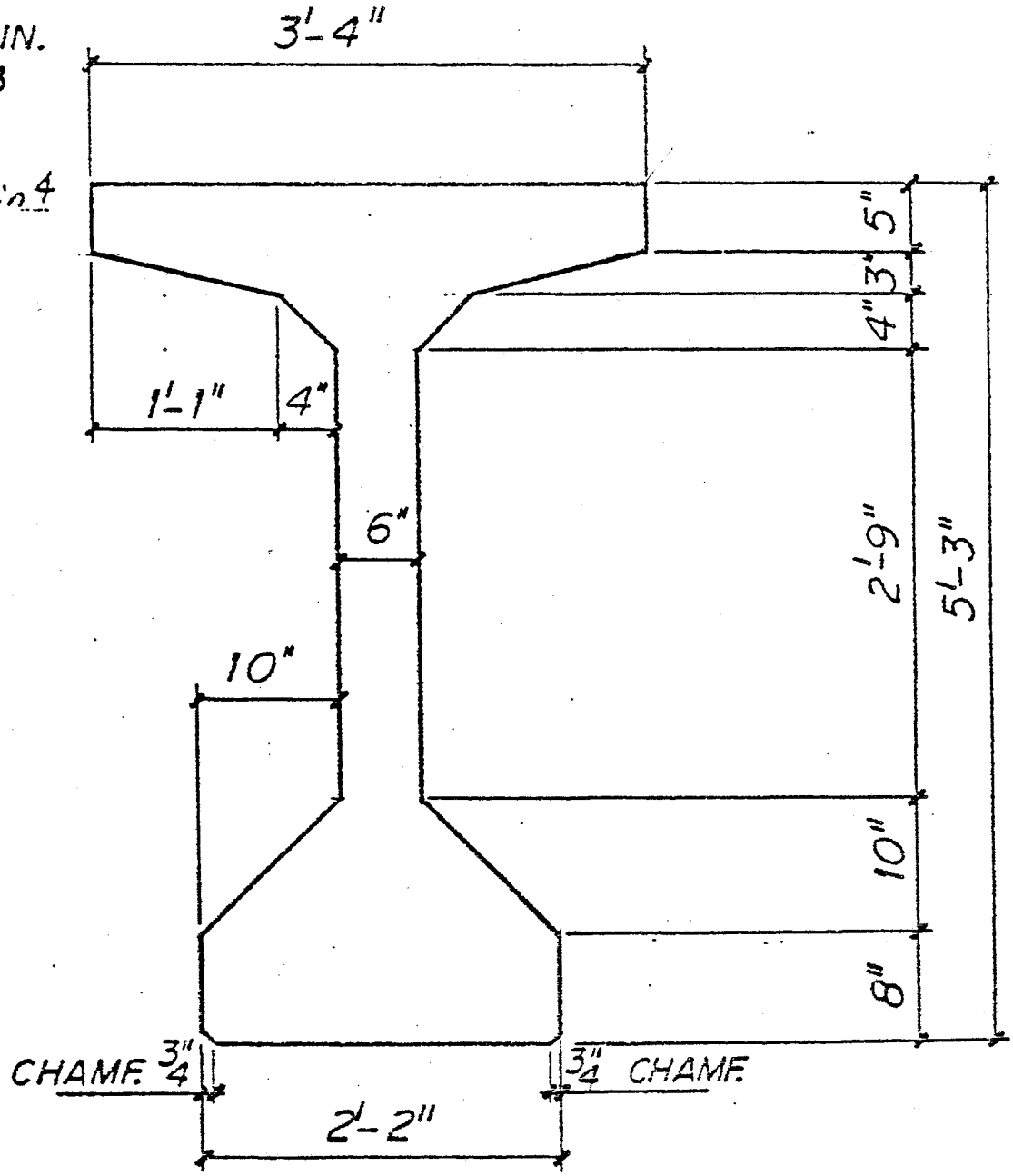
TUCSON, ARIZONA

Drafting Standard

REVISIONS	DATE	DRAWN BY:	SHEET NO
	JOB NO:	CHECKED BY:	OF

APPROVED FOR PRODUCTION - DATE ISSUED:

AREA = 887 SQ. IN.
CU. YDS/FT. = .23
WT./FT. = 924#
y_b = 32.02 in
I = 479,459 in^4



TYPE "V" MODIFIED BEAM

15

PHOENIX, ARIZONA
TUCSON, ARIZONA

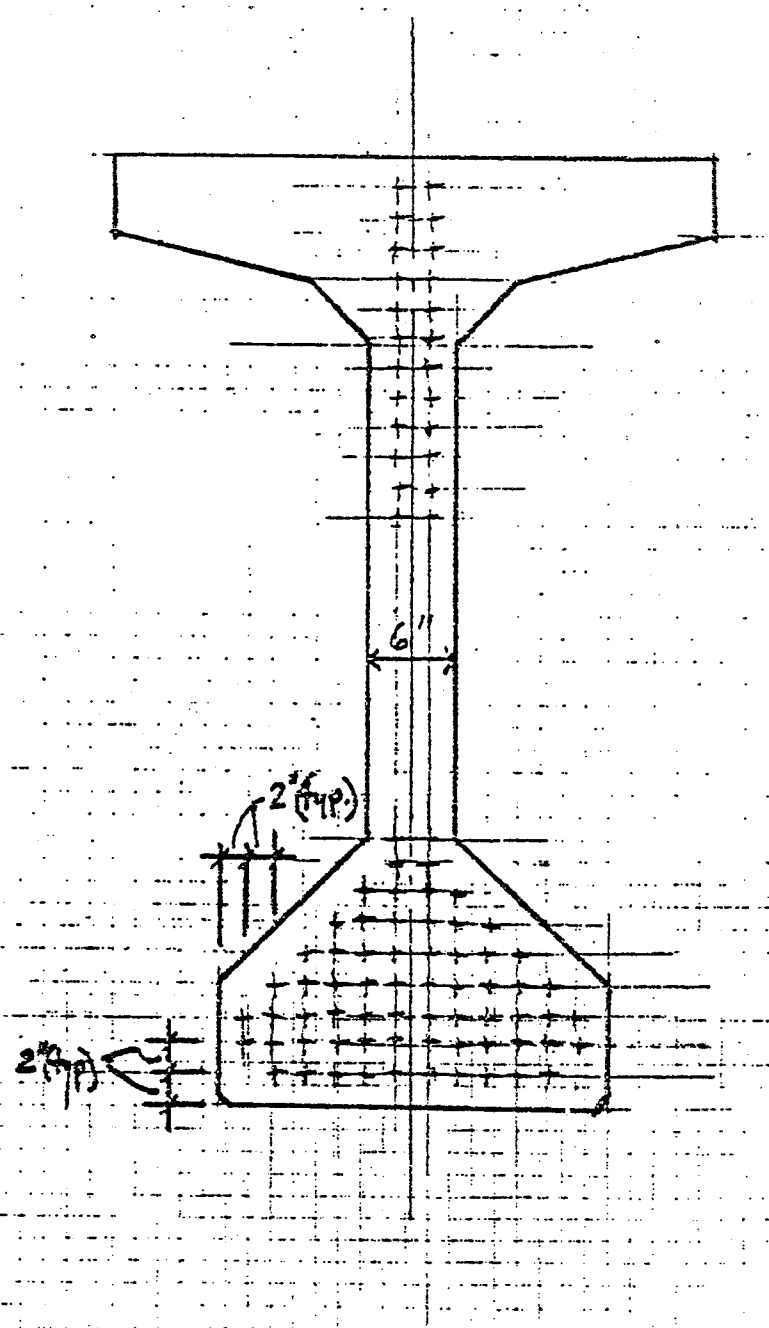
THE TANNER COMPANIES

T-PAC
PRESTRESSED DIVISION

RANCHO CUCAMONGA, CALIF.

REVISIONS	DATE	DRAWN BY HTJ	SHEET NO
	JOB NO	CHECKED BY	OF

APPROVED FOR PRODUCTION - DATE ISSUED



AASHTO TYPE V "MODIFIED" GIRDER 16

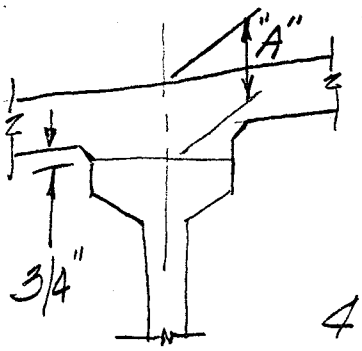
New River Channel

℄ Station of Bridge = 46+14.70

Critical Station for Clearance are ±162.5
or
Sta 47+77.20 & Sta 44+52.20

Calculate "A" Dimension at ℄ Pier

3 Span



Slab Thickness + 3/4" Fillet	=	8 3/4"
Excess Camber (TS PS Run)	=	1.60"
Top Flange Width Effect (.02)(40/2)	=	0.40
Vertical Curve Effect	=	*0
		10.75"

Elev @ Girder Bottom 1077.454

Min Elev @ PGL = 1077.454 + 6' + $\frac{10.75}{12}$ + 37.748(.02)
= 1085.105

4 Span

Slab Thickness + 3/4" Fillet	=	8 3/4"
Excess Camber (TS PS Run)	=	1.29"
Top Flange Width Effect (.02)(40/2)	=	0.40
Vertical Curve Effect	=	*0
Elev @ Girder Bottom 1077.454		10.44

Min Elev PGL = 1077.454 + 5.25' + $\frac{10.5}{12}$ + 37.083(.02) Use 10 1/2"
= 1084.32'

* The vertical curve for a crest VC will actually reduce "A". Ignor this to provide greater clearance

Raise Elevations 1/2" to Compensate for some variation in Camber

PGL Elevations @ (Minimum)

Sta	3 Span	4 Span
44+52.20	1085.15	1084.36
47+77.20	1085.15	1084.36

BRW

JOB Northern Avenue Bridge SHEET NO. V2
MADE BY Lee Arnold DATE 9/14/89 JOB NO. 33-8962
CHECKED BY T Stones DATE 9-14-89 SEC. SHT. NO. 17
CALCULATIONS FOR Vertical

✓ T Stones 2/90

BRW, INC. PHOENIX, ARIZONA JOB NUMBER 33-8962 1/23/90 TIME 11:36:35

*** PRESTRESSED GIRDER CHECK *** REV. 10/07/87 ***

*** NO END BLOCKS *** SIMPLE SPAN SHEAR DESIGN *** FINAL LOSSES - 48.

IDENT: 10 PROBLEM 2 12 HARPED STRANDS 30 STRAIGHT STRANDS
(LOW RELAXATION STRANDS)

SPAN 95.00 SERIES 2 9. GIRDERS AT 9.42 8.00 SLAB

WEB THICKNESS 6.0 IN., 2.5 IN. ASPHALT OVERLAY

STRANDS 0.153 @ 270. K A- 10.75 N. J. BARRIER S/ 5.5 AVFY - 24.80

FINAL DESIGN STRESS, BOT. CENTERLINE 0.214 (ALLOWED 0.232)

FINAL DESIGN STRESS, TOP CENTERLINE -1.636 (ALLOWED -2.400)

FINAL DESIGN STRESS, TOP AT D/2 -0.412 (ALLOWED 0.232)

STRESS TOP AT HARP PT. AT RELEASE -0.048 (ALLOWED 0.212)

STRESS BOT. AT HARP POINT AT RELEASE -2.699 (ALLOWED -3.000)

STRESS TOP AT D/2 AT RELEASE -0.374 (ALLOWED 0.212)

STRESS BOT AT D/2 AT RELEASE -2.408 (ALLOWED -3.000)

(INCLUDES ELASTIC SHORTENING AND RELAXATION)

GIR. CONCRETE STRENGTH AT STRAND RELEASE- 5.000 AT 28 DAYS- 6.000

WS(SLAB+DIAPH) - 1.191 WC(CURB+ASPH) - 0.451 E - 4.400 FCL - 7.000

RISE IN HARPED STRANDS - 49.000 FD - 7.001 FLG W - 113.00

COMPOSITE SECTION I - 976340. Y BOT - 47.91 Q SLAB - 14092.

ULTIMATE MOMENT REQ'D. 3963. AVAILABLE 6510. (GIRDER ONLY)

ULTIMATE MOMENT REQ'D. 7049. AVAILABLE 8931. (COMPOSITE)

SHEAR @ D/2: VU-293.7 VC-180.4 VS-165.1 SPACING- 7.6 (HS)
CHECK STANDARD STIRRUP SPACING AT GIRDER END ON STANDARD SHEET.
SHEAR @ D: VU-280.9 VC-186.8 VS-143.7 SPA. (S)- 8.7 (HS)

SHEAR @ HARP. PT.: VU-116.4 VC- 74.2 VS- 62.8 SPACING- 22.9 (18" MAX) (HS)

BEGIN 18" STIRRUP SPA. 35.3 FEET FROM CL BEARING (LS - 30.1)
INTERPOLATE FOR A MULTI-STEP INCREMENT IN STIRRUP SPACING.

GIR. D.L. END REACTION- 124.6 WITHOUT END DIAPHRAGM

GIR. D.L. END REACTION- 102.7 WITHOUT CURB, ASPHALT, OR END DIAPHRAGM
BREAKDOWN OF STRESSES AT CENTERLINE SPAN

✓ T Stones 2/92

SBDG- 0.935 SBDS- 1.096 SBDC- 0.300 SBXML- 0.885 SBPRE- -2.901

BRW, INC. PHOENIX, ARIZONA JOB NUMBER 33-8962 1/23/90 TIME 11:36:35

*** PRESTRESSED GIRDER DEFLECTIONS *** REV 10/07/87

PROBLEM NO. 2

GIRDER WT.	0.924	K/FT.	Y BOTTOM (GIRDER)	32.02	IN.
GIRDER AREA	887.0	IN.**2	S TOP (GIRDER)	15476.0	IN.**3
I (GIRDER)	479459.	IN.**4	Y TOP (GIRDER)	30.98	IN.
GIRDER SERIES	2		SPAN LENGTH	95.00	FT.
STRAND AREA	0.153	IN.**2	ULTIMATE STRENGTH	270.0	KSI
E	4.40	IN.	FCL	7.00	IN.
RISE / HARPED STND	49.00	IN.	NO. STRAIGHT STND	30	
NO. HARPED STND	12		I (COMPOSITE)	976339.	IN.**4
Y BOTTOM (COMP)	47.91	IN.	GROSS SLAB THICK	8.00	IN.
GIRDER SPACING	9.42	FT.	SBXML	0.885	KSI
WS	1.191	K/FT.	WC	0.451	K/FT.
DAYS	90		FC (GIRDER)	7.20	KSI
NO. CONT. ENDS	0		HARP. DIST.	38.00	FT.
FC (SLAB)	4.50	KSI	SHRINK	0.00030	
CREEP	2.00				

GIRDER IS SIMPLE SPAN FOR LL AND CURB DL.
STRANDS ARE LOW RELAXATION.

GIRDER CAMBER AT TIME OF SLAB POUR(90 DAYS)- 1.97 INCHES

DEFLECTION DUE TO SLAB WT - 0.84 INCHES
DEFLECTION DUE TO CURB + ASPH - 0.16 INCHES

SET SCREEDS FOR A TOTAL OF * 1.00 * INCHES CAMBER
ABOVE GRADE AT C.L. SPAN.

FINAL CAMBER OF GIRDER - 0.99 INCHES ABOVE THE HORIZONTAL POSITION.

EXCESS GIRDER CAMBER ALLOWANCE FOR ESTABLISHING THE 'A' DIMENSION
PER BRIDGE DESIGN MANUAL, VOLUME 2, PAGE 6-360 IS 1.13 INCHES.

APPROXIMATE LIVE LOAD + IMPACT DEFL. IS 0.46 INCHES

Northern Avenue Bridge Over the New River
Prestressed Girder Analysis

The ADOT current policy is to design prestressed I girders as simply supported composite sections for live load and impact and all superimposed dead loads. Negative moment reinforcement is then designed over intermediate supports considering span continuity for all composite dead loads and live load plus impact. It does not appear that a line of girders could be saved by a less conservative approach, so ADOT's policy will be followed.

Preliminary Depth at supports

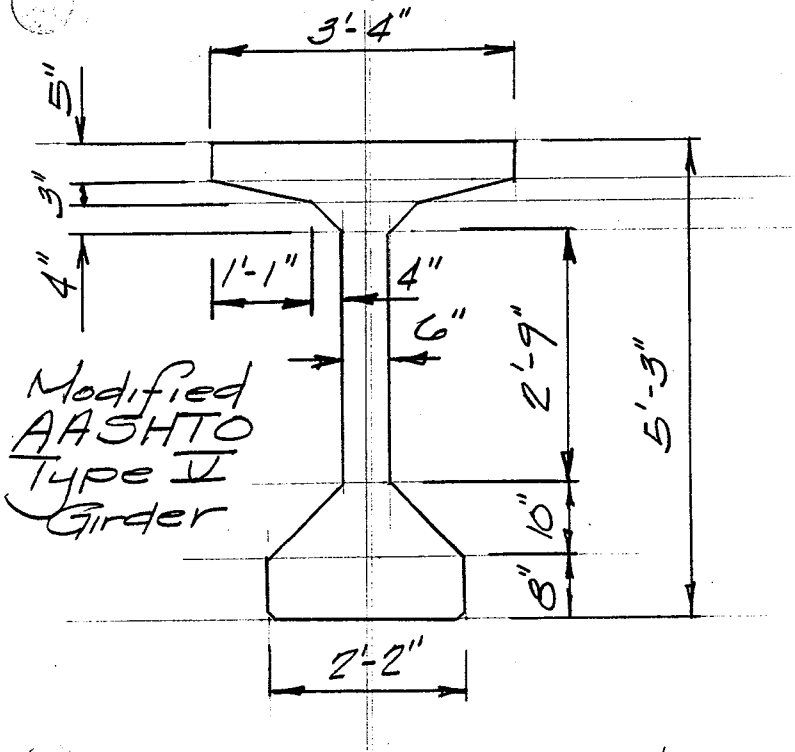
Slab Thickness plus 3/4" Fillet = 8 3/4"

Top Flange Effect = $.02\left(\frac{40''}{2}\right)$ = + 3/8"

Vertical Curve Effect = $-\frac{1.5(G)(S)^2}{L} = \frac{1.5(3.54)(.95)^2}{8} = -5/8''$

Excess Girder Camber (Assume & verify) = +1 1/4"

9 3/4"



Design Criteria

Girder

Area = 887 Sq. In.

Wt/Ft = 924#/Lf

$I = 479,459 \text{ in}^4$

$y_b = 32.02$ in

$S_T = 15,476 \text{ in}^3$

$S_B = 14,974 \text{ in}^3$

BRW

JOB Northern Avenue Br SHEET NO. G-1

MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 20

CALCULATIONS FOR Prestressed Girders

Northern Avenue Bridge Over the New River
Prestressed Girder (Cont'd)

Design Criteria (Cont'd)

	Concrete Strength	E
Girders @ Release	5000 psi	4,030,000 psi
Girders @ 28 Days	6000 psi	4,415,000 psi
Slab and Diaphragms	4000 psi	3,605,000 psi

Impact for 95' Span = 22.7%

Simple Beam Moment for HS20 Truck = 1434'K

Live Load Lanes = $\frac{9.42}{5.5 \times 2}$ = 0.857 lanes

I of Composite Section @ Mid-Span

15

Area = 1801 in^2
y_b = 50.28 y_t = 21.97
I_{xx} = 1,067,762 in^4
ST = 48,600 in^3
SB = 21,236 in^3

Taken from Sheet 5-10
These require reduction to effective flange width

Revised I for composite Section with reduced effective width & 1/2" Slab Wearing

	Area	y	Ay	Ay^2	I_{oo}
Slab 8(12)7½	720	68.0	48,960	3,329,280	3375
Filler Pad	50	63.625	3,181	202,407	6
Girder	887	32.02	28,402	909,424	479,459
	*1657		80,543	4,441,111	482,840

$y_b = \frac{80,543}{1657} = {}^*48.61$

$I_{xx} = 482,840 + 4,441,111 - 1657(48.61)^2 = {}^*1,008,572$

Note: * These values will be slightly higher the program, because program does not account for PAD

BRW

JOB Northern Avenue Br SHEET NO. G-2
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 21
CALCULATIONS FOR Prestressed Girders

Northern Avenue Bridge Over the New River
Prestressed Girder (Cont'd)

Recalc Composite Section properties reducing top slab & pad by Modular ratio

$$\frac{E_s}{E_g} = \frac{\sqrt{4000}}{\sqrt{6000}} = 0.817$$

	Area	y	Ay	Ay^2	I_{oo}
Slab	588	68	39,976	2,718,346	2756
Filler Pad	41	63.625	2,598	165,264	5
Girder	887	32.02	28,402	909,424	479,459
	1516		70,976	3,793,054	482,220

$y_{bc} = \frac{70,976}{1516} = 46.82$ $\quad y_{tc} = 16.18$ $\quad y_{TSC} = 24.93$

$I_{xxc} = 482,220 + 3,793,054 - 1516(46.82)^2 = 952,012 \text{ in}^4$

$\therefore S_{bc} = 20,333 \text{ in}^3$ $\quad S_{TC} = 58,838 \text{ in}^3$ $\quad S_{TSC} = 38,187 \text{ in}^3$

Dead Load Stress (w/o prestress)

		Moment	Top	Bottom
DL Girder	$.924\left(\frac{95}{8}\right)^2$ =	1043'k	809 psi	836 psi
Slab	$.67(9.42).150\left(\frac{95}{8}\right)^2$ =	1068'k	828 psi	856 psi
PAD	$0.10(3.33).150\left(\frac{95}{8}\right)^2$ =	56'k	43 psi	45 psi
Diaphragms	$3.6(95)\frac{1}{3}$ =	114'k	88 psi	91 psi
$3.75(.75)6.5(.150)=3.6$	Totals	2,281'k	1768 psi	1828 psi

Added DL = FWS + Barrier + Utilities

$= 236 + \frac{2(372)}{6 \text{ Gdr}} + 59 = 419 \frac{lb}{LF}$ $\quad M = \frac{.419(95)^2}{8} = 473'k$

Composite Stress (w/o Prestress)

		Moment	Top	Bottom
LL+I = 1434 (1.227) 0.857	=	1508'k	306 psi	690 psi
Superimposed DL	=	473'k	97 psi	279 psi
		1981'k	403 psi	1169 psi

BRW

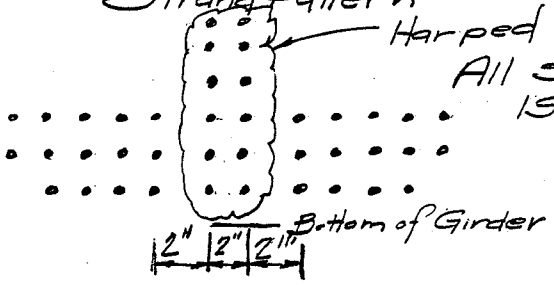
JOB Northern Avenue Bridge SHEET NO. G-3
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 22
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girder (Cont'd)

Total Stresses (w/o Prestress) for Girder

	Top	Bottom
	2171 psi	2997 psi
Allowable Tension $3\sqrt{f'c}$	232 psi	232 psi
	2403 psi	2765 psi

Try 28 Straight & 12 Harped
Strand Pattern



All strand spacing is 2"

Determine cg @ midspan to harping point

10	x 2	=	20
12	x 4	=	48
12	x 6	=	72
2	x 8	=	16
2	x 10	=	20
2	x 12	=	24
40			200

$cg = \frac{200}{40} = 5.0$

$E = Y_B - cg = 32.02 - 5.0 = 27.02''$

Assume Total System losses of 35 KSI & losses after transfer of 22 KSI for low-Lax strand

$$P_f = \frac{f_{des} \times \frac{1}{1000}}{\frac{1}{A} + \frac{E y_b}{I_g}}$$

$$= \frac{2765 \times \frac{1}{1000}}{\frac{1}{887} + \frac{27.02(32.02)}{479,459}} = 943 \text{ KIPS}$$

$$A_s = \frac{943}{(202.5 - 45)} = 6.00 \square'' \approx 40 \text{ Strands}$$

Check Losses $\Delta f_s = SH + ES + CR_c + CR_s$

Shrinkage $SH = 17,000 - 150(RH)$
from Fig. 9.16.2.1.1 the Relative Humidity in Phoenix is 40%

$\therefore SH = 17,000 - 150(40) = 11,000$ psi

BRW

JOB Northern Avenue Bridge SHEET NO. G-4
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 23
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

Elastic Shortening

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

$E_s = 28 \times 10^6$ psi

$E_{ci} = 33(150)^{1.5}(5000)^{1/2}$
$= 4.286 \times 10^6$ psi

Reduced tendon Stress $= 0.69 f'_s$

$$f_{cir} = \frac{F}{A} + \frac{Fe^2}{I_g} - \frac{M_n^{db} \times e}{I_g}$$

where M_n^{db} = Moment due dead load of the beam @ point of maximum moment, e.g. - midspan

$$ES = \frac{28 \times 10^6}{4.286 \times 10^6}\left[\frac{0.69(270)40(.1531)}{887} + \frac{.69(270)(27.02)^2(40)(.1531)}{479,459} - \frac{1043(12)(27.02)}{479,459}\right] = 15.144 \text{ ksi}$$

$f_{cir} = 2318$ psi

Relaxation of Prestressing Steel

for low relaxation Strand Pretensioned Member

$$C_{Rs} = 5,000 - 0.10\,ES - 0.05(SH + CR_c)$$

∴ Calculate Concrete Creep (CR_c)

$$C_{RC} = 12 f_{cir} - 7 f_{cds} \quad \text{where } f_{cir}\ 2,318 \text{ psi}$$

$$f_{cds} = \frac{M_n^s e}{I_g} + \frac{M_c^d\left[e + (\bar{y}' - \bar{y})\right]}{I'}$$

$$= \frac{(2,281 - 1043)12(27.02)}{479,459} + \frac{473(12)\left[27.02 + (46.82 - 32.02)\right]}{952,012}$$

$$= 1087 \text{ psi} \quad (1.087 \text{ ksi})$$

$$C_{RC} = 12(2318) - 7(1087) = 20,207 \text{ psi}$$

(Adjusted for 40 Strand)

$$C_{RS} = 5,000 - 0.10(15,144) - 0.05(11,000 + 20,207)$$
$$= 1926$$

$$\Delta f_s = 11,000 + 15,144 + 20,207 + 1926 = 48,277 \text{ psi}$$

BRW

JOB Northern Avenue Bridge SHEET NO. G-5
MADE BY Lee Arnold DATE _____ JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 24
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

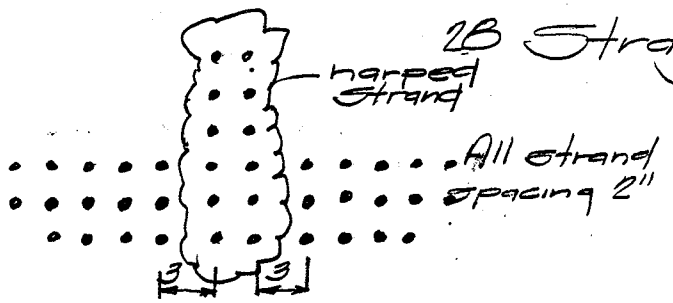
Prestressing Steel

$$A_s = \frac{P_f}{0.75(f'_s) - \Delta f_s} = \frac{943^{K.P}}{.75(270) - 48.277}$$

$$= 6.12 \ \square''$$ 40 – ½"φ Strand

Use 40 Strand

28 Straight & 12 Harped



Strand Pattern

$$A_s = 40(.1531) = 6.12 \ \square''$$

$$E = Y_b - c_g = 32.02 - 5.00 = 27.02$$

$$F_f = \text{Final Force} = 40(.1531)[.75(270) - 48.277]$$
$$= 944^K$$

$$F_t = \text{Force @ Transfer} = F_f + [\Delta f_s - ES](A_s)$$

$$\text{Where } ES = \frac{28 \times 10^6}{4.286 \times 10^6}\left[\frac{0.69(270)40(.1531)}{887} + \frac{0.69(270)(27.02)^2(.153}{479,459} - \frac{1043(12)(27.02)}{479,459}\right] = 15.144 \text{ KSI}$$

@ transfer

$$F_t = 943 + [48.277 - 15.144](40)(.1531) = 1,153^K$$

@ Jacking

$$F_j = [(F_t) + ES(40)(.1531)] = [1,153 + 15,144(40)(.1531)]$$

$$= 1246 \text{ KIPS} < \underset{1240}{0.75(270)40(.1531)}$$ NG ∴ Add 2 Strand

BRW

JOB Northern Avenue Bridge SHEET NO. J-6
MADE BY Lee Arnold DATE _____ JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 25
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

$$ES = \frac{28\times10^6}{4,286}\times10^6\left[\frac{0.69(270)42(.1531)}{887}+\frac{0.69(270)(26.88)^2(42)(.1531)}{479,459} - \frac{1043(12)(26.88)}{479,459}\right] = 16.033 \text{ Ksi}$$

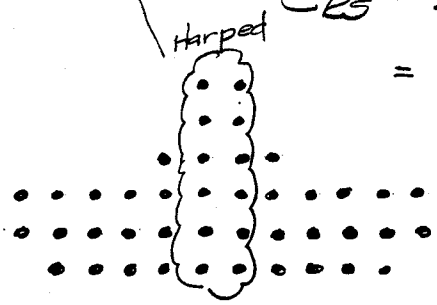
$$f_{cds} = \frac{(2281-1043)12(26.88)}{479,459} + \frac{473(12)[26.88+(47.34-32.02)]}{952,012}$$

$$= 1061 \text{ psi } (1.061 \text{ Ksi})$$

$$C_{RC} = 12 f_{cir} - 7 f_{cds} = 12(2454) - 7(1061) = 22,021 \text{ psi}$$

$$C_{ES} = 5,000 - 0.10(16,033) - 0.05(11,000 + 22,021)$$

$$= 1746 \text{ psi}$$



2 x 12	=	24
2 x 10	=	20
4 x 8	=	32
12 x 6	=	72
12 x 4	=	48
10 x 2	=	20
42		214

$$cg = \frac{214}{42} = 5.14$$

$$E = Y_b - cg$$

$$= 32.02 - 5.14 = 26.88$$

F_f = Final Force $= 42(.1531)[.75(270) - 50.80] = 975^K$

F_t = Force @ Transfer $= F_f + [\Delta f_s - ES]A_s$

where $ES = 16.033$ $\quad \Delta f_s = 11,000 + 16,033 + 22,021 + 1746 = 50,800$

$$F_t = 943 + [50,800 - 16,033]42(.1531)$$

$$= 1166^{KIP}$$

@ Jacking

$$F_J = [(F_t) + ES(42)(.1531)]$$

$$= 1166 + \frac{16,033(42)(.1531)}{1000}$$

$$= 1270^{KIP} < 0.75(270)42(.1531) = 1302 \therefore OK$$

BRW

JOB Northern Avenue Bridge SHEET NO. G-7
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 26
CALCULATIONS FOR Prestressed Girders

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

$P_J = 42(.1531)\,270\,(.75) = 1302^{KIPS}$

$P_t = P_J - ES(A_s) = 1302 - 16.033(42)(.1531) = 1199^{KIPS}$

$P_f = 1302 - 42(.1531)50800 = 975 > 943^K \therefore OK$

$FAI = \frac{P_t}{A_g} = \frac{1199}{887} = 1352$ psi

$FA = \frac{P_f}{A_g} = \frac{975}{887} = 1100$ psi

Combined Stresses

$R^2 = \frac{I_g}{A_g} = \frac{479,479}{887} = 540$

At Transfer

Top Fiber

$f_{ti} = FAI\left(1 - \frac{Ey_t}{R^2}\right) + f_t$ ← ℄ Girder

$1 - \frac{Ey_t}{R^2} = 1 - \frac{26.88(30.98)}{540} = -0.5421$

Bottom Fiber

$f_{bi} = FAI\left(1 + \frac{Ey_b}{R^2}\right) - f_b$ ← ℄ Girder

$1 + \frac{Ey_b}{R^2} = 1 + \frac{26.88(32.02)}{540} = 2.5939$

Stresses at the 4/10 Pt (harping point) = 96% of the stress @ ℄

@ ℄

$f_{ti} = 1352(-0.5421) + 809 = 76$ psi

$f_{bi} = 1352(2.5939) - 836 = 267$ psi

@ 4/10 Pt

$f_{ti} = 1352(-0.5421) + (.96)809 = 44$ psi

$f_{bi} = 1352(2.5939) - .96(836) = 2705$

Critical release stress

After placement of the slab

$f_t = FA\left(1 - \frac{Ey_t}{R^2}\right) + f_t$ ← ℄ Gir, Slab, Pavg & Diaph

$f_b = FA\left(1 + \frac{Ey_b}{R^2}\right) + f_b$ ← ℄ Gdr, Slab, Pavg & diaph

This loading will not control design

BRW

JOB Northern Avenue Bridge SHEET NO. G-8
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 27
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

✓ Stresses under full design Load

$f_{t+} = FA\left(1 - \frac{Ey_t}{r^2}\right) + f_t$ (all ℄ & ⊥)

$f_{b+} = FA\left(1 + \frac{Ey_b}{r^2}\right) - f_b$ (all ℄ & ⊥)

@ ℄

$f_{t+} = 1100(-0.5421) + (1768 + 403) = 1575$ psi

$f_{b+} = 1100(2.5939) - (1828 + 1169) = -144$

Allowable $= 3\sqrt{6{,}000}$
232 psi ∴ OK

@ harping Point

$f_{t+} = 1100(.5421) + 0.96(1768 + 403) = 1488$ psi

$f_{b+} = 1100(2.5939) - .96(1828 - 1169) = -24$ psi

Required Concrete Strength

@ release

$f'_{c1} = \frac{f_b}{0.60} = \frac{2705}{0.60} = 4600$ psi Use 5000

@ Final Design Stress

$f'_c = \frac{1575}{0.4} = 4000$ psi Use $f'_c = 6000$ for more durability.

✓ Cable Path

harped strand cg is 6" below the top of girder @ the end

$$
\begin{aligned}
cg = 12 \times (63 - 7) &= 672 \\
8 \times 2 &= 16 \\
10 \times 4 &= 40 \\
10 \times 6 &= 60 \\
2 \times 8 &= 16 \\
\hline
42 \quad & \quad 804
\end{aligned}
$$

$cg = \frac{804}{42} = 19.14$ off bottom

JOB Northern Avenue Bridge SHEET NO. G-9
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 28
CALCULATIONS FOR Prestressed Girder

BRW

Northern Avenue Bridge Over the New River
Prestressed Girder (Cont'd)

Kern bottom Limit $= \frac{R^2}{y_t} = \frac{540}{(63-32.02)} = 17.43''$ below cg

Center of jacking force $= 32.02 - 19.14$
$= 12.88''$

Which is within Kern ∴ no tension exists at beam end

Check Ultimate Strength

M_u applied $= 1.3\left[M_{DL} + \frac{5}{3}(M_{LL+I})\right]$

$= 1.3\left[(2281+473) + \frac{5}{3}(1508)\right] = 6,850^{'k}$

$\rho = \frac{A_s}{bd} = \frac{40(0.1531)}{96(71.75-5.40)} = 0.0010$

$f^*_{su} = f_s\left(1 - 0.5\rho\frac{f_s}{f'_c}\right) = 270\left[1-(0.5)\frac{270}{6}(0.0010)\right]$
$= 264.16$ ksi

$a = 1.4\,d\rho\frac{f^*_{su}}{f'_c} = 1.4(66.35)(.0010)\frac{(264.16)}{6} = 4.09''$
$< 7\frac{1}{2}''$ ∴ it is a rectangular section

$\phi M_n = \phi\left[A^*_s f^*_{su}\, d\left(1 - 0.6\rho\frac{f^*_{su}}{f'_c}\right)\right]\frac{1}{12}$

$= 1.0\left[42(0.1531)\,264.16(66.35)\left(1 - 0.6(0.0010)\frac{264.16}{6}\right)\right]\frac{1}{12}$

$= 9152^{'k} > 6850^{'k}$

∴ Ultimate Strength Capacity is adequate

Shear Reinforcement

✓ Shear requirement @ D/2

V_{DL} @ End $= 0.924\left(\frac{95}{2}\right) + 0.67(9.42)(.150)\frac{95}{2} + .05\left(\frac{95}{2}\right) + 3.6$
$+ 0.415\left(\frac{95}{2}\right) = 114.54$

JOB Northern Avenue Br SHEET NO. G-10
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-83967
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 29
CALCULATIONS FOR Prestressed Girder

BRW

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

Shear Reinforcement

$\mathbb{L}$ Reaction = 64.9^K @ End

With Impact 79.63^{KIP}

$V_{u\mathbb{L}} = 146.30$

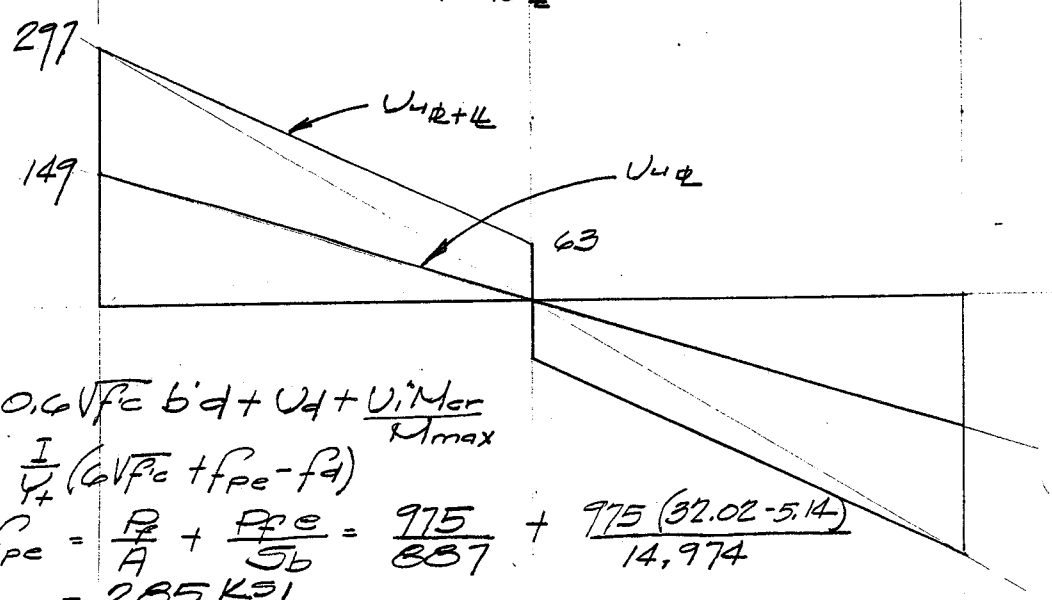
Shear @ Midspan = 29.1 / Lane

35.7 $\frac{KIPS}{Lane}$

$V_{u\mathbb{L}} = 1.227 \times .857 \times 64.9 \times 1.3 \times 5/3 = 148$

$V_u = 1.3[114.54 + 5/3(64.9)1.227(.857)] = 297$

$V_{u\,mid\,span}$ due to $\mathbb{L}$ = 66.3 ← $1.3 \times 5/3 \times 29.1 \times 1.227 \times .857$



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

$M_{cr} = \frac{I}{y_t}(6\sqrt{f'_c} + f_{pe} - f_d)$

$f_{pe} = \frac{P_z}{A} + \frac{P_z e}{S_b} = \frac{975}{887} + \frac{975(32.02-5.14)}{14,974}$

$= 2.85$ KSI

$f_d = \frac{856+45+91+279}{1000} = 1.271$ KSI

$M_{cr} = \frac{952,012}{24.93}\left[6(6000)^{1/2} + 2850 - 1271\right]\frac{1}{12000} = 6503^{IK}$

$V_{ci} = 0.6\frac{(6000)^{1/2}}{1000}(6)(.8)(63) + 114.54 + \frac{6503 \times 143.6}{3882} = 369^K$

where $V_i = 1.3[5/3 \times 66.3] = 143.6^K$

$M_{max} = 1.3[473 + 5/3(1508)] = 3882$

$V_{cw} = [3.5(f'_c)^{1/2} + 0.3 f_{pc}]b'd + V_p$

$f_{pc} = \frac{P}{A} + \frac{Pe}{S} =$

where $P = 42(.1531)[270(.75) - 50.8] = 975^K$

$\frac{975}{887} = 1100$ PSI

P/S @ centroid of Cross Section

$1100\left(1 - \frac{Ey'_t}{r^2}\right) = 1100\left[1 - \frac{26.88(46.82-32.02)}{540}\right] = 290$ PSI

JOB Northern Avenue Bridge SHEET NO. G-11

MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 30

CALCULATIONS FOR Prestressed Girder

BRW

Northern Avenue Bridge Over the New River
Prestressed Girder (Cont'd)

S @ NA of Composite $= \frac{479,459}{46.82 - 32.02} = 32,396 \text{ in}^3$

Stresses

℄ Gdr	$= \frac{1043 \times 12000}{32,396}$	$= 386$ psi	
Slab	$= \frac{1068 \times 12000}{32,396}$	$= 396$	
Pad	$= \frac{56 \times 12,000}{32,396}$	$= 21$	
Diaph	$= \frac{114 \times 12,000}{32,396}$	$= 42$	
		845 psi	
Plus P/S		290	
		1,135	

$$V_{pls} = 975\left(\left(\frac{19.14 - 5.14}{\left[(14)^2 + (.4 \times 95 \times 12)^2\right]^{1/2}}\right)\right) = 29.9^{\text{kips}}$$

$$V_{cw} = \frac{\left[3.5(6000)^{1/2} + 0.3(1135)\right] 6(.8)43}{1000} + 29.9 = 214^{K}$$

∴ Use V_{cw}

$$V_s = \frac{V_u}{\phi} - V_{cw} = \frac{297}{.85} - 214 = 135^{K}$$

$$S = \frac{A_v f_{sy} d}{V_s} = \frac{2(.2)60(50.4)}{135} = 8.9'' \text{ @ Ends}$$

Minimum Web Reinforcement

$$S = \frac{A_v f_{sy}}{50 b'} = \frac{(.40)60,000}{50(6)}$$

Use 18" maximum

Space stirrups @ 6" for ≈ 3' @ Girder ends, the 8" for 0.15 x S, then at 12" to 0.3 x S, then @ 18" in the center 0.45

BRW

JOB Northern Avenue Bridge SHEET NO. G-12
MADE BY Lee Arnold DATE 2/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 31
CALCULATIONS FOR Prestressed Girder

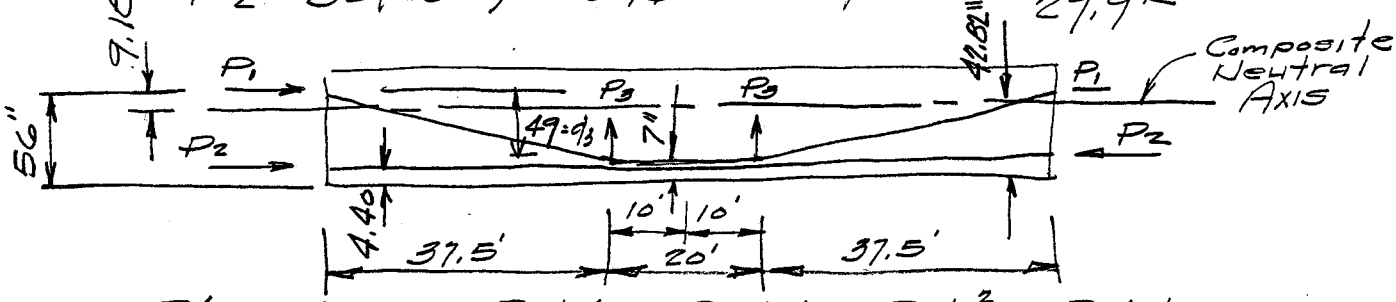
Northern Avenue Bridge Over the New River
Prestressed Girder - Positive Moment
@ Piers

Final P/S = $0.1531[270(.75) - 50.8] = 23.2 \frac{K}{Strand}$

$P_1 = 12(23.2) = 278.4^K$

$P_2 = 30(23.2) = 696.0^K$

P_3 is upward P/S force from G-12 29.9^K



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

$$P/S \text{ Slope} = 278.4(9.18)\,97\times\frac{12}{2} - 696(42.42)\,97\times\frac{12}{2} - 29.9(37.5)^2\times\frac{144}{2}$$

$$- 29.9(37.5)\,20\times 144/2$$

$$= 1{,}487{,}425 - 17{,}183{,}155 - 3{,}027{,}375 - 1{,}614{,}600$$

$$= \frac{20{,}338{,}000 \text{ in-Kips}}{EI}$$

Ref PCA "Design of Continuous Hwy Bridges W/PC PS Concrete Girders" Pgs 10 & 11

@ Piers 1 & 3 $= \frac{-2.57EI}{97\times 12}\left[-\frac{20{,}338{,}000}{EI}\right] = 44\times10^6 \text{ in lb} = 3742^{ft\text{-}lb}$

@ Pier 2 $= \frac{-1.72EI}{97\times 12}\left[-\frac{20{,}338{,}000}{EI}\right] = 30\times10^6 \text{ in lb} = 2504^{ft\text{-}lb}$

Creep Restraint Moment (not adjusted by creep adjust factor)

@ Piers 1 & 3 $= -0.86(3742) = -3218^{1K}$

@ Pier 2 $= -0.572(3742) = -2140^{1K}$

Phoenix Relative Humidity = 40%

Creep correction factor $1.10\times 0.600\times10^6$
$= 0.660\times10^6$

Continuity Connection @ 45 days

$\epsilon_s = 0.00066\times 0.5 = 0.00033$

BRW

JOB Northern Avenue Bridge SHEET NO. G-13
MADE BY Lee Arnold DATE 2/90 JOB NO.
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 32
CALCULATIONS FOR Positive Connection

Northern Avenue Bridge Over the New River
Prestressed Girder - Positive Moment

deck $f'_c = 4000$
$E_d = 3.6 \times 10^6$ psi

$$M_s = \epsilon_s E_d A_B \left(e_2' + \frac{t}{2}\right)$$

$$M_s = .00033 \times 3.6 \times 10^6 \times 8 \times 113 \times 24.93 \times \frac{1}{12{,}000}$$

$$= 2{,}231$$

Shrinkage Restraint Moments

@ Piers 1 & 3 $\quad -1.286(2{,}231) = -2869^{'K}$

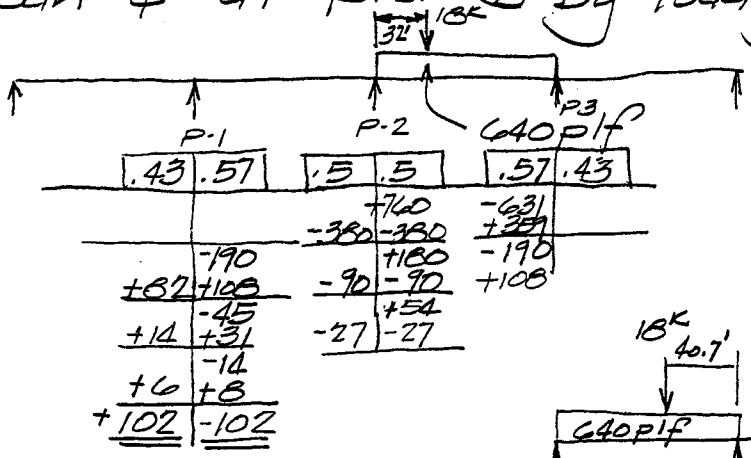
@ Pier 2 $\quad -0.857(2{,}231) = -1912^{'K}$

Positive Moments due to L+I — see below

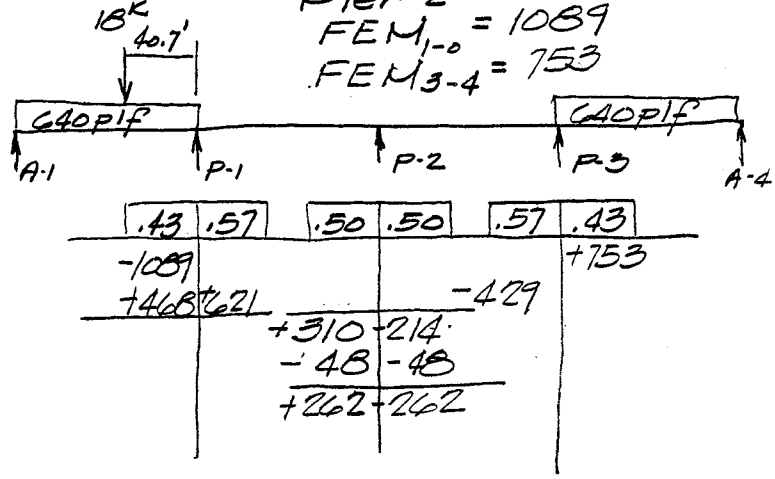
@ Piers 1 & 3 $= 0.857 \times 1.227 \times 102 = 108^{'K}$

@ Pier 2 $= 0.857 \times 1.227 \times 262 = 276^{'K}$

Maximum Positive Live load Moments at Piers 1 & 3 occur by loading opposite interior span & at pier 2 by loading exterior spans



Piers 1 & 3
$FEM_{23} = 760^{'K}$
$FEM_{32} = 631$



Pier 2
$FEM_{1-2} = 1089$
$FEM_{3-4} = 753$

BRW

JOB Northern Avenue Bridge SHEET NO. G-14
MADE BY Lee Arnold DATE 2/90 JOB NO.
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 33
CALCULATIONS FOR Positive Connection

Northern Avenue Bridge

Prestressed Girder - Positive Moment Connection

Evaluation of Girder Creep (Ref PCA "Design of Continuous Highway Bridges with Precast Prestressed Concrete Girders")

$E_{ci} = (5000)^{1/2} 57{,}000 = 4.03 \times 10^6$ psi

Use 20 years for ultimate creep

from figure 5 Creep $= 0.35 \times 10^{-6}$ in/in/psi

from figure 6 and 2 day release

Adjusted Specific Creep $= 1.66 \times 0.35 \times 10^{-6}$

$= 0.58 \times 10^{-6}$ in/in/psi

from Table 1, adjust for volume-surface ration

$C_s = 1.18 \times 0.58 \times 10^{-6} = 0.68 \times 10^{-6}$ in/in/psi

In all likelihood, the girder will not be set for several months after release, use 28 days. This would leave 60% of the creep remain to occur (Fig 8)

Design Creep $= 0.6(1.68 \times 10^{-6}) = 0.41 \times 10^{-6}$ in/in/psi

$\phi = \epsilon_c \times E = 0.41 \times 10^{-6} \times (5000)^{1/2} 57{,}000 = 1.65$

$1 - e^{-\phi} = 1 - e^{-1.65} = 0.808$

$\left(\frac{1-e^{\phi}}{\phi}\right) = 0.490$

Sum of Shrinkage plus L+I positive moment

at Piers 1 & 3 $(3742-3218).808 + 0.49(2869) + 108 = 874^{1K}$

at Piers 2 $(2504-2140).808 + 0.49(-1912) + 276 = -366^{1K}$

The resultant of all moments is negative, such that positive moments never occur at the bents. Extend all strands in the bottom row of straight strands into the cap diaphragm & anchor

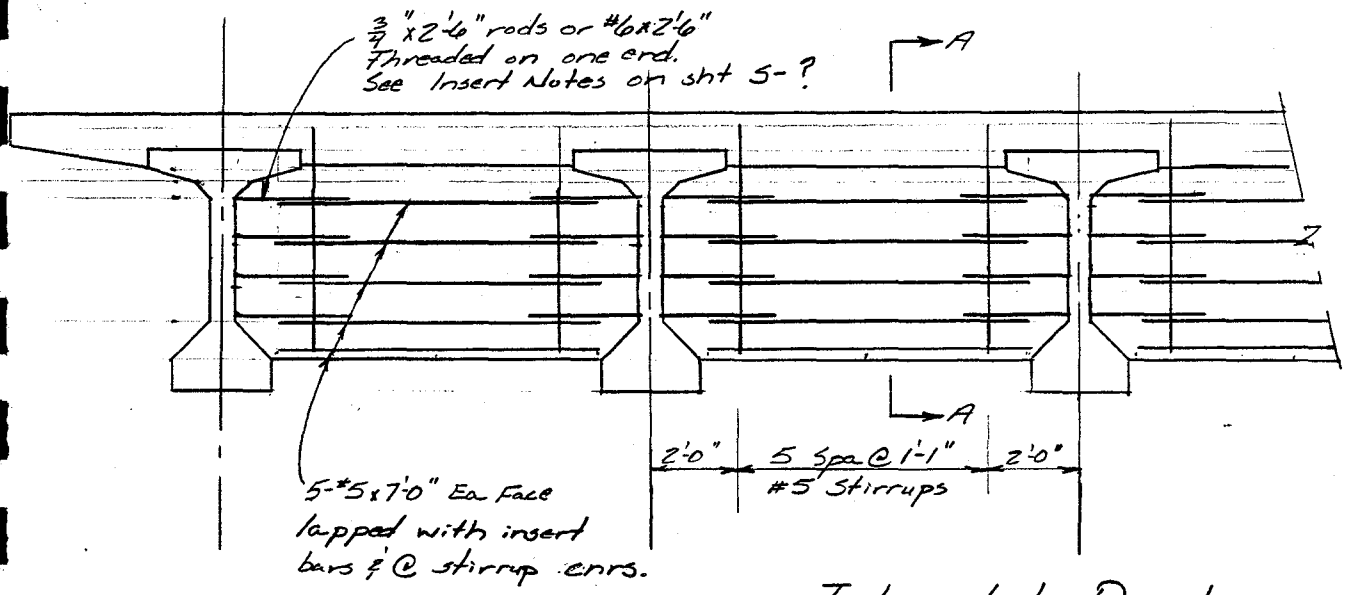
BRW

JOB Northern Avenue Bridge SHEET NO. G-15

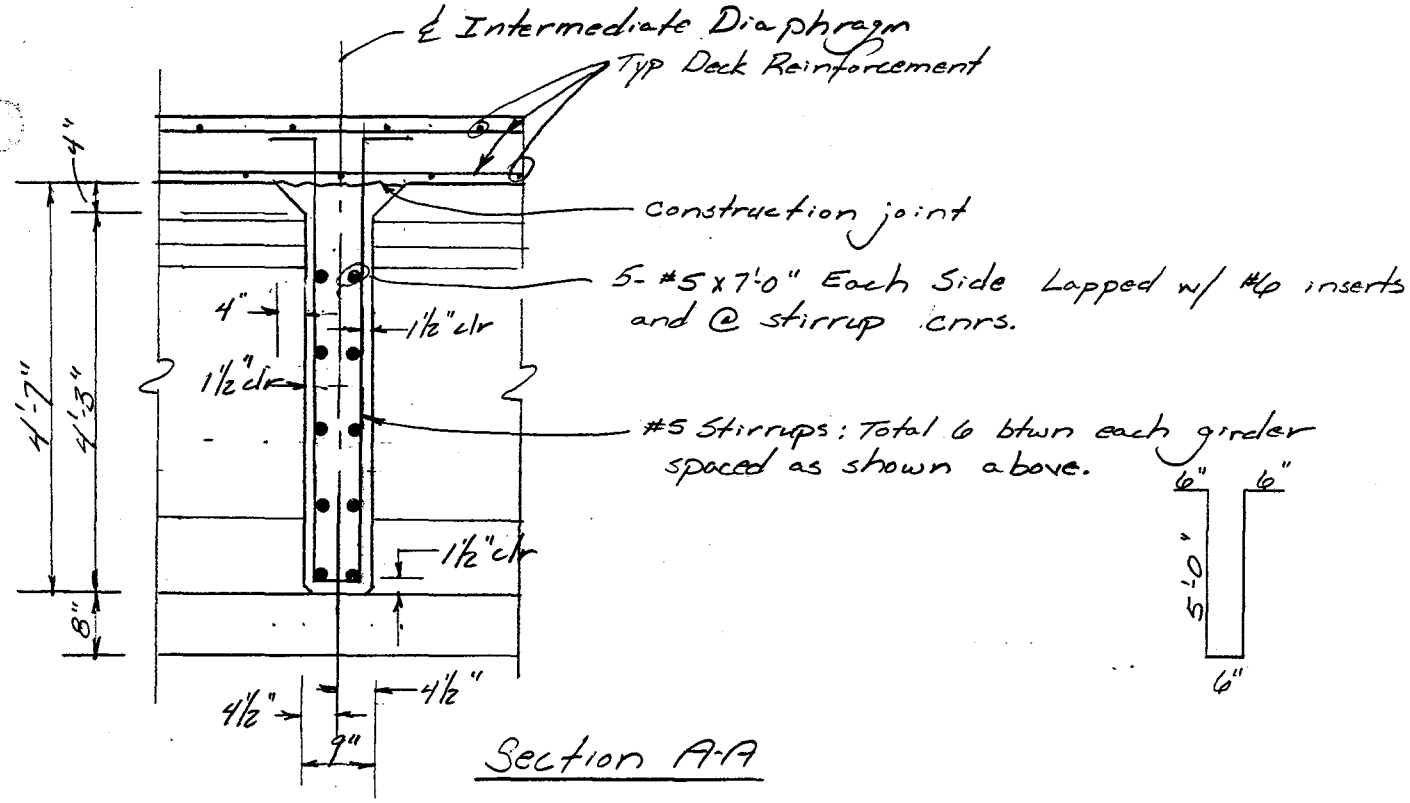
MADE BY Lee Arnold DATE 2/90 JOB NO.

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 34

CALCULATIONS FOR Positive Connection



Intermediate Diaphragm

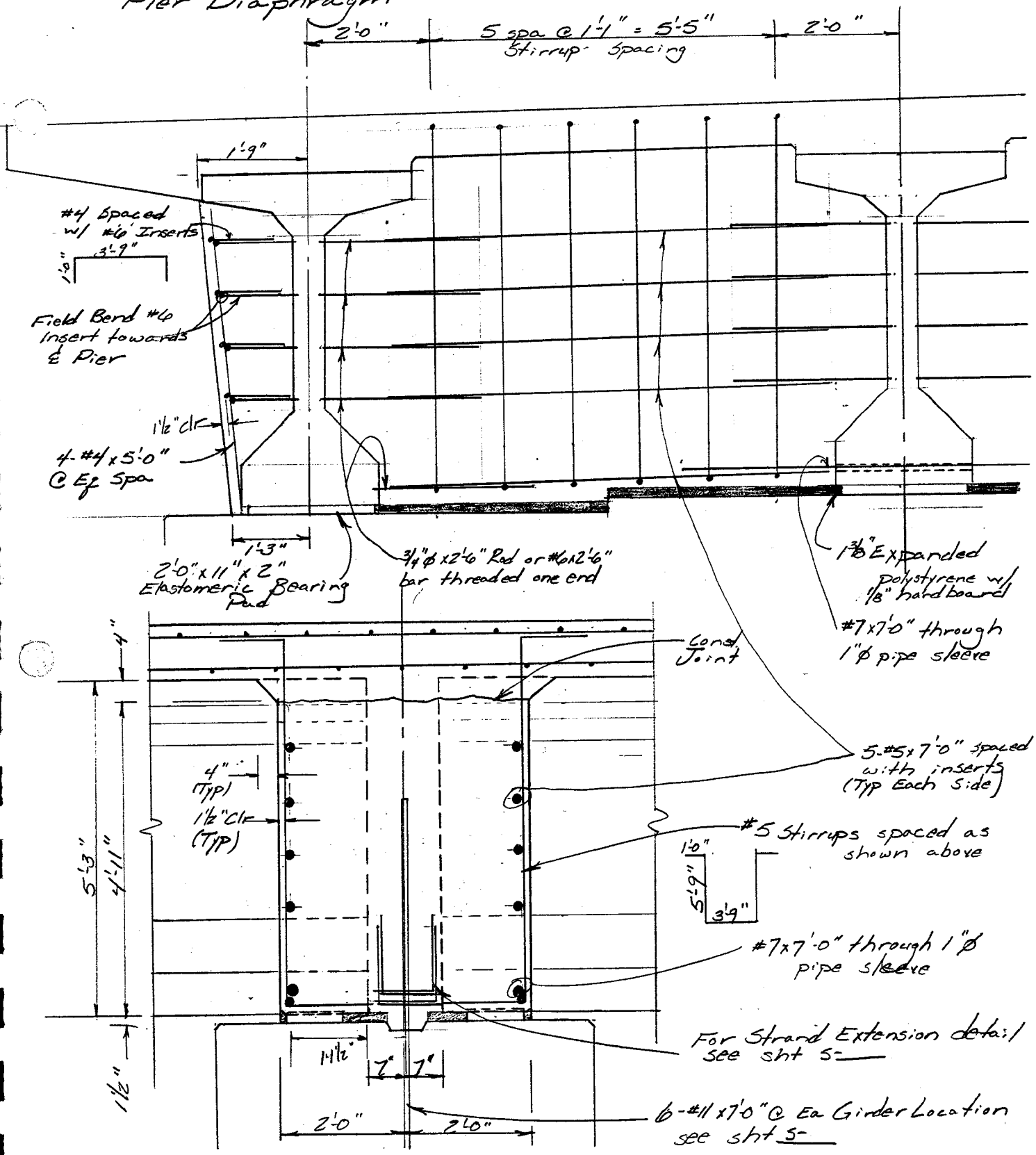


Section A-A

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. D-1
MADE BY T Stones DATE 1-30-90 JOB NO. 23-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 35
CALCULATIONS FOR Diaphragm Details

Pier Diaphragm



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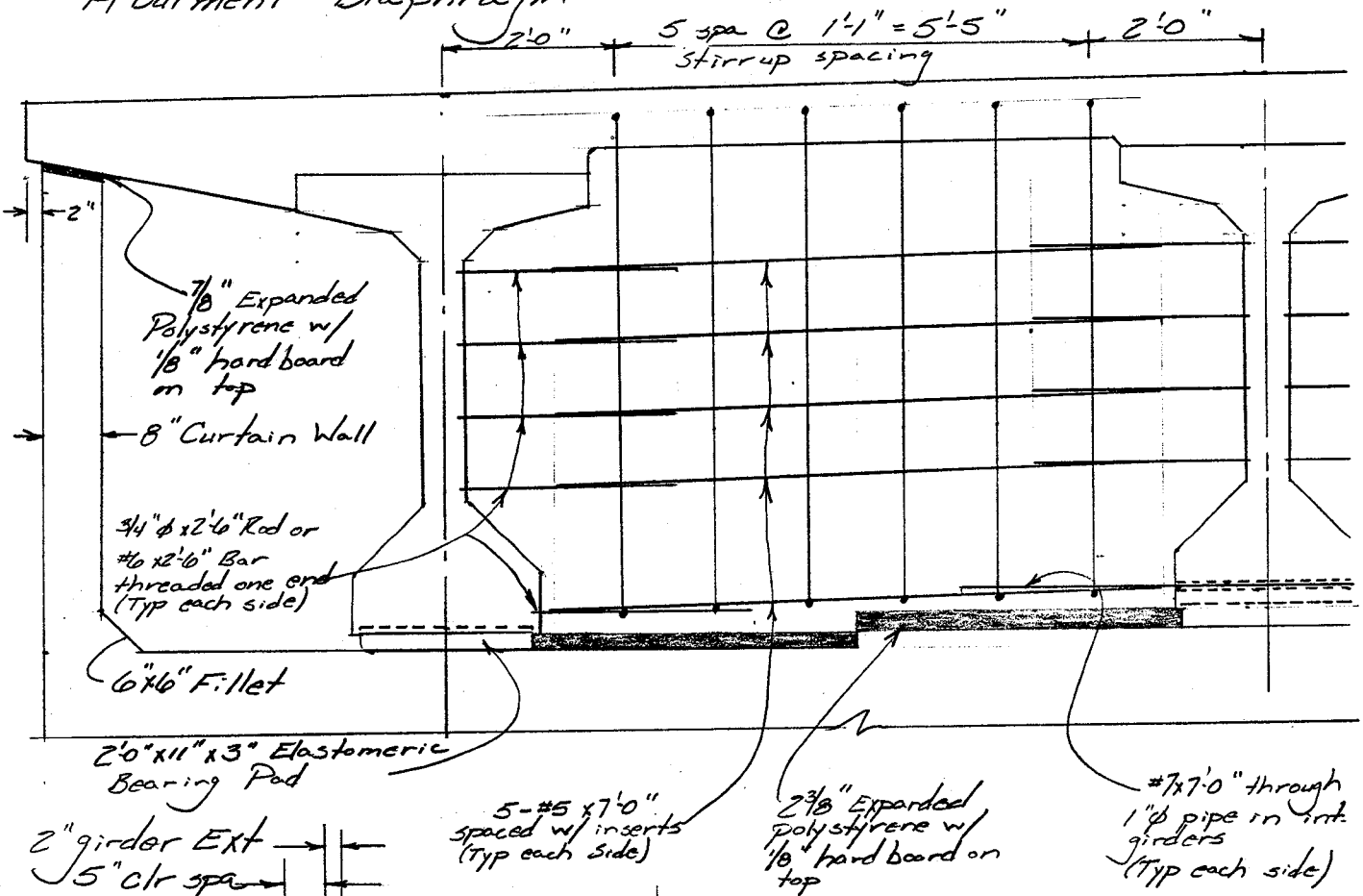
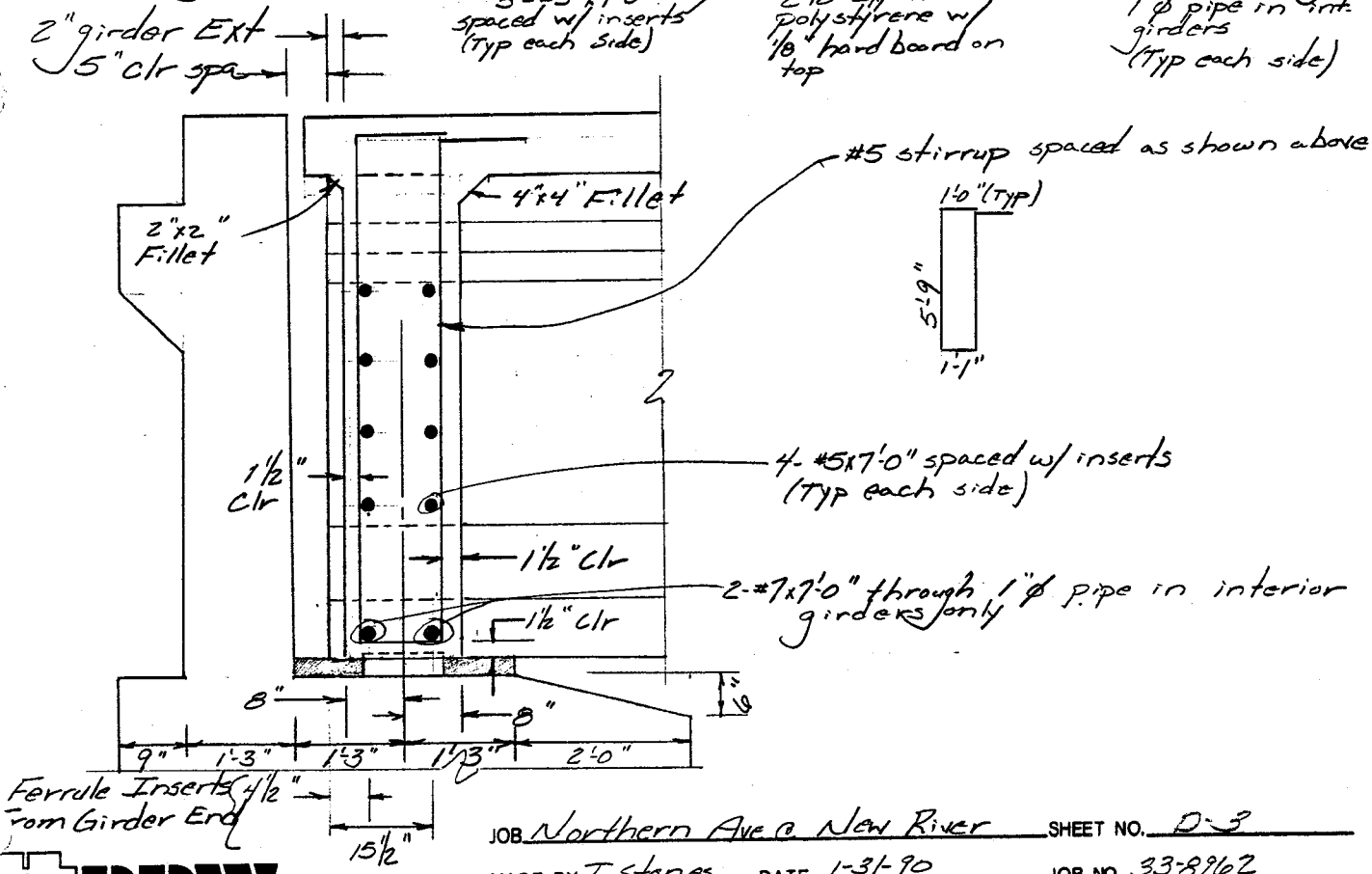
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. D2
MADE BY T Stones DATE 1-30-90 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 36
CALCULATIONS FOR Diaphragm Details

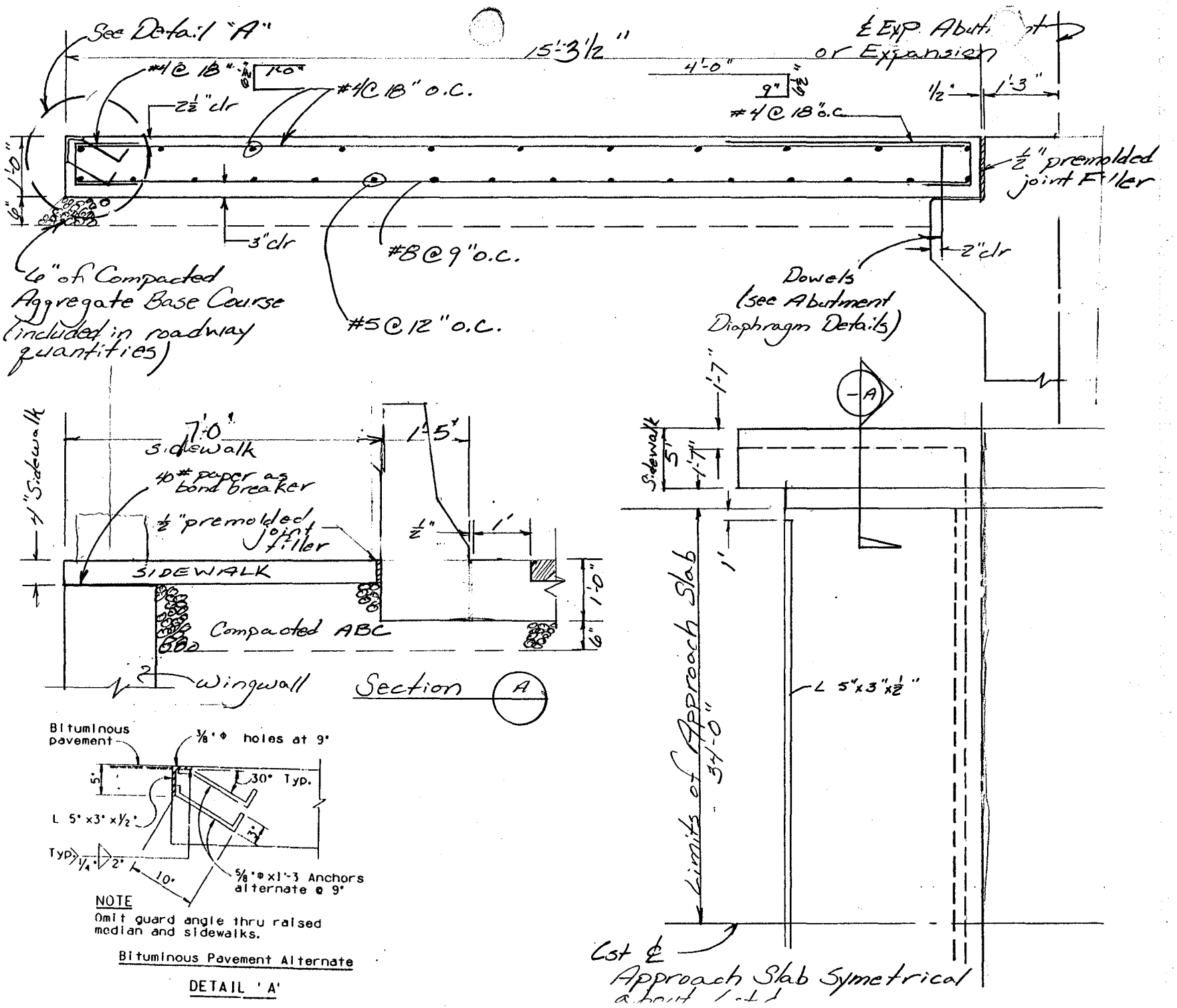
Abutment Diaphragm





BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. D-3
MADE BY T. Stones DATE 1-31-90 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 37
CALCULATIONS FOR Diaphragm Details

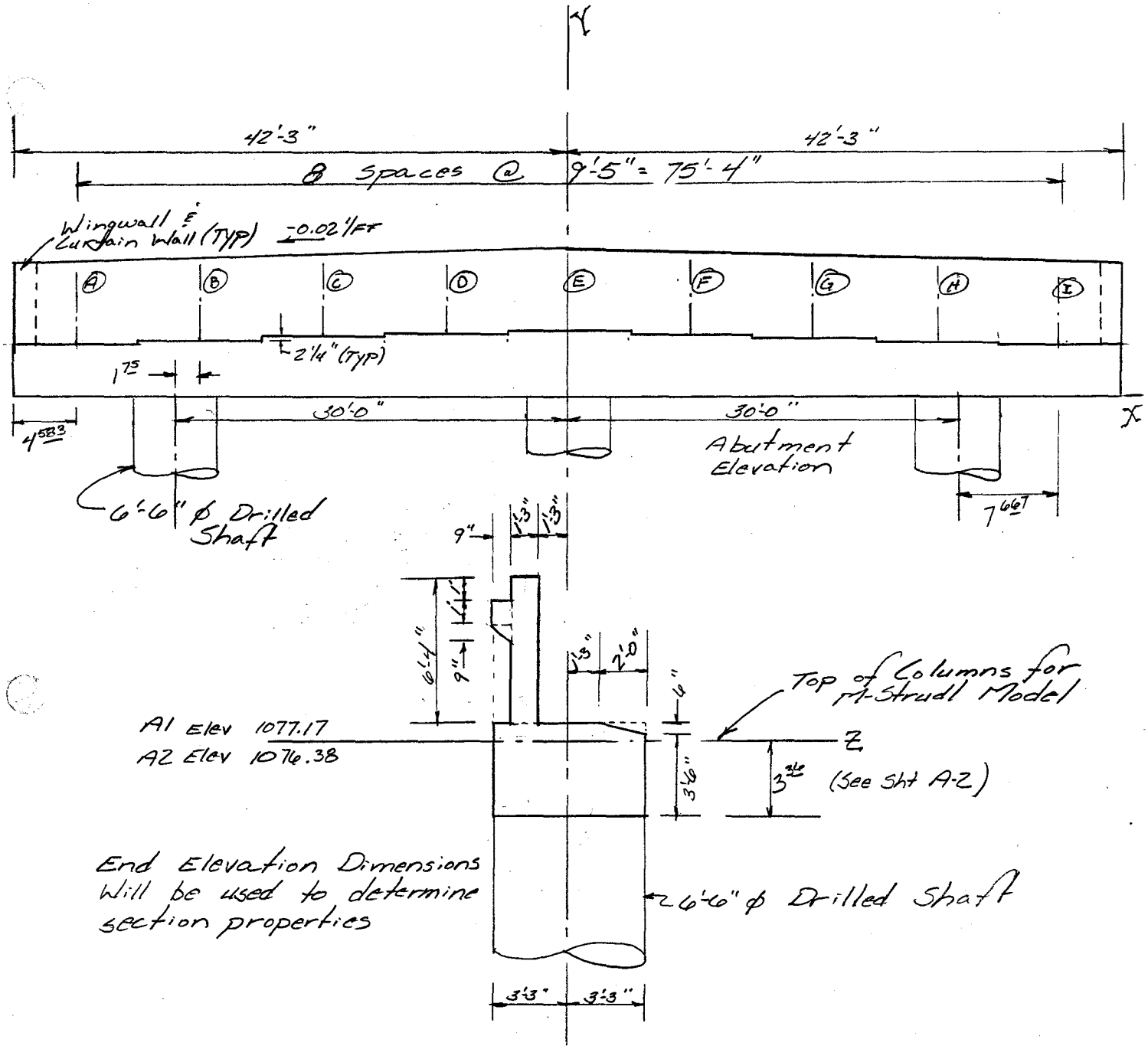


JOB Northern Ave. @ New River SHEET NO.

MADE BY T. Stones DATE 9-14-89 JOB NO. 33-8962

CHECKED BY Lee Arnold DATE 10/89 SEC. SHT. NO. 38

CALCULATIONS FOR Approach Slab



BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-1
MADE BY T Stones DATE 1-24-90 JOB NO. 33-8962
CHECKED BY L. Arnold DATE 1/90 SEC. SHT. NO. 39
CALCULATIONS FOR Abutment Design

Section Properties

$f'_c = 4000$ psi density = 0.015 ksf therm = 0.000006

Abutment Cap

A (ft²)	y_b (ft)	Ay_b (ft³)	d (ft²)	Ad^2 (ft⁴)	I_{zz} (ft⁴)	
$4(6^{5})=26^{0}$	2^{0}	52^{0}	-1^{359}	48^{021}	$\frac{6^{5}}{12}(4)^3=$	34^{447}
$-0^{5}(2)(0^{3})=-0^{5}$	3^{833}	-1^{917}	0^{474}	-0^{112}	$-2/36(0^{5})^3=$	-0^{007}
$6^{333}(1^{25})=7^{916}$	7^{167}	56^{73}	3^{808}	114^{786}	$1^{25}/12(6^{333})^3=$	26^{458}
$1^{0}(0^{25})=0^{25}$	8^{833}	6^{625}	5^{474}	22^{473}	$0^{25}/12(1)^3=$	0^{063}
$0^{3}(0^{25})^2=0^{281}$	8^{083}	2^{271}	4^{724}	6^{271}	$0^{25}/36(0^{25})^3=$	0^{009}
34^{447}		115^{709}		$191.^{439}$		61^{189}

$$y_c = \frac{115^{709}}{34^{447}} = 3^{359} \qquad d = y_b - y_c$$

$$I_{zz} = 61^{189} + 191^{439} = 252.628 \text{ ft}^4 \qquad \text{Area} = 34.447 \text{ ft}^2$$

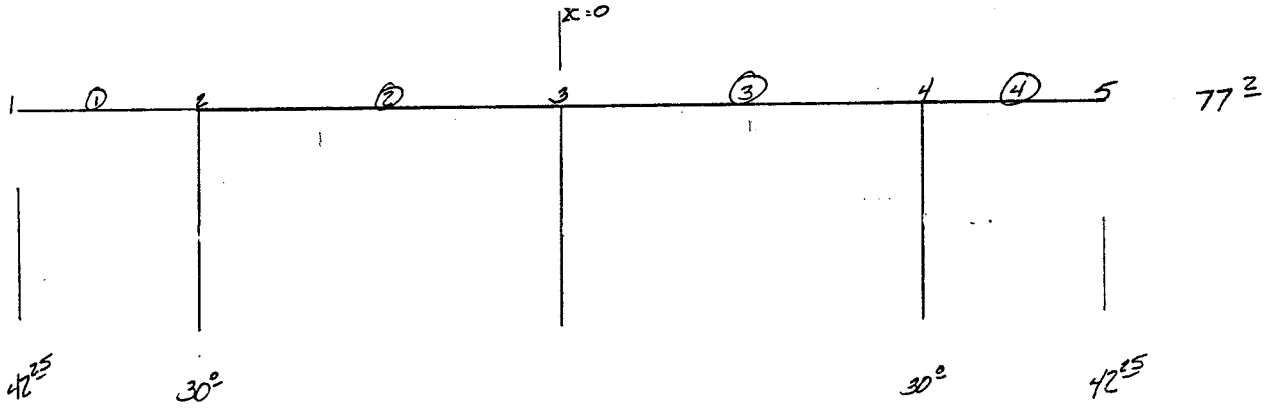
Shaft:

$$\text{Area} = \frac{\pi}{4}(6^{5})^2 = 33.183 \text{ ft}^2$$

$$I_{zz} = \frac{\pi}{64}(6^{5})^4 = 87.624 \text{ ft}^4$$

Very little lateral load will be applied to the abutment shafts: no stream and only wind from superstructure

Assume point of fixity @ 4x shaft ϕ = 26 ft.



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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-2
MADE BY T Stones DATE 1-24-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 40
CALCULATIONS FOR Abutment Design

Abutment will be modelled as a 3-D frame to apply moments caused by wingwalls and longitudinal force due to wind

poisson's ratio = 0.2

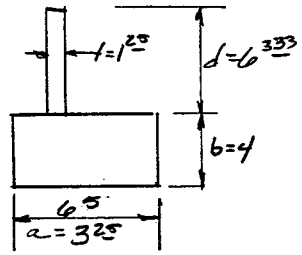
Abutment Cap

A (ft^2)	x_ℓ (ft)	Ax_ℓ (ft^3)	d (ft)	Ad^2 (ft^4)	I_{yy} (ft^4)
26^{0}	0	0	-0^{55}	7^{865}	$4(6^{5})^3/12 = 91^{542}$
-0^{5}	2^{583}	-1^{292}	-0^{742}	-0^{275}	$-0^{5}(2)^3/36 = -0^{111}$
7^{916}	-1^{875}	-14^{843}	-1^{325}	13^{898}	$6^{33}(1^{25})^3/12 = 1^{03}$
0^{75}	-2^{875}	-2^{156}	-2^{325}	4^{054}	$1(0^{75})^3/12 = 0^{035}$
0^{281}	-2^{75}	-0^{773}	-2^{20}	1^{360}	$0^{75}(0^{75})^3/36 = 0^{009}$
34^{447}		-19^{064}		$26^{902}\ ft^4$	$92^{505}\ ft^4$

$$x_c = \frac{-19^{064}}{34^{447}} = -0.^{55}\ ft \qquad d = x_\ell - x_c$$

$$I_{yy} = 92^{505} + 26^{902} = 119^{407}\ ft^4$$

I_{xx}



$$I_{xx} \approx ab^3\left[\frac{16}{3} - 3.36\frac{b}{a}\left(1 - \frac{b^4}{12a^4}\right)\right] + \frac{dt^3}{3}$$

$$\approx 3^{25}(4)^3\left[\frac{16}{3} - 3.36\left(\frac{4}{3^{25}}\right)\left(1 - \frac{4^4}{12(3^{25})^4}\right)\right] + \frac{6^{333}(1^{25})^3}{3}$$

$$I_{xx} \approx 417^{11}\ ft^4$$

Shaft:

$$I_{yy} = I_{zz} = 87.^{624}\ ft^4$$

$$I_{xx} = \frac{\pi r^4}{2} = \frac{\pi}{2}(3^{25})^4 = 175^{248}\ ft^4$$

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-3
MADE BY T Stones DATE 1-25-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 41
CALCULATIONS FOR Abutment Design

Soil Properties @ Abut: For soil profile see sheets S-1 & S-2

	Depth	Elev.	K	γ	ϕ	depth p-y curve
	0	1072				
1			44.0	62pcf - 0.036pci	32°	84 in
	168 in	1058				
2			44.0	62pcf - 0.036pci	40°	258 in
	348 in	1043				
3			25.5	53 - 0.031	34°	408 in
	468 in	1033				
4			44.0	56 - 0.032	40°	498 in
	528 in	1028				
5			44.0	62 0.036	32°	588 in
	648 in	1018				
6			25.5	53 0.031	34°	678 in
	708 in	1013				
7			44.0	62 0.036	32°	768 in
	828 in	1003				
8			44.0	62 0.036	40	864 in
	900 in	997				

COM624 Input:

```
1     Northern Avenue Abutment Bent
2     Engl
3.    75, 8, 1, 0
4.    9, 9, 0
5.    900, 3605000, 0
6.    1, 1
7.    3, 2, 1, 0
8.    0, 0, 12
9.
10.   0.0, 78, 1816972, 4778.4
11.   1, 4, 0, 168, 44.0, 0, 0
      2, 4, 168, 348, 44.0, 0, 0
      3, 4, 348, 468, 25.5, 0, 0
      4, 4, 468, 528, 44.0, 0, 0
      5, 4, 528, 648, 44.0, 0, 0
      6, 4, 648, 708, 25.5, 0, 0
      7, 4, 708, 828, 44.0, 0, 0
      8, 4, 828, 900, 44.0, 0, 0
```

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-4
MADE BY T Stones DATE 1-25-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 42
CALCULATIONS FOR Abutment Design

COM624G Input (Cont)

```
12    0    , 0.036
      168  , 0.036
      348  , 0.031
      468  , 0.032
      528  , 0.036
      648  , 0.031
      708  , 0.036
      828  , 0.036
      900  , 0.036

13    0   , 0, 32 , 0
      168 , 0, 40 , 0
      348 , 0, 34 , 0
      468 , 0, 40 , 0
      528 , 0, 32 , 0
      648 , 0, 34 , 0
      708 , 0, 32 , 0
      828 , 0, 40 , 0
      900 , 0, 40 , 0

14
15
16
17    8
18    84
      258
      408
      498
      588
      678
      768
      864
19    -1, 0, 0, 0
20    End
```

The following spring constant taken from COM624G output shown on sht A-6 will be used for M-Strudl Analysis

Depth	Elev.	lb/in COM624G p value	k/ft M-Strudl k value
84 in	65 ft	2372	28.5
258	50.5	6705	80.5
408	38	10864	130.4
498	30.5	14293	171.5
588	23	15261	183.1
678	15.5	18730	224.8
768	8	32357	388.3
864	0	41184	494.2

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-5
MADE BY T Stones DATE 1-25-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 43
CALCULATIONS FOR Abutment Design

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 8
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
84.00	78.00	36.0	.36D-01

Y IN	P LBS/IN
.000	.000
.108	400.400
.217	800.800
.325	1201.200
.433	1601.600
.542	1929.694
.650	2037.342
.758	2133.029
.867	2219.542
.975	2298.760
1.083	2372.015
1.192	2440.292
1.300	2504.338

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
258.00	78.00	37.0	.36D-01

Y IN	P LBS/IN
.000	.000
.108	1229.800
.217	2459.600
.325	3689.400
.433	4499.551
.542	4958.491
.650	5368.010
.758	5740.532
.867	6084.057
.975	6404.091
1.083	6704.611
1.192	6988.595
1.300	7258.342

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
408.00	78.00	37.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	1127.100
.217	2254.200
.325	3381.300
.433	4508.400
.542	5635.500
.650	6762.600
.758	7889.700
.867	9016.800
.975	10143.900
1.083	10664.435

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
498.00	78.00	36.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	2373.800
.217	4747.600
.325	6874.004
.433	8187.909
.542	9377.645
.650	10476.977
.758	11506.405
.867	12479.545
.975	13406.008
1.083	14292.890
1.192	15145.611
1.300	15968.431

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
588.00	78.00	33.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	2802.800
.217	5605.600
.325	7339.763
.433	8742.685
.542	10013.043
.650	11186.862
.758	12266.041
.867	13325.118
.975	14314.355
1.083	15261.328

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
678.00	78.00	33.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	1872.975
.217	3745.950
.325	5618.925
.433	7491.900
.542	9364.875
.650	11237.850
.758	13110.825
.867	14983.800
.975	16856.775
1.083	18729.750

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
768.00	78.00	36.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	3660.800
.217	7321.600
.325	10982.400
.433	14643.200
.542	18304.000
.650	21964.800
.758	25625.600
.867	28251.865
.975	30349.242
1.083	32357.012

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
864.00	78.00	40.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	4118.400
.217	8236.800
.325	12355.200
.433	16473.600
.542	20592.000
.650	24710.400
.758	28828.800
.867	32947.200
.975	37065.600
1.083	41184.000

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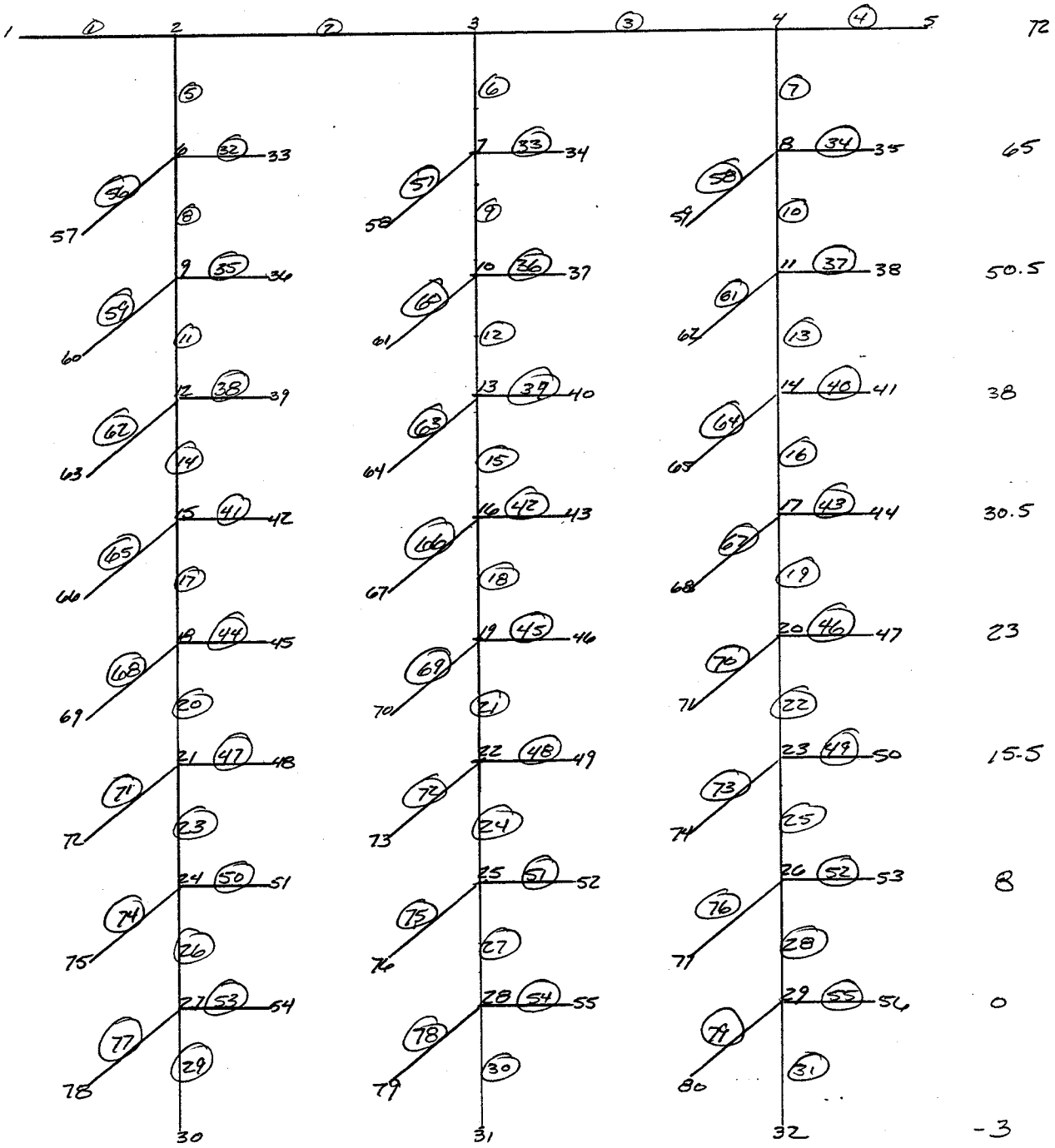
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-6
MADE BY T Stones DATE 1-26-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 44
CALCULATIONS FOR Abutment Design

Shaft Joint & Member No.'s



BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-7
MADE BY T Stones DATE 1-26-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 45
CALCULATIONS FOR Abutment Design

Loads:

Dead Load: DL from substructure will be calculated by M-Strudl

Additional DL in Cap for girder steps will be linearly distributed over members 2 & 3

End dimension $2'\frac{1}{4}'' \times 2'\text{-}6''$ $\quad WA = 2.5(\frac{2.25}{12})(0.15) = 0.07$ k/ft

℄ dimension $9'' \times 2'\text{-}6''$ $\quad WB = 2.5(\frac{9}{12})(0.15) = 0.281$ k/ft

Diaphragm: $1'\text{-}4'' \times 5'\text{-}3''$ $\quad$ Girder Area $= 6.16\ ft^2$

Total Weight $= \left[(1.333)(5.25)(84.5 - 1.333) - 9(6.16)(1.333)\right](0.15)$

2.8" Curtain Walls

$= 76.22$ k

$W = 76.22 / 84.5 = 0.9$ k/ft

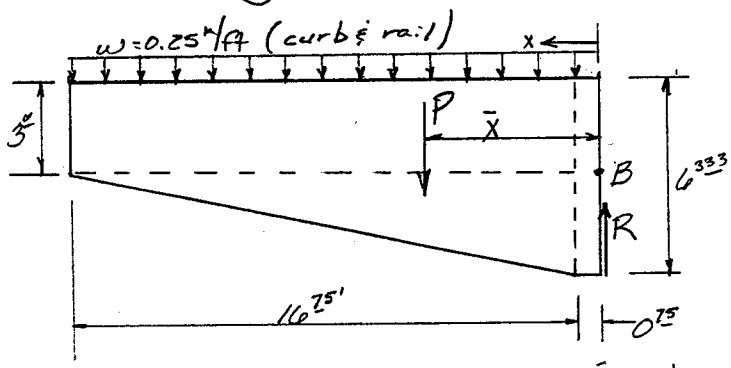
Girder + Slab DL from PSGDES $= 102.1$ k/Girder

For Barrier, Curb, and Railing Load Breakdown see sht. P4

$DL_{BAR\ CURB\ RAIL}$ to Girders A & I $= 25.0$ k

$DL_{BAR\ CURB\ RAIL}$ to Girders B & H $= 5.3$ k

Wingwalls: Assume 1'-3" Thick — actually deeper at Abut connection, but thinner (12")



A	x	Ax
$0.75(6.333) = 4.75$	0.375	1.781
$0.5(16.75)(3.333) = 27.94$	6.333	176.89
$3.0(16.75) = 50.25$	9.125	458.531
82.94		637.101

$\bar{x} = \frac{637.101}{82.94} = 7.684$

$P = 1.25(82.94)(0.15) = 15.546$ k

$R = 15.546 + 0.25(16.75) = 19.75$ k

$M_{B_1} = 15.546(7.684) + 0.25\frac{(16.75)^2}{2} = 154.5$ ft·k

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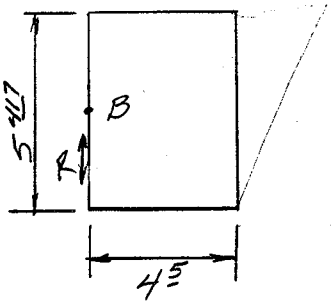
JOB Northern Ave @ New River SHEET NO. A-8
MADE BY T Stones DATE 1-24-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 46
CALCULATIONS FOR Abutment Design

Loads (cont.)

Curtain Wall : 8" Thickness



$R = (5^{4/7})(4^{5})(0^{6/7})(0.15) = 2^{44}$ k

$M_{B_2} = 2^{44}(2^{25}) = 5^{5}$ ft·k

This Moment is in opposite direction from wingwall moment.

Resulting Moment $= 154^{5} - 5^{5} = 149^{0}$ ft·k

Future Wearing Surface and Utility Allowance is modelled as a $2^{5/12}$ overlay

$w = (2^{5}/12)(.15\ k/ft^2)(97/2) = 1.5$ k/ft

Live Load:

Sidewalk Load applied on Upstream Side Only:

$w = (6\ ft)(60\ psf) = 360$ plf

From Beam Diagrams: AISC Manual pg 2-127 for 4 continuous spans: Max Reaction @ 1st support:

$R_A = 0.393\ wl = 0.393(.360)(97) = 13^{72}$ k

R_A will be applied to Girder Location A

Traffic Load: $I = \frac{50}{97+125} = 0.225$

From Continuous Highway Bridge Tables:

$R_{LL\ 95ft\ Span} = 63^{1}$ k

$R_{LL\ 100ft\ Span} = 63^{6}$ k

$R_{LL\ 97ft\ Span} = 63^{1} + 0^{5}(\frac{2}{5}) = 63^{3}$ k

$I = .225(63^{3}) = 14^{24}$

$R_{LL+I} = 77^{54}$ k

Tire Load $= \frac{R_{LL+I}}{2}$ $T = 38^{77}$ k

JOB Northern Ave @ New River SHEET NO. A-9

MADE BY T Stones DATE 1-24-90 JOB NO. 33-8962

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 47

CALCULATIONS FOR Abutment Design

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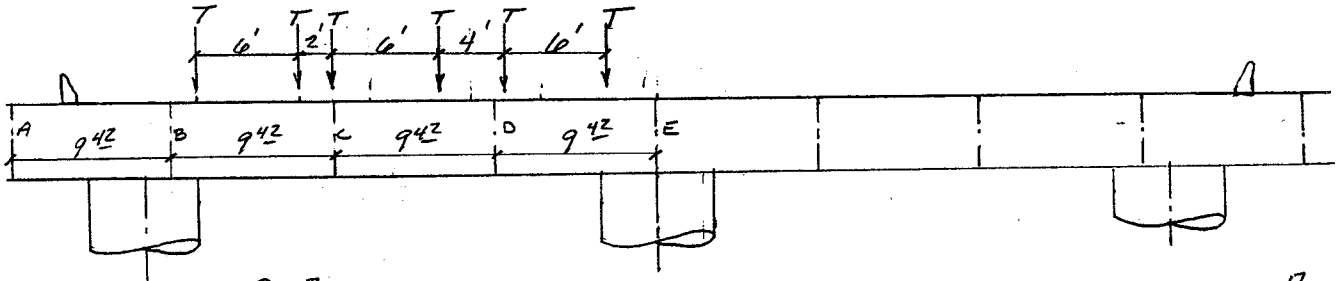
Live Load (cont.)

No. of Traffic Lanes = $68/12 = 5^{67}$ use 5 lanes

Reduction for improbability = $5(.75) = 3^{75}$ lanes

I'll conservatively load a maximum of 4 lanes.

I. Max Moment in Shaft, Max Positive Moment in Loaded Span & Max Neg. Mom in Unloaded Span



$R_B = T\left[\frac{8+2}{9^{42}}\right] = 1^{062}\,T$ — 41^{17} k

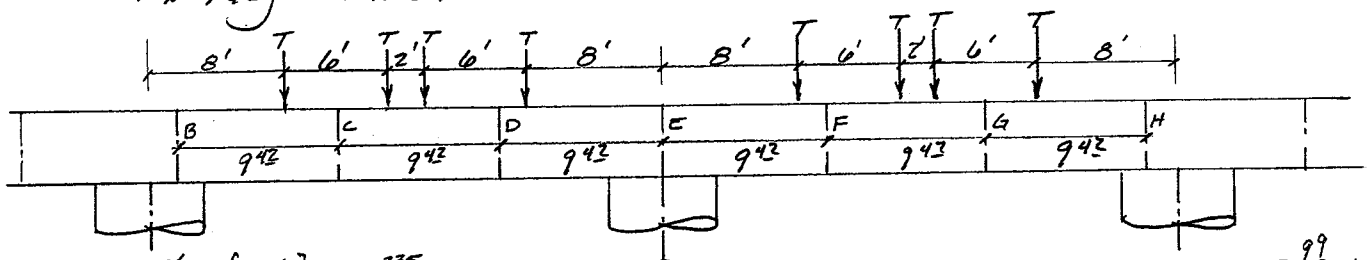
$R_C = T\left[\frac{1^{42}+7^{42}+3^{42}}{9^{42}}\right]+T = 2^{302}\,T$ — 89^{25} k

$R_D = T\left[\frac{6+8^{84}+2^{84}}{9^{42}}\right] = 1^{877}\,T$ — 72^{77} k

$R_E = T\left[\frac{0^{58}+6^{58}}{9^{42}}\right] = 0^{760}\,T$ — 29^{47} k

Total 6.00 T ✓

II. Max Neg Moment Over Center Shaft



$R_H = R_B = T/9^{42}\left[3^{16}\right] = 0^{335}\,T$ — 12^{99} k

$R_G = R_C = T/9^{42}\left[6^{26}+6^{58}+4^{58}\right] = 1^{849}\,T$ — 71^{69} k

$R_F = R_D = T/9^{42}\left[2^{84}+4^{84}+8\right] = 1^{665}\,T$ — 64^{55} k

$R_E = T/9^{42}\left[1^{42}\right](2) = 0^{301}\,T$ — 11^{67} k

Total = $2(3^{849}+0^{304}) = 8^{00}$ ✓

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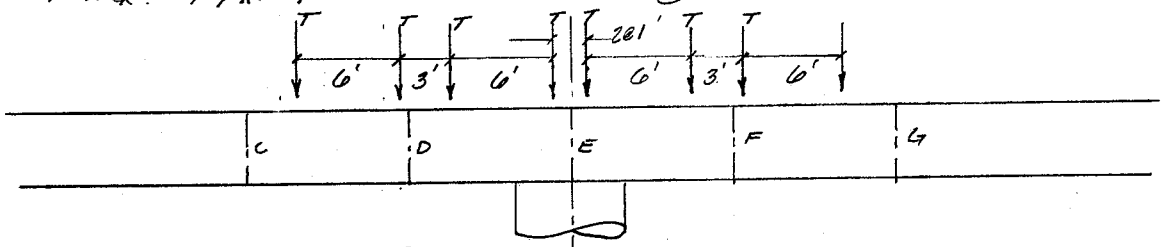
JOB Northern Ave @ New River SHEET NO. A-10
MADE BY T Stones DATE 1-24-90 JOB NO. 33-8962
CHECKED BY LArnold DATE 2/90 SEC. SHT. NO. 48
CALCULATIONS FOR Abutment Design

Live Load (Cont.)

III Max. Axial Load Over Center Column



$R_C = R_G = \frac{T}{9.42}\left[1.0^{58} + 6^{58}\right] = 0.76\,T$ — 29^{47} k

$R_D = R_F = \frac{T}{9.42}\left[8^{84} + 2^{84} + 7 + 1\right] = 2^{08}\,T$ — 81^{00} k

$R_E = \frac{T}{9.42}\left[2^{42} + 8^{42}\right](2) = 2^{30}\,T$ — 89^{23} k

Total $= 2(2^{84}) + 2^{30} = 8^{08}$ ✓

Differential Settlement of 3/32 (0.008 ft) will be analyzed for shafts 1 & 2 (Shaft 3 will be similar to Shaft 1)

Thermal Forces are not considered in abutment design (Ref AASHTO 7.4). Min Temperature reinforcement will be provided per AASHTO 7.4.2

Wind:

Force from superstructure: AASHTO 3.15.2.1.3

$W = 50$ psf Transverse
$= 12$ psf Longitudinal

Effective Area: Structure $\frac{10^5}{12} + 5^{25} = 6^{12.5}$ ft
Barrier 2^{67} ft
8^{80} ft

Length $= \frac{97}{2} + 2(1^{25}) = 51^{0}$ ft

Area = 449 ft

$W_T = 0.05\,(449) = 22^{45}$ k

$W_L = 0.012\,(449) = 5^{39}$ k

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JOB Northern Ave @ New River SHEET NO. A-11
MADE BY T Stones DATE 1-24-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 49
CALCULATIONS FOR Abutment Design

Wind on Superstructure (cont.)

W_T & W_L will act at ℄ of superstructure Area

distance of cap c.g. is $\frac{8^{80}}{2} + (4^{0} - 3^{36}) = 5^{04}$ ft

Effective W_T & W_L Applied at c.g. cap

$$\left.\begin{aligned} W_T &= 22^{45}\left(\frac{26+5^{04}}{26}\right) = 26^{80}\ k \\ W_L &= 5^{39}\left(\frac{26+5^{04}}{26}\right) = 6^{44}\ k \end{aligned}\right\}\ \text{Both applied @ joint \# 1}$$

Wind on Substructure: AASHTO 3.15.2.2.

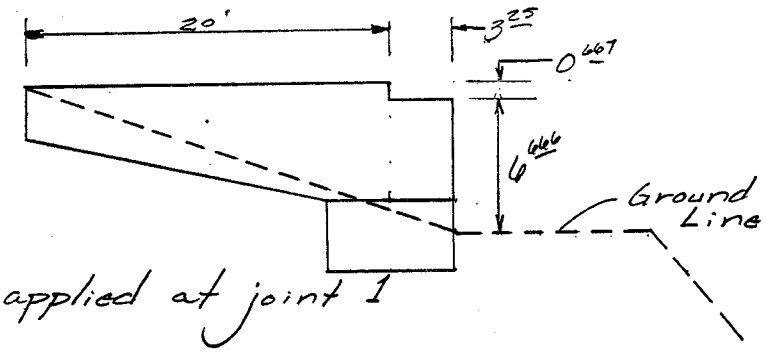
$W = 40$ psf

Effective Area

$A = (23^{25})(7^{33})(0^{5}) - 3^{25}(0^{67})$

$A = 83^{07}$ ft²

$W = 83^{07}(0.04) = 3^{33}$ k applied at joint 1



Overturning: AASHTO 3.15.3 Group 2 and 5

$W_{OT} = 20$ psf acting @ windward 1/4 pt

Effective Area $= (97/2)(84^{83}) = 4114^{25}$ ft²

$W = 0.02(4114^{25}) = 82^{3}$ k acting up @ $30 - \frac{84^{5}}{4} = 8^{88}$ ft from left end of member 2

Group 3 and 6 Forces will be 30% of those determined from the loading above.

Forces from Stream current will not impact Abutment Analysis

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JOB Northern Ave @ New River SHEET NO. A-12
MADE BY T. Stones DATE 1-25-90 JOB NO. 33-8962
CHECKED BY L. Arnold DATE 2/90 SEC. SHT. NO. 50
CALCULATIONS FOR Abutment Design

Wind on Live Load:

$W_{LL} = .40$ plf longitudinal
$W_{LT} = 100$ plf transverse
} acting 6° ft above deck

distance from c.g. of cap $= 6 + 6^{\underline{333}} + 4 - 3^{\underline{36}} = 12^{97}$

Effective W_{LL} & W_{LT} acting at c.g. of cap at joint 3

$$W_{LL} = 0.04\left(\tfrac{97}{2}\right)\left(\frac{26 + 12^{97}}{26}\right) = 2^{91}\ k$$

$$W_{LT} = 0.1\left(\tfrac{97}{2}\right)\left(\frac{26 + 12^{97}}{26}\right) = 7^{27}\ k$$

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JOB Northern Ave @ New River SHEET NO. A-13
MADE BY T Stones DATE 1-25-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 51
CALCULATIONS FOR Abutment Design

Determine Bearing Capacity (Ref NAVFAC DM 7.1)

Devided soil into 2 layers (see sht 5-3)

Layer 1 1056 to 1073.8 $\gamma_{sat} = 63$ pcf ϕ 33°

Layer 2 below 1056 $\gamma_{sat} = 85$ pcf $\phi = 38°$

From NAVFAC DM 7.2 pg 192 to 194

Bearing Capacity $Q_{ult} = P_T N_f A_T$ No side friction

From Fig 1 pg 194

$\phi = 33°$ $N_f = 17$ $\phi = 38°$ $N_f = 43$

$A_T = \frac{\pi(6.5)^2}{4} = 33.2 \text{ ft}^2$

Check Capacity @ Elev 997±

$P_{T_{997}} = 17.8(63) + 59(85) = 6139$ psf

$Q_{ult} = 6139\,(43)(33.2)/1000 = 8764^k$ FS = 10.8

$Q_{req} = 573 + \frac{205}{1.225} = 740$ Group I No Impact Below Ground

$= (573 + \frac{205}{1.225} + 277)/1.25 = 814$ Group IV

$= (573 + \frac{205}{1.225} + 3 + 1 + (1.3)(56) + 227)/1.4 = 741$ Group VI

For Factor of safety of 3 $Q_{req} = 3(814) = 2442^k$

Can decrease shaft depth considerably

Elev.	P_T	Q_{ult}	FS
1056	$17.8(63) = 1121$ pcf	1601	1.97
1042	$17.8(63) + 14(85) = 2311$	3299	4.05
1049	$17.8(63) + 7(85) = 1716$	2450	3.01 ✓

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-13 A
MADE BY T Stones DATE 1-26-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 52
CALCULATIONS FOR Abutment Design

Check settlement (Ref NAVFAC DM 7.2 pp 207-209)

1. Settlement due to axial deformation

$W_s = (Q_p + \alpha_s Q_s) L / A E_p$

$Q_p = 814^k$ $\quad Q_s = 0$ No Skin Friction

$L_{1044} = 1077.2 - 3^{36} - 1049 = 24^{84}$ ft

$A = 33.2$ ft^2 $\quad E_p = 57\sqrt{4000}\,144 = 519,120$ ksf 0.00363

$W_s = (814^k)(24^{84}\text{ ft}) / (33.2\text{ ft}^2)(519,120\text{ ksf}) = 0.00117$ ft $= 0.014$ in

2. Settlement of pile tip by end bearing only

$W_{PP} = \frac{C_P Q_P}{B q_o}$ $\quad C_P = 0.02$ $\quad B = 6.5$ ft

$q_o = \frac{2450^k}{33.2\text{ ft}^2} = 73.8$ ksf (FS = 3.0)

$W_{PP} = \frac{0.02(814)}{(6.5)(73.8)} = 0.034$ ft $= 0.407$ in

3. Settlement for friction load

$W_{SP} = 0$ End Bearing Only

Total Settlement $= 0.014 + 0.407 = 0.421$ in

Differential Settlement $= TS/2 = 0.211$ in $\approx$ this differential settlement causes excessive moments.

Determine Settlements @ Tip Elev 997±

$L = 1077.2 - 3^{36} - 997 = 76^{84}$ ft $\quad W_s = \frac{814(76^{84})}{33.2(519,120)} = 0.0036$ ft $= 0.044$ in

$q_o = \frac{8764}{33.2} = 264$ ksf $\quad W_{PP} = \frac{0.02(814)}{(6.5)(264)} = 0.009$ ft $= 0.114$ in

$TS = 0.044 + 0.114 = 0.158$ in

Differential Settlement $= 0.158/2 = 0.08$ in

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-13B
MADE BY T Stones DATE 1-26-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 53
CALCULATIONS FOR Abutment Design

2/90

```
*********************************************************************
*                                                                   *
* TYPE OF THE STRUCTURE                  : SPACE FRAME              *
* RESTART STATUS                         :   NONE GIVEN             *
* DESIGNATED UNITS                       : KIPS    FEET   DEGREES   *
* NO. OF JOINTS DEFINED                  :    0                     *
* NO. OF MEMBERS DEFINED                 :    0                     *
* NO. OF PLATE ELEMENTS DEFINED          :    0                     *
* NO. OF DIFF. MATERIAL DEFINED          :    0                     *
* NO. OF DIFF. SECTION PROPERTIES DEFINED:    0                     *
* NO. OF DIFF. TYPES OF SPRINGS DEFINED  :    0                     *
*                                                                   *
*********************************************************************
```

```
*****************************
* LISTING OF THE INPUT FILE *
*****************************

  1) $ THE FOLLOWING DATA IS THE INPUT FILE FOR THE ANALYSIS OF THE
  2) $ NORTHERN AVENUE BRIDGE, ABUTMENT BENTS
  3) $
  4) TITLE ANALYSIS OF ABUTMENT BENT CAPS
  5) TYPE SPACE FRAME
  6) UNIT KIP FEET DEGREE
  7) REPORT DEVICE NAABTOUT
  8) REPORT PAGE 14
  9) $ FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
 10) $ NORTHERN AVENUE BRIDGE OVER THE NEW RIVER
 11) $ BRW, INC.; PHOENIX, ARIZONA; JANUARY, 1990
 12) REPORT LOGO
 13) PRINT INPUT
 14) OUTPUT DEC 2
 15) OUTPUT BY MEMBER
 16) JOINT COORDINATES
 17) 1 -42.25 77.20 0.0 F
 18) 2 -30.0 77.20 0.0
 19) 3 0.0 77.20 0.0
 20) 4 30.0 77.20 0.0
 21) 5 42.25 72.20 0.0 F
 22) 6 -30. 65. 0.
 23) 7 0. 65. 0.
 24) 8 30. 65. 0.
 25) 9 -30. 50.5 0.
 26) 10 0. 50.5 0.
 27) 11 30. 50.5 0.
 28) 12 -30. 38. 0.
 29) 13 0. 38. 0.
 30) 14 30. 38. 0.
 31) 15 -30. 30.5 0.
 32) 16 0. 30.5 0.
 33) 17 30. 30.5 0.
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* LISTING OF THE INPUT FILE *
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34) 18 -30. 23. 0.
35) 19 0. 23. 0.
36) 20 30. 23. 0.
37) 21 -30. 15.5 0.
38) 22 0. 15.5 0.
39) 23 30. 15.5 0.
40) 24 -30. 8. 0.
41) 25 0. 8. 0.
42) 26 30. 8. 0.
43) 27 -30. 0. 0.
44) 28 0. 0. 0.
45) 29 30. 0. 0.
46) 30 -30. -3. 0.
47) 31 0. -3. 0.
48) 32 30. -3. 0.
49) 33 -29.9 65. 0.
50) 34 0.1 65. 0.
51) 35 30.1 65. 0.
52) 36 -29.9 50.0 0.
53) 37 0.1 50.5 0.
54) 38 30.1 50.5 0.
55) 39 -29.9 38. 0.
56) 40 0.1 38. 0.
57) 41 30.1 38. 0.
58) 42 -29.9 30.5 0.
59) 43 0.1 30.5 0.
60) 44 30.1 30.5 0.
61) 45 -29.9 23. 0.
62) 46 0.1 23. 0.
63) 47 30.1 23. 0.
64) 48 -29.9 15.5 0.
65) 49 0.1 15.5 0.
66) 50 30.1 15.5 0.
67) 51 -29.9 8. 0.
68) 52 0.1 8. 0.
69) 53 30.1 8. 0.
70) 54 -29.9 0. 0.
71) 55 0.1 0. 0.
72) 56 30.1 0. 0.
73) 57 -30. 65. 0.1
74) 58 0. 65. 0.1
75) 59 30. 65. 0.1
76) 60 -30. 50.5 0.1
77) 61 0. 50.5 0.1
78) 62 30. 50.5 0.1
79) 63 -30. 38. 0.1
80) 64 0. 38. 0.1
81) 65 30. 38. 0.1
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* LISTING OF THE INPUT FILE *
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 82) 66 -30. 30.5 0.1
 83) 67 0. 30.5 0.1
 84) 68 30. 30.5 0.1
 85) 69 -30. 23. 0.1
 86) 70 0. 23. 0.1
 87) 71 30. 23. 0.1
 88) 72 -30. 15.5 0.1
 89) 73 0. 15.5 0.1
 90) 74 30. 15.5 0.1
 91) 75 -30. 8. 0.1
 92) 76 0. 8. 0.1
 93) 77 30. 8. 0.1
 94) 78 -30. 0. 0.1
 95) 79 0. 0. 0.1
 96) 80 30. 0. 0.1
 97) $
 98) JOINT RELEASE
 99) 30 TO 80 AS SUPPORT
100) 30 TO 32 MOM X MOM Y MOM Z KFX 494.2 KFZ 494.2
101) 33 TO 35 KFX 28.5
102) 57 TO 59 KFZ 28.5
103) 36 TO 38 KFX 80.5
104) 60 TO 62 KFZ 80.5
105) 39 TO 41 KFX 130.4
106) 63 TO 65 KFZ 130.4
107) 42 TO 44 KFX 171.5
108) 66 TO 68 KFZ 171.5
109) 45 TO 47 KFX 183.1
110) 69 TO 71 KFZ 183.1
111) 48 TO 50 KFX 224.8
112) 72 TO 74 KFZ 224.8
113) 51 TO 53 KFX 388.3
114) 75 TO 77 KFZ 388.3
115) 54 TO 56 KFX 494.2
116) 78 TO 80 KFZ 494.2
117) $
118) MEMBER INCIDENCES
119) 1 1 2
120) 2 2 3
121) 3 3 4
122) 4 4 5
123) 5 6 2
124) 6 7 3
125) 7 8 4
126) 8 9 6
127) 9 10 7
128) 10 11 8
129) 11 12 9
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* LISTING OF THE INPUT FILE *
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130) 12 13 10
131) 13 14 11
132) 14 15 12
133) 15 16 13
134) 16 17 14
135) 17 18 15
136) 18 19 16
137) 19 20 17
138) 20 21 18
139) 21 22 19
140) 22 23 20
141) 23 24 21
142) 24 25 22
143) 25 26 23
144) 26 27 24
145) 27 28 25
146) 28 29 26
147) 29 30 27
148) 30 31 28
149) 31 32 29
150) 32 6 33
151) 33 7 34
152) 34 8 35
153) 35 9 36
154) 36 10 37
155) 37 11 38
156) 38 12 39
157) 39 13 40
158) 40 14 41
159) 41 15 42
160) 42 16 43
161) 43 17 44
162) 44 18 45
163) 45 19 46
164) 46 20 47
165) 47 21 48
166) 48 22 49
167) 49 23 50
168) 50 24 51
169) 51 25 52
170) 52 26 53
171) 53 27 54
172) 54 28 55
173) 55 29 56
174) 56 6 57
175) 57 7 58
176) 58 8 59
177) 59 9 60
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* LISTING OF THE INPUT FILE *
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178) 60 10 61
179) 61 11 62
180) 62 12 63
181) 63 13 64
182) 64 14 65
183) 65 15 66
184) 66 16 67
185) 67 17 68
186) 68 18 69
187) 69 19 70
188) 70 20 71
189) 71 21 72
190) 72 22 73
191) 73 23 74
192) 74 24 75
193) 75 25 76
194) 76 26 77
195) 77 27 78
196) 78 28 79
197) 79 29 80
198) $
199) MEMBER RELEASE
200) 32 TO 55 START FORCE Y Z MOM X Y Z
201) 56 TO 79 START FORCE X Y MOM X Y Z
202) $
203) MATERIAL PROPERTY
204) CONCR E 519120. DEN .15 CT 0.000006 POI 0.2 ALL
205) $
206) MEMBER PROPERTY
207) CAP AX 34.45 IX 417.77 IY 119.41 IZ 252.63 1 TO 4 $ CAP MEMBERS
208) SHAFT AX 33.18 IX 175.25 IY 87.62 IZ 87.62 5 TO 31 $ SHAFT MEMBER
     S
209) CONN AX 33.18 IX 175.25 IY 87.62 IZ 87.62 32 TO 79 $ FALSE CONNEC
     TORS
210) $
211) STIFFNESS ANALYSIS
212) $
213) LOAD DL $ ADDITIONAL D.L. FROM SUPERSTR., BARRIERS, AND WALLS
214) DEADLOAD Y -1.0 $ DEAD LOAD OF 1.0 G'S IN THE GLOBAL Y DIRECTION
215) JOINT LOADS
216) 3 FORCE Y -102.7 $ GIRDER 5 D.L.
217) 1 FORCE Y -22.2 MOM X -149.0 $ WINGWALL + CURTAIN WALL FORCE & MO
     MENT
218) 5 FORCE Y -22.2 MOM X 149.0 $ WINGWALL + CURTAIN WALL FORCE & MOM
     ENT
219) MEMBER LOADS
220) 1 FORCE Y CONC P -127.7 L 4.58 $ GIRDER 1 D.L. + BAR. + CURB + RA
     IL
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* LISTING OF THE INPUT FILE *
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221) 2 FORCE Y CONC P -108.0 L 1.75 $ GIRDER 2 D.L. + BAR.
222) 2 FORCE Y CONC P -102.7 L 11.17 $ GIRDER 3 D.L.
223) 2 FORCE Y CONC P -102.7 L 20.58 $ GIRDER 4 D.L.
224) 3 FORCE Y CONC P -102.7 L 9.42 $ GIRDER 6 D.L.
225) 3 FORCE Y CONC P -102.7 L 18.83 $ GIRDER 7 D.L.
226) 3 FORCE Y CONC P -108.0 L 28.25 $ GIRDER 8 D.L. + BAR.
227) 4 FORCE Y CONC P -127.7 L 7.67 $ GIRDER 9 D.L. + BAR. + CURB + RA
     IL
228) 2 FORCE Y LIN WA -0.07 WB -0.28 $ ADDITIONAL D.L. FOR GIRDER STEP
     S
229) 3 FORCE Y LIN WA -0.28 WB -0.07 $ ADDITIONAL D.L. FOR GIRDER STEP
     S
230) 1 TO 4 FORCE Y UNIF W -0.9 $ DIAPHRAGM D.L.
231) 1 TO 4 FORCE Y UNIF W -1.5 $ FWS AND UTILITY ALLOWANCE
232) $
233) LOAD SETTLE_1 $ DIFFERENTIAL SETTLEMENT OF AT SHAFT 1
234) JOINT DISPLACEMENT
235) 30 DIS Y -0.006
236) $
237) LOAD SETTLE_2 $ DIFFERENTIAL SETTLEMENT OF AT SHAFT 2
238) JOINT DISPLACEMENT
239) 31 DIS Y -0.006
240) $
241) LOAD SDWK_LL
242) MEMBER LOAD
243) 1 FORCE Y CONC P -13.72 L 4.58 $ SIDEWALK L.L. ON ONE SIDE ONLY
244) $
245) LOAD LL_I_1
246) $ LIVE LOAD PLUS IMPACT FOR MAX POS MOM IN LOADED SPAN
247) $ & MAX NEG MOM IN UNLOADED SPAN & MAX MOM IN SHAFTS
248) MEMBER LOAD
249) 2 FORCE Y CONC P -41.17 L 1.75 $ L.L. AT GIRDER 2
250) 2 FORCE Y CONC P -89.25 L 11.17 $ L.L. AT GIRDER 3
251) 2 FORCE Y CONC P -72.77 L 20.58 $ L.L. AT GIRDER 4
252) JOINT LOADS
253) 3 FORCE Y -29.47 $ L.L. AT GIRDER 5
254) $
255) LOAD LL_I_2
256) $ LIVE LOAD PLUS IMPACT FOR MAX NEG MOM OVER THE CENTER SHAFT
257) JOINT LOADS
258) 3 FORCE Y -11.67 $ L.L. AT GIRDER 5
259) MEMBER LOADS
260) 2 FORCE Y CONC P -12.99 L 1.75 $ L.L. AT GIRDER 2
261) 2 FORCE Y CONC P -71.69 L 11.17 $ L.L. AT GIRDER 3
262) 2 FORCE Y CONC P -64.55 L 20.58 $ L.L. AT GIRDER 4
263) 3 FORCE Y CONC P -64.55 L 9.42 $ L.L. AT GIRDER 6
264) 3 FORCE Y CONC P -71.69 L 18.83 $ L.L. AT GIRDER 7
265) 3 FORCE Y CONC P -12.99 L 28.25 $ L.L. AT GIRDER 8
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* LISTING OF THE INPUT FILE *
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266) $
267) LOAD LL_I_3
268) $ LIVE LOAD PLUS IMPACT FOR MAX AXIAL LOAD IN SHAFTS
269) JOINT LOADS
270) 3 FORCE Y -89.23 $ L.L. AT GIRDER 5
271) MEMBER LOADS
272) 2 FORCE Y CONC P -29.47 L 11.17 $ L.L. AT GIRDER 3
273) 2 FORCE Y CONC P -81.0 L 20.58 $ L.L. AT GIRDER 4
274) 3 FORCE Y CONC P -81.0 L 9.42 $ L.L. AT GIRDER 6
275) 3 FORCE Y CONC P -29.47 L 18.83 $ L.L.AT GIRDER 7
276) $
277) LOAD WIND
278) JOINT LOADS
279) 1 FORCE X 26.8 Z 6.44 $ WIND FORCE FROM SUPERSTRUCTURE
280) 1 FORCE X 3.33 $ WIND FORCE FROM SUBSTRUCTURE
281) $
282) LOAD OVERTRN
283) MEMBER LOADS
284) 2 FORCE Y CONC P 82.3 L 8.88 $ OVERTURNING FORCE FOR AASHTO GROUP
       2
285) $
286) LOAD WIND_LL
287) JOINT LOADS
288) 3 FORCE X 7.27 Z 2.91 $ WIND ON A MOVING LIVE LOAD
289) $
290) SELECT LOAD 1 TO 10
291) LIST MEM FOR 1 SECT NS 2 6.125 9.0
292) LIST MEM FOR 2 3 SECT NS 5 3.25 7.5 15.0 22.5 26.75
293) LIST MEM FOR 4 SECT NS 2 3.25 6.125
294) LIST MEM FOR 5 TO 7
295) LIST MEM FOR 29 TO 31
296) UNIT KIP INCH DEGREE
297) LIST JOI DISP 1 TO 5
298) LIST JOI DISP 30 TO 32
299) FINISH
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* RESULTS OF ANALYSIS *
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TYPE OF THE PROBLEM : SPACE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
1	DL	1	-0.00	22.20	-0.00	149.00	0.00	-0.00
		6.13	-0.00	196.25	-0.00	149.00	0.00	-475.22
		9.00	-0.00	218.01	-0.00	149.00	0.00	-1070.72
CAP		2	-0.00	242.60	-0.00	149.00	0.00	-1819.21
	SETTLE_1	1	-0.00	0.00	0.00	0.00	0.00	0.00
		6.13	-0.00	0.00	0.00	0.00	0.00	-0.00
		9.00	-0.00	0.00	0.00	0.00	0.00	-0.00
		2	-0.00	0.00	0.00	0.00	0.00	-0.00
	SETTLE_2	1	-0.00	0.00	0.00	0.00	0.00	0.00
		6.13	-0.00	0.00	0.00	0.00	0.00	0.00
		9.00	-0.00	0.00	0.00	0.00	0.00	-0.00
		2	-0.00	0.00	0.00	0.00	0.00	0.00
	SDWK_LL	1	0.00	-0.00	0.00	0.00	0.00	-0.00
		6.13	0.00	13.72	0.00	0.00	0.00	-21.20
		9.00	0.00	13.72	0.00	0.00	0.00	-60.64
		2	0.00	13.72	0.00	0.00	0.00	-105.23
	LL_I_1	1	0.00	-0.00	0.00	0.00	0.00	0.00
		6.13	0.00	-0.00	0.00	0.00	0.00	0.00
		9.00	0.00	-0.00	0.00	0.00	0.00	0.00
		2	0.00	-0.00	0.00	0.00	0.00	0.00
	LL_I_2	1	-0.00	-0.00	0.00	0.00	0.00	-0.00
		6.13	-0.00	-0.00	0.00	0.00	0.00	0.00
		9.00	-0.00	-0.00	0.00	0.00	0.00	0.00
		2	-0.00	-0.00	0.00	0.00	0.00	0.00
	LL_I_3	1	0.00	-0.00	0.00	0.00	0.00	-0.00
		6.13	0.00	-0.00	0.00	0.00	0.00	0.00
		9.00	0.00	-0.00	0.00	0.00	0.00	0.00
		2	0.00	-0.00	0.00	0.00	0.00	0.00
	WIND	1	-30.13	0.00	-6.44	0.00	0.00	0.00
		6.13	-30.13	0.00	-6.44	0.00	-39.45	-0.00
		9.00	-30.13	0.00	-6.44	0.00	-57.96	-0.00

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
		2	-30.13	0.00	-6.44	0.00	-78.89	0.00
	OVERTRN	1	0.00	0.00	0.00	0.00	0.00	0.00
		6.13	0.00	0.00	0.00	0.00	0.00	-0.00
		9.00	0.00	0.00	0.00	0.00	0.00	-0.00
		2	0.00	0.00	0.00	0.00	0.00	-0.00
	WIND_LL	1	0.00	0.00	-0.00	0.00	0.00	0.00
		6.13	0.00	0.00	-0.00	0.00	-0.00	0.00
		9.00	0.00	0.00	-0.00	0.00	-0.00	0.00
		2	0.00	0.00	-0.00	0.00	0.00	-0.00

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* RESULTS OF ANALYSIS *

TYPE OF THE PROBLEM : SPACE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
2	DL	2	9.49	-330.01	0.06	66.71	-1.11	-1683.41
		3.25	9.49	-197.15	0.06	66.71	-0.91	-813.25
		7.50	9.49	-164.53	0.06	66.71	-0.65	-44.62
		15.00	9.49	-3.96	0.06	66.71	-0.20	579.27
		22.50	9.49	157.00	0.06	66.71	0.25	193.56
		26.75	9.49	190.20	0.06	66.71	0.51	-544.20
CAP		3	9.49	215.66	0.06	66.71	0.70	-1203.70
	SETTLE_1	2	71.76	81.31	0.00	0.00	0.00	982.97
		3.25	71.76	81.31	0.00	0.00	0.00	718.73
		7.50	71.76	81.31	0.00	0.00	0.00	373.18
		15.00	71.76	81.31	0.00	0.00	0.00	-236.62
		22.50	71.76	81.31	0.00	0.00	0.00	-846.41
		26.75	71.76	81.31	0.00	0.00	0.00	-1191.96
		3	71.76	81.31	0.00	0.00	0.00	-1456.21
	SETTLE_2	2	-153.86	-131.51	0.00	0.00	0.00	-1416.61
		3.25	-153.86	-131.51	0.00	0.00	0.00	-989.19
		7.50	-153.86	-131.51	0.00	0.00	0.00	-430.27
		15.00	-153.86	-131.51	0.00	0.00	0.00	556.08
		22.50	-153.86	-131.51	0.00	0.00	0.00	1542.42
		26.75	-153.86	-131.51	0.00	0.00	0.00	2101.35
		3	-153.86	-131.51	0.00	0.00	0.00	2528.77
	SDWK_LL	2	5.03	-1.72	0.00	0.00	0.00	-50.79
		3.25	5.03	-1.72	0.00	0.00	0.00	-45.21
		7.50	5.03	-1.72	0.00	0.00	0.00	-37.91
		15.00	5.03	-1.72	0.00	0.00	0.00	-25.03
		22.50	5.03	-1.72	0.00	0.00	0.00	-12.14
		26.75	5.03	-1.72	0.00	0.00	0.00	-4.84
		3	5.03	-1.72	0.00	0.00	0.00	0.74
	LL_I_1	2	-39.82	-117.01	0.00	0.00	0.00	-344.86
		3.25	-39.82	-75.84	0.00	0.00	0.00	-26.35
		7.50	-39.82	-75.84	0.00	0.00	0.00	295.95
		15.00	-39.82	13.41	0.00	0.00	0.00	522.88
		22.50	-39.82	86.18	0.00	0.00	0.00	282.56
		26.75	-39.82	86.18	0.00	0.00	0.00	-83.73

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
		3	-39.82	86.18	0.00	0.00	0.00	-363.83
	LL_I_2	2	-34.20	-75.37	0.00	0.00	0.00	-314.97
		3.25	-34.20	-62.38	0.00	0.00	0.00	-89.51
		7.50	-34.20	-62.38	0.00	0.00	0.00	175.58
		15.00	-34.20	9.31	0.00	0.00	0.00	368.82
		22.50	-34.20	73.86	0.00	0.00	0.00	175.02
		26.75	-34.20	73.86	0.00	0.00	0.00	-138.90
		3	-34.20	73.86	0.00	0.00	0.00	-378.96
	LL_I_3	2	-38.64	-52.63	0.00	0.00	0.00	-355.82
		3.25	-38.64	-52.63	0.00	0.00	0.00	-184.78
		7.50	-38.64	-52.63	0.00	0.00	0.00	38.90
		15.00	-38.64	-23.16	0.00	0.00	0.00	320.74
		22.50	-38.64	57.84	0.00	0.00	0.00	338.91
		26.75	-38.64	57.84	0.00	0.00	0.00	93.08
		3	-38.64	57.84	0.00	0.00	0.00	-94.90
	WIND	2	-17.07	3.32	-3.82	-1.37	35.76	64.41
		3.25	-17.07	3.32	-3.82	-1.37	23.34	53.64
		7.50	-17.07	3.32	-3.82	-1.37	7.09	39.55
		15.00	-17.07	3.32	-3.82	-1.37	-21.57	14.68
		22.50	-17.07	3.32	-3.82	-1.37	-50.24	-10.19
		26.75	-17.07	3.32	-3.82	-1.37	-66.49	-24.28
		3	-17.07	3.32	-3.82	-1.37	-78.91	-35.05
	OVERTRN	2	15.42	56.06	0.00	0.00	0.00	136.72
		3.25	15.42	56.06	0.00	0.00	0.00	-45.48
		7.50	15.42	56.06	0.00	0.00	0.00	-283.75
		15.00	15.42	-26.24	0.00	0.00	0.00	-200.56
		22.50	15.42	-26.24	0.00	0.00	0.00	-3.78
		26.75	15.42	-26.24	0.00	0.00	0.00	107.72
		3	15.42	-26.24	0.00	0.00	0.00	192.99
	WIND_LL	2	2.01	0.65	0.96	0.05	-8.60	10.38
		3.25	2.01	0.65	0.96	0.05	-5.49	8.25
		7.50	2.01	0.65	0.96	0.05	-1.42	5.48
		15.00	2.01	0.65	0.96	0.05	5.76	0.58
		22.50	2.01	0.65	0.96	0.05	12.94	-4.32
		26.75	2.01	0.65	0.96	0.05	17.01	-7.10
		3	2.01	0.65	0.96	0.05	20.12	-9.22
3	DL	3	-10.21	-224.59	0.06	66.71	-0.70	-1316.84
		3.25	-10.21	-199.12	0.06	66.71	-0.51	-628.33
		7.50	-10.21	-165.93	0.06	66.71	-0.25	147.36

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
		15.00	-10.21	-4.96	0.06	66.71	0.20	600.03
		22.50	-10.21	155.61	0.06	66.71	0.65	43.09
		26.75	-10.21	188.23	0.06	66.71	0.91	-687.60
CAP		4	-10.21	321.09	0.06	66.71	1.11	-1528.76
	SETTLE_1	3	81.15	-49.39	0.00	0.00	0.00	-1056.90
		3.25	81.15	-49.39	0.00	0.00	0.00	-896.38
		7.50	81.15	-49.39	0.00	0.00	0.00	-686.47
		15.00	81.15	-49.39	0.00	0.00	0.00	-316.05
		22.50	81.15	-49.39	0.00	0.00	0.00	54.37
		26.75	81.15	-49.39	0.00	0.00	0.00	264.28
		4	81.15	-49.39	0.00	0.00	0.00	424.80
	SETTLE_2	3	-153.86	131.50	0.00	0.00	0.00	2528.60
		3.25	-153.86	131.50	0.00	0.00	0.00	2101.23
		7.50	-153.86	131.50	0.00	0.00	0.00	1542.36
		15.00	-153.86	131.50	0.00	0.00	0.00	556.11
		22.50	-153.86	131.50	0.00	0.00	0.00	-430.13
		26.75	-153.86	131.50	0.00	0.00	0.00	-989.00
		4	-153.86	131.50	0.00	0.00	0.00	-1416.37
	SDWK_LL	3	2.20	-0.88	0.00	0.00	0.00	-14.30
		3.25	2.20	-0.88	0.00	0.00	0.00	-11.44
		7.50	2.20	-0.88	0.00	0.00	0.00	-7.70
		15.00	2.20	-0.88	0.00	0.00	0.00	-1.10
		22.50	2.20	-0.88	0.00	0.00	0.00	5.50
		26.75	2.20	-0.88	0.00	0.00	0.00	9.24
		4	2.20	-0.88	0.00	0.00	0.00	12.10
	LL_I_1	3	-5.27	0.38	0.00	0.00	0.00	-58.94
		3.25	-5.27	0.38	0.00	0.00	0.00	-60.17
		7.50	-5.27	0.38	0.00	0.00	0.00	-61.77
		15.00	-5.27	0.38	0.00	0.00	0.00	-64.60
		22.50	-5.27	0.38	0.00	0.00	0.00	-67.42
		26.75	-5.27	0.38	0.00	0.00	0.00	-69.03
		4	-5.27	0.38	0.00	0.00	0.00	-70.25
	LL_I_2	3	-34.21	-73.87	0.00	0.00	0.00	-379.06
		3.25	-34.21	-73.87	0.00	0.00	0.00	-138.97
		7.50	-34.21	-73.87	0.00	0.00	0.00	174.99
		15.00	-34.21	-9.32	0.00	0.00	0.00	368.84
		22.50	-34.21	62.37	0.00	0.00	0.00	175.66
		26.75	-34.21	62.37	0.00	0.00	0.00	-89.40
		4	-34.21	75.36	0.00	0.00	0.00	-314.83
	LL_I_3	3	-38.64	-57.85	0.00	0.00	0.00	-94.97

65

2/90

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
		3.25	-38.64	-57.85	0.00	0.00	0.00	93.03
		7.50	-38.64	-57.85	0.00	0.00	0.00	338.88
		15.00	-38.64	23.15	0.00	0.00	0.00	320.75
		22.50	-38.64	52.62	0.00	0.00	0.00	38.95
		26.75	-38.64	52.62	0.00	0.00	0.00	-184.70
		4	-38.64	52.62	0.00	0.00	0.00	-355.72
	WIND	3	-6.36	2.02	-1.74	-1.08	3.52	27.34
		3.25	-6.36	2.02	-1.74	-1.08	-2.14	20.79
		7.50	-6.36	2.02	-1.74	-1.08	-9.54	12.21
		15.00	-6.36	2.02	-1.74	-1.08	-22.59	-2.93
		22.50	-6.36	2.02	-1.74	-1.08	-35.64	-18.07
		26.75	-6.36	2.02	-1.74	-1.08	-43.03	-26.65
		4	-6.36	2.02	-1.74	-1.08	-48.69	-33.21
	OVERTRN	3	0.27	1.86	0.00	0.00	0.00	63.64
		3.25	0.27	1.86	0.00	0.00	0.00	57.58
		7.50	0.27	1.86	0.00	0.00	0.00	49.66
		15.00	0.27	1.86	0.00	0.00	0.00	35.68
		22.50	0.27	1.86	0.00	0.00	0.00	21.71
		26.75	0.27	1.86	0.00	0.00	0.00	13.79
		4	0.27	1.86	0.00	0.00	0.00	7.73
	WIND_LL	3	-2.01	0.65	-0.96	-0.05	20.12	9.21
		3.25	-2.01	0.65	-0.96	-0.05	17.01	7.09
		7.50	-2.01	0.65	-0.96	-0.05	12.94	4.32
		15.00	-2.01	0.65	-0.96	-0.05	5.76	-0.58
		22.50	-2.01	0.65	-0.96	-0.05	-1.42	-5.48
		26.75	-2.01	0.65	-0.96	-0.05	-5.49	-8.25
		4	-2.01	0.65	-0.96	-0.05	-8.60	-10.37

✓ Lee Ahrell 2/90

****
* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : SPACE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
4	DL	4	34.23	-243.31	-0.00	137.95	56.31	-1880.26
		3.25	27.88	-219.96	-0.00	137.95	56.31	-1127.44
		6.13	22.27	-199.31	-0.00	137.95	56.31	-524.75
CAP		5	8.39	-20.55	-0.00	137.95	56.31	-0.00
	SETTLE_1	4	-0.00	-0.00	0.00	0.00	0.00	-0.00
		3.25	-0.00	-0.00	0.00	0.00	0.00	-0.00
		6.13	-0.00	-0.00	0.00	0.00	0.00	0.00
		5	-0.00	-0.00	0.00	0.00	0.00	-0.00
	SETTLE_2	4	0.00	0.00	0.00	0.00	0.00	-0.00
		3.25	0.00	0.00	0.00	0.00	0.00	-0.00
		6.13	0.00	0.00	0.00	0.00	0.00	-0.00
		5	0.00	0.00	0.00	0.00	0.00	-0.00
	SDWK_LL	4	-0.00	-0.00	0.00	0.00	0.00	-0.00
		3.25	-0.00	-0.00	0.00	0.00	0.00	-0.00
		6.13	-0.00	-0.00	0.00	0.00	0.00	-0.00
		5	-0.00	-0.00	0.00	0.00	0.00	-0.00
	LL_I_1	4	-0.00	0.00	0.00	0.00	0.00	-0.00
		3.25	-0.00	0.00	0.00	0.00	0.00	-0.00
		6.13	-0.00	0.00	0.00	0.00	0.00	-0.00
		5	-0.00	0.00	0.00	0.00	0.00	-0.00
	LL_I_2	4	-0.00	0.00	0.00	0.00	0.00	0.00
		3.25	-0.00	0.00	0.00	0.00	0.00	-0.00
		6.13	-0.00	0.00	0.00	0.00	0.00	-0.00
		5	-0.00	0.00	0.00	0.00	0.00	-0.00
	LL_I_3	4	-0.00	0.00	0.00	0.00	0.00	-0.00
		3.25	-0.00	0.00	0.00	0.00	0.00	-0.00
		6.13	-0.00	0.00	0.00	0.00	0.00	-0.00
		5	-0.00	0.00	0.00	0.00	0.00	-0.00
	WIND	4	-0.00	-0.00	-0.00	-0.00	0.00	0.00
		3.25	-0.00	-0.00	-0.00	-0.00	-0.00	0.00
		6.13	-0.00	-0.00	-0.00	-0.00	-0.00	0.00

✓ [illegible] 2/90

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
		5	-0.00	-0.00	-0.00	-0.00	0.00	-0.00
	OVERTRN	4	-0.00	0.00	0.00	0.00	0.00	0.00
		3.25	-0.00	0.00	0.00	0.00	0.00	0.00
		6.13	-0.00	0.00	0.00	0.00	0.00	0.00
		5	-0.00	0.00	0.00	0.00	0.00	0.00
	WIND_LL	4	0.00	0.00	0.00	-0.00	0.00	0.00
		3.25	0.00	0.00	0.00	-0.00	0.00	0.00
		6.13	0.00	0.00	0.00	-0.00	0.00	-0.00
		5	0.00	0.00	0.00	-0.00	0.00	0.00

Northern Avenue Bridge Over The New Rives
- Structural Calculations -
Table of Contents

Item	Pages
Deck Slab Design	1 to 11
Precast Prestressed Girder Design	12 to 37
Abutment & Wingwall Design	38 to 101
Drilled Shaft Analysis	102 to 186
Drilled Shaft Design	187 to 208
Pier Cap Design	209 to 251
Quantities	252 to 287

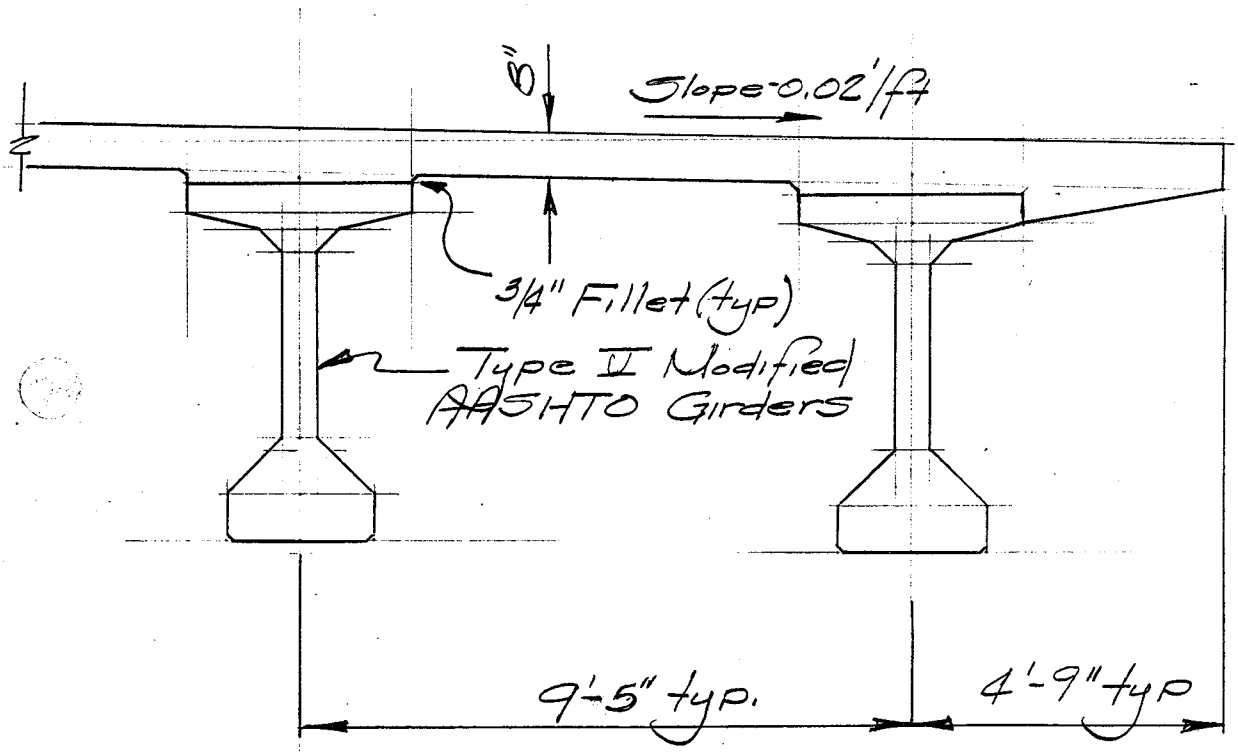
BRW
MINNEAPOLIS PHOENIX

JOB _____ SHEET NO. _____
MADE BY _____ DATE _____ JOB NO. _____
CHECKED BY _____ DATE _____ SEC. SHT. NO. _____
CALCULATIONS FOR _____

Northern Avenue Bridge
Over the New River
Superstructure Design

Deck Slab

The protective system provided will be 2 1/2 inches of cover for top bars of which the top 1/2 inch will be an integral wearing surface not included in computing section properties of either the slab or the composite effect with the girders.



Effective Slab Span = (9'-5") - (3'-4") + (1'-5") = 7'-6" (= S

(Ref ADOT "Procedures for Development of Structure Plans by Consultant) Section 200.5.4.E

$DL\ M = \frac{1}{10} WS^2$ (Ft-Kips)

W includes 25#/ft³ for future Wearing Surface

W = 0.67(150) + 25 = 125#/Lf

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JOB Northern Avenue Bridge SHEET NO. S-1
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 1
CALCULATIONS FOR Deck Slab Design

Northern Avenue Bridge Over the New River

Deck Slab Design (Cont'd)

$$DM = 0.125\left(\frac{7.50}{10}\right)^2 = 0.703 \frac{K}{ft}$$

$$LM_{+I} = \frac{P(S+2)}{40} \times 1.30 \quad \text{where } P = 16^{kips}$$

$$LM_{+I} = 16\left(\frac{7.50+2}{40}\right) \times 1.30 = 4.94 \frac{K}{ft}$$

Assume #5 bars will be used both transverse and as longitudinal distribution steel. It is also assumed that #6 will be used as longitudinal continuity bars for negative moment.

$$DM + LM_{+I} = 0.703 + 4.94 = 5.64 \frac{K}{ft}$$

Slab Design Allowables $f'_c = 4{,}000$ psi
$f_c = 1600$ psi
$n = 8$

$$K = \frac{f_c}{\frac{f_s}{n} + f_c} = \frac{1600}{\frac{24{,}000}{8} + 1600} = 0.348$$

$$j = 1 - \frac{K}{3} = 1 - \frac{0.348}{3} = 0.884$$

$$\rho = \frac{f_c}{2f_s} \times K = \frac{1600}{2(24{,}000)}(0.348) = 0.0116$$

$$R_{allow} = \tfrac{1}{2} jKf_c = \tfrac{1}{2}(.884)(.348)1600 = 246 \text{ psi}$$

$$\text{Required } bd^2 = \frac{M_{DL+LL}}{R} = \frac{5.64(12{,}000)}{246} = 275$$

$$d = \left(\frac{275}{12}\right)^{1/2} = 4.78''$$

$$\text{Minimum Slab Depth} = 4.78 + 2.5 + (.5)(5/8) = 7.60 \therefore 8'' \text{ is OK}$$

BRW
MINNEAPOLIS PHOENIX DENVER

JOB Northern Avenue Bridge SHEET NO. S-2
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 2
CALCULATIONS FOR Deck Slab Design

Northern Avenue Bridge Over the New River

Deck Slab Design (Cont'd)

$d_{top} = 8 - 2\frac{1}{2} - \frac{1}{2}(\frac{5}{8}) = 5.18''$

$d_{bot} = 8 - 1 - \frac{1}{2}(5/8) = 6.68''$

Required Steel Area

$$A_s = \frac{M_{D+L+I}}{f_s j d} = \frac{5.64(12)}{24(.884)5.18} = 0.62 \frac{\square''}{ft}$$

for top use #5 @ 12" Straight bars with a #5 truss @ 12" in between

$A_{s\,prov} = 0.62 \frac{\square''}{ft} \geq 0.62 \frac{\square''}{ft}$ ∴ ok

Bottom Steel

$$A_s = \frac{5.64(12)}{24(.884)6.68} = 0.48 \frac{\square''}{ft}$$

for bottom use #4 @ 12" Straight bars with the #5 truss @ 12" in between.

$A_{prov} = 0.51 \frac{\square''}{ft} > 0.48 \frac{\square''}{ft}$ ∴ ok

Distribution Reinforcement

$$\frac{220}{\sqrt{S}} = \frac{220}{(7.50)^{1/2}} = 80\% > 67\%$$

∴ Use 67% max

$0.67(.48) = 0.32 \frac{\square''}{ft}$ Use #5 @ 10" oc in the middle half & #5 @ 1'-4" o.c in the outer quarter

Use #5 @ 12" centers longitudinal in the top of the top slab, which is an ADOT minimum

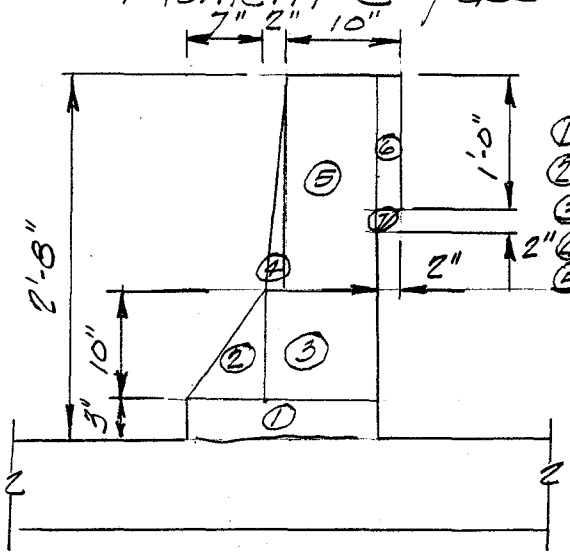
The deck overhang is not critical for design, by inspection

BRW
MINNEAPOLIS PHOENIX DENVER

JOB Northern Avenue Bridge SHEET NO. 5-3
MADE BY Lee Arnold DATE _____ JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 3
CALCULATIONS FOR Deck Slab Design

Northern Avenue Bridge Over the New River
Slab Design (Cont'd)

✓ Moment @ face of Rail



Rail Weight	about face of Curb	cg X Wt
① 17 x 3 x $\frac{150}{144}$ = 53.12	8.5	451.6
② 5 x 7 x " = 36.46	4.67	170.3
③ 10 x 10 x " = 104.17	12.0	1250.0
④ 1 x 19 x " = 19.00	8.33	158.3
⑤ 19 x 8 x " = 158.33	13.00	2058.3
372		4089

c.g. of Rail from face of Rail $= \frac{4089}{372} = 11''$

$M_{@ \text{ face of Rail}} = 10^k \times \frac{32}{12} \times \left[\frac{1}{5+(0.8)\left(\frac{11}{12}\right)}\right] = 4.65 \frac{k}{ft}$

this is slightly less than the M used in design, ∴ reinforcement is OK

Longitudinal Slab Reinforcement

The girders will be designed as simply supported. The superstructure will be effectively pinned at intermediate supports, but continuous for live load and superimposed DL (Barrier, FWS & Utilities). Longitudinal deck reinforcement will be added to resist these loads

Distribution of Wheel loads to longitudinal beams $= \frac{S}{5.5}$ (Table 3.23.1) $= \frac{9.42}{5.5} = 1.712$ Wheel Lines $= 0.856$ lanes

FWS = $25^{lb}/ft^2 \times 9.42' = 236$ lbs/Lf

Utilities = $6.25^{lb}/ft^2 \times 9.42 = 59$ lbs/ft

Total Added DL = 295 lbs/ft Plus Barrier

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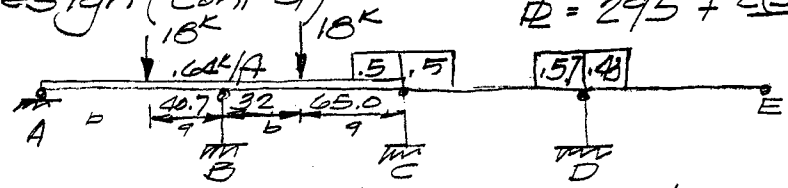
JOB Northern Avenue Bridge SHEET NO. S-4
MADE BY Lee Arnold DATE ____ JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 4
CALCULATIONS FOR Deck Slab Design

Northern Avenue Bridge Over the New River
Slab Design (Cont'd)

$\Phi = 295 + \frac{2(372)}{6\text{ girs}}$ Barriers $= 419 \frac{lb}{Lf}$



Maximum Negative Moment @ B / per lane

Span 1
FEM

Uniform $= \frac{Wl^2}{8} = 0.64\frac{(97)^2}{8} = 753^{'K}$

Concentrated $= \frac{P}{L^2}\left[b^2a + \frac{a^2b}{2}\right] = \frac{18}{(97)^2}\left[(56.26)^2(40.74) + \frac{(40.74)^2(56.26)}{2}\right]$
$= 336^{'K}$

Span 2
FEM

Uniform $= \frac{Wl^2}{12} = 0.64\frac{(97)^2}{12} = 502^{'K}$

Concentrated $M_B = \frac{Pa^2b}{L^2} = 18\frac{(65.0)^2(32.0)}{(97)^2} = 259^{'K}$

$M_C = \frac{Pb^2a}{L^2} = 18\frac{(32.0)^2(65.0)}{(97)^2} = 127^{'K}$

.43	.57	.5	.5	.57	.43
-1089	+761	-629			
+141	+187	+314	+315		
	+157	+94		+157	
-67	-90	-47	-47	-90	-67
	-23	-45	-45		
+10	+13	+45	+45		
	+23				
-10	-13				
1015	(*1066)				

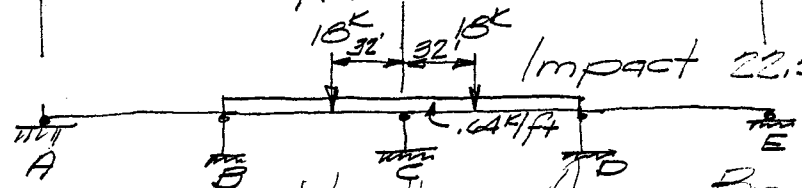
Impact 22.5%

* Multiply all LL by # of Lanes plus Impact = 0.856 x 1.225 = 1.05

Maximum Negative Moment @ C / per lane

.43	.57	.5	.5	.57	.43
	629	-761	761	-629	
	-359			+359	
		-180	+180		
	(*988)	-941	941	(*988)	

Impact 22.5%



BRW
MINNEAPOLIS PHOENIX DENVER

JOB Northern Avenue Br SHEET NO. S-5
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 5
CALCULATIONS FOR Longitudinal Deck Reinforcement

Northern Avenue Bridge Over the New River

Loading Spans 1 & 2, find reaction at Abut 1

$\Sigma M_B = -1015$

$18(40.7) = 732.6$

$.64\left(\frac{97}{2}\right)^2 = 3010.9$

$R_A(97)$

$R_A = \frac{3010.9 + 732.6 - 1015}{97} = 28.13^K$

Calculate Moment @ 9/10 Pt of Span 1

$\Sigma M_{9/10\ S\text{-}1}$

$18(31.0) = 558$

$.64\left(\frac{87.3}{2}\right)^2 = 2438.8$

$28.13(87.3) = 2455.6$

$M_{9/10} = 541^{IK}$ (*568)

Find reaction at Pier C (Loading Span 1 & 2)

$\Sigma M_B = +1015$

$R_C(97)$

$18(32) = 576$

$.64\left(\frac{97}{2}\right)^2 = 3010.9$

$R_C = \frac{3010.9 + 576 - 1015}{97} = 26.51^K$

Calculate Moment @ 0.10 Pt of Span 2

$\Sigma M_{1/10\ S\text{-}2}$

$26.51(87.3) = 2314.3$

$18(22.3) = 401.4$

$0.64\left(\frac{87.3}{2}\right)^2 = 2438.8$

$M_{1/10\ S\text{-}2} = 526^{IK}$ (*552)

Find Reaction at Pier B Loading Spans 2 & 3

$\Sigma M_C = -941$

$18(32) = 576$

$0.64(97) = 3010.9$

$R_B(97)$

Similar $R_B = 27.28$

Calculate Moment @ 9/10 Pt of Span 2

$\Sigma M_{9/10\ S\text{-}2}$

401.4

2438.8

$27.28(87.3)$

$M_{9/10\ S\text{-}2} = +459$ (*483)

JOB Northern Avenue Bridge SHEET NO. S-6

MADE BY Lee Arnold DATE _____ JOB NO. 33-8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 6

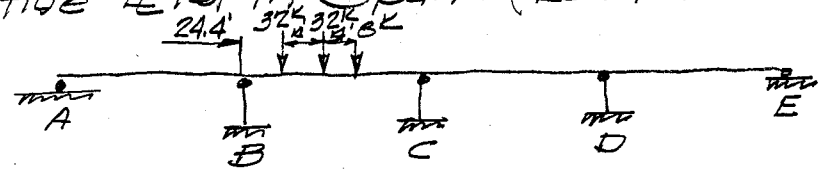
CALCULATIONS FOR Longitudinal Deck Reinforcement

BRW

MINNEAPOLIS PHOENIX DENVER

Northern Avenue Bridge Over the New River
Slab Longitudinal Reinforcement

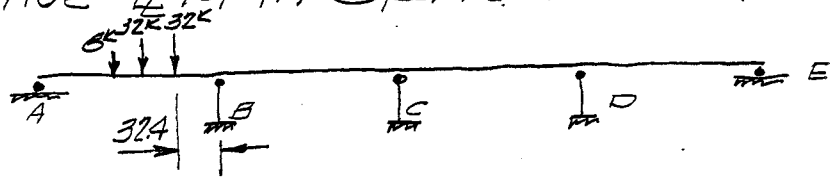
Negative ℄M in Span 1 (Load Truck in Span 2)



$FEM_{BC} = +974^{'K}$ $FEM_{CB} = -544^{'K}$

.43	.57	.5	.5	.57	.43
	+974	-544			
-418	-556	+272	+272		
	+136	-278		+136	
-58	-78	+139	+139	-78	-58
	+70		-39	+70	
-30	-40	+20	+19		
	+10				
-4	-6				
(535) -510					

Negative ℄M in Span 2 1st Pier (Truck in Span 1)



$FEM_{BA} = 1290^{'K}$

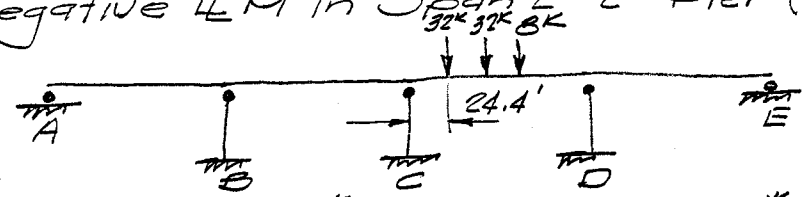
.43	.57	.5	.5	.57	.43
-1290					
+553	+737				
		+368			
		-184	-184		
	-92			-92	
+39	+53			+53	
		+26	+26		
		-26	-26		
	-13				
+6	+7				
-692	+692'K (727)				

JOB Northern Avenue Bridge SHEET NO. 5.7
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 7
CALCULATIONS FOR Longitudinal Deck Steel

BRW
MINNEAPOLIS PHOENIX DENVER

Northern Avenue Bridge Over the New River
Slab Longitudinal Reinforcement

Negative ℄M in Span 2 2nd Pier (Truck in Span 3)



$FEM_{CD} = 974^{'K}$ $FEM_{DC} = -544^{'K}$

.43	.57	.5	.5	.57	.43
			+974	-544	
		-487	-487	+311	
	-243		+156		
	+139	-78	-78		
	-39	+70		-39	
	+22	-35	-35	+22	
		+11	+11		
		-11	-11		
		(556'K) -530			

Superimposed Φ = FWS + Utilities + Barrier

$= 236^{lb} + 59^{lb} + \frac{2(372)}{6\,Gdrs} = 419\,\frac{lb}{Lf}$ per girder

Dead Load Moments

@ B $= -0.1071 \times WL^2 = -0.1071(.419)(97)^2 = -423^{'K}$

@ C $= -0.0714 \times WL^2 = -0.0714(.419)(97)^2 = -282^{'K}$

@ 38.1' from Abut ℄ in Span 1

$M_{pos} = 0.0772 \times WL^2 = 0.0772(.419)(97)^2 = +304^{'K}$

@ 52.0' from Pier 2 in Span 2

$M_{pos} = 0.0364 \times WL^2 = 0.0364(.419)(97)^2 = +144^{'K}$

BRW
MINNEAPOLIS PHOENIX DENVER

JOB Northern Avenue Bridge SHEET NO. 5-8
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 8
CALCULATIONS FOR Longitudinal Deck Steel

Northern Avenue Bridge Over the New River
Slab Longitudinal Reinforcement
Superimposed D + L_I Moment Diagram
per Girder

There are 9 Longitudinal #5 in top mat & 8 in bottom mat

$d \approx 63 + 1 + \frac{8-3.5}{2} = 66''$

Resisting Neg Moment

$M = A_s f_s j d$

$= 17(.31)24(.884)\frac{66}{12}$

614^{K}

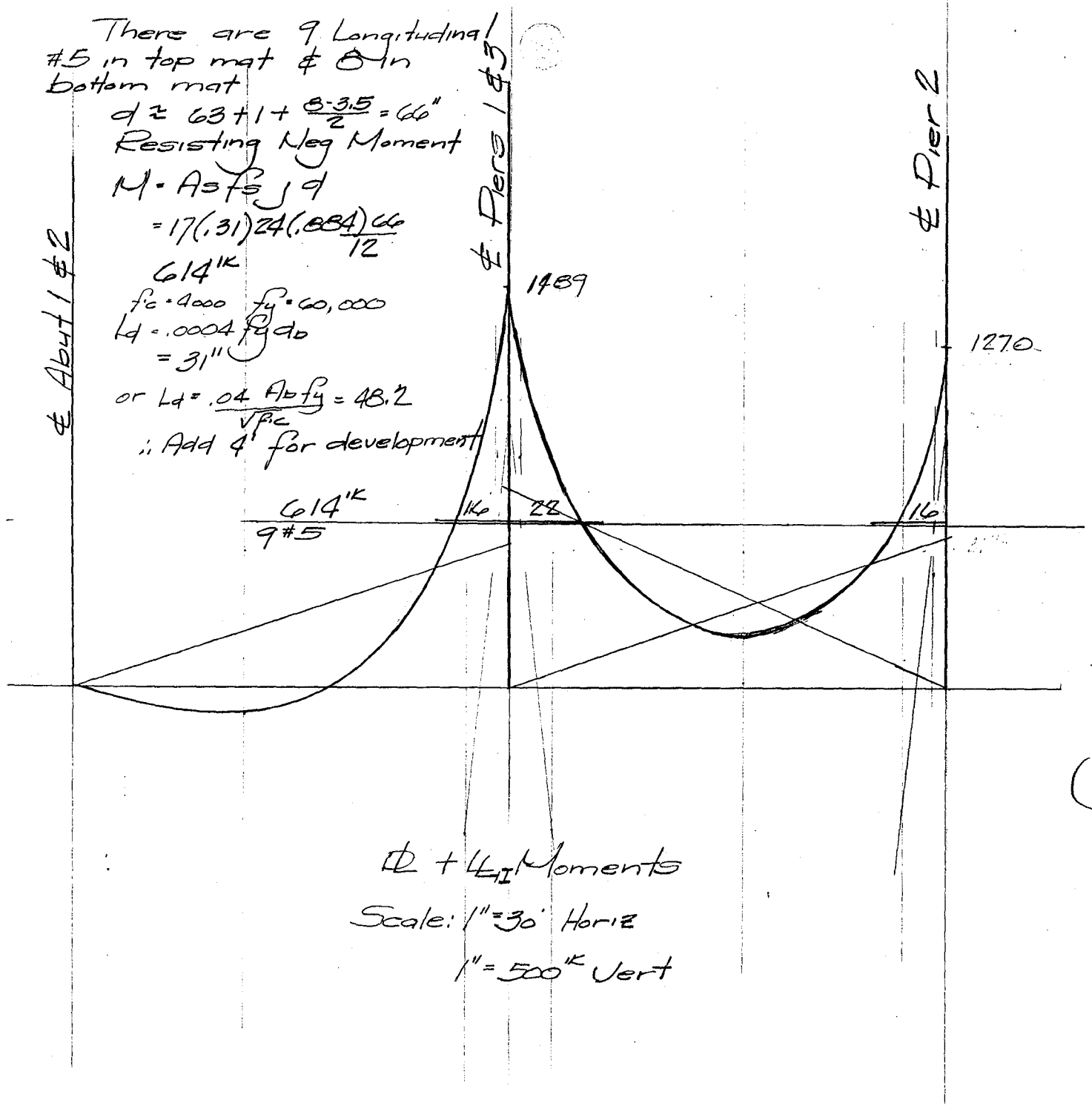
$f'_c = 4000$ $f_y = 60{,}000$

$l_d = .0004 f_y d_b$

$= 31''$

or $l_d = \frac{.04 A_b f_y}{\sqrt{f'_c}} = 48.2$

∴ Add 4' for development



D + L_I Moments

Scale: 1" = 30' Horiz

1" = 500^{K} Vert

IBRW
MINNEAPOLIS PHOENIX DENVER

JOB Northern Avenue Bridge SHEET NO. S-9
MADE BY Lee Arnold DATE JOB NO. 33-89612
CHECKED BY Tibbens DATE 2/90 SEC. SHT. NO. 9
CALCULATIONS FOR Longitudinal Deck Steel

Northern Avenue Bridge Over the New River
Slab Longitudinal Reinforcement

Settlement is projected to be less than .89" occuring during application of dead load. Differential settlement is estimated to be 1/2 of total settlement.

Consider
1. Abut 1 settling & Pier 2 Settling
2. Pier 1 & Pier 3 Settling

$E = 4{,}000$ Ksi

Calculate I for 1 Line of Girders

$I_{girder} = 479{,}459 \text{ in}^4$ $A_{girder} = 887 \text{ in}^2$ $y_b = 32.02$ in

I of Composite Section

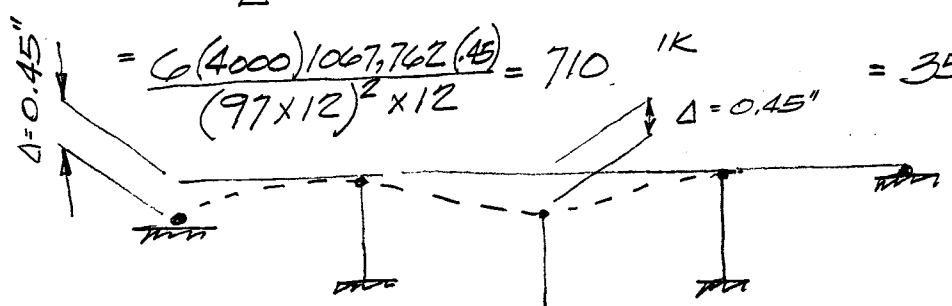
	A	y	Ay	Ay^2	I_{oo}
Slab	864	88.75	58,968	4,024,566	4,608
Filler	50	63.625	3181	202,407	6
Girder	887	32.02	28,402	909,423	479,459
	1801		90,551	5,136,396	484,073

$y_b = 50.278$

$I_{xx} = 484{,}073 + 5{,}136{,}396 - 1801(50.278)^2$
$= 1{,}067{,}762 \text{ in}^4$

$FEM = \frac{6EI\Delta}{L^2}$ (Ends Fixed) $FEM = \frac{3EI\Delta}{L^2}$ (one end hinged)

$= \frac{6(4000)1067{,}762(.45)}{(97 \times 12)^2 \times 12} = 710$ ik $= 355$ ik



Settlement Case 1

JOB Northern Avenue Bridge SHEET NO. 5.10
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 10
CALCULATIONS FOR Longitudinal Deck Steel

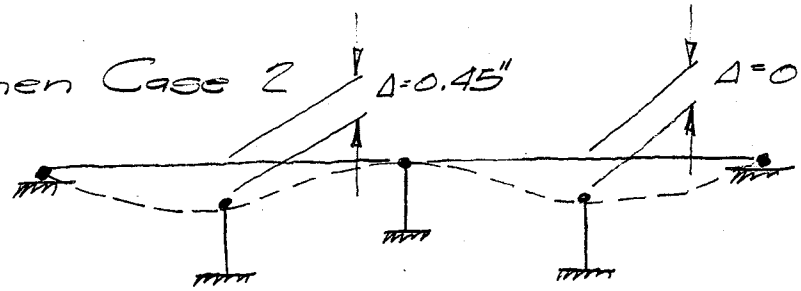
BRW
MINNEAPOLIS PHOENIX DENVER

Northern Avenue Bridge Over the New River
Slab Longitudinal Reinforcement
Settlement Case 1 (Cont'd)

.43	.57	.50	.50	.57	.43
-355	+710	+710	-710	-710	
-153	-205			+404	+306
		-101	+202		
		-50	-50		
	-25			-25	
+11	+14			+14	+11
		+7	+7		
		-7	-7		
	-4			-4	
+2	+2			+2	+2
-495	+495	+550			

This case produces the maximum negative moment at B

Settlemen Case 2



excess girder camber

Filler

$d = 63 + 1.25 + 1.19 + 8 - 2.5 - 5/8 - 1/2(1.27)$

$= 69.68''$

.43	.57	.50	.50	.57	.43
+355	-710	-710	+710	+710	-355
+153	+202			-255	-169
		+112	-112		
		-609	+609		

Maximum Moments due to Superimposed DL plus LL+I and settlement at each pier

Pier 1 & 3 $M = -1489 - 495 = -1984^{'k}/Gdr$

Pier 2 $M = -1270 - 609 = -1879^{'k}/Gdr$

This can be reduced to the moment at face of the diaphragm, but the diaphragm is 2'-0" wide and will reduce the moment only slightly. Design for maximum moment

$$A_s = \frac{M}{f_s j d} = \frac{1984(12)}{24(.884)69.68} = 16.10 \frac{\square''}{girder}$$

Use 10 #11 or #11 @ 12" oc

JOB Northern Ave Bridge SHEET NO. S-11

MADE BY Lee Arnold DATE JOB NO. 33-8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 11

CALCULATIONS FOR Longitudinal Deck Steel

BRW MINNEAPOLIS PHOENIX DENVER

Verify Geometrics:

New River Channel: from Coe & Van Loo Plans of New River Channelization.

Channel width between toe of slope (not bottom of soil cement) is 325 ft.

Channel bottom slope -0.002 ft/ft downstream south

Elevations

sta. 167+80	Lt TOB 1077.70	℄ 1060.00	Rt TOB 1077.7
160+25	1076.16	1058.46	1076.16
160+00		1058.41	1076.86
159+75	1076.06	1058.36	
159+45	1075.30	1058.30	
159+00	1075.20	1058.22	1076.7
148+00	1073.03	1056.03	1074.53

Channel Bearing N 0° 11'02"E

Northern Ave. Bearing N 12°23'52"W

Ties: ℄ intersection w/ ⅊ Rt (East) 848.15 ft to NE Cnr 5 T2N R1E

Lt (West) 1777.53 ft to N ¼ 5

NE Cnr 5 N 23,058.1166 E 15,385.4924

N ¼ Cnr 5 N 23,008.3339 E 12,760.2844

Intersection ℄ New River @ ⅊ Northern sta 159+85.42

Elev.'s. Lt TOB 1076.081 ℄ 1058.381 Rt TOB 1076.837

BRW

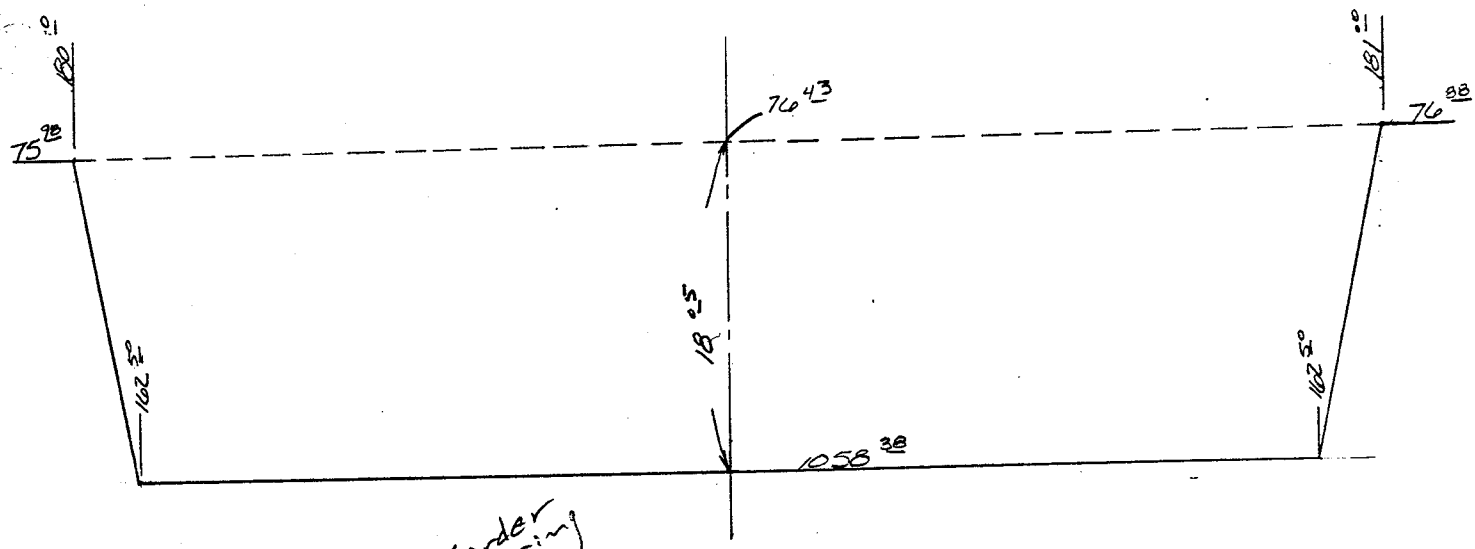
JOB Northern Ave. @ New River SHEET NO. G 1

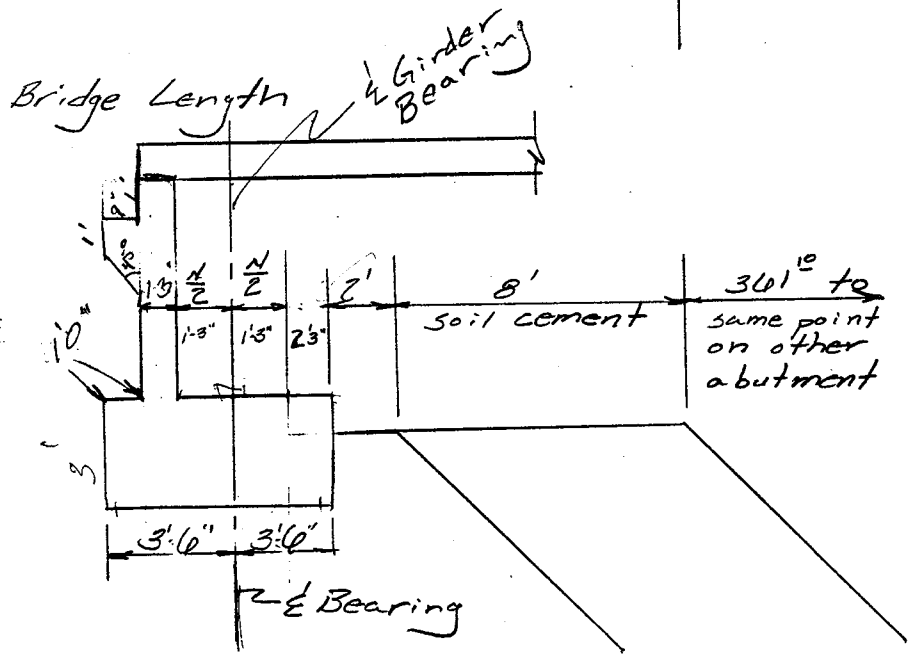
MADE BY T. Stones DATE 9-5-89 JOB NO. 33-8962

CHECKED BY Lee Arnold DATE 10/89 SEC. SHT. NO. 12

CALCULATIONS FOR Layout

From Typical X-sections the following ℄ X-section is interpolated





From California DOT Memo to Designers 5-1 January 1988

$$N = (12 + 0.03L + 0.12H)\left(1 + \frac{S^2}{8000}\right)$$

N = support width in inches; $N_{min} = 2'6"$
L = Length of bridge deck from abutment to adjacent expansion jt.
H = Average Height of columns
S = Skew of Abutment in degrees

BRW

JOB Northern Ave. @ New River SHEET NO. G 2
MADE BY T. Stones DATE 9-5-89 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 10/89 SEC. SHT. NO. 13
CALCULATIONS FOR Layout

Bridge Length (cont.)

Assume $N = 2'\text{-}6''$ $\quad L = 361^{10} + 2\left(8 + 2 + \frac{N}{2} + 3^{25}\right) = 390^{10}$ ft

$S = 0°$ $\quad H \cong 18$ ft

$N = \left[12 + 0.03(390^{10}) + 0.12(18)\right] = 25^{88} < 2'\text{-}6''$ use $N_{min} = 2'\text{-}6''$

℄ Brg Abut to ℄ Brg Abut $= 361^{10} + 2(8 + 2 + 3^{25}) = 387^{60}$ ft

make an even 388 ft.

Hinge @ Piers

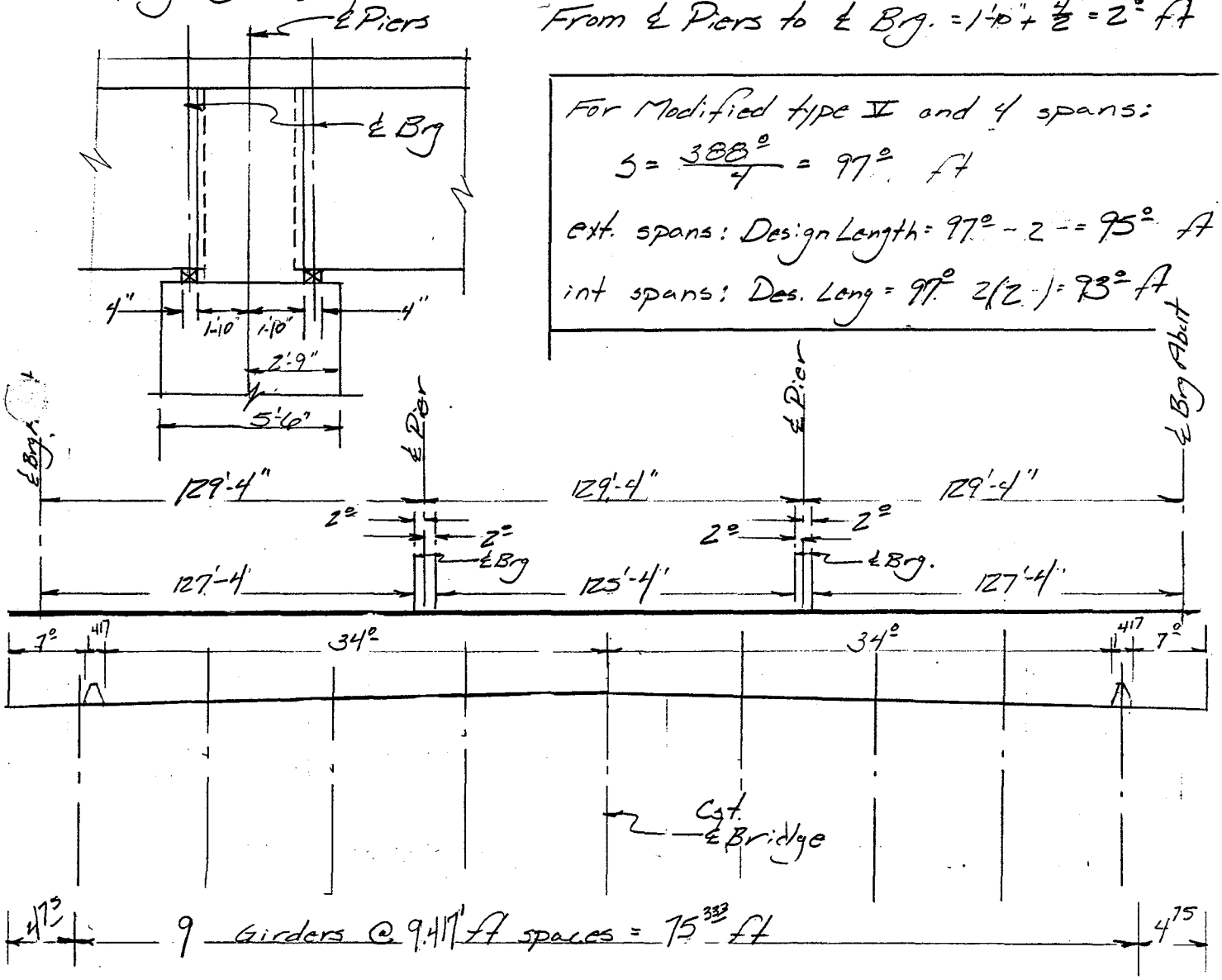
From ℄ Piers to ℄ Brg. $= 1'\text{-}10'' + \frac{4''}{2} = 2^{0}$ ft

For Modified type V and 4 spans:

$S = \frac{388^{0}}{4} = 97^{0}$ ft

ext. spans: Design Length $= 97^{0} - 2 = 95^{0}$ ft

int spans: Des. Leng $= 97^{0} - 2(2) = 93^{0}$ ft



JOB Northern Ave. @ New River	SHEET NO. G3
MADE BY T. Stones DATE 9-6-89	JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 10/89	SEC. SHT. NO. 14
CALCULATIONS FOR Layout	

BRW

TUCSON, ARIZONA

T-PAC
PRESTRESS DIVISION

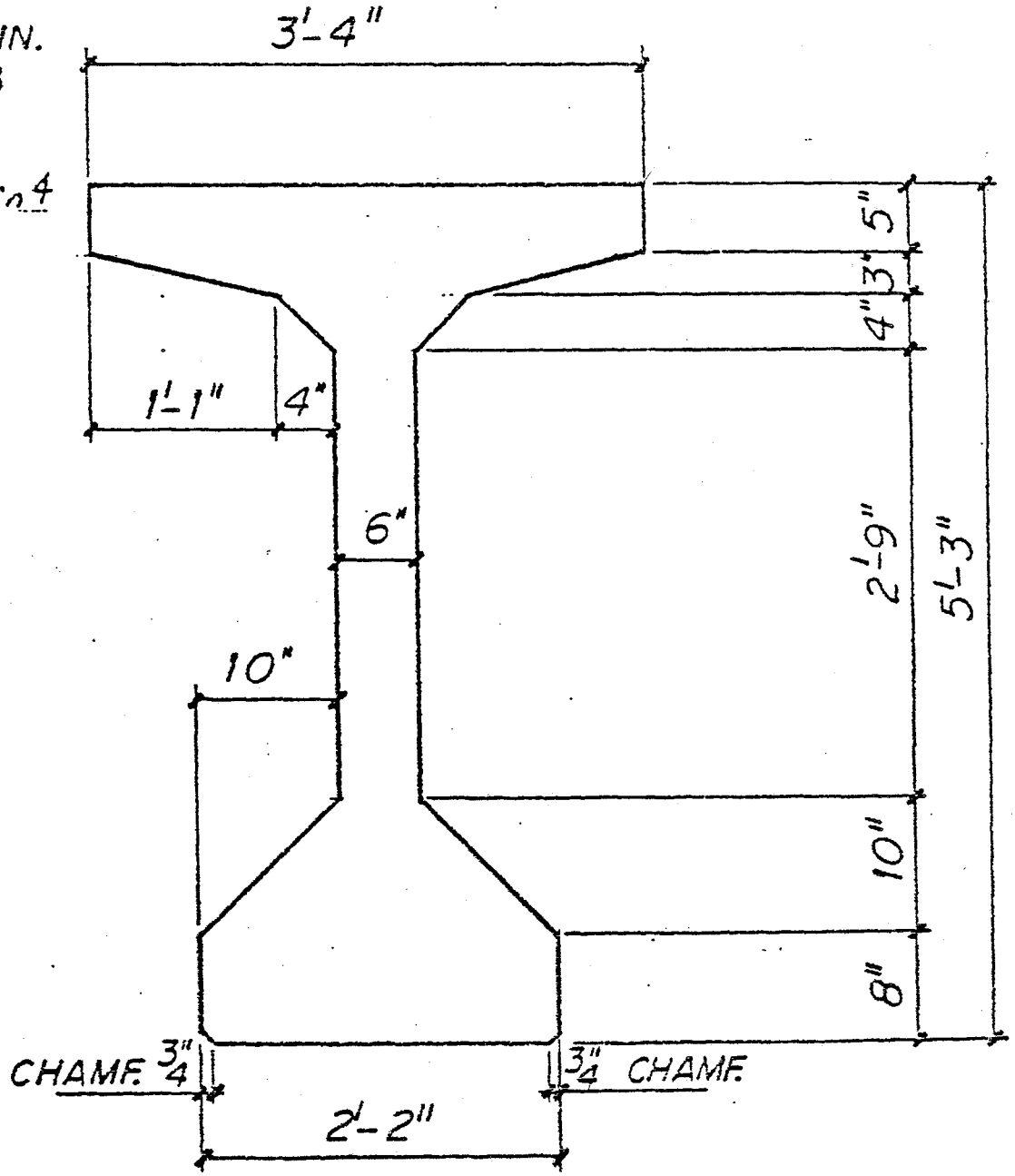
RANCHO CUCAMONGA, CALIF.

Drafting Standard

REVISIONS	DATE	DRAWN BY	SHEET NO
	JOB NO:	CHECKED BY	OF

APPROVED FOR PRODUCTION - DATE ISSUED:

AREA = 887 SQ. IN.
CU. YDS/FT. = .23
WT./FT. = 924#
y_b = 32.02 in
I = 479,459 in^4



TYPE "V" MODIFIED BEAM

PHOENIX, ARIZONA
TUCSON, ARIZONA

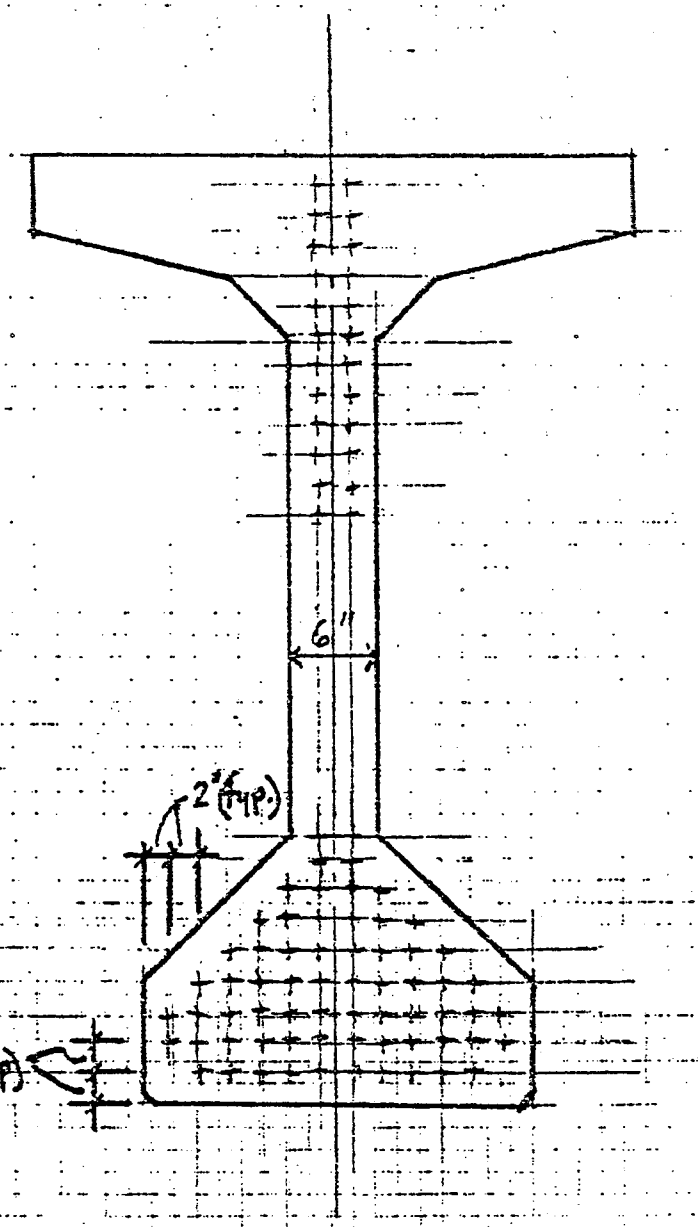
THE TANNER COMPANIES

T-PAC
PRESTRESSED DIVISION

RANCHO CUCAMONGA, CALIF.

REVISIONS	DATE	DRAWN BY HTB	SHEET NO
	JOB NO	CHECKED BY	OF

APPROVED FOR PRODUCTION - DATE ISSUED



AASHTO TYPE V "MODIFIED" GIRDER 16

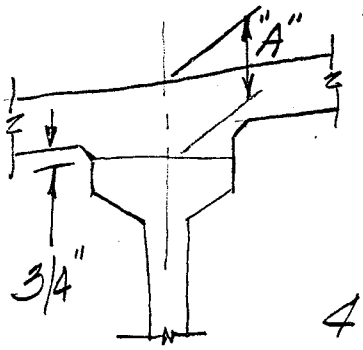
New River Channel

℄ Station of Bridge = 46+14.70

Critical Station for Clearance are ±162.5
or
Sta 47+77.20 & Sta 44+52.20

Calculate "A" Dimension at ℄ Pier

3 Span



Slab Thickness + 3/4" Fillet	=	8 3/4"
Excess Camber (TS PS Run)	=	1.60"
Top Flange Width Effect (.02)(40/2)	=	0.40
Vertical Curve Effect	=	*0
		10.75"

Elev @ Girder Bottom 1077.454

Min Elev @ PGL = 1077.454 + 6' + 10.75/12 + 37.748(.02)
= 1085.105

4 Span

Slab Thickness + 3/4" Fillet	=	8 3/4"
Excess Camber (TS PS Run)	=	1.29"
Top Flange Width Effect (.02)(40/2)	=	0.40
Vertical Curve Effect	=	*0
Elev @ Girder Bottom 1077.454		10.44

Min Elev PGL = 1077.454 + 5.25' + 10.5/12 + 37.083(.02) Use 10 1/2"
= 1084.32'

* The vertical curve for a crest vc will actually reduce "A". Ignor this to provide greater clearance

Raise Elevations 1/2" to Compensate for some variation in Camber

PGL Elevations @ (Minimum)

Sta	3 Span	4 Span
44+52.20	1085.15	1084.36
47+77.20	1085.15	1084.36

BRW

JOB Northern Avenue Bridge SHEET NO. V2
MADE BY Lee Arnold DATE 9/14/89 JOB NO. 33-8962
CHECKED BY T Stones DATE 9-14-89 SEC. SHT. NO. 17
CALCULATIONS FOR Vertical

✓ T Stones 2/90

BRW, INC. PHOENIX, ARIZONA JOB NUMBER 33-8962 1/23/90 TIME 11:36:35

*** PRESTRESSED GIRDER CHECK *** REV. 10/07/87 ***

*** NO END BLOCKS *** SIMPLE SPAN SHEAR DESIGN *** FINAL LOSSES - 48.

IDENT: 10 PROBLEM 2 12 HARPED STRANDS 30 STRAIGHT STRANDS
(LOW RELAXATION STRANDS)

SPAN 95.00 SERIES 2 9. GIRDERS AT 9.42 8.00 SLAB

WEB THICKNESS 6.0 IN., 2.5 IN. ASPHALT OVERLAY

STRANDS 0.153 @ 270. K A- 10.75 N. J. BARRIER S/ 5.5 AVFY - 24.80

FINAL DESIGN STRESS, BOT. CENTERLINE 0.214 (ALLOWED 0.232)

FINAL DESIGN STRESS, TOP CENTERLINE -1.636 (ALLOWED -2.400)

FINAL DESIGN STRESS, TOP AT D/2 -0.412 (ALLOWED 0.232)

STRESS TOP AT HARP PT. AT RELEASE -0.048 (ALLOWED 0.212)

STRESS BOT. AT HARP POINT AT RELEASE -2.690 (ALLOWED -3.000)

STRESS TOP AT D/2 AT RELEASE -0.374 (ALLOWED 0.212)

STRESS BOT AT D/2 AT RELEASE -2.408 (ALLOWED -3.000)

(INCLUDES ELASTIC SHORTENING AND RELAXATION)

GIR. CONCRETE STRENGTH AT STRAND RELEASE- 5.000 AT 28 DAYS- 6.000

WS(SLAB+DIAPH) - 1.191 WC(CURB+ASPH) - 0.451 E - 4.400 FCL - 7.000

RISE IN HARPED STRANDS - 49.000 FO - 7.001 FLG W - 113.00

COMPOSITE SECTION I - 976340. Y BOT - 47.91 Q SLAB - 14092.

ULTIMATE MOMENT REQ'D. 3963. AVAILABLE 6510. (GIRDER ONLY)

ULTIMATE MOMENT REQ'D. 7049. AVAILABLE 8931. (COMPOSITE)

SHEAR @ D/2: VU-293.7 VC-180.4 VS-165.1 SPACING- 7.6 (HS)
CHECK STANDARD STIRRUP SPACING AT GIRDER END ON STANDARD SHEET.
SHEAR @ D: VU-280.9 VC-186.8 VS-143.7 SPA. (S)- 8.7 (HS)

SHEAR @ HARP. PT.: VU-116.4 VC- 74.2 VS- 62.8 SPACING- 22.9 (18" MAX) (HS)

BEGIN 18" STIRRUP SPA. 35.3 FEET FROM CL BEARING (LS - 30.1)
INTERPOLATE FOR A MULTI-STEP INCREMENT IN STIRRUP SPACING.

GIR. D.L. END REACTION- 124.6 WITHOUT END DIAPHRAGM

GIR. D.L. END REACTION- 102.7 WITHOUT CURB, ASPHALT, OR END DIAPHRAGM
BREAKDOWN OF STRESSES AT CENTERLINE SPAN

✓ T Stones 2/95

SBDG- 0.835 SBDS- 1.096 SBDC- 0.300 SBXML- 0.885 SBPRE- -2.901

BRW, INC. PHOENIX, ARIZONA JOB NUMBER 33-8962 1/23/90 TIME 11:36:35

*** PRESTRESSED GIRDER DEFLECTIONS *** REV 10/07/87

PROBLEM NO. 2

GIRDER WT.	0.924	K/FT.	Y BOTTOM (GIRDER)	32.02	IN.
GIRDER AREA	897.0	IN.**2	S TOP (GIRDER)	15476.0	IN.**3
I (GIRDER)	479459.	IN.**4	Y TOP (GIRDER)	30.98	IN.
GIRDER SERIES	2		SPAN LENGTH	95.00	FT.
STRAND AREA	0.153	IN.**2	ULTIMATE STRENGTH	270.0	KSI
E	4.40	IN.	FCL	7.00	IN.
RISE / HARPED STND	49.00	IN.	NO. STRAIGHT STND	30	
NO. HARPED STND	12		I (COMPOSITE)	976339.	IN.**4
Y BOTTOM (COMP)	47.91	IN.	GROSS SLAB THICK	8.00	IN.
GIRDER SPACING	9.42	FT.	SBXML	0.885-	KSI
WS	1.191	K/FT.	WC	0.451	K/FT.
DAYS	90		FC (GIRDER)	7.20	KSI
NO. CONT. ENDS	0		HARP. DIST.	38.00	FT.
FC (SLAB)	4.50	KSI	SHRINK	0.00030	
CREEP	2.00				

GIRDER IS SIMPLE SPAN FOR LL AND CURB DL.
STRANDS ARE LOW RELAXATION.

GIRDER CAMBER AT TIME OF SLAB POUR(90 DAYS)- 1.97 INCHES

DEFLECTION DUE TO SLAB WT - 0.84 INCHES
DEFLECTION DUE TO CURB + ASPH - 0.16 INCHES

SET SCREEDS FOR A TOTAL OF * 1.00 * INCHES CAMBER
ABOVE GRADE AT C.L. SPAN.

FINAL CAMBER OF GIRDER - 0.98 INCHES ABOVE THE HORIZONTAL POSITION.

EXCESS GIRDER CAMBER ALLOWANCE FOR ESTABLISHING THE 'A' DIMENSION
PER BRIDGE DESIGN MANUAL, VOLUME 2, PAGE 6-360 IS 1.13 INCHES.

APPROXIMATE LIVE LOAD + IMPACT DEFL. IS 0.46 INCHES

Northern Avenue Bridge Over the New River
Prestressed Girder Analysis

The ADOT current policy is to design prestressed I girders as simply supported composite sections for live load and impact and all superimposed dead loads. Negative moment reinforcement is then designed over intermediate supports considering span continuity for all composite dead loads and live load plus impact. It does not appear that a line of girders could be saved by a less conservative approach, so ADOT's policy will be followed.

Preliminary Depth at Supports

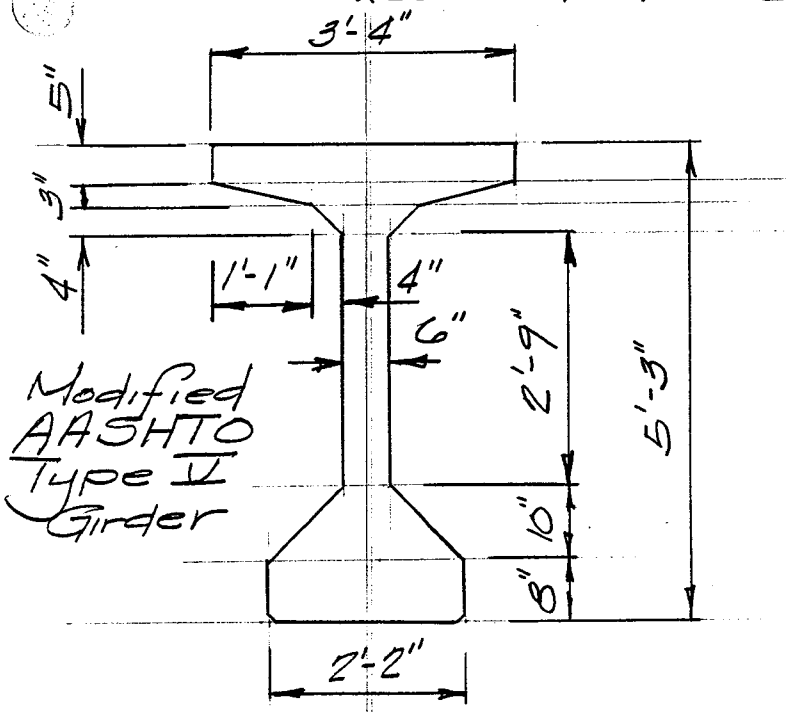
Slab Thickness plus 3/4" Fillet = 8 3/4"

Top Flange Effect = $.02\left(\frac{40''}{2}\right)$ = + 3/8"

Vertical Curve Effect = $-\frac{1.5(G)(S)^2}{L} = \frac{1.5(3.54)(.95)^2}{8} = -5/8''$

Excess Girder Camber (Assume & verify) = +1 1/4"

9 3/4"



Design Criteria

Girder

Area = 667 Sq. In.

Wt/Ft = 924#/Lf

$I = 479,459 \text{ in}^4$

$y_b = 32.02$ in

$S_T = 15,476 \text{ in}^3$

$S_B = 14,974 \text{ in}^3$

BRW

JOB Northern Avenue Br SHEET NO. G-1

MADE BY Lee Arnold DATE 1/90 JOB NO. 93-89-2

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 20

CALCULATIONS FOR Prestressed Girders

Northern Avenue Bridge Over the New River
Prestressed Girder (Cont'd)

Design Criteria (Cont'd)

	Concrete Strength	E
Girders @ Release	5000 psi	4,030,000 psi
Girders @ 28 Days	6000 psi	4,415,000 psi
Slab and Diaphragms	4000 psi	3,605,000 psi

Impact for 95' Span = 22.7%

Simple Beam Moment for HS20 Truck = 1434'K

Live Load Lanes = $\frac{9.42}{5.5 \times 2}$ = 0.857 lanes

I of Composite Section @ Mid-Span

15

Area = 1801 in^2
y_b = 50.28 y_t = 21.97
I_{xx} = 1,067,762 in^4
ST = 48,600 in^3
SB = 21,236 in^3

Taken from Sheet 5-10
These require reduction to effective flange width

Revised I for composite section with reduced effective width & 1/2" Slab Wearing

	Area	y	Ay	Ay^2	I_{oo}
Slab 8(12)7½	720	68.0	48,960	3,329,280	3375
Filler Pad	50	63.625	3,181	202,407	6
Girder	887	32.02	28,402	909,424	479,459
	*1657		80,543	4,441,111	482,840

$y_b = \frac{80,543}{1657} = {}^*48.61$

$I_{xx} = 482,840 + 4,441,111 - 1657(48.61)^2 = {}^*1,008,572$

Note: * These values will be slightly higher the program, because program does not account for PAD

BRW

JOB Northern Avenue Br SHEET NO. G-2
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 21
CALCULATIONS FOR Prestressed Girders

Northern Avenue Bridge Over the New River
Prestressed Girder (Cont'd)

Recalc Composite Section Properties reducing Top slab & pad by Modular ratio

$$\frac{E_s}{E_g} = \frac{\sqrt{4000}}{\sqrt{6000}} = 0.817$$

	Area	y	Ay	Ay^2	I_{oo}
Slab	588	68	39,976	2,718,346	2756
Filler Pad	41	63.625	2,598	165,264	5
Girder	887	32.02	28,402	909,424	479,459
	1516		70,976	3,793,054	482,220

$y_{bc} = \frac{70,976}{1516} = 46.82$ $\quad y_{tc} = 16.18$ $\quad y_{TSC} = 24.93$

$I_{xxc} = 482,220 + 3,793,054 - 1516(46.82)^2 = 952,012 \text{ in}^4$

$\therefore S_{bc} = 20,333 \text{ in}^3$ $\quad S_{TC} = 58,838 \text{ in}^3$ $\quad S_{TSC} = 38,187 \text{ in}^3$

Dead Load Stress (w/o prestress)

		Moment	Top	Bottom
DL Girder	$.924\left(\frac{95}{8}\right)^2$ =	1043'k	809 psi	836 psi
Slab	$.67(9.42).150\left(\frac{95}{8}\right)^2$ =	1068'k	828 psi	856 psi
PAD	$0.10(3.33).150\left(\frac{95}{8}\right)^2$ =	56'k	43 psi	45 psi
Diaphragms	$3.6(95)\frac{1}{3}$ =	114'k	88 psi	91 psi
$3.75(.75)6.5(.150)=3.6$	Totals	2,281 k	1768 psi	1828 psi

Added DL = FWS + Barrier + Utilities

$= 236 + \frac{2(372)}{6 \text{ Gdr}} + 59 = 419 \frac{lb}{Lf}$ $\quad M = \frac{.419(95)^2}{8} = 473'k$

Composite Stress (w/o Prestress)

		Moment	Top	Bottom
LL+I = 1434 (1.227) 0.857	=	1508'k	306 psi	690 psi
Superimposed DL	=	473'k	97 psi	279 psi
		1981'k	403 psi	1169 psi

BRW

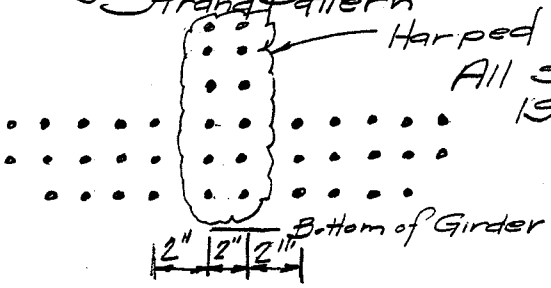
JOB Northern Avenue Bridge SHEET NO. G-3
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 22
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girder (Cont'd)

Total Stresses (W/O Prestress) for Girder

	Top	Bottom
	2171 psi	2997 psi
Allowable Tension $3\sqrt{f'c}$	232 psi	232 psi
	2403 psi	2765 psi

Try 28 Straight & 12 Harped Strand Pattern



All strand spacing is 2"

Determine cg @ midspan to harping point

10	x	2	=	20
12	x	4	=	48
12	x	6	=	72
2	x	8	=	16
2	x	10	=	20
2	x	12	=	24
40				200

$cg = \frac{200}{40} = 5.0$

$E = Y_B - cg = 32.02 - 5.0 = 27.02''$

Assume Total System losses of 35 KSI & losses after transfer of 22 KSI for low-Lax Strand

$$P_f = \frac{f_{des} \times \frac{1}{1000}}{\frac{1}{A} + \frac{E y_b}{I_g}}$$

$$= \frac{2765 \times \frac{1}{1000}}{\frac{1}{887} + \frac{27.02(32.02)}{479,459}} = 943 \text{ KIPS}$$

$$A_s = \frac{943}{(202.5 - 45)} = 6.00 \square'' \approx 40 \text{ Strands}$$

Check Losses $\Delta f_s = SH + ES + CR_c + CR_s$

Shrinkage $SH = 17,000 - 150(RH)$
from Fig. 9.16.2.1.1 the Relative Humidity in Phoenix is 40%

$\therefore SH = 17,000 - 150(40) = 11,000$ psi

BRW

JOB Northern Avenue Bridge SHEET NO. G-4
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 23
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

Elastic Shortening

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

$E_s = 28 \times 10^6$ psi

$E_{ci} = 33(150)^{1.5}(5000)^{1/2}$
$= 4.286 \times 10^6$ psi

Reduced tendon Stress $= 0.69 f'_s$

$$f_{cir} = \frac{F}{A} + \frac{Fe^2}{I_g} - \frac{M_n^{db} \times e}{I_g}$$

where M_n^{db} = Moment due dead load of the beam @ point of maximum moment, e.g. - midspan

$$ES = \frac{28 \times 10^6}{4.286 \times 10^6}\left[\frac{0.69(270)40(.1531)}{887} + \frac{.69(270)(27.02)^2(40)(.1531)}{479,459} - \frac{1043(12)(27.02)}{479,459}\right] = 15.144 \text{ ksi}$$

$f_{cir} = 2318$ psi

Relaxation of Prestressing Steel

for low relaxation Strand Pretensioned Member

$$C_{RS} = 5,000 - 0.10\,ES - 0.05(SH + CR_c)$$

∴ Calculate Concrete Creep (CR_c)

$$C_{RC} = 12 f_{cir} - 7 f_{cds} \quad \text{where } f_{cir}\ 2,318 \text{ psi}$$

$$f_{cds} = \frac{M_n^s e}{I_g} + \frac{M_c^d\left[e + (\bar{y}' - \bar{y})\right]}{I'}$$

$$= \frac{(2,281 - 1043)12(27.02)}{479,459} + \frac{473(12)\left[27.02 + (46.82 - 32.02)\right]}{952,012}$$

$= 1087$ psi (1.087 ksi)

$$C_{RC} = 12(2318) - 7(1087) = 20,207 \text{ psi}$$

Adjusted for 40 Strand

$$C_{RS} = 5,000 - 0.10(15,144) - 0.05(11,000 + 20,207)$$
$= 1926$

$$\Delta f_s = 11,000 + 15,144 + 20,207 + 1926 = 48,277 \text{ psi}$$

BRW

JOB Northern Avenue Bridge SHEET NO. G-5
MADE BY Lee Arnold DATE _____ JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 24
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

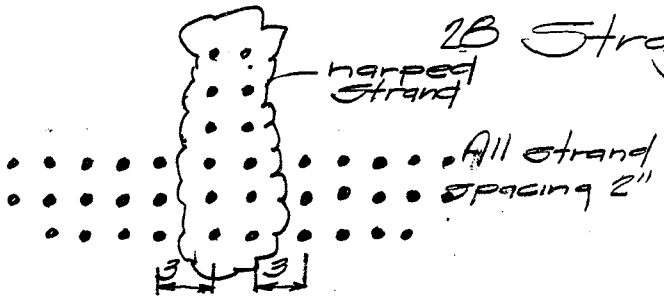
Prestressing Steel

$$A_s = \frac{P_f}{0.75(f_s) - \Delta f_s} = \frac{943^{K.P}}{.75(270) - 48.277}$$

$= 6.12 \square''$ 40 – ½" ϕ Strand

Use 40 Strand

28 Straight & 12 Harped



Strand Pattern

$A_s = 40(.1531) = 6.12 \square''$

$E = Y_b - cg = 32.02 - 5.00 = 27.02$

$F_f =$ Final Force $= 40(.1531)[.75(270) - 48.277]$
$= 944^K$

$F_t =$ Force @ Transfer $= F_f + [\Delta f_s - ES](A_s)$

Where $ES = \frac{28 \times 10^6}{4.286 \times 10^6}\left[\frac{0.69(270)40(.1531)}{887} + \frac{0.69(270)(27.02)^2}{479,459}(.153 - \frac{1043(12)(27.02)}{479,459}\right] = 15.144$ KSI

@ transfer

$F_t = 943 + [48.277 - 15.144](40)(.1531) = 1,153^K$

@ Jacking

$F_J = [(F_t) + ES(40)(.1531)] = [1,153 + 15.144(40)(.1531)]$

$= 1246$ KIPS $< 0.75(270)40(.1531)$ NG ∴ Add 2 Strand
1240

BRW

JOB Northern Avenue Bridge SHEET NO. J-6
MADE BY Lee Arnold DATE JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 25
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

$$ES = \frac{28\times10^6}{4,286}\times10^6\left[\frac{0.69(270)42(.1531)}{887}+\frac{0.69(270)(26.88)^2(42)(.1531)}{479,459}-\frac{1043(12)(26.88)}{479,459}\right] = 16.033 \text{ Ksi}$$

$$f_{cds} = \frac{(2281-1043)12(26.88)}{479,459}+\frac{473(12)[26.88+(47.34-32.02)]}{952,012}$$

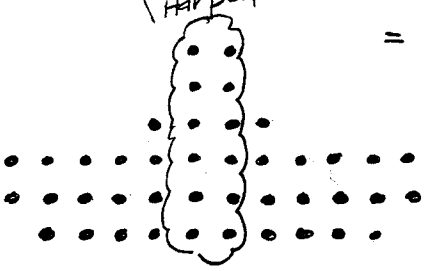
$$= 1061 \text{ psi } (1.061 \text{ Ksi})$$

$$C_{RC} = 12 f_{cir} - 7 f_{cds} = 12(2454)-7(1061) = 22,021 \text{ psi}$$

$$C_{RS} = 5,000 - 0.10(16,033) - 0.05(11,000+22,021)$$

$$= 1746 \text{ psi}$$

Harped



2 x 12	= 24
2 x 10	= 20
4 x 8	= 32
12 x 6	= 72
12 x 4	= 48
10 x 2	= 20
42	216

$$cg = \frac{216}{42} = 5.14$$

$$E = Y_b - cg$$

$$= 32.02 - 5.14 = 26.88$$

$$F_f = \text{Final Force} = 42(.1531)[.75(270)-50.80] = 975^K$$

$$F_t = \text{Force @ Transfer} = F_f + [\Delta f_s - ES]A_s$$

where $ES = 16,033$ $\quad \Delta f_s = 11,000+16,033+22,021+1746 = 50,800$

$$F_t = 943 + [50,800-16,033]42(.1531)$$

$$= 1166^{KIP}$$

@ Jacking

$$F_J = [(F_t) + ES(42)(.1531)]$$

$$= 1166 + \frac{16,033(42)(.1531)}{1000}$$

$$= 1270^{KIP} < 0.75(270)42(.1531) = 1302 \therefore \text{OK}$$

BRW

JOB Northern Avenue Bridge SHEET NO. G-7
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 26
CALCULATIONS FOR Prestressed Girders

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

$P_J = 42(.1531)\,270\,(.75) = 1302^{\text{KIPS}}$

$P_t = P_J - ES(A_s) = 1302 - 16.033(42)(.1531) = 1199^{\text{KIPS}}$

$P_f = 1302 - 42(.1531)50800 = 975 > 943^K \therefore \text{OK}$

$FAI = \frac{P_t}{A_g} = \frac{1199}{887} = 1352 \text{ psi}$

$FA = \frac{P_f}{A_g} = \frac{975}{887} = 1100 \text{ psi}$

Combined Stresses

$R^2 = \frac{I_g}{A_g} = \frac{479,479}{887} = 540$

At Transfer

Top Fiber

$f_{ti} = FAI\left(1 - \frac{Ey_t}{R^2}\right) + f_t$ ← ℄ Girder

$1 - \frac{Ey_t}{R^2} = 1 - \frac{26.88(30.98)}{540} = -0.5421$

Bottom Fiber

$f_{bi} = FAI\left(1 + \frac{Ey_b}{R^2}\right) - f_b$ ← ℄ Girder

$1 + \frac{Ey_b}{R^2} = 1 + \frac{26.88(32.02)}{540} = 2.5939$

Stresses at the 4/10 Pt (harping point) = 96% of the stress @ ℄

@ ℄

$f_{ti} = 1352(-0.5421) + 809 = 76 \text{ psi}$

$f_{bi} = 1352(2.5939) - 836 = 2671 \text{ psi}$

@ 4/10 Pt

$f_{ti} = 1352(-0.5421) + (.96)809 = 44 \text{ psi}$

$f_{bi} = 1352(2.5939) - .96(836) = 2705$

Critical release stress

After placement of the slab

$f_t = FA\left(1 - \frac{Ey_t}{R^2}\right) + f_t$ ← ℄ Gir, Slab, Pad & Diaph

$f_b = FA\left(1 + \frac{Ey_b}{R^2}\right) + f_b$ ← ℄ Gdr, Slab, pad & diaph

This loading will not control design

BRW

JOB Northern Avenue Bridge SHEET NO. G-8
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 27
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

✓ Stresses under full design Load

$f_{t+} = FA\left(1 - \frac{Ey_t}{r^2}\right) + f_t$ (all DL & LL)

$f_{b+} = FA\left(1 + \frac{Ey_b}{r^2}\right) - f_b$ (all DL & LL)

@ ℄

$f_{t+} = 1100(-0.5421) + (1768+403) = 1575$ psi

$f_{b+} = 1100(2.5939) - (1828+1169) = -144$

Allowable $= 3\sqrt{6{,}000}$
232 psi ∴ OK

@ harping Point

$f_{t+} = 1100(.5421) + 0.96(1768+403) = 1488$ psi

$f_{b+} = 1100(2.5939) - .96(1828-1169) = -24$ psi

Required Concrete Strength

@ release

$f'_{c1} = \frac{f_b}{0.60} = \frac{2705}{0.60} = 4600$ psi Use 5000

@ Final Design Stress

$f'_c = \frac{1575}{0.4} = 4000$ psi Use $f'_c = 6000$ for more durability.

✓ Cable Path
harped strand cg is 6" below the top of girder @ the end

$cg = 12 \times (63-7) = 672$
$8 \times 2 = 16$
$10 \times 4 = 40$
$10 \times 6 = 60$
$2 \times 8 = 16$
42 804

$cg = \frac{804}{42} = 19.14$ off bottom

BRW

JOB Northern Avenue Bridge SHEET NO. G-9
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 28
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girder (Cont'd)

Kern bottom Limit $= \frac{R^2}{y_t} = \frac{540}{(63-32.02)} = 17.43''$ below cg

Center of jacking force $= 32.02 - 19.14$
$= 12.88''$

Which is within Kern ∴ no tension exists at beam end

Check Ultimate Strength

M_u applied $= 1.3\left[M_{DL} + \frac{5}{3}(M_{LL+I})\right]$

$= 1.3\left[(2281+473) + \frac{5}{3}(1508)\right] = 6,850'^k$

$\rho = \frac{A_s}{bd} = \frac{40(0.1531)}{96(71.75-5.40)} = 0.0010$

$f^*_{su} = f_s\left(1 - 0.5\rho\frac{f_s}{f'_c}\right) = 270\left[1-(0.5)\frac{270}{6}(0.0010)\right]$
$= 264.16$ ksi

$\delta = 1.4\,d\rho\frac{f^*_{su}}{f'_c} = 1.4(66.35)(.0010)\frac{(264.16)}{6} = 4.09''$
$< 7\frac{1}{2}''$ ∴ it is a rectangular section

$\phi M_n = \phi\left[A^*_s f^*_{su}\, d\left(1 - 0.6\rho\frac{f^*_{su}}{f'_c}\right)\right]\frac{1}{12}$

$= 1.0\left[42(0.1531)264.16(66.35)\left(1-0.6(0.0010)\frac{264.16}{6}\right)\right]\frac{1}{12}$

$= 9152^k > 6850^k$

∴ Ultimate Strength Capacity is adequate

Shear Reinforcement

✓ Shear requirement @ D/2

V_{DL} @ End $= 0.924\left(\frac{95}{2}\right) + 0.67(9.42)(.150)\frac{95}{2} + .05\left(\frac{95}{2}\right) + 3.6$
$+ 0.415\left(\frac{95}{2}\right) = 114.54$

BRW

JOB Northern Avenue Br SHEET NO. G-10
MADE BY Lee Arnold DATE 1/90 JOB NO. 33-63962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 29
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girders (Cont'd)

Shear Reinforcement

℄ Reaction = 64.9^K @ End

With Impact $79.63^{K/P}$

$V_{u\,DL} = 146.30$

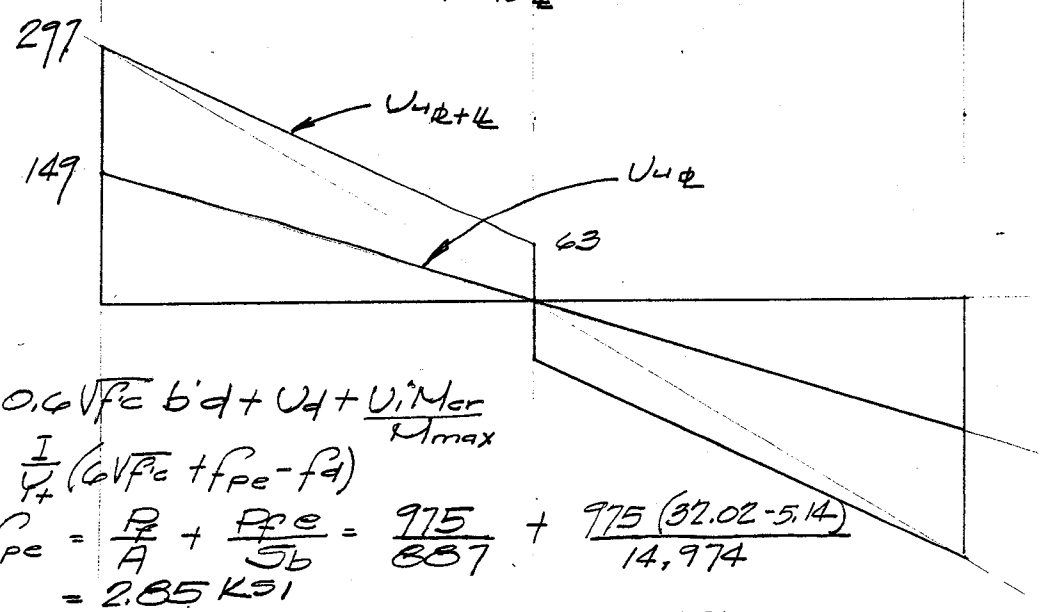
Shear @ Midspan = 29.1 / Lane

$35.7 \frac{KIPS}{Lane}$

$V_{u\,LL} = 1.227 \times .857 \times 64.9 \times 1.3 \times 5/3 = 148$

$V_u = 1.3[114.54 + 5/3(64.9)1.227(.857)] = 297$

$V_{u\,midspan\ due\ to\ LL} = 66.3 \leftarrow 1.3 \times 5/3 \times 29.1 \times 1.227 \times .857$



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

$M_{cr} = \frac{I}{y_t}(6\sqrt{f'_c} + f_{pe} - f_d)$

$f_{pe} = \frac{P_z}{A} + \frac{P_z e}{S_b} = \frac{975}{887} + \frac{975(32.02-5.14)}{14{,}974}$

$= 2.85$ KSI

$f_d = \frac{856 + 45 + 91 + 279}{1000} = 1.271$ KSI

$M_{cr} = \frac{952{,}012}{24.93}\left[6(6000)^{1/2} + 2850 - 1271\right]\frac{1}{12000} = 6503^{IK}$

$V_{ci} = 0.6\frac{(6000)^{1/2}}{1000}(6)(.8)(63) + 114.54 + \frac{6503 \times 143.6}{3882} = 369^K$

where $V_i = 1.3[5/3 \times 66.3] = 143.6^K$

$M_{max} = 1.3[473 + 5/3(1508)] = 3882$

$V_{cw} = [3.5(f'_c)^{1/2} + 0.3 f_{pc}]b'd + V_p$

$f_{pc} = \frac{P}{A} + \frac{Pe}{S} =$

where $P = 42(.1531)[270(.75) - 50.8] = 975^K$

$\frac{975}{887} = 1100$ PSI

P/S @ centroid of Cross Section

$1100\left(1 - \frac{E y'_t}{P_z}\right) = 1100\left[1 - \frac{26.88(46.82-32.02)}{540}\right] = 290$ PSI

JOB Northern Avenue Bridge SHEET NO. G-11

MADE BY Lee Arnold DATE 1/90 JOB NO. 33-8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 30

CALCULATIONS FOR Prestressed Girder

BRW

Northern Avenue Bridge Over the New River
Prestressed Girder (Cont'd)

$S_{@\ NA\ of\ Composite} = \frac{479,459}{46.82 - 32.02} = 32,396\ in^3$

Stresses

℄ Gdr	$= \frac{1043 \times 12000}{32,396}$		$= 386$ psi
Slab	$= \frac{1068 \times 12000}{32,396}$		$= 396$
Pad	$= \frac{56 \times 12,000}{32,396}$		$= 21$
Diaph	$= \frac{114 \times 12,000}{32,396}$		$= 42$
			845 PS
Plus P/S			290
			1,135

$V_{p/s} = 975 \left(\frac{19.14 - 5.14}{[(14)^2 + (.4 \times 95 \times 12)^2]^{1/2}} \right) = 29.9^{kips}$

$V_{cw} = \frac{[3.5(6000)^{1/2} + 0.3(1135)]\,6(.8)43}{1000} + 29.9 = 214^k$

∴ Use V_{cw}

$V_s = \frac{V_u}{\phi} - V_{cw} = \frac{297}{.85} - 214 = 135^k$

$S = \frac{A_v f_{sy} d}{V_s} = \frac{2(.2)60(50.4)}{135} = 8.9''$ @ Ends

Minimum Web Reinforcement

$S = \frac{A_v f_{sy}}{50\ b_w} = \frac{(.40)60,000}{50(6)}$

Use 18" maximum

Space stirrups @ 6" for ≈ 3' @ Girder ends, the 8" for 0.15 x S, then at 12" to 0.3 x S, then @ 18" in the center 0.45

BRW

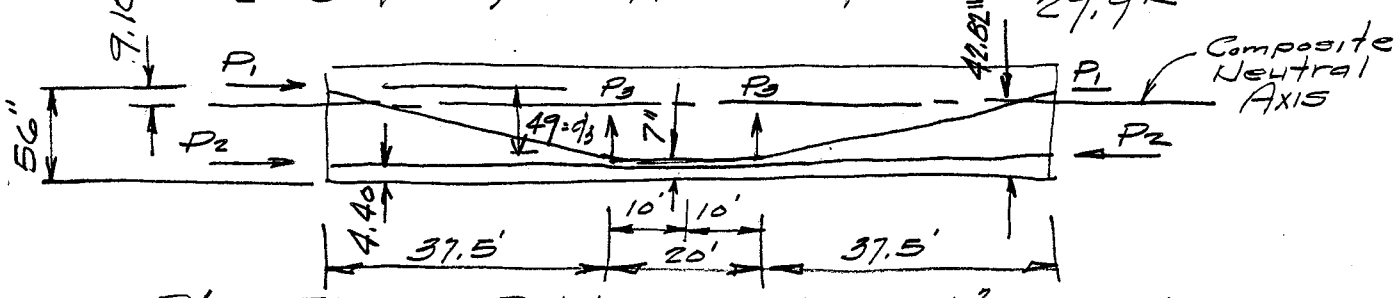
JOB Northern Avenue Bridge SHEET NO. G-12
MADE BY Lee Arnold DATE 2/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 31
CALCULATIONS FOR Prestressed Girder

Northern Avenue Bridge Over the New River
Prestressed Girder - Positive Moment
@ Piers

Final P/S = $0.1531[270(.75)-50.8] = 23.2 \frac{K}{Strand}$

$P_1 = 12(23.2) = 278.4^K$ P_3 is upward P/S force from G-12 29.9K

$P_2 = 30(23.2) = 696.0^K$



$$P/S\ Slope = \frac{P_1 d_1 L}{2EI} - \frac{P_2 d_2 L}{2EI} - \frac{P_3 L_1^2}{2EI} - \frac{P_3 L_1 L_2}{2EI}$$

$$P/S\ Slope = 278.4(9.18)97\times\frac{12}{2} - 696(42.42)97\times\frac{12}{2} - 29.9(37.5)^2\times\frac{144}{2}$$

$$- 29.9(37.5)20\times 144/2$$

$$= 1,487,425 - 17,183,155 - 3,027,375 - 1,614,600$$

$$= \frac{20,338,000 \text{ in-Kips}}{EI}$$

Ref PCA "Design of Continuous Hwy Bridges W/PC PS Concrete Girders" Pgs 10 & 11

$$@\ Piers\ 1\ \&\ 3 = \frac{-2.57EI}{97\times 12}\left[-\frac{20,338,000}{EI}\right] = 44\times 10^6 \text{ in lb} = 3742^{ft\text{-}lb}$$

$$@\ Pier\ 2 = \frac{-1.72EI}{97\times 12}\left[-\frac{20,338,000}{EI}\right] = 30\times 10^6 \text{ in lb} = 2504^{ft\text{-}k}$$

Creep Restraint Moment (not adjusted by creep adjust factor)

$$@\ Piers\ 1\ \&\ 3 = -0.86(3742) = -3218^{'K}$$

$$@\ Pier\ 2 = -0.572(3742) = -2140^{'K}$$

Phoenix Relative Humidity = 40%
Creep correction factor $1.10\times 0.600\times 10^6$
$= 0.660\times 10^6$
Continuity Connection @ 45 days
$\epsilon_s = 0.00066\times 0.5 = 0.00033$

BRW

JOB Northern Avenue Bridge SHEET NO. G-13
MADE BY Lee Arnold DATE 2/90 JOB NO.
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 32
CALCULATIONS FOR Positive Connection

Northern Avenue Bridge Over the New River
Prestressed Girder - Positive Moment

$M_s = \epsilon_s E_d A_B \left(e_2' + \frac{t}{2}\right)$ deck $f'_c = 4000$, $E_d = 3.6 \times 10^6$ psi

$M_s = .00033 \times 3.6 \times 10^6 \times 8 \times 113 \times 24.93 \times \frac{1}{12{,}000}$

$= 2{,}231$

Shrinkage Restraint Moments

@ Piers 1 & 3 $\quad -1.286(2{,}231) = -2869^{'K}$

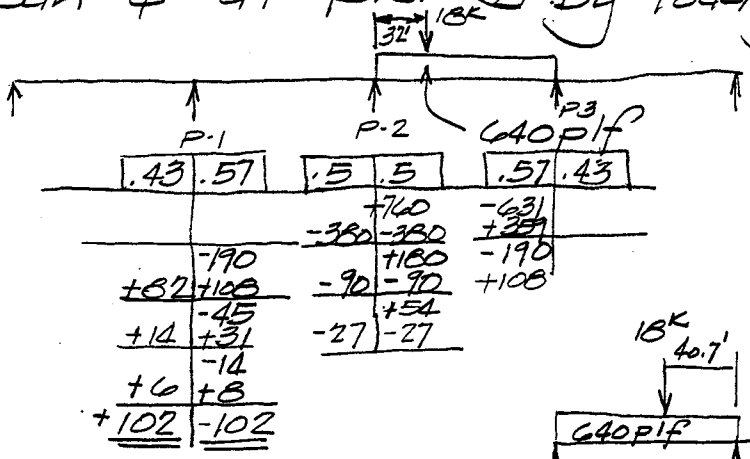
@ Pier 2 $\quad -0.857(2{,}231) = -1912^{'K}$

Positive Moments due to L+I — see below

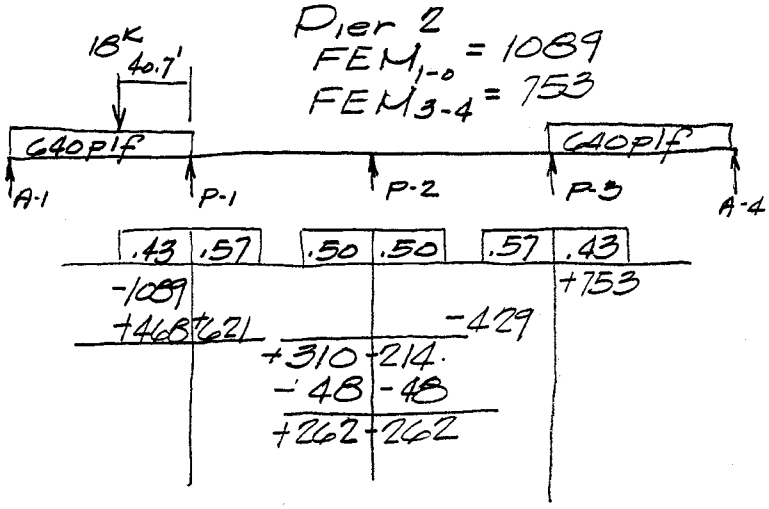
@ Piers 1 & 3 $= 0.857 \times 1.227 \times 102 = 108^{'K}$

@ Pier 2 $= 0.857 \times 1.227 \times 262 = 276^{'K}$

Maximum Positive Live load Moments at Piers 1 & 3 occur by loading opposite interior span & at pier 2 by loading exterior spans



Piers 1 & 3
$FEM_{23} = 760^{'K}$
$FEM_{32} = 631$



Pier 2
$FEM_{1-0} = 1089$
$FEM_{3-4} = 753$

JOB Northern Avenue Bridge SHEET NO. G-14
MADE BY Lee Arnold DATE 2/90 JOB NO.
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 33
CALCULATIONS FOR Positive Connection

BRW

Northern Avenue Bridge

Prestressed Girder - Positive Moment Connection

Evaluation of Girder Creep (Ref PCA "Design of Continuous Highway Bridges with Precast Prestressed Concrete Girders")

$E_{ci} = (5000)^{1/2} 57{,}000 = 4.03 \times 10^6$ psi

Use 20 years for ultimate creep

from figure 5 Creep $= 0.35 \times 10^{-6}$ in/in/psi

from figure 6 and 2 day release

Adjusted Specific creep $= 1.66 \times 0.35 \times 10^{-6}$

$= 0.58 \times 10^{-6}$ in/in/psi

from Table 1, adjust for volume-surface ration

$C_s = 1.18 \times 0.58 \times 10^{-6} = 0.68 \times 10^{-6}$ in/in/psi

In all likelihood, the girder will not be set for several months after release, use 28 days. This would leave 60% of the creep remain to occur (Fig 8)

Design Creep $= 0.6(1.68 \times 10^{-6}) = 0.41 \times 10^{-6}$ in/in/psi

$\phi = \epsilon_c \times E = 0.41 \times 10^{-6} \times (5000)^{1/2} 57{,}000 = 1.65$

$1 - e^{-\phi} = 1 - e^{-1.65} = 0.808$

$\left(\frac{1 - e^{\phi}}{\phi}\right) = 0.490$

Sum of Shrinkage plus L+I positive moment

at Piers 1 & 3 $(3742 - 3218).808 + 0.49(-2869) + 108 = -874^{k}$

at Piers 2 $(2504 - 2140).808 + 0.49(-1912) + 276 = -366^{k}$

The resultant of all moments is negative, such that positive moments never occur at the bents. Extend all strands in the bottom row of straight strands into the cap diaphragm & anchor

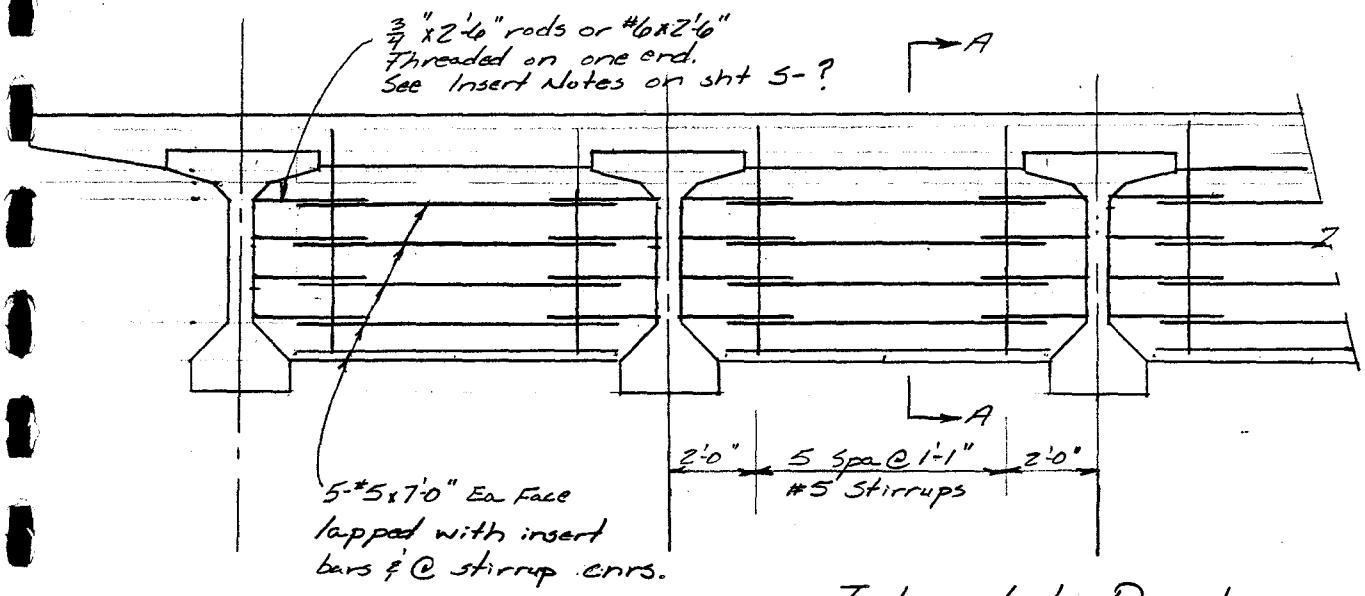
BRW

JOB Northern Avenue Bridge SHEET NO. G-15

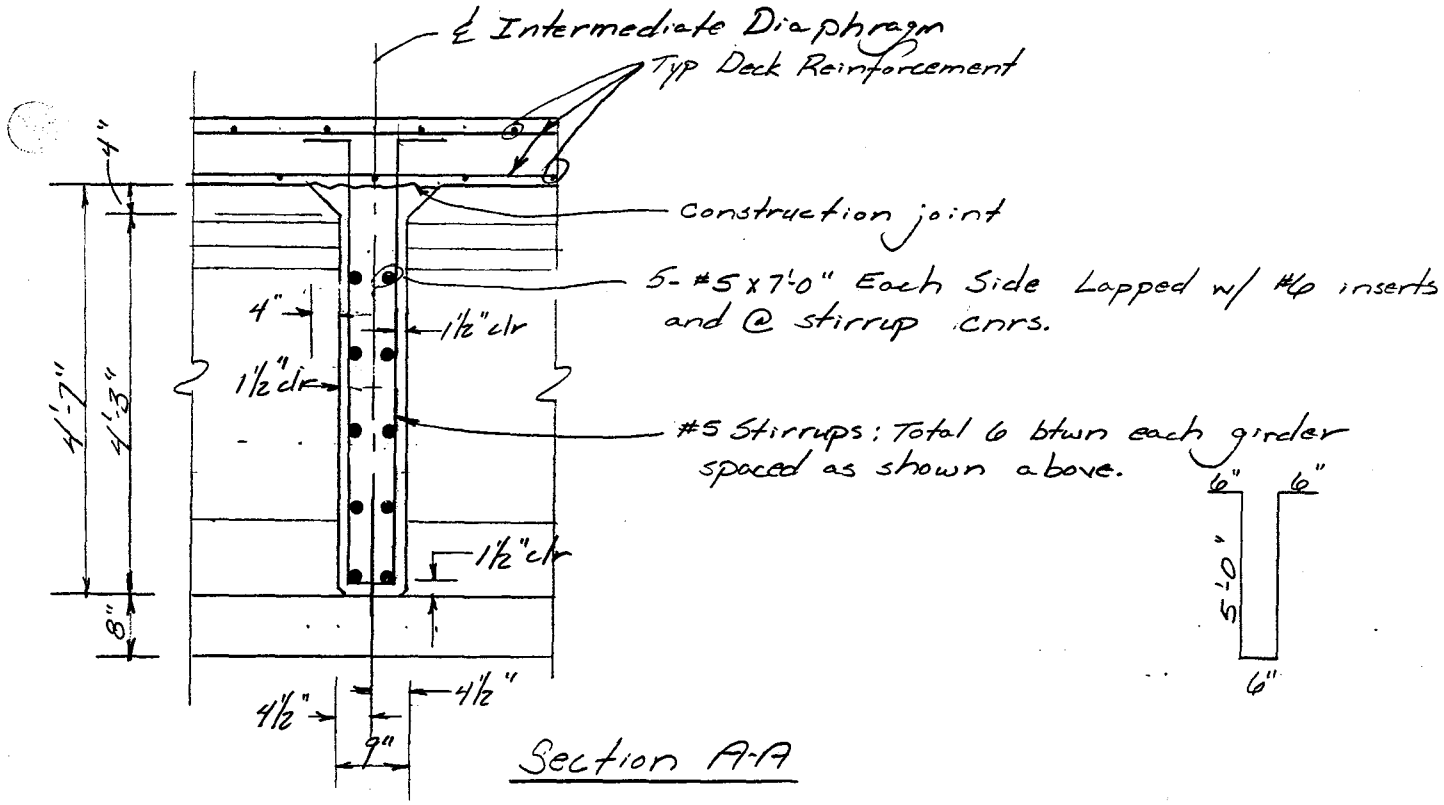
MADE BY Lee Arnold DATE 2/90 JOB NO.

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 34

CALCULATIONS FOR Positive Connection



Intermediate Diaphragm

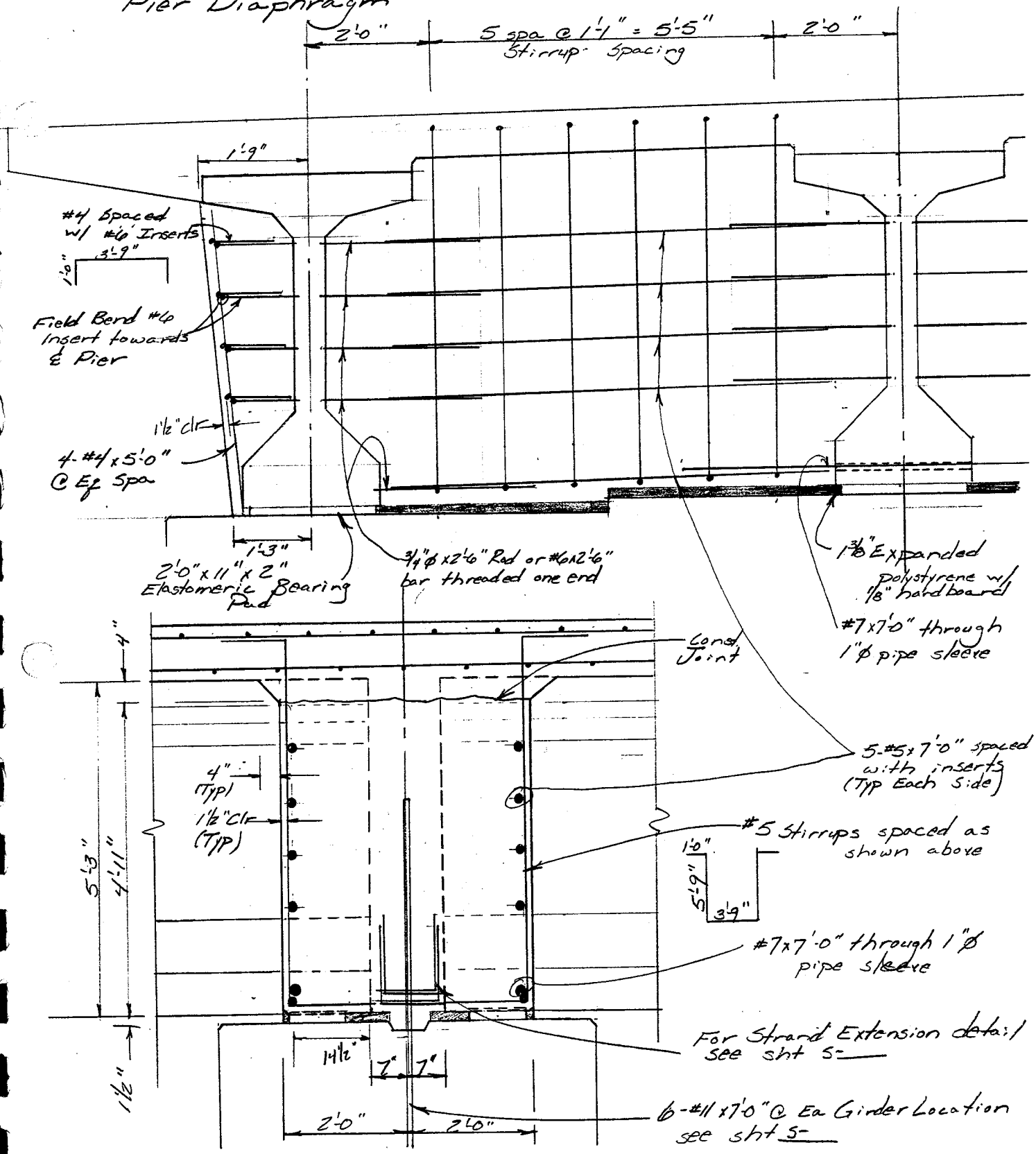


Section A-A

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. D-1
MADE BY T Stones DATE 1-30-90 JOB NO. 23-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 35
CALCULATIONS FOR Diaphragm Details

Pier Diaphragm



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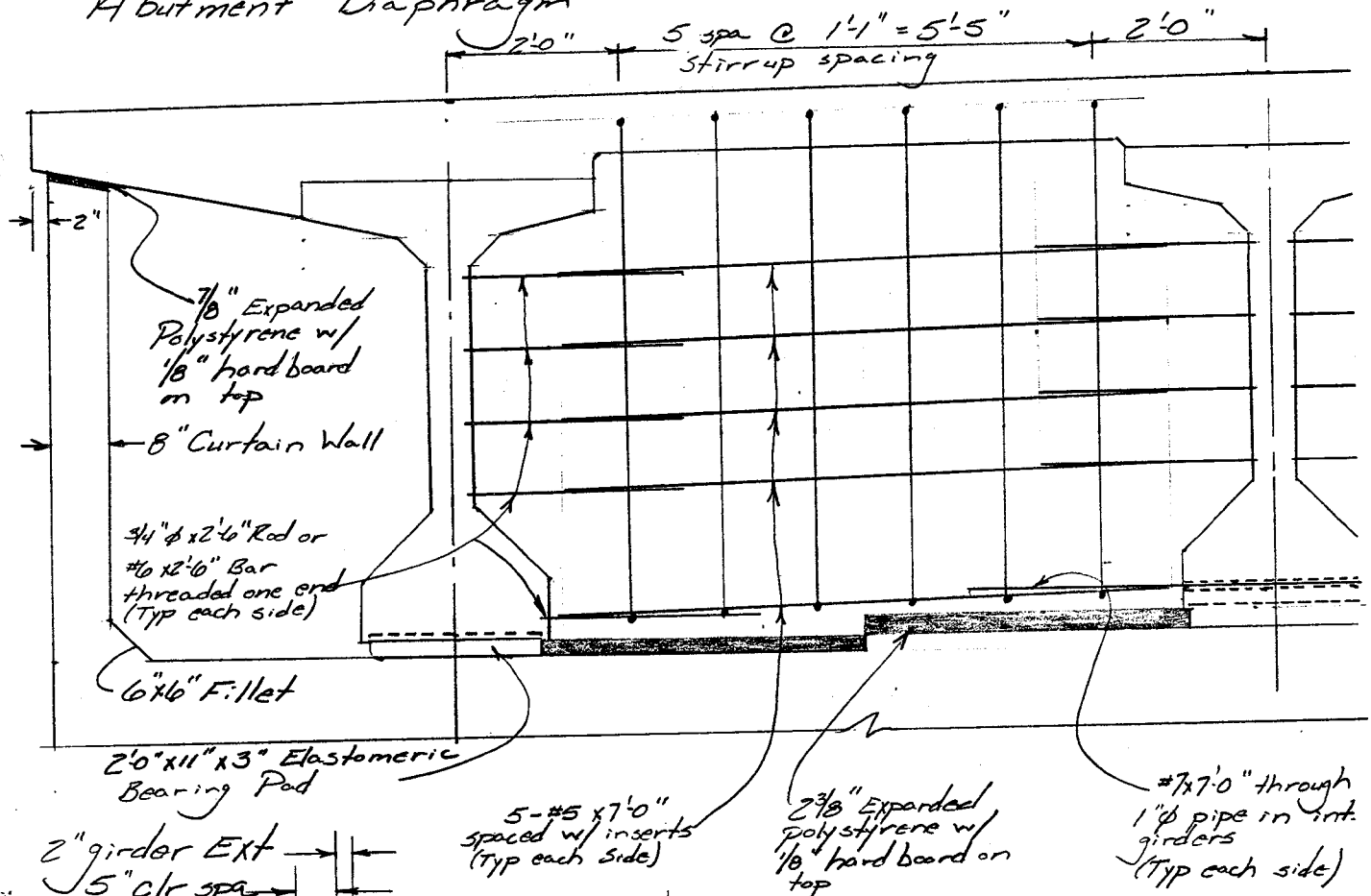
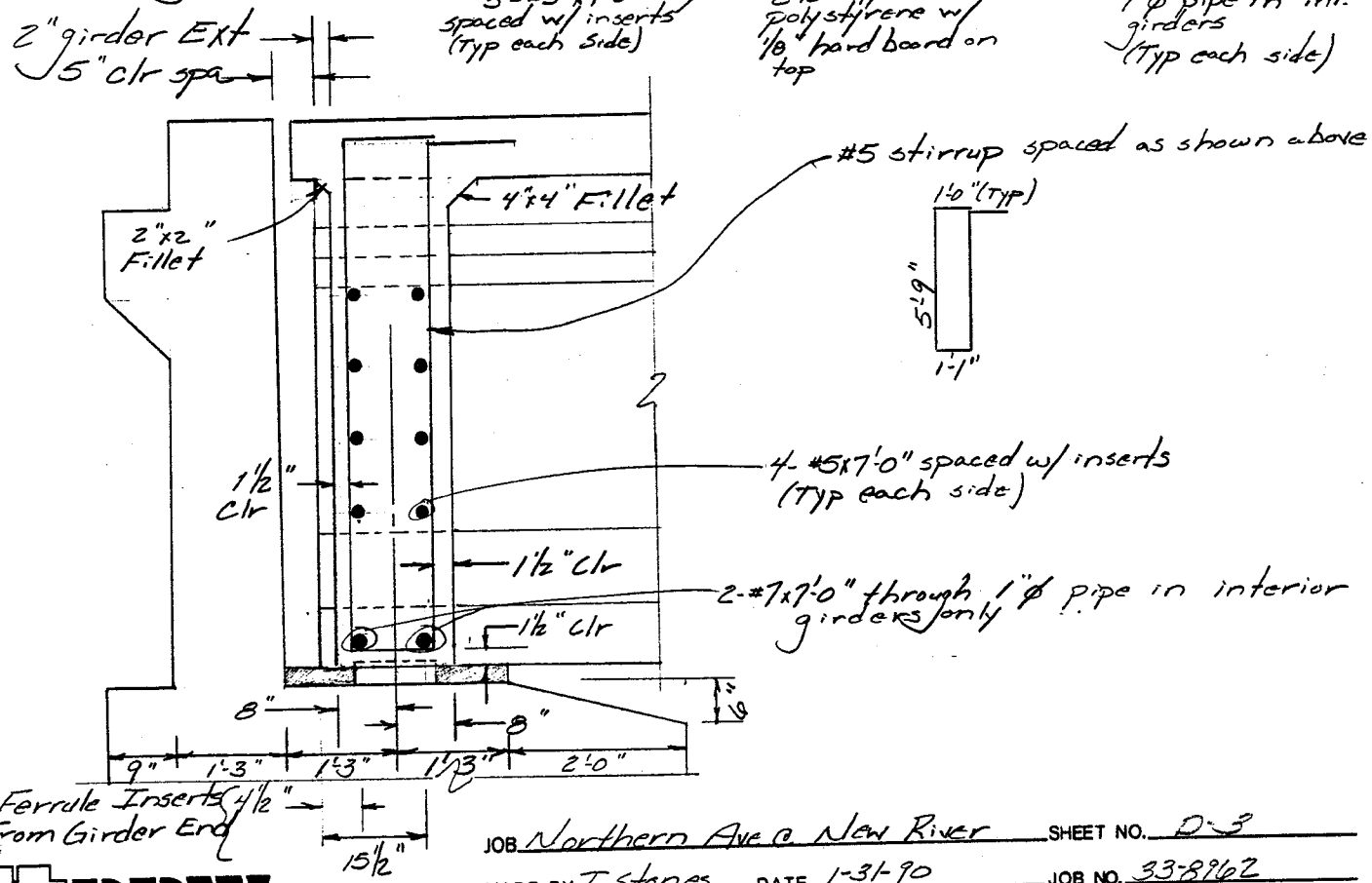
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. D2
MADE BY T Stones DATE 1-30-90 JOB NO. 33-89162
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 36
CALCULATIONS FOR Diaphragm Details

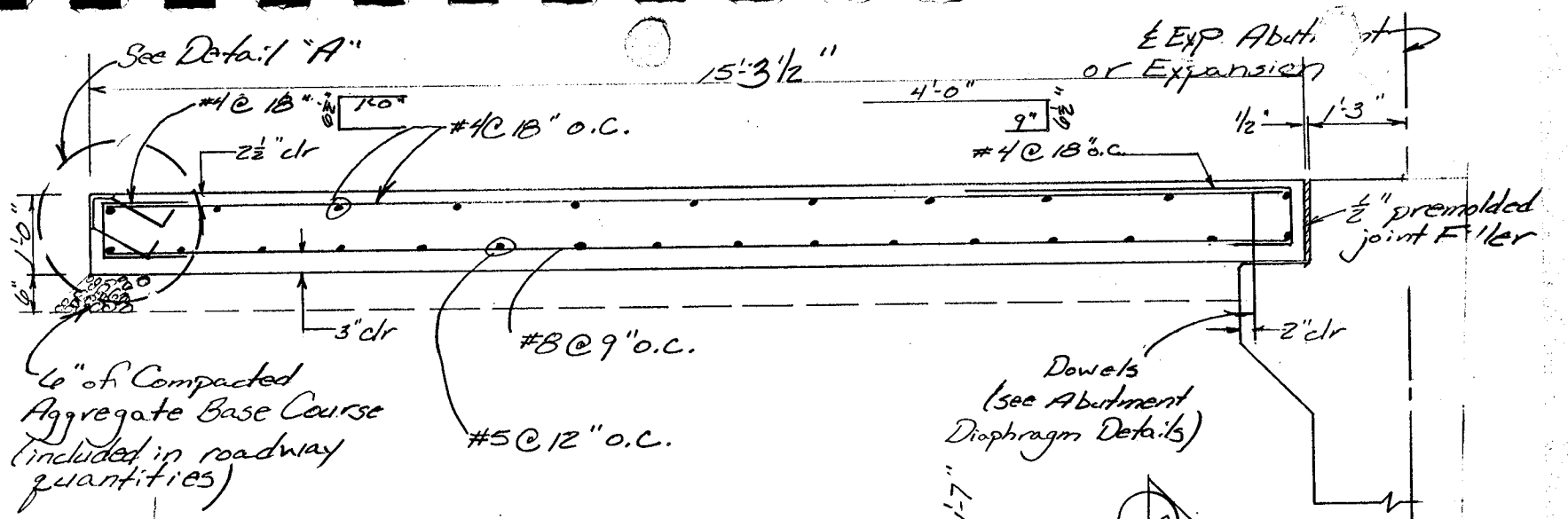
Abutment Diaphragm

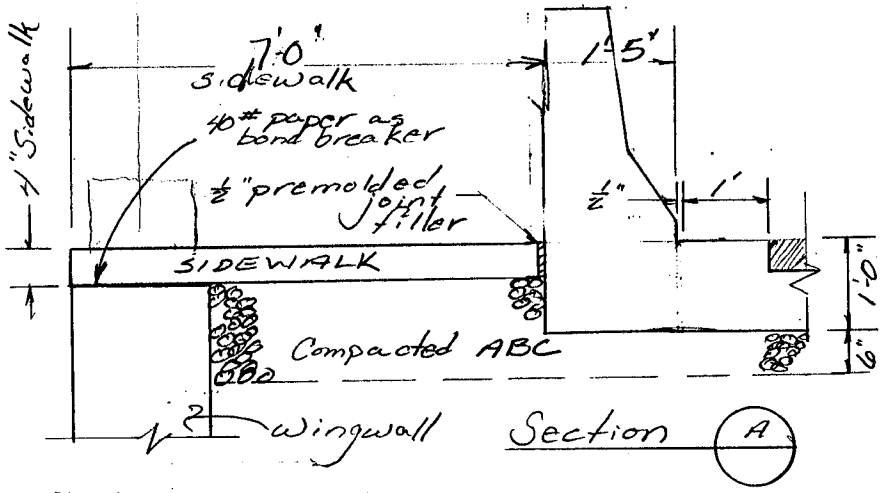


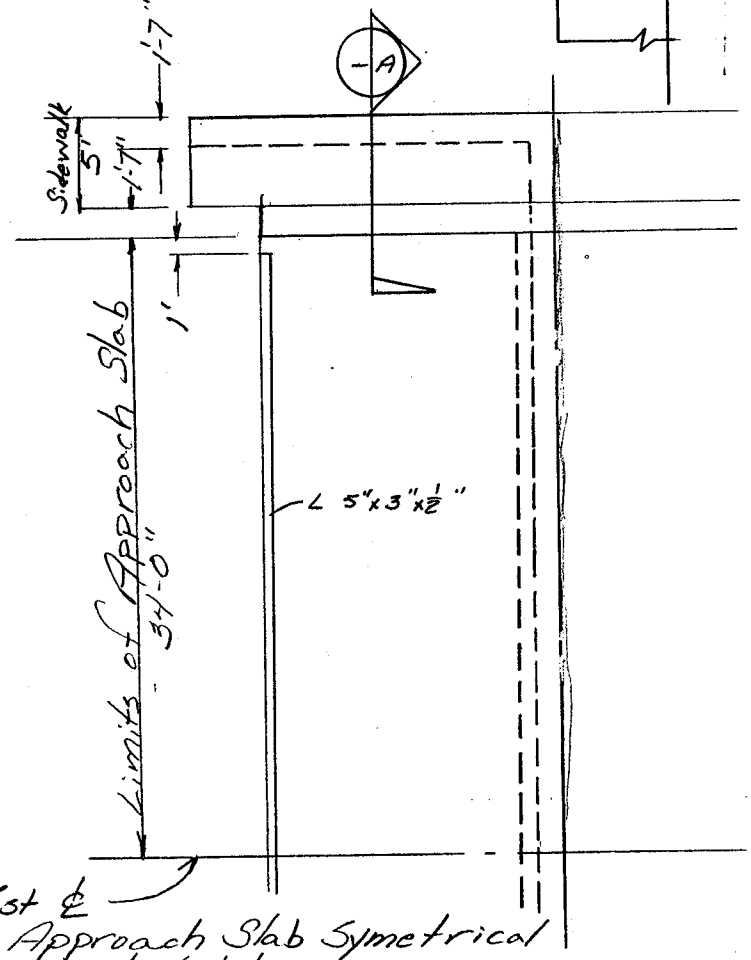


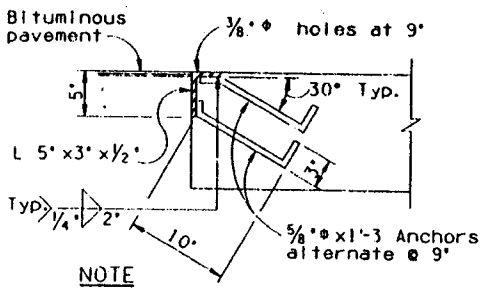
BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. D-3
MADE BY I Stones DATE 1-31-90 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 37
CALCULATIONS FOR Diaphragm Details







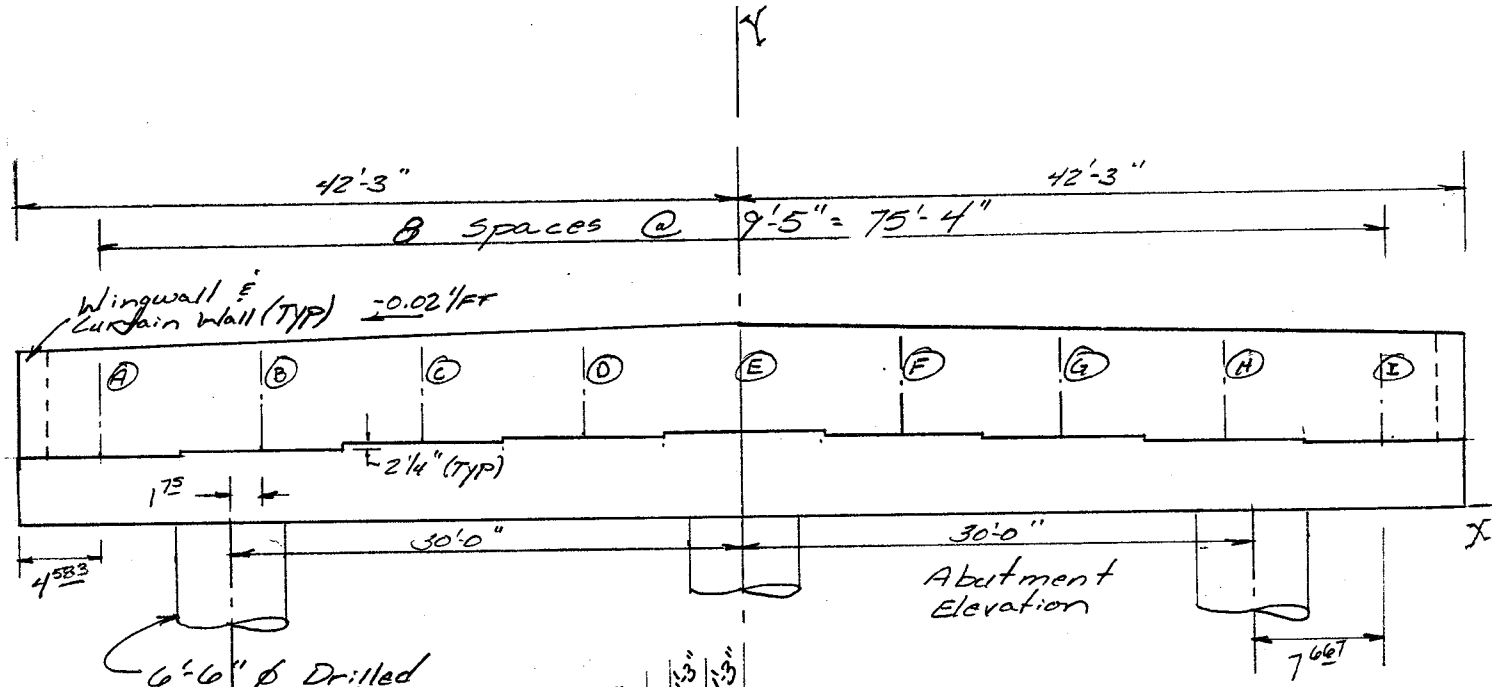


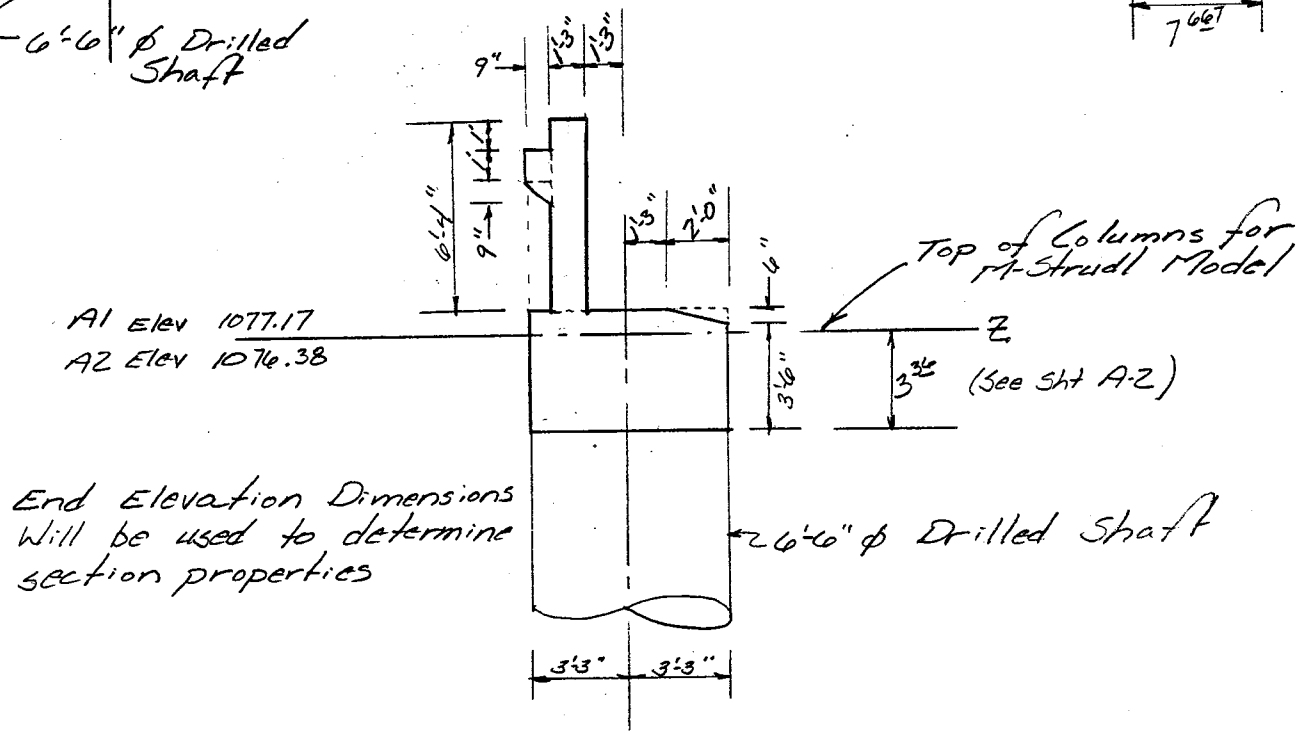
NOTE
Omit guard angle thru raised median and sidewalks.

Bituminous Pavement Alternate

DETAIL 'A'

JOB Northern Ave. @ New River SHEET NO.
MADE BY T. Stones DATE 9-14-89 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 10/89 SEC. SHT. NO. 38
CALCULATIONS FOR Approach Slab





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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-1
MADE BY T Stones DATE 1-24-90 JOB NO. 33-8912
CHECKED BY L. Arnold DATE 1/90 SEC. SHT. NO. 39
CALCULATIONS FOR Abutment Design

Section Properties

$f'_c = 4000$ psi density = 0.015 ksf therm = 0.000006

Abutment Cap

A (ft^2)	y_b (ft)	Ay_b (ft^3)	d (ft^2)	Ad^2 (ft^4)	I_{zz} (ft^4)
$4(6.5) = 26.0$	2.0	52.0	-1.359	48.021	$\frac{6.5}{12}(4)^3 = 34.667$
$-0.5(2)(0.5) = -0.5$	3.833	-1.917	0.474	-0.112	$-2/36(0.5)^3 = -0.007$
$6.333(1.25) = 7.916$	7.167	56.73	3.808	114.786	$1.25/12(6.333)^3 = 26.458$
$1.0(0.25) = 0.25$	8.833	6.625	5.474	22.473	$0.25/12(1)^3 = 0.063$
$0.5(0.25)^2 = 0.281$	8.083	2.271	4.724	6.271	$0.5/36(0.25)^3 = 0.001$
34.447		115.709		191.439	61.189

$$y_c = \frac{115.709}{34.447} = 3.359 \qquad d = y_b - y_c$$

$$I_{zz} = 61.189 + 191.439 = 252.628\ ft^4 \qquad Area = 34.447\ ft^2$$

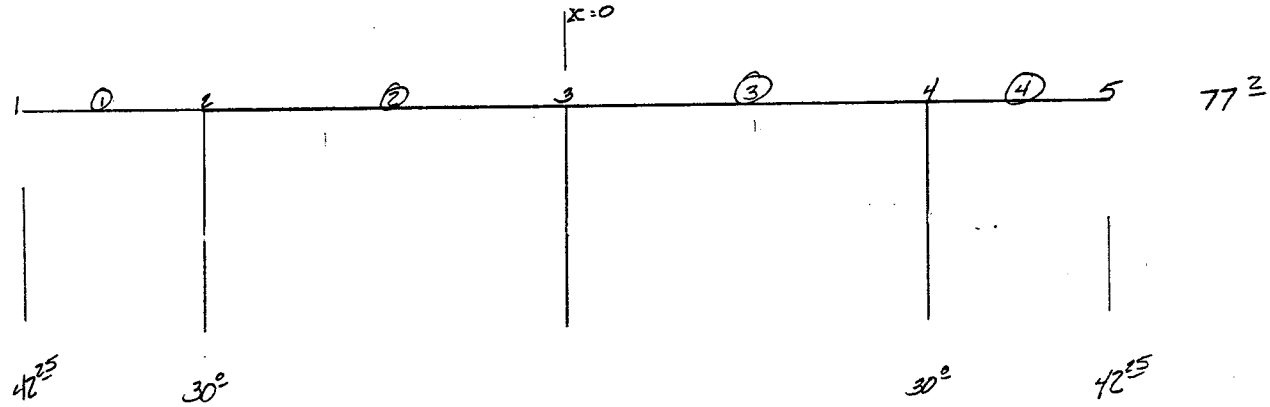
Shaft:

$$Area = \frac{\pi}{4}(6.5)^2 = 33.183\ ft^2$$

$$I_{zz} = \frac{\pi}{64}(6.5)^4 = 87.624\ ft^4$$

Very little lateral load will be applied to the abutment shafts: no stream and only wind from superstructure

Assume point of fixity @ 4x shaft ϕ = 26 ft.



JOB Northern Ave @ New River SHEET NO. A-2

MADE BY T Stones DATE 1-24-90 JOB NO. 33-8962

CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 40

CALCULATIONS FOR Abutment Design

BRW MINNEAPOLIS PHOENIX

Abutment will be modelled as a 3-D frame to apply moments caused by wingwalls and longitudinal force due to wind

poisson's ratio = 0.2

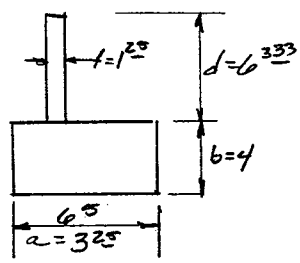
Abutment Cap

ft^2 A	ft x_ℓ	ft^3 Ax_c	ft d	ft^4 Ad^2	ft^4 I_{yy}
26^{0}	0	0	-0^{55}	7^{865}	$4(6^{5})^3/12 = 91^{542}$
-0^{5}	2^{583}	-1^{292}	-0^{742}	-0^{275}	$-0^{5}(2)^3/36 = -0^{111}$
7^{916}	-1^{875}	-14^{843}	-1^{325}	13^{898}	$6^{33}(1^{25})^3/12 = 1^{03}$
0^{75}	-2^{875}	-2^{156}	-2^{325}	4^{054}	$1(0^{75})^3/12 = 0^{035}$
0^{281}	-2^{75}	-0^{773}	-2^{20}	1^{360}	$0^{75}(0^{75})^3/36 = 0^{009}$
34^{447}		-19^{064}		$26^{902}\,ft^4$	$92^{505}\,ft^4$

$$x_c = \frac{-19^{064}}{34^{447}} = -0.^{55}\,ft \qquad d = x_\ell - x_c$$

$$I_{yy} = 92^{505} + 26^{902} = 119^{407}\,ft^4$$

I_{xx}



$$I_{xx} \approx ab^3\left[\frac{16}{3} - 3.36\frac{b}{a}\left(1-\frac{b^4}{12a^4}\right)\right] + \frac{dt^3}{3}$$

$$\approx 3^{25}(4)^3\left[\frac{16}{3} - 3.36\left(\frac{4}{3^{25}}\right)\left(1-\frac{4^4}{12(3^{25})^4}\right)\right] + \frac{6^{333}(1^{25})^3}{3}$$

$$I_{xx} \approx 417^{11}\,ft^4$$

Shaft:

$$I_{yy} = I_{zz} = 87.^{624}\,ft^4$$

$$I_{xx} = \frac{\pi r^4}{2} = \frac{\pi}{2}(3^{25})^4 = 175^{248}\,ft^4$$

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-3
MADE BY T Stones DATE 1-25-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 41
CALCULATIONS FOR Abutment Design

Soil Properties @ Abut: For soil profile see sheets S-1 & S-2

	Depth	Elev.	K	γ	ϕ	depth p-y curve
	0	1072				
1			44.0	62pcf = 0.036pci	32°	84 in
	168 in	1058				
2			44.0	62 pcf = 0.036pci	40°	258 in
	348 in	1043				
3			25.5	53 - 0.031	34°	408 in
	468 in	1033				
4			44.0	56 - 0.032	40°	498 in
	528 in	1028				
5			44.0	62 0.036	32°	588 in
	648 in	1018				
6			25.5	53 0.031	34°	678 in
	708 in	1013				
7			44.0	62 0.036	32°	768 in
	828 in	1003				
8			44.0	62 0.036	40	864 in
	900 in	997				

COM624 Input:

```
1     Northern Avenue Abutment Bent
2     Engl
3.    75, 8, 1, 0
4.    9, 9, 0
5.    900, 3605000, 0
6.    1, 1
7.    3, 2, 1, 0
8.    0, 0, 12
9.
10.   0.0, 78, 1816972, 4778.4
11.   1, 4, 0, 168, 44.0, 0, 0
      2, 4, 168, 348, 44.0, 0, 0
      3, 4, 348, 468, 25.5, 0, 0
      4, 4, 468, 528, 44.0, 0, 0
      5, 4, 528, 648, 44.0, 0, 0
      6, 4, 648, 708, 25.5, 0, 0
      7, 4, 708, 828, 44.0, 0, 0
      8, 4, 828, 900, 44.0, 0, 0
```

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-4
MADE BY T Stones DATE 1-25-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 42
CALCULATIONS FOR Abutment Design

COM624G Input (Cont)

```
12    0    , 0.036
      168  , 0.036
      348  , 0.031
      468  , 0.032
      528  , 0.036
      648  , 0.031
      708  , 0.036
      828  , 0.036
      900  , 0.036

13    0   , 0, 32 , 0
      168 , 0, 40 , 0
      348 , 0, 34 , 0
      468 , 0, 40 , 0
      528 , 0, 32 , 0
      648 , 0, 34 , 0
      708 , 0, 32 , 0
      828 , 0, 40 , 0
      900 , 0, 40 , 0

14
15
16
17    8
18    84
      258
      408
      498
      588
      678
      768
      864
19    -1, 0, 0, 0
20    End
```

The following spring constant taken from COM624G output shown on sht A-6 will be used for M-Strudl Analysis

Depth	Elev.	lb/in COM624G P value	k/ft M-Strudl K value
84 in	65 ft	2372	28.5
258	50.5	6705	80.5
408	38	10864	130.4
498	30.5	14293	171.5
588	23	15261	183.1
678	15.5	18730	224.8
768	8	32357	388.3
864	0	41184	494.2

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-5
MADE BY T Stones DATE 1-25-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 43
CALCULATIONS FOR Abutment Design

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 8
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
84.00	78.00	36.0	.36D-01

Y IN	P LBS/IN
.000	.000
.108	400.400
.217	800.800
.325	1201.200
.433	1601.600
.542	1929.694
.650	2037.342
.758	2133.029
.867	2219.542
.975	2298.760
1.083	2372.015
1.192	2440.292
1.300	2504.338

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
258.00	78.00	37.0	.36D-01

Y IN	P LBS/IN
.000	.000
.108	1229.800
.217	2459.600
.325	3689.400
.433	4499.551
.542	4958.491
.650	5368.010
.758	5740.532
.867	6084.057
.975	6404.091
1.083	6704.611
1.192	6988.595
1.300	7258.342

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
408.00	78.00	37.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	1127.100
.217	2254.200
.325	3381.300
.433	4508.400
.542	5635.500
.650	6762.600
.758	7889.700
.867	9016.800
.975	10143.900
1.083	10864.435

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
498.00	78.00	36.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	2373.800
.217	4747.600
.325	6874.004
.433	8187.900
.542	9377.645
.650	10476.977
.758	11506.405
.867	12479.545
.975	13406.008
1.083	14292.890
1.192	15145.611
1.300	15968.431

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
588.00	78.00	35.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	2802.800
.217	5605.600
.325	7339.763
.433	8742.685
.542	10013.043
.650	11186.862
.758	12296.041
.867	13325.118
.975	14314.355
1.083	15261.328

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
678.00	78.00	33.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	1872.975
.217	3745.950
.325	5618.925
.433	7491.900
.542	9364.875
.650	11237.850
.758	13110.825
.867	14983.800
.975	16856.775
1.083	18729.750

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
768.00	78.00	36.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	3660.800
.217	7321.600
.325	10982.400
.433	14643.200
.542	18304.000
.650	21964.800
.758	25625.600
.867	28251.865
.975	30349.242
1.083	32357.012

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
864.00	78.00	40.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	4118.400
.217	8236.800
.325	12355.200
.433	16473.600
.542	20592.000
.650	24710.400
.758	28828.800
.867	32947.200
.975	37065.600
1.083	41184.000

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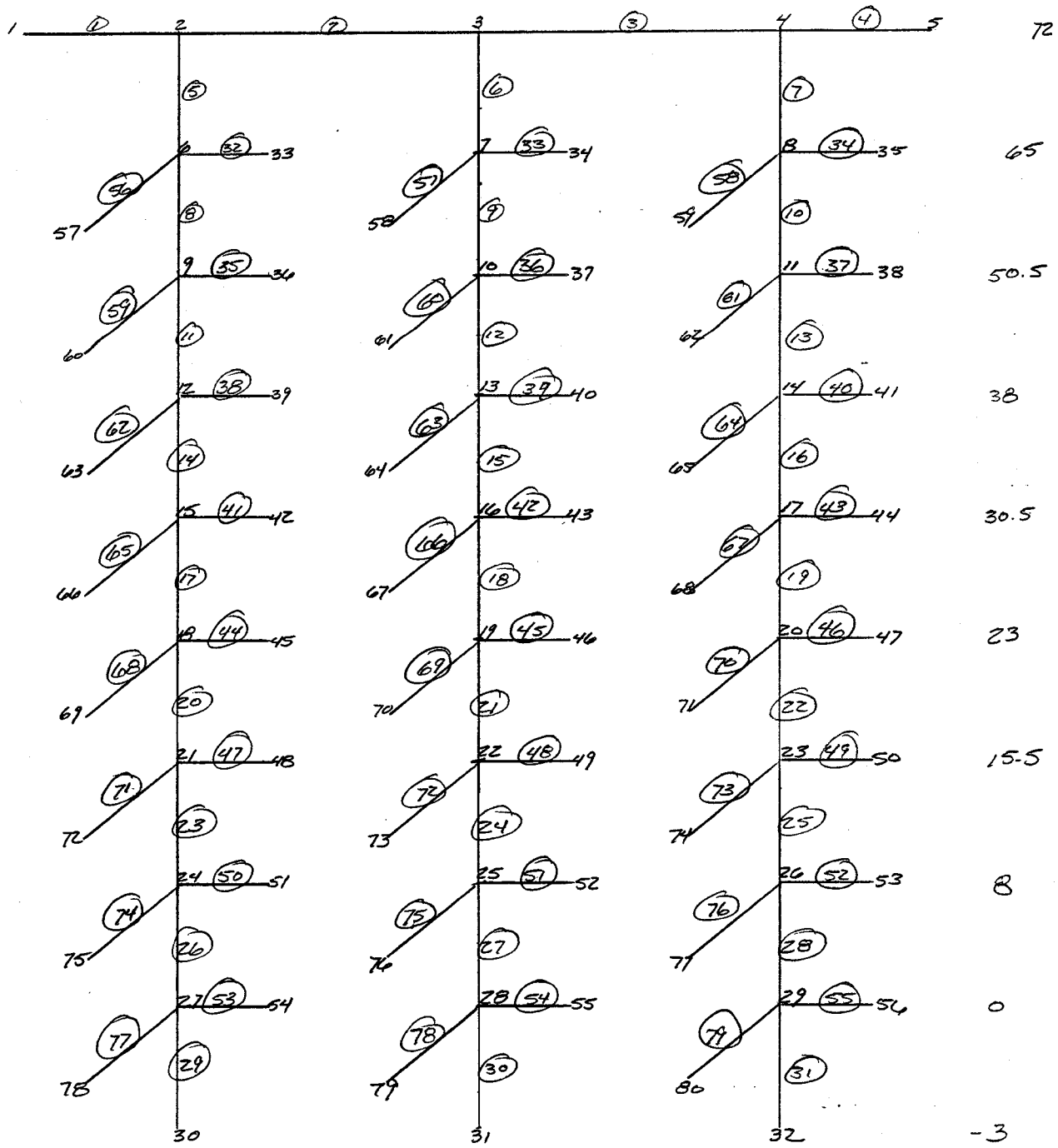
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-6
MADE BY T Stones DATE 1-26-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 44
CALCULATIONS FOR Abutment Design

Shaft Joint & Member No.'s



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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-7
MADE BY T Stones DATE 1-26-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 45
CALCULATIONS FOR Abutment Design

Loads:

Dead Load: DL from substructure will be calculated by M-Strudl

Additional DL in Cap for girder steps will be linearly distributed over members 2 & 3

End dimension $2'\frac{11}{4}'' \times 2'\text{-}6''$ $\quad WA = 2.5(\frac{2.25}{12})(0.15) = 0.07$ k/ft

℄ dimension $9'' \times 2'\text{-}6''$ $\quad WB = 2.5(\frac{9}{12})(0.15) = 0.281$ k/ft

Diaphragm: $1'\text{-}4'' \times 5'\text{-}3''$ $\quad$ Girder Area $= 6.16\ ft^2$

Total Weight $= \left[(1.333)(5.25)(84.5 - 1.333) - 9(6.16)(1.333)\right](0.15)$

$= 76.22$ k $\quad$ 2-8" Curtain Walls

$W = 76.22/84.5 = 0.9$ k/ft

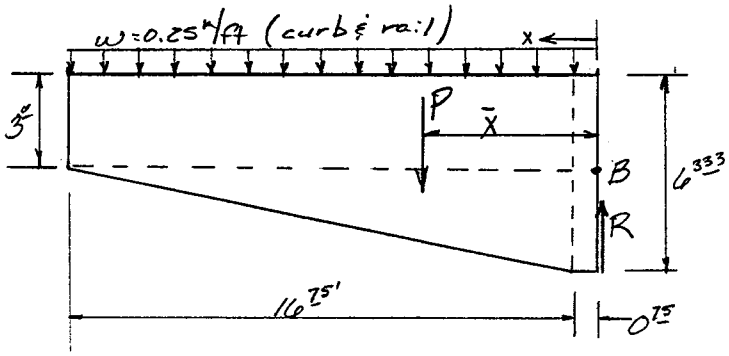
Girder + Slab DL from PSGDES $= 102.1$ k/Girder

For Barrier, Curb, and Railing Load Breakdown see sht. P4

$DL_{\substack{BAR\\CURB\\RAIL}}$ to Girders A & I $= 25.0$ k

$DL_{\substack{BAR\\CURB\\RAIL}}$ to Girders B & H $= 5.3$ k

Wingwalls: Assume 1'-3" Thick — actually deeper at Abut connection, but thinner (12")



A	x	Ax
$0.75(6.333) = 4.75$	0.375	1.781
$0.5(16.75)(3.333) = 27.914$	6.333	176.789
$3.0(16.75) = 50.25$	9.125	458.531
82.914		637.101

$\bar{x} = \frac{637.101}{82.914} = 7.684$

$P = 1.25(82.914)(0.15) = 15.546$ k

$R = 15.546 + 0.25(16.75) = 19.75$ k

$M_{B_1} = 15.546(7.684) + 0.25\frac{(16.75)^2}{2} = 154.5$ ft·k

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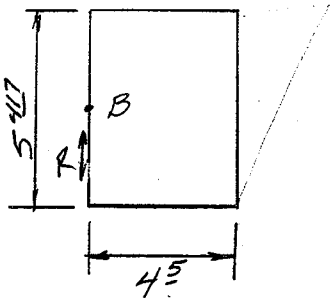
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-8
MADE BY T Stones DATE 1-24-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 1/90 SEC. SHT. NO. 46
CALCULATIONS FOR Abutment Design

Loads (cont.)

Curtain Wall : 8" Thickness



$R = (5.417)(4.5)(0.667)(0.15) = 2.44\ k$

$M_{B_2} = 2.44(2.25) = 5.5\ ft\cdot k$

This Moment is in opposite direction from wingwall moment.

Resulting Moment $= 154.5 - 5.5 = 149.0\ ft\cdot k$

Future Wearing Surface and Utility Allowance is modelled as a 2.5" overlay

$w = (2.5/12)(.15\ k/ft^2)(97/2) = 1.5\ k/ft$

Live Load:

Sidewalk Load applied on Upstream Side Only:

$w = (6\ ft)(60\ psf) = 360\ plf$

From Beam Diagrams: AISC Manual pg 2-127 for 4 continuous spans: Max Reaction @ 1st support:

$R_A = 0.393\ wl = 0.393(.360)(97) = 13.72\ k$

R_A will be applied to Girder Location A

Traffic Load: $I = \frac{50}{97+125} = 0.225$

From Continuous Highway Bridge Tables:

R_{LL} 95 ft Span $= 63.1\ k$

R_{LL} 100 ft Span $= 63.6\ k$

$R_{LL\ 97\ ft\ span} = 63.1 + 0.5(\frac{2}{5}) = 63.3\ k$

$I = .225(63.3) = 14.24$

$R_{LL+I} = 77.54\ k$

Tire Load $= \frac{R_{LL+I}}{2}$ $T = 38.77\ k$

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-9
MADE BY T Stones DATE 1-24-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 47
CALCULATIONS FOR Abutment Design

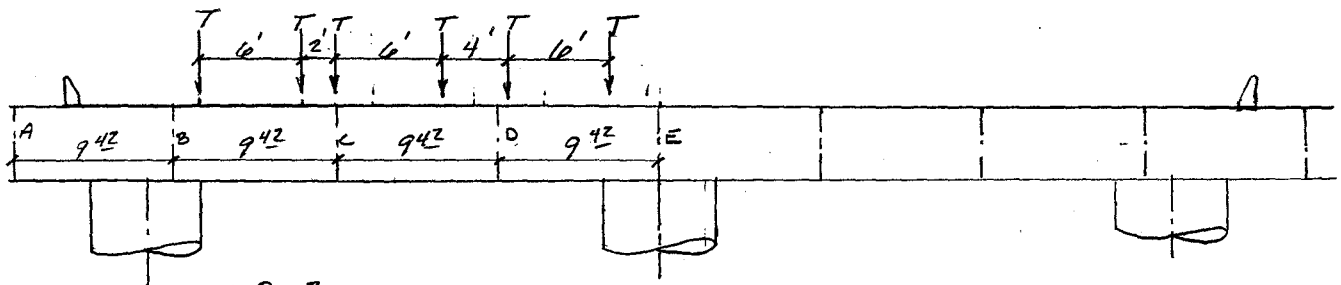
Live Load (cont.)

No. of Traffic Lanes = $68/12 = 5^{67}$ use 5 lanes

Reduction for improbability = 5(.75) = 3^{75} lanes

I'll conservatively load a maximum of 4 lanes.

I. Max Moment in Shaft, Max Positive Moment in Loaded Span & Max Neg. Mom in Unloaded Span



$R_B = T\left[\frac{8+2}{9^{42}}\right] = 1^{062}\,T$ — 41^{12} k

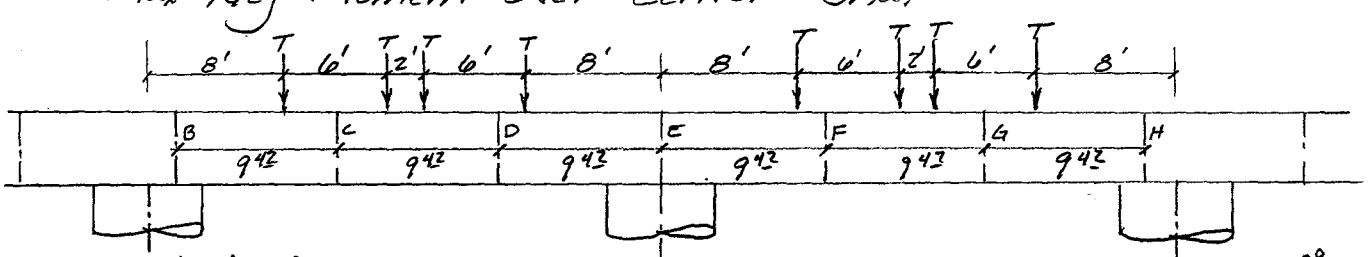
$R_C = T\left[\frac{1^{42}+7^{42}+3^{42}}{9^{42}}\right]+T = 2^{302}\,T$ — 89^{25} k

$R_D = T\left[\frac{6+8^{84}+2^{84}}{9^{42}}\right] = 1^{817}\,T$ — 72^{77} k

$R_E = T\left[\frac{0^{58}+6^{58}}{9^{42}}\right] = 0^{760}\,T$ — 29^{47} k

Total 6.00 T ✓

II. Max Neg Moment over Center Shaft



$R_H = R_B = T/9^{42}\left[3^{16}\right] = 0^{335}\,T$ — 12^{99} k

$R_G = R_C = T/9^{42}\left[6^{26}+6^{58}+4^{58}\right] = 1^{849}\,T$ — 71^{69} k

$R_F = R_D = T/9^{42}\left[2^{84}+4^{84}+8\right] = 1^{665}\,T$ — 64^{55} k

$R_E = T/9^{42}\left[1^{42}\right](2) = 0^{301}\,T$ — 11^{67} k

Total = $2(3^{849}+0^{304}) = 8^{00}$ ✓

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave c. New River SHEET NO. A-10
MADE BY T Stones DATE 1-24-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 48
CALCULATIONS FOR Abutment Design

✓ 2/90

****
* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : SPACE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
5 SHAFT	DL	6	633.33	9.49	-0.06	1.11	-81.55	-19.98
		2	-572.61	-9.49	0.06	-1.11	82.29	135.80
	SETTLE_1	6	-81.31	71.76	0.00	0.00	0.00	-107.50
		2	81.31	-71.76	0.00	0.00	0.00	982.97
	SETTLE_2	6	131.51	-153.86	0.00	0.00	0.00	-460.45
		2	-131.51	153.86	0.00	0.00	0.00	-1416.61
	SDWK_LL	6	15.44	5.03	0.00	0.00	0.00	6.91
		2	-15.44	-5.03	0.00	0.00	0.00	54.44
	LL_I_1	6	117.01	-39.82	0.00	0.00	0.00	-140.91
		2	-117.01	39.82	0.00	0.00	0.00	-344.86
	LL_I_2	6	75.37	-34.20	0.00	0.00	0.00	-102.31
		2	-75.37	34.20	0.00	0.00	0.00	-314.97
	LL_I_3	6	52.63	-38.64	0.00	0.00	0.00	-115.63
		2	-52.63	38.64	0.00	0.00	0.00	-355.82
	WIND	6	-3.32	13.06	-2.62	-114.65	30.57	94.88
		2	3.32	-13.06	2.62	114.65	1.37	64.41
	OVERTRN	6	-56.06	15.42	0.00	0.00	0.00	51.36
		2	56.06	-15.42	0.00	0.00	0.00	136.72
	WIND_LL	6	-0.65	2.01	-0.96	8.60	11.74	14.18
		2	0.65	-2.01	0.96	-8.60	-0.05	10.38
6 SHAFT	DL	7	603.67	-19.71	0.00	1.41	-0.00	-127.28
		3	-542.95	19.71	-0.00	-1.41	-0.00	-113.14
	SETTLE_1	7	130.70	9.39	0.00	0.00	0.00	-284.73
		3	-130.70	-9.39	0.00	0.00	0.00	399.31
	SETTLE_2	7	-263.01	-0.00	0.00	0.00	0.00	0.12
		3	263.01	0.00	0.00	0.00	0.00	-0.17
	SDWK_LL	7	-0.84	-2.83	0.00	0.00	0.00	-13.47
		3	0.84	2.83	0.00	0.00	0.00	-15.04
	LL_I_1	7	115.28	34.55	0.00	0.00	0.00	116.56
		3	-115.28	-34.55	0.00	0.00	0.00	304.89
	LL_I_2	7	159.41	-0.00	0.00	0.00	0.00	0.07
		3	-159.41	0.00	0.00	0.00	0.00	-0.10
	LL_I_3	7	204.92	-0.00	0.00	0.00	0.00	0.05
		3	-204.92	0.00	0.00	0.00	0.00	-0.07
	WIND	7	1.30	10.71	-2.08	-82.42	25.69	68.25
		3	-1.30	-10.71	2.08	82.42	-0.28	62.40
	OVERTRN	7	-28.10	-15.14	0.00	0.00	0.00	-55.41

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
		3	28.10	15.14	0.00	0.00	0.00	-129.35
	WIND_LL	7	0.00	3.24	-0.99	0.00	12.03	21.14
		3	-0.00	-3.24	0.99	-0.00	0.11	18.43
7 SHAFT	DL	8	620.01	-50.04	0.06	1.11	81.55	-259.02
		4	-559.29	50.04	-0.06	-1.11	-82.29	-351.50
	SETTLE_1	8	-49.39	-81.15	0.00	0.00	0.00	-565.26
		4	49.39	81.15	0.00	0.00	0.00	-424.80
	SETTLE_2	8	131.50	153.86	0.00	0.00	0.00	460.74
		4	-131.50	-153.86	0.00	0.00	0.00	1416.37
	SDWK_LL	8	-0.88	-2.20	0.00	0.00	0.00	-14.73
		4	0.88	2.20	0.00	0.00	0.00	-12.10
	LL_I_1	8	0.38	5.27	0.00	0.00	0.00	-5.93
		4	-0.38	-5.27	0.00	0.00	0.00	70.25
	LL_I_2	8	75.36	34.21	0.00	0.00	0.00	102.48
		4	-75.36	-34.21	0.00	0.00	0.00	314.83
	LL_I_3	8	52.62	38.64	0.00	0.00	0.00	115.74
		4	-52.62	-38.64	0.00	0.00	0.00	355.72
	WIND	8	2.02	6.36	-1.74	-48.69	22.31	44.44
		4	-2.02	-6.36	1.74	48.69	-1.08	33.21
	OVERTRN	8	1.86	-0.27	0.00	0.00	0.00	4.39
		4	-1.86	0.27	0.00	0.00	0.00	-7.73
	WIND_LL	8	0.65	2.01	-0.96	-8.60	11.74	14.18
		4	-0.65	-2.01	0.96	8.60	-0.05	10.37

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* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : SPACE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
29 SHAFT	DL	30	970.30	-0.00	-0.06	-0.00	0.00	0.00
		27	-955.37	0.00	0.06	0.00	0.18	-0.00
	SETTLE_1	30	-92.80	-0.00	0.00	0.00	0.00	0.00
		27	92.80	0.00	0.00	0.00	0.00	-0.00
	SETTLE_2	30	130.70	-0.00	0.00	0.00	0.00	-0.00
		27	-130.70	0.00	0.00	0.00	0.00	-0.00
	SDWK_LL	30	15.34	0.00	0.00	0.00	0.00	0.00
		27	-15.34	-0.00	0.00	0.00	0.00	0.00
	LL_I_1	30	116.28	-0.00	0.00	0.00	0.00	-0.00
		27	-116.28	0.00	0.00	0.00	0.00	-0.00
	LL_I_2	30	74.90	-0.00	0.00	0.00	0.00	0.00
		27	-74.90	0.00	0.00	0.00	0.00	-0.00
	LL_I_3	30	52.30	-0.00	0.00	0.00	0.00	-0.00
		27	-52.30	0.00	0.00	0.00	0.00	-0.00
	WIND	30	-3.29	0.00	-2.62	0.00	-0.00	-0.00
		27	3.29	-0.00	2.62	-0.00	7.85	0.00
	OVERTRN	30	-55.72	0.00	0.00	0.00	0.00	-0.00
		27	55.72	-0.00	0.00	0.00	0.00	0.00
	WIND_LL	30	-0.65	0.00	-0.96	-0.00	0.00	-0.00
		27	0.65	-0.00	0.96	0.00	2.87	0.00
30 SHAFT	DL	31	945.09	-0.00	0.00	-0.00	0.00	-0.00
		28	-930.16	0.00	-0.00	0.00	-0.00	-0.00
	SETTLE_1	31	130.70	-0.00	0.00	0.00	0.00	-0.00
		28	-130.70	0.00	0.00	0.00	0.00	-0.00
	SETTLE_2	31	-263.01	0.00	0.00	0.00	0.00	-0.00
		28	263.01	-0.00	0.00	0.00	0.00	0.00
	SDWK_LL	31	-0.84	-0.00	0.00	0.00	0.00	-0.00
		28	0.84	0.00	0.00	0.00	0.00	-0.00
	LL_I_1	31	115.28	0.00	0.00	0.00	0.00	-0.00
		28	-115.28	-0.00	0.00	0.00	0.00	0.00
	LL_I_2	31	159.41	0.00	0.00	0.00	0.00	-0.00
		28	-159.41	-0.00	0.00	0.00	0.00	0.00
	LL_I_3	31	204.92	0.00	0.00	0.00	0.00	-0.00
		28	-204.92	-0.00	0.00	0.00	0.00	0.00
	WIND	31	1.30	0.00	-2.08	0.00	-0.00	-0.00
		28	-1.30	-0.00	2.08	-0.00	6.25	0.00
	OVERTRN	31	-28.10	-0.00	0.00	0.00	0.00	0.00

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z SHEAR FORCE	TORSIONAL MOMENT	LOCAL Y MOMENT	LOCAL Z MOMENT
		28	28.10	0.00	0.00	0.00	0.00	-0.00
	WIND_LL	31	0.00	0.00	-0.99	0.00	-0.00	-0.00
		28	-0.00	-0.00	0.99	-0.00	2.98	0.00
31 SHAFT	DL	32	961.43	-0.00	0.06	-0.00	-0.00	0.00
		29	-946.50	0.00	-0.06	0.00	-0.18	-0.00
	SETTLE_1	32	-49.39	-0.00	0.00	0.00	0.00	0.00
		29	49.39	0.00	0.00	0.00	0.00	-0.00
	SETTLE_2	32	131.50	0.00	0.00	0.00	0.00	-0.00
		29	-131.50	-0.00	0.00	0.00	0.00	0.00
	SDWK_LL	32	-0.88	-0.00	0.00	0.00	0.00	0.00
		29	0.88	0.00	0.00	0.00	0.00	-0.00
	LL_I_1	32	0.38	-0.00	0.00	0.00	0.00	-0.00
		29	-0.38	0.00	0.00	0.00	0.00	-0.00
	LL_I_2	32	75.36	0.00	0.00	0.00	0.00	0.00
		29	-75.36	-0.00	0.00	0.00	0.00	0.00
	LL_I_3	32	52.62	0.00	0.00	0.00	0.00	-0.00
		29	-52.62	-0.00	0.00	0.00	0.00	0.00
	WIND	32	2.02	0.00	-1.74	0.00	0.00	-0.00
		29	-2.02	-0.00	1.74	-0.00	5.22	0.00
	OVERTRN	32	1.86	0.00	0.00	0.00	0.00	-0.00
		29	-1.86	-0.00	0.00	0.00	0.00	0.00
	WIND_LL	32	0.65	0.00	-0.96	-0.00	0.00	-0.00
		29	-0.65	-0.00	0.96	0.00	2.87	0.00

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****
* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : SPACE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS INCHES DEGREES

*** LOAD INDEX : 1 LOAD TAG : DL ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
1	-0.00	-0.05	-0.00	-0.00	-0.00	0.00
2	-0.00	-0.04	-0.00	-0.00	-0.00	0.00
3	-0.00	-0.04	-0.00	-0.00	-0.00	0.00
4	-0.00	-0.04	0.00	0.00	-0.00	-0.00
5	-0.00	-0.05	-0.00	0.00	0.00	-0.00

*** LOAD INDEX : 2 LOAD TAG : SETTLE_1 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
1	-0.01	-0.09	0.00	0.00	0.00	0.01
2	-0.01	-0.07	0.00	0.00	0.00	0.01
3	-0.01	-0.01	0.00	0.00	0.00	0.01
4	-0.00	0.00	0.00	0.00	0.00	0.00
5	-0.00	0.01	0.00	0.00	0.00	0.00

*** LOAD INDEX : 3 LOAD TAG : SETTLE_2 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
1	0.00	0.01	0.00	0.00	0.00	-0.01
2	0.00	-0.01	0.00	0.00	0.00	-0.01
3	0.00	-0.06	0.00	0.00	0.00	-0.00
4	-0.00	-0.01	0.00	0.00	0.00	0.01

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RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
5	0.00	0.01	0.00	0.00	0.00	0.01

*** LOAD INDEX : 4 LOAD TAG : SDWK_LL ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
1	-0.00	-0.00	0.00	0.00	0.00	0.00
2	-0.00	-0.00	0.00	0.00	0.00	0.00
3	-0.00	0.00	0.00	0.00	0.00	0.00
4	-0.00	0.00	0.00	0.00	0.00	0.00
5	-0.00	0.00	0.00	0.00	0.00	0.00

*** LOAD INDEX : 5 LOAD TAG : LL_I_1 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
1	0.00	-0.00	0.00	0.00	0.00	-0.00
2	0.00	-0.01	0.00	0.00	0.00	-0.00
3	-0.00	-0.01	0.00	0.00	0.00	0.00
4	-0.00	-0.00	0.00	0.00	0.00	0.00
5	0.00	0.00	0.00	0.00	0.00	0.00

*** LOAD INDEX : 6 LOAD TAG : LL_I_2 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
1	0.00	-0.00	0.00	0.00	0.00	-0.00
2	0.00	-0.00	0.00	0.00	0.00	-0.00
3	0.00	-0.01	0.00	0.00	0.00	-0.00
4	-0.00	-0.00	0.00	0.00	0.00	0.00
5	0.00	-0.00	0.00	0.00	0.00	0.00

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*** LOAD INDEX : 7 LOAD TAG : LL_I_3 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
1	0.00	0.00	0.00	0.00	0.00	-0.00
2	0.00	-0.00	0.00	0.00	0.00	-0.00
3	0.00	-0.01	0.00	0.00	0.00	-0.00
4	-0.00	-0.00	0.00	0.00	0.00	0.00
5	0.00	0.00	0.00	0.00	0.00	0.00

*** LOAD INDEX : 8 LOAD TAG : WIND ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
1	0.00	0.00	0.07	0.00	0.00	-0.00
2	0.00	0.00	0.06	0.00	0.00	-0.00
3	0.00	-0.00	0.05	0.00	0.00	-0.00
4	0.00	-0.00	0.04	0.00	0.00	-0.00
5	0.00	-0.00	0.04	0.00	0.00	-0.00

*** LOAD INDEX : 9 LOAD TAG : OVERTRN ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
1	-0.00	0.00	0.00	0.00	0.00	0.00
2	-0.00	0.00	0.00	0.00	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	-0.00
4	0.00	-0.00	0.00	0.00	0.00	-0.00
5	0.00	-0.00	0.00	0.00	0.00	-0.00

*** LOAD INDEX : 10 LOAD TAG : WIND_LL ***

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RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
1	0.00	0.00	0.02	0.00	-0.00	-0.00
2	0.00	0.00	0.02	0.00	-0.00	-0.00
3	0.00	-0.00	0.02	0.00	-0.00	-0.00
4	0.00	-0.00	0.02	0.00	0.00	-0.00
5	0.00	-0.00	0.02	0.00	0.00	-0.00

✓ Lee Ahmad 2/90

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* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : SPACE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS INCHES DEGREES

*** LOAD INDEX : 1 LOAD TAG : DL ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
30	-0.00	-0.00	0.00	0.00	-0.00	-0.00
31	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00
32	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00

*** LOAD INDEX : 2 LOAD TAG : SETTLE_1 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
30	-0.00	-0.07	0.00	0.00	0.00	-0.00
31	-0.00	-0.00	0.00	0.00	0.00	-0.00
32	-0.00	0.00	0.00	0.00	0.00	-0.00

*** LOAD INDEX : 3 LOAD TAG : SETTLE_2 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
30	-0.00	-0.00	0.00	0.00	0.00	-0.00
31	0.00	-0.07	0.00	0.00	0.00	0.00
32	0.00	-0.00	0.00	0.00	0.00	0.00

*** LOAD INDEX : 4 LOAD TAG : SDWK_LL ***

✓ [illegible] 2/90

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
30	0.00	-0.00	0.00	0.00	0.00	0.00
31	-0.00	0.00	0.00	0.00	0.00	-0.00
32	-0.00	0.00	0.00	0.00	0.00	-0.00

*** LOAD INDEX : 5 LOAD TAG : LL_I_1 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
30	-0.00	-0.00	0.00	0.00	0.00	-0.00
31	0.00	-0.00	0.00	0.00	0.00	0.00
32	-0.00	-0.00	0.00	0.00	0.00	-0.00

*** LOAD INDEX : 6 LOAD TAG : LL_I_2 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
30	-0.00	-0.00	0.00	0.00	0.00	-0.00
31	0.00	-0.00	0.00	0.00	0.00	0.00
32	0.00	-0.00	0.00	0.00	0.00	0.00

*** LOAD INDEX : 7 LOAD TAG : LL_I_3 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
30	-0.00	-0.00	0.00	0.00	0.00	-0.00
31	0.00	-0.00	0.00	0.00	0.00	0.00
32	0.00	-0.00	0.00	0.00	0.00	0.00

*** LOAD INDEX : 8 LOAD TAG : WIND ***

✓ [illegible] 2/90

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
30	0.00	0.00	0.06	0.00	-0.00	0.00
31	0.00	-0.00	0.05	0.00	0.00	0.00
32	0.00	-0.00	0.04	0.00	0.00	0.00

*** LOAD INDEX : 9 LOAD TAG : OVERTRN ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
30	0.00	0.00	0.00	0.00	0.00	0.00
31	-0.00	0.00	0.00	0.00	0.00	-0.00
32	0.00	-0.00	0.00	0.00	0.00	0.00

*** LOAD INDEX : 10 LOAD TAG : WIND_LL ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	Z DIRECTION	GLOBAL ROTATIONS X DIRECTION	Y DIRECTION	Z DIRECTION
30	0.00	0.00	0.02	0.00	-0.00	0.00
31	0.00	-0.00	0.02	0.00	-0.00	0.00
32	0.00	-0.00	0.02	0.00	0.00	0.00

MINNEAPOLIS PHOENIX

IBRW

JOB Northern Ave @ New River SHEET NO. A-40

MADE BY T Stokes DATE 1-26-90 JOB NO. 33896.2

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 80

CALCULATIONS FOR Abutment Design

M-Strut Moment Results

LOAD	Member 2 0.0	Member 2 Face of Shaft	Member 2 0.25	Member 2 0.5	Member 2 0.75	Member 2 Face of Shaft	Member 2 1.0	Member 3 0.0	Member 3 Face of Shaft	Member 3 0.25	Member 3 0.5	Member 3 0.75	Member 3 Face of Shaft	Member 3 1.0
DL	-1683	-813	-45	579	194	-544	-1204	-1317	-628	147	600	43	-688	-1529
SETTLE-1	983	719	373	-237	-846	-1192	-1456	-1057	-896	-686	-316	54	264	425
SETTLE-2	-1417	-989	-430	556	1542	2101	2529	2529	2101	1542	556	-430	-989	-1416
SDWK-LL	-51	-45	-38	-25	-12	-5	1	-14	-11	-8	-1	6	9	12
LL-I-1	-345	-26	296	523	283	-84	-364	-59	-60	-62	-65	-67	-69	-70
LL-I-2	-315	-90	176	369	175	-139	-379	-379	-139	175	369	176	-89	-315
LL-I-3	-356	-185	39	321	339	93	-95	-95	93	339	321	39	-185	-356
WIND	64	54	10	15	-10	-24	-35	27	21	12	-3	-18	-26	-33
OVERTURN	137	-45	-284	-201	-4	108	193	64	58	50	36	22	14	8
WIND-LL	10	8	5	1	-4	-7	-9	9	7	4	-1	-5	-8	-10
I Group Neg	-3507*	-2032*	-580*	173*	-734*	-1891*	-3053*							
I Group Pos								1213*	1566*	2028*	1679*	718*	40	-534
II Group Neg	-2507	-1499	-777	59	-574	-1408	-2156							
II Group Pos								1146	1282	1401	966	359	80	-276
III Group Neg	-2822	-1649	-541	111	-594	-1524	-2458							
III Group Pos								1031	1289	1641	1356	586	55*	-371*

125% overstress incl.

* Controls Moment Design

Check 1.2 M_{cr}

$M_{cr} = f_r I_g / y_t$ AASHTO 8.13.3

$f_r = 7.5\sqrt{f'_c} = 474$ psi

$I_g = 252.6$ ft^4 (Sht A-2)

$y_t = 10^{33} - 3^{36} = 6^{97}$ ft

$M_{cr} = (0.474 \text{ ksi})(252.6)(144) / 6^{97} = 2474$ ft-k

1.2 $M_{cr} > M_{des}$ increase positive & Negative Design Moments by 1/3.

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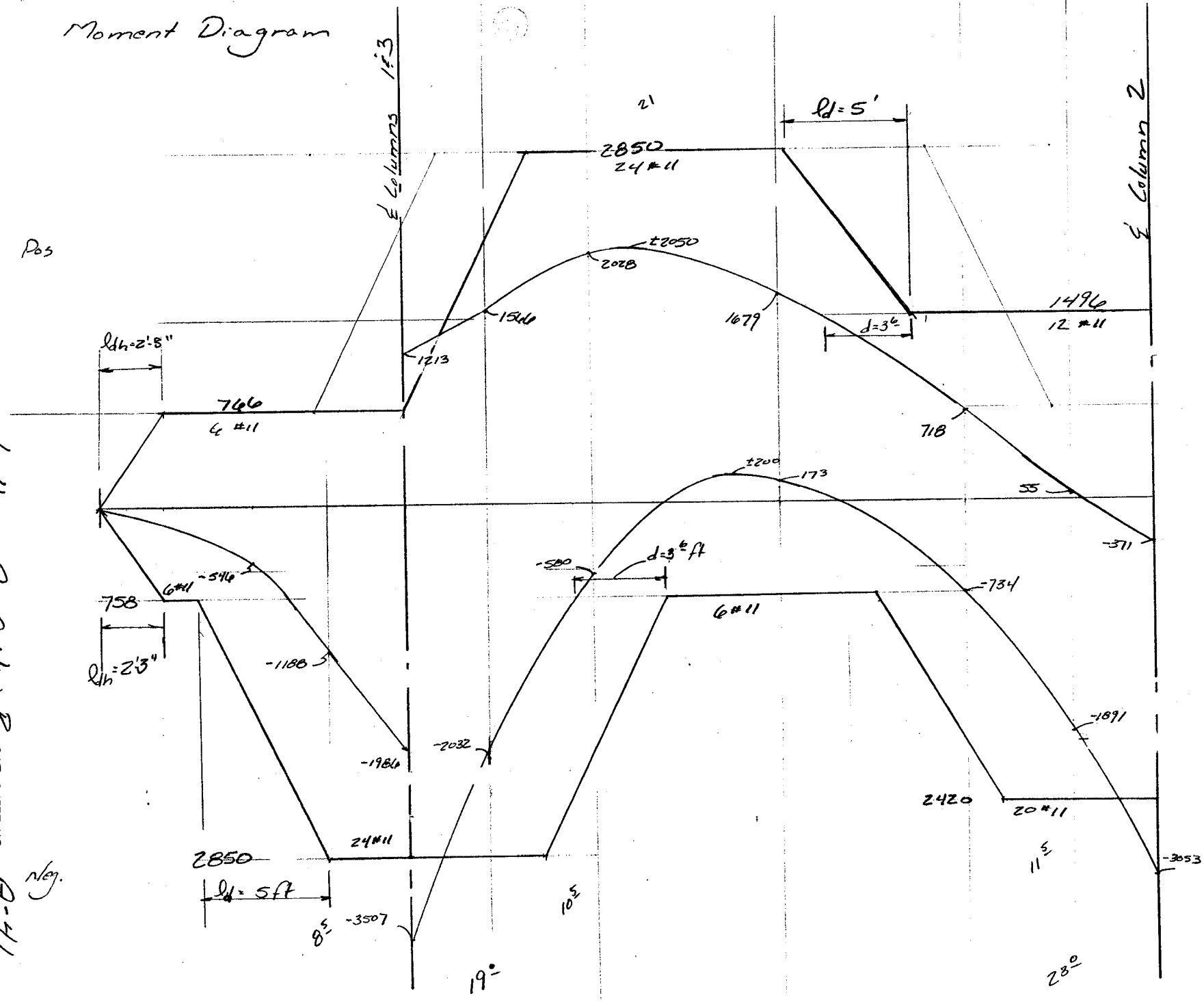
JOB Northern Ave @ New River SHEET NO. A-40A
MADE BY T Stones DATE 1-29-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 81
CALCULATIONS FOR Abutment Design

Moment Diagram



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JOB Northern Avenue New River SHEET NO. A-41

MADE BY T Stones DATE 1-29-90 JOB NO. 33-8862

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 82

CALCULATIONS FOR Abutment Design

Abutment Cap Design by Allowable Stress

Cap Dimension 6^{5} ft x 4^{0} ft

Assume #11 longitudinal bars bundled vertical

Neg d_{avail} = 48 - 2 - 0.625 - 1.41 = $43.^{97}$ in

Pos d_{avail} = 48 - 3 - 0.625 - 1.41 = 42^{97}

Max Design Moment = 2050 ft-k

$$bd_{req}^2 = \frac{2M}{f_c j k} \qquad k = \frac{n}{n + f_s/f_c} \qquad n = \frac{E_s}{E_c} = \frac{29{,}000}{57\sqrt{4000}} = 8$$

$$k = \frac{8}{8 + 24/1.6} = 0.348 \qquad j = 1 - \frac{k}{3} = 0.884$$

$$bd_{req}^2 = \frac{2(1.33)(2050\,ft\text{-}k)(12\ in/ft)}{1.4\,ksi\,(.884)(0.348)} = 133{,}343\ in^3 \Rightarrow d_{req} = 41^{33}\ in \quad OK$$

Positive Moment Reinforcement Req'd

$$A_s = \frac{M}{f_s j d} = \frac{1.33(2050\,ft\text{-}k)(12\,in/ft)}{(24\,ksi)(.884)(42^{97}\,in)} = 35^{97}\ in^2$$

#11 req = 23^{06} use 24 #11

Negative Moment Reinforcement Req'd

$$A_s = \frac{1.33(2032)(12)}{24\,(.884)(43^{97})} = 34^{84}$$ req's 23 #11 bar. Due to decreased d @ Front face for neg mom. reinforcement, increase this to 24 #11 @ same spacing as bottom steel.

Negative Moment Reinforcement Req'd in Cantilever

$$A_s = \frac{1.33(1180)(12)}{24(.884)(43^{97})} = 20^{23}\ in$$ req's 13 - #11 for same reasoning as above increase this to 14 - #11 min

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JOB Northern Ave @ New River SHEET NO. A-42
MADE BY T Stones DATE 1-29-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 83
CALCULATIONS FOR Abutment Design

Abutment Cap Shear

	Members 1 & 4				Members 2 & 3						
	0.0	0.5	Face of Shaft	1.0	0.0	Face of Shaft	0.25	0.5	0.75	Face of Shaft	1.0
DL	23	197	218	243	-330	-197	-165	-4	157	190	216
SETTLE_1					81	81	81	81	81	81	81
SETTLE_2					-132	-132	-132	-132	-132	-132	-132
SDWK_LL	0	14	14	14	-2	-2	-2	-2	-2	-2	-2
LL_I_1					-117	-76	-76	13	86	86	86
LL_I_2					-75	-62	-62	9	74	74	74
LL_I_3					-53	-53	-53	-23	58	58	58

Torsional Moments caused by cantilever wingwalls can be neglected per ACI 11.6.1 if $T_n \leq 0.5\sqrt{f'_c}\,\Sigma x^2 y$.

Conservatively say only cap section resists torsion

$\therefore\ T_n \leq 0.5\sqrt{4000}\,(78)^2(48)/12000 = 769$ ft-k

Maximum Torsional Moment is in member 1:

$T_n = 150$ ft-k < 769 ft-k so can be neglected.

Assume 4-#5 legs of ties will be required for shear. $A_v = 4(0.31) = 1.24$ in^2/set

Design shear a distance d from face of support

$d_{min} = 43'' = 3'\text{-}7''$

$V_c = 0.95\sqrt{f'_c} = 0.060$ ksi

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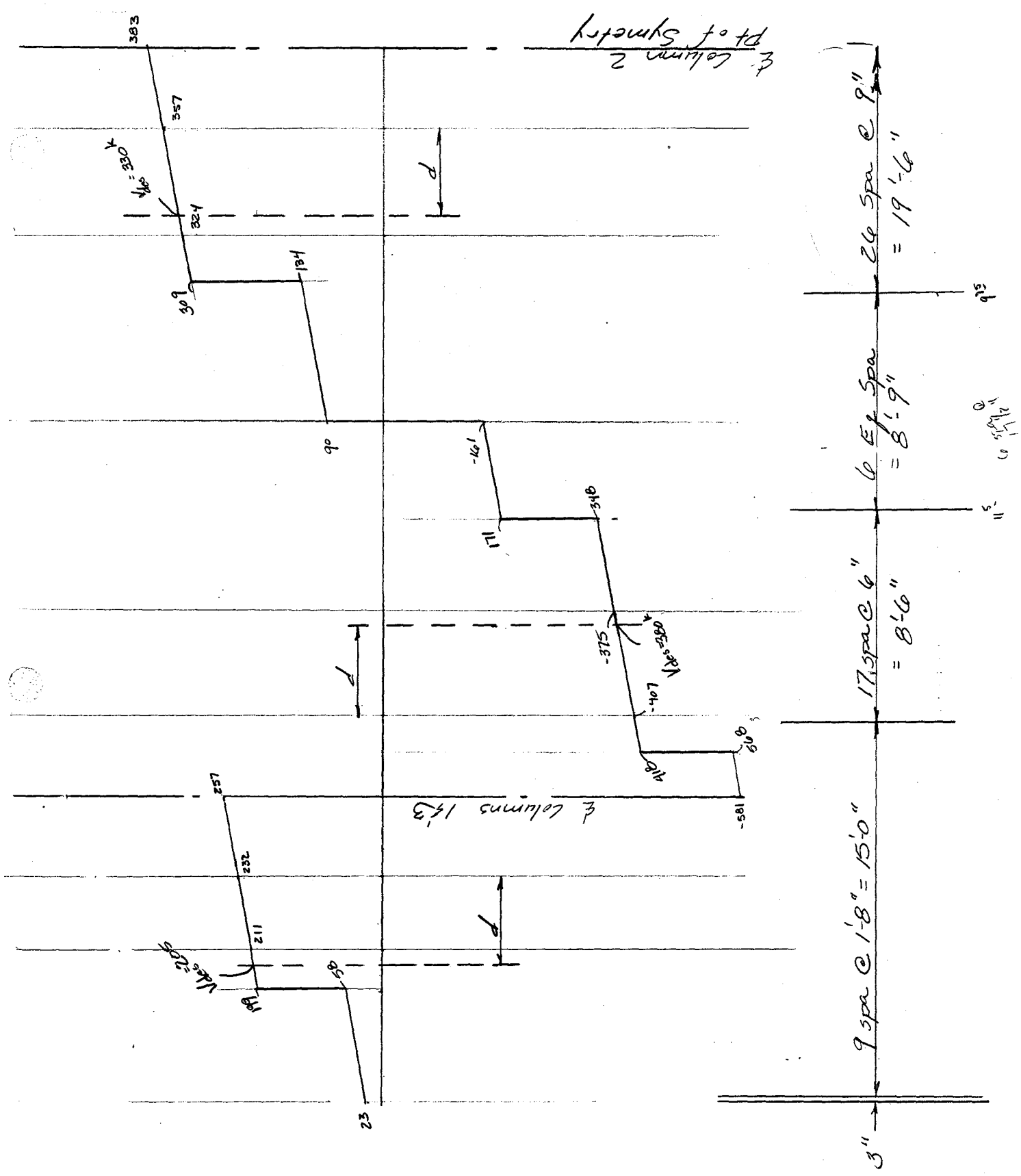
JOB Northern Ave. New River SHEET NO. A-43
MADE BY T Stones DATE 1-29-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 84
CALCULATIONS FOR Abutment Design



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JOB Northern Avenue @ New River SHEET NO. A-44
MADE BY T Stones DATE 1-29-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 85
CALCULATIONS FOR Abutment Design

Shear Design: Member 1 (Cantilever)

$v = \frac{V}{b_w d} = \frac{205^k}{78(43)} = 0.061$

$v_s = v - v_c = 0.061 - 0.06 = 0.001$ ksi

$s_{req} = \frac{A_v f_s}{v_s b_w} = \frac{(1.24 in^2)(24 ksi)}{0.001 ksi (78 in)} = 381$ in

$s_{max} = d/2$ or $24''$ $\frac{d}{2} = \frac{43}{2} = 21.5$ in

Member 2 $v = \frac{380}{78(43)} = 0.113$ ksi $v_s = 0.113 - 0.06 = 0.053$ ksi

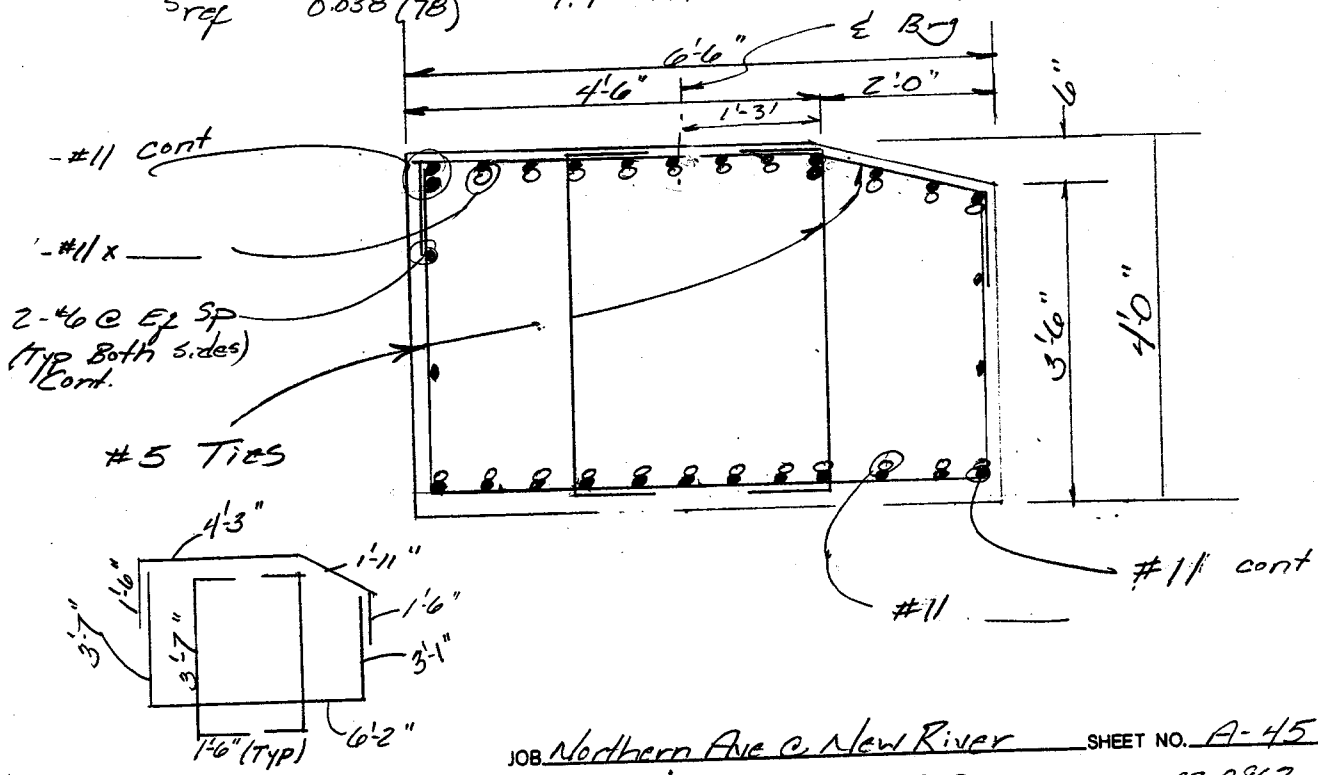
$s_{req} = \frac{(1.24)(24)}{0.053(78)} = 7.2$ use $s = 6$ in

can reduce to $s = 20''$ @ $v_s = \frac{1.24(24)}{20(78)} = 0.019$ ksi

$v = 0.019 + 0.06 = 0.079$ $V = 0.079(43)(78) = 265$ k

At middle column: $v = \frac{330}{78(43)} = 0.098$ ksi $v_s = 0.038$ ksi

$s_{req} = \frac{1.24(24)}{0.038(78)} = 9.9$ in use $s = 9$ in



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JOB Northern Ave @ New River SHEET NO. A-45
MADE BY T Stones DATE 1-29-90 JOB NO. 32-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 86
CALCULATIONS FOR Abutment Design

Examine Terminating some longitudinal Reinforcement.

Refer to moment diagram on sht.

$\ell_{d\#11} = 59.2$ inches use 5 ft

Neg. steel capacity: $A_s = 24(1^{56}) = 37^{44}$ in^2 18 #11

$d_{1-18} = (48-2-0.625-1.41) = 43.^{97}$ in

$d_{23-24} = (48-6-2-0.625-1.41) = 37.^{97}$ in

$d_{19-20} = 41^{97}$ $d_{21-22} = 39^{97}$

$d_{ave} = \frac{1}{24}\left[18(43^{97}) + 2(41^{97} + 39^{97} + 37^{97})\right]$

$d_{ave} = 42^{97}$ in

$\rho = \frac{A_s}{bd} = \frac{37^{44}}{78(42^{97})} = 0.001117$ $n = 8$

$k = \sqrt{2\rho n + (\rho n)^2} - \rho n = 0.343$ $j = 1 - \frac{k}{3} = 0.886$

$M_{cap} = 24^{kai}(37^{44\,in^2})(0.886)(42^{97\,in})/12^{in/ft} = \underline{2850 \text{ ft-k}}$

14 #18 $A_s = 14(1^{56}) = 21^{84}$ in^2

$d_{3-9} = 48-2-0^{625} - \frac{1^{41}}{2} = 44^{67}$ in

$d_{1-2, 10-11} = 43^{97}$ in

$d_{14} = 48-6-2-0^{625} - \frac{1^{41}}{2} = 38^{67}$

$d_{13} = 40^{67}$ $d_{12} = 42^{67}$

$d_{ave} = \frac{1}{14}\left[7(44^{67}) + 4(43^{97}) + 38^{67} + 40^{67} + 42^{67}\right]$

$d_{ave} = 43^{61}$ in

$\rho = 0.00642$ $k = 0.273$ $j = 0.909$

$M = 24(21^{84})(0.909)(43^{61})/12 = \underline{1731 \text{ ft-k}}$

Try 6 #11 $A_s = 9^{36}$ in^2 $d = \frac{1}{6}\left[(44^{67}) + 38^{67} + 4(43^{97})\right] = 43^{20}$

$\rho = 0.00278$ $j = 0.190$ $k = 0.937$

$M = 24(9^{36})(.937)(43^{20})/12 = \underline{758 \text{ ft-k}}$

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. A-46
MADE BY T Stones DATE 1-29-90 JOB NO. 32-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 87
CALCULATIONS FOR Abutment Design

Northern Avenue Bridge Over the New River
Abutment Walls

Active Pressure is $30^{\#}/ft^2/ft$ per foundation study & report

Abutment backwall

Tractive force $= 0.05[18 + \frac{1}{2}(0.640)16] = 1.156^{K}$, over 12'

Moment/ft $\frac{1.156(6.375)}{12} = 0.62\frac{^{1}K}{ft}$ ⊥

2ft of Surcharge $2(.030)\frac{(6.375)^2}{2} = 1.22\frac{^{1}K}{ft}$

∴ Surcharge governs ⊥ Moments

Active Pressure $0.030\frac{(6.375)^3}{6} = 1.295\frac{^{1}K}{ft}$

Total Moment $= 1.22 + 1.30 = 2.52\frac{^{1}K}{ft}$

$f_r = 7.5(4000)^{1/2} = 475psi$ $\quad 1.2\,M_{cr} = 1.2(475)\frac{12(15)^2}{6} \times \frac{1}{12000}$
$= 21.3^{K}$

∴ increase design moment by 1/3 $M = 2.52 \times 1\frac{1}{3} = 3.36^{K}$

$f'_c = 4{,}000$ $\quad n = 8$
$f_y = 60KSI$
$f_s = 1600$
$= 0.884$

$A_s = \frac{M}{f_s j d}$ where $d = 15 - 2 - \frac{1}{2}(5/8)$ $\quad K = 0.348$
$d = 12.6$

$A_s = \frac{3.36 \times 12}{24 \times 12.6 \times .884} = 0.15\frac{\square''}{ft}$ ∴ Use #4@12" both front & back

Corbel Reinforcement

$\frac{15.29}{2} \times \frac{12}{12}(.150) = 1.15^{K}/Lf$ D

$L = \frac{32 \times 1.3}{12} = 3.47^{K}/Lf$ L

$V_{D+L} = 4.62\frac{K}{Lf}$

$A_{vf} = \frac{4.62}{12 \times 1.4} = 0.27\frac{\square''}{ft}$

Use #5@12"

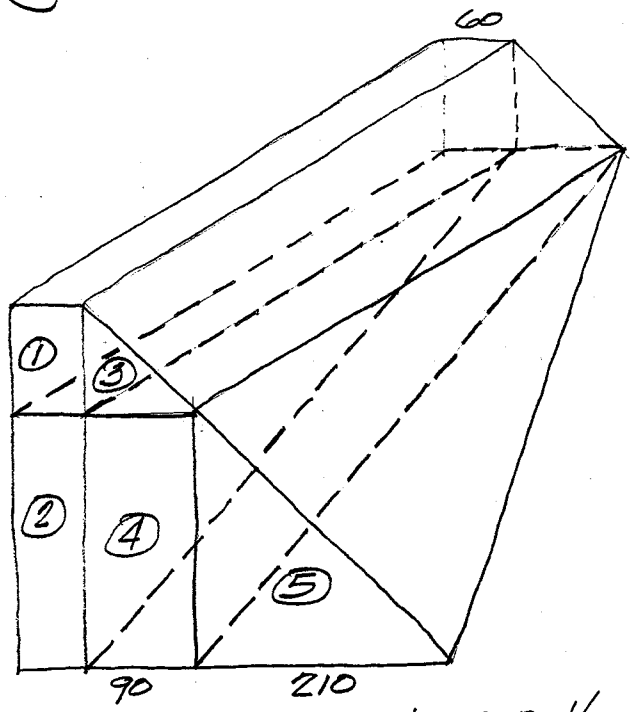
Use #4@12" with two legs penetrating crack plane

$A_{vf} = \frac{V}{f_s \mu}$

BRW

JOB Northern Avenue Br SHEET NO. A-49
MADE BY Lee Arnold DATE 2/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 100
CALCULATIONS FOR Wall Design

Northern Avenue Bridge Over the New River
Wingwalls & Curtains



Place 2' surcharge for construction loading

Length from back of abutment wall to end of wing wall = 17.5'

$\Sigma M_{about\ back\ of\ wall}$

① $60(3)\frac{(17.5)^2}{2} = 27,563$

② $\frac{60(7)}{2}\frac{(17.5)^2}{3} = 21,438$

③ $\frac{90}{2}(7.0)\frac{(17.5)^2}{2} = 48,235$

④ $\frac{90(7.0)}{2}\frac{(17.5)^2}{3} = 32,156$

⑤ $\frac{210(7)}{2\times 3}\frac{(17.5)^2}{4} = 18,757$

$148,149$

$d = 12 - 2 - \frac{1}{2} - \frac{1}{2}(1) = 9$

$f'_c = 4000 \quad f_c = 1600 \quad n = 8$

$K = \frac{f_c}{\frac{f_s}{n} + f_c} = \frac{1600}{\frac{24,000}{8} + 1600} = 0.348 \quad J = 0.884$

$A_s = \frac{M_w}{f_s J d} = \frac{148,149 \times 12}{24,000(.884)9} = 9.32\ \square''$ Use 12 #8

Vertical

$0.10\frac{(18)^2}{2} = 16.2^{k}$ $\quad d = 117''$

$3.5(1.0)(.150)\frac{(18)^2}{2} = 85.05^{k}$ $\quad A_s = 1\frac{1}{3}\frac{(158,000)12}{24,000(.884)117} = 1.02\ \square''$

$\frac{7.0}{2}(1.0)(.150)\frac{(18)^2}{3} = 56.7$ $\quad$ Use 2 #8 min

158^{k}

Lateral Earthquake Restrainer (Ref G-10)

$EQ = 9 \times 114.54 \times 0.20 = 207^{kips}$ Lateral EQ Force

Shear friction $= A_{vf} = \frac{V}{f_s \mu}$ assume poured after girders are set

$A_{vf} = \frac{207}{24 \times 1.0} = 8.62$ 20 #6 bars required to penetrate crack plane

BRW

JOB Northern Avenue SHEET NO. A-50
MADE BY Lee Arnold DATE 2/90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 101
CALCULATIONS FOR Wall Design

NOTE: ALL ELEV ARE TO TOP OF BRIDGE SEAT AND TOP OF BEARING PAD

J	H	G	F	E	D	C	B	A
1078.02 / 1077.77	1078.21 / 1077.96	1078.39 / 1078.14	1078.58 / 1078.33	1078.77 / 1078.52	1078.58 / 1078.33	1078.39 / 1078.14	1078.21 / 1077.96	1078.02 / 1077.77
1078.45 / 1078.28	1078.63 / 1078.46	1078.82 / 1078.65	1079.01 / 1078.84	1079.20 / 1079.03	1079.01 / 1078.84	1078.82 / 1078.65	1078.63 / 1078.46	1078.45 / 1078.28
1078.47 / 1078.30	1078.65 / 1078.48	1078.84 / 1078.67	1079.03 / 1078.86	1079.22 / 1079.05	1079.03 / 1078.86	1078.84 / 1078.67	1078.65 / 1078.48	1078.47 / 1078.30
1078.06 / 1077.89	1078.25 / 1078.08	1078.43 / 1078.26	1078.63 / 1078.45	1078.81 / 1078.64	1078.63 / 1078.45	1078.43 / 1078.26	1078.25 / 1078.08	1078.06 / 1077.89
1077.20 / 1076.95	1077.39 / 1077.14	1077.57 / 1077.32	1077.76 / 1077.51	1077.95 / 1077.70	1077.76 / 1077.51	1077.57 / 1077.32	1077.39 / 1077.14	1077.20 / 1076.95

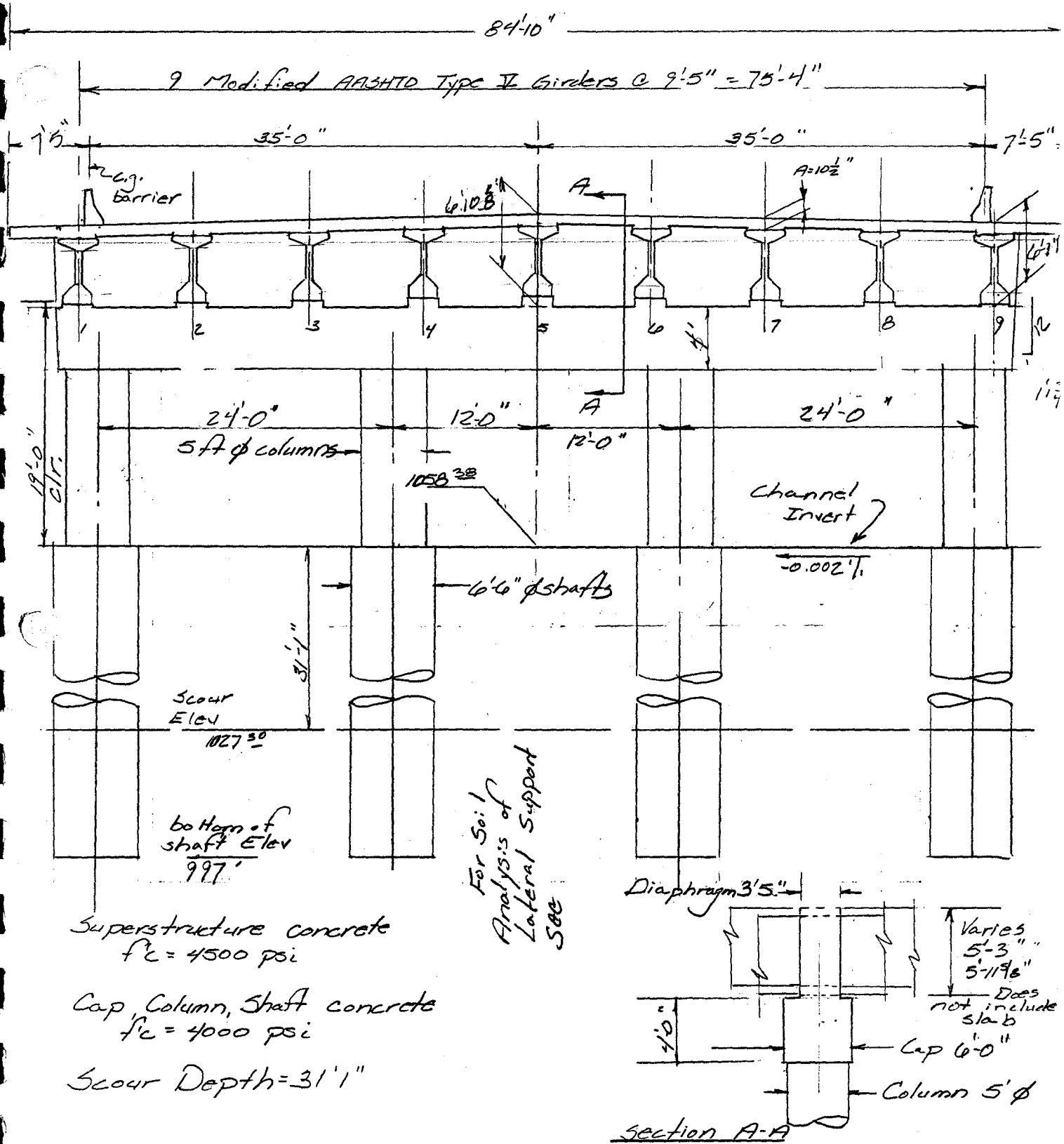
BRW

JOB NORTHERN AV. BRIDGE SHEET NO.

MADE BY MAB DATE 7 FEB 90 JOB NO.

CHECKED BY Lee Arnold DATE 2/8/90 SEC. SHT. NO. 102

CALCULATIONS FOR ABUT AND PIER SEAT ELEV.



Superstructure concrete
$f'_c = 4500$ psi

Cap, Column, Shaft concrete
$f'_c = 4000$ psi

Scour Depth = 31'1"

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave. @ New River SHEET NO. P 1
MADE BY T Stones DATE 10-18-89 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 103
CALCULATIONS FOR Center Bent Frame Analysis

Section Properties

Cap: $f'_c = 4000$ psi

$E = 57{,}000\sqrt{4000} = 3{,}605{,}000$ psi (519,120 ksf) Ref. AASHTO 8.7.1

$I_z = \frac{bh^3}{12} = \frac{(72\text{ in})(48\text{ in})^3}{12} = 663{,}552\ in^4$ (32.0 ft^4)

$A = bh = (72\text{ in})(48\text{ in}) = 3456\ in^2$ (24.0 ft^2)

Column: $f'_c = 4000$ psi

$E = 3{,}605{,}000$ psi (519,120 ksf)

$I_z = \frac{\pi d^4}{64} = \frac{\pi(60)^4}{64} = 636{,}172\ in^4$ (30.68 ft^4)

$A = \frac{\pi d^2}{4} = \frac{\pi(60)^2}{4} = 2827\ in^2$ (19.632 ft^2)

Shaft: $f'_c = 4000$ psi $\phi = 6'\text{-}6" = 78"$

$E = 3{,}605{,}000$ psi (519,120 ksf)

$I_z = \frac{\pi(78)^4}{64} = 1{,}816{,}972\ in^4$ (87.624 ft^4)

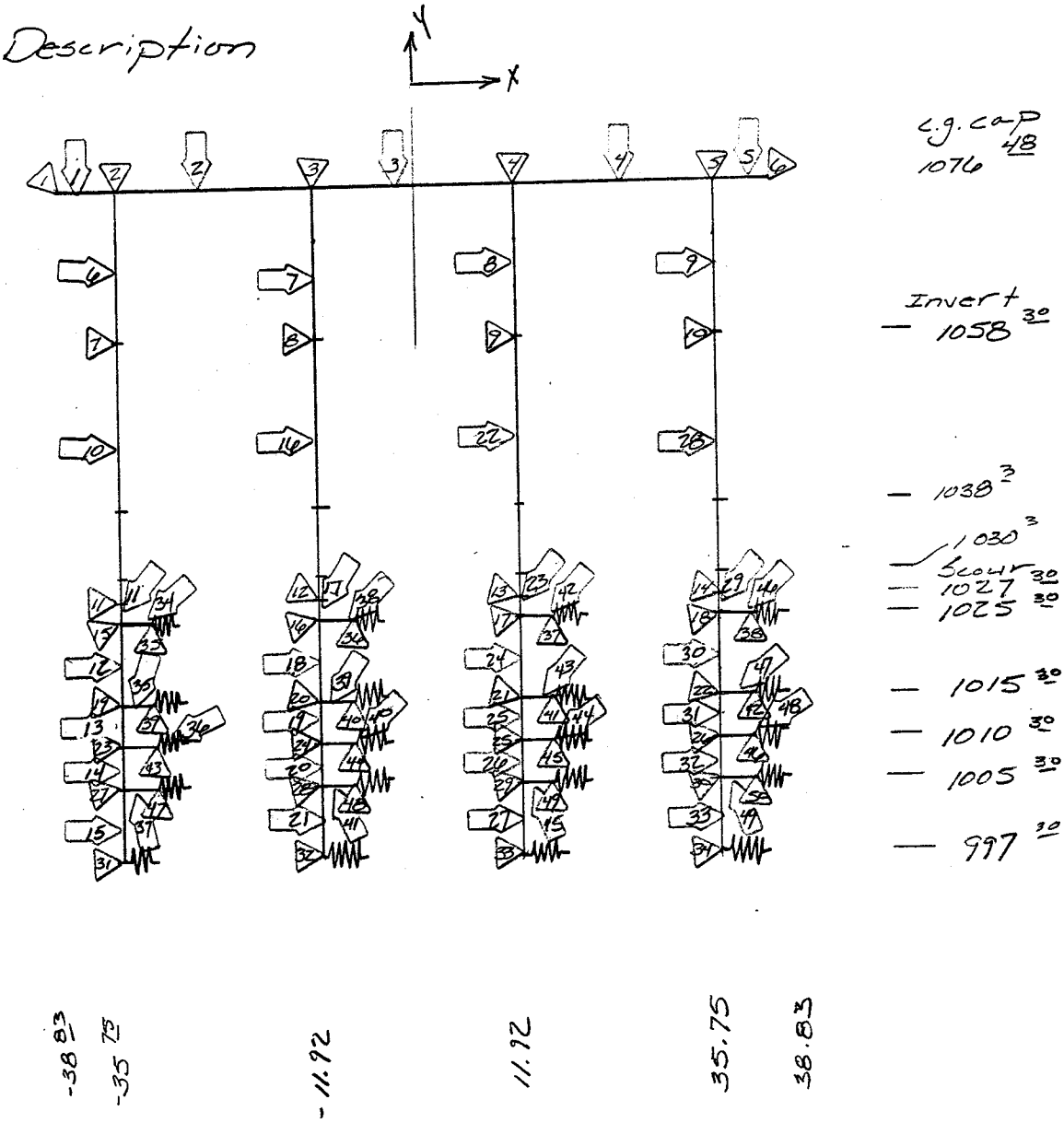
$A = \frac{\pi(78)^2}{4} = 4778\ in^2$ (33.183 ft^2)

DENSITY $= \left(\frac{150\text{ lb}}{ft^3}\right)\frac{1\ ft^3}{12^3\ in^3} = 0.0868\ lb/in^3$ (0.15 k/ft^3)

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. P 2
MADE BY T Stones DATE 10-19-89 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 104
CALCULATIONS FOR Center Bent Frame Analysis

Frame Description



Joint Release
31 TO 50 As Support

31 TO 34 Mom Z KFX 60.434
47 TO 50 KFX 34.47
43 TO 46 KFX 33.376
39 TO 42 KFX 21.673
35 TO 38 KFX 4.039

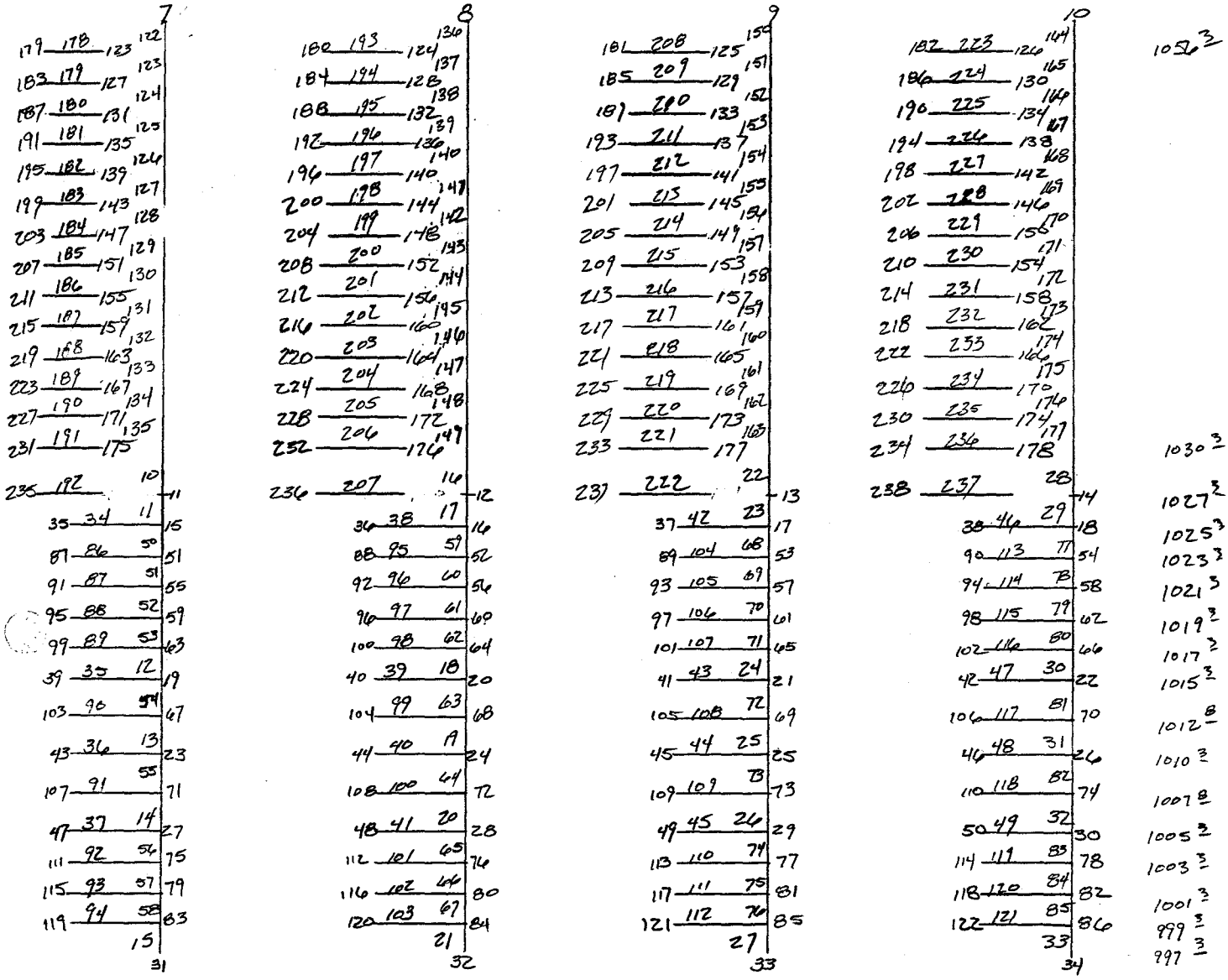
Member Release
34 TO 49 start Force Y Mom Z

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. P3
MADE BY T. Stones DATE 12-22-89 JOB NO. 338962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 105
CALCULATIONS FOR First Bent Frame Analysis

Spring Connector & Shaft M-Strudl Joint & Members

Only member no.'s through #121 included in analysis with scour



JOB Northern Ave @ New River SHEET NO. P3A

MADE BY T. Stones DATE 1-12-90 JOB NO. 33-8962

CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 106

CALCULATIONS FOR First Bent Frame Analysis

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The following p-y curves are generated from COM624G Computer Program.

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
12.00	78.00	38.0	.32D-01

Y IN	P LBS/IN
.000	.000
.108	33.150
.217	66.300
.325	99.450
.433	132.600
.542	165.750
.650	198.900
.758	232.050
.867	246.941
.975	254.776
1.083	261.994
1.192	268.699
1.300	274.971
2.925	366.111

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
84.00	78.00	36.0	.35D-01

Y IN	P LBS/IN
.000	.000
.108	400.400
.217	800.800
.325	1201.200
.433	1601.600
.542	1883.749
.650	1988.834
.758	2082.242
.867	2166.696
.975	2244.027
1.083	2315.538
1.192	2382.190
1.300	2444.711
2.925	3354.575

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. P-3 B
MADE BY T. Stones DATE 1-12-90 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 107
CALCULATIONS FOR First Bent Frame Analysis

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
234.00	78.00	34.5	.34D-01

Y IN	P LBS/IN
.000	.000
.108	1115.400
.217	2230.800
.325	3045.178
.433	3411.020
.542	3724.795
.650	4002.476
.758	4253.342
.867	4483.326
.975	4696.487
1.083	4895.740
1.192	5083.259
1.300	5260.715
2.925	7854.025

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
174.00	78.00	34.0	.35D-01

Y IN	P LBS/IN
.000	.000
.108	829.400
.217	1658.800
.325	2488.200
.433	2978.860
.542	3209.682
.650	3411.492
.758	3591.988
.867	3756.042
.975	3906.956
1.083	4047.084
1.192	4178.170
1.300	4301.546
2.925	6099.874

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
360.00	78.00	33.0	.33D-01

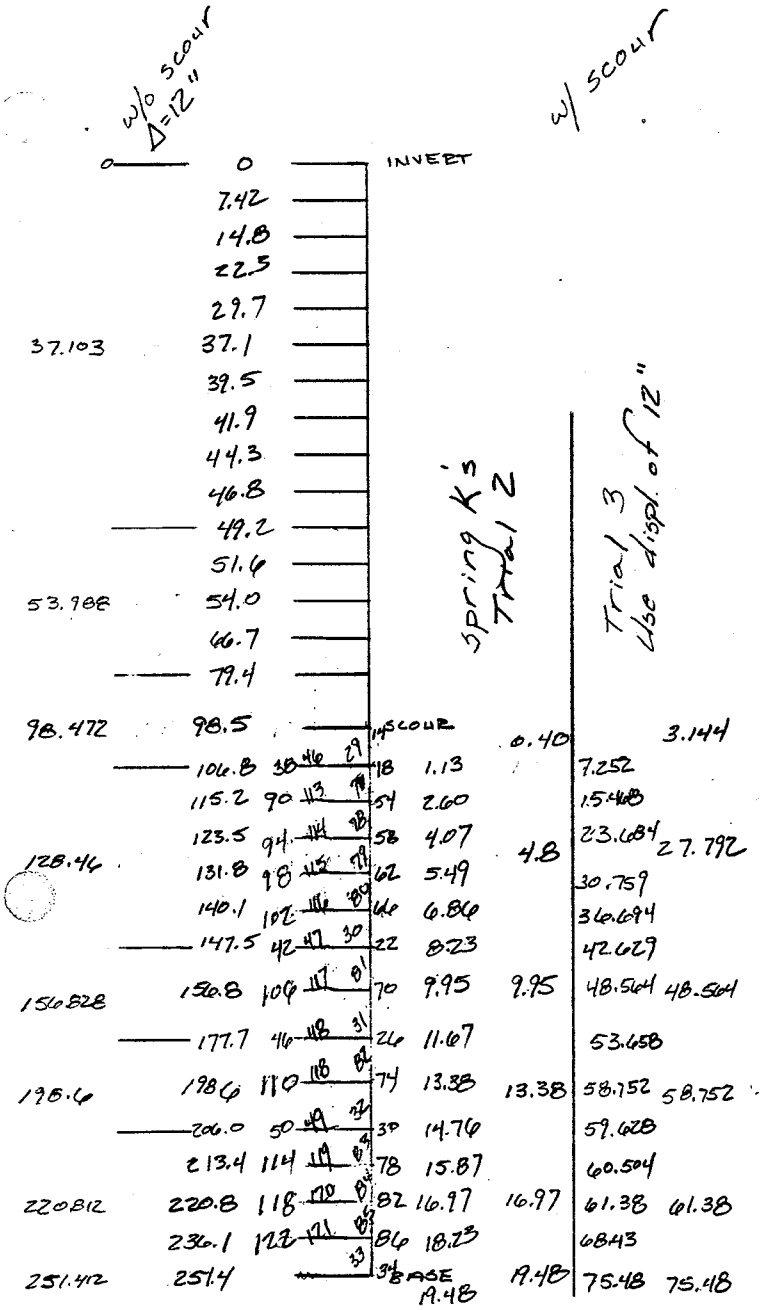
Y IN	P LBS/IN
.000	.000
.108	1623.295
.217	2440.532
.325	3097.946
.433	3669.202
.542	4183.897
.650	4657.587
.758	5099.691
.867	5516.439
.975	5912.217
1.083	6290.253
1.192	6653.011
1.300	7002.420
2.925	12151.567

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
312.00	78.00	33.0	.34D-01

Y IN	P LBS/IN
.000	.000
.108	1413.777
.217	2082.108
.325	2611.251
.433	3066.378
.542	3473.354
.650	3845.661
.758	4191.411
.867	4515.940
.975	4822.990
1.083	5115.305
1.192	5394.969
1.300	5663.612
2.925	9617.454

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. P3C
MADE BY T. Stones DATE 1-12-90 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 108
CALCULATIONS FOR First Bent Frame Analysis

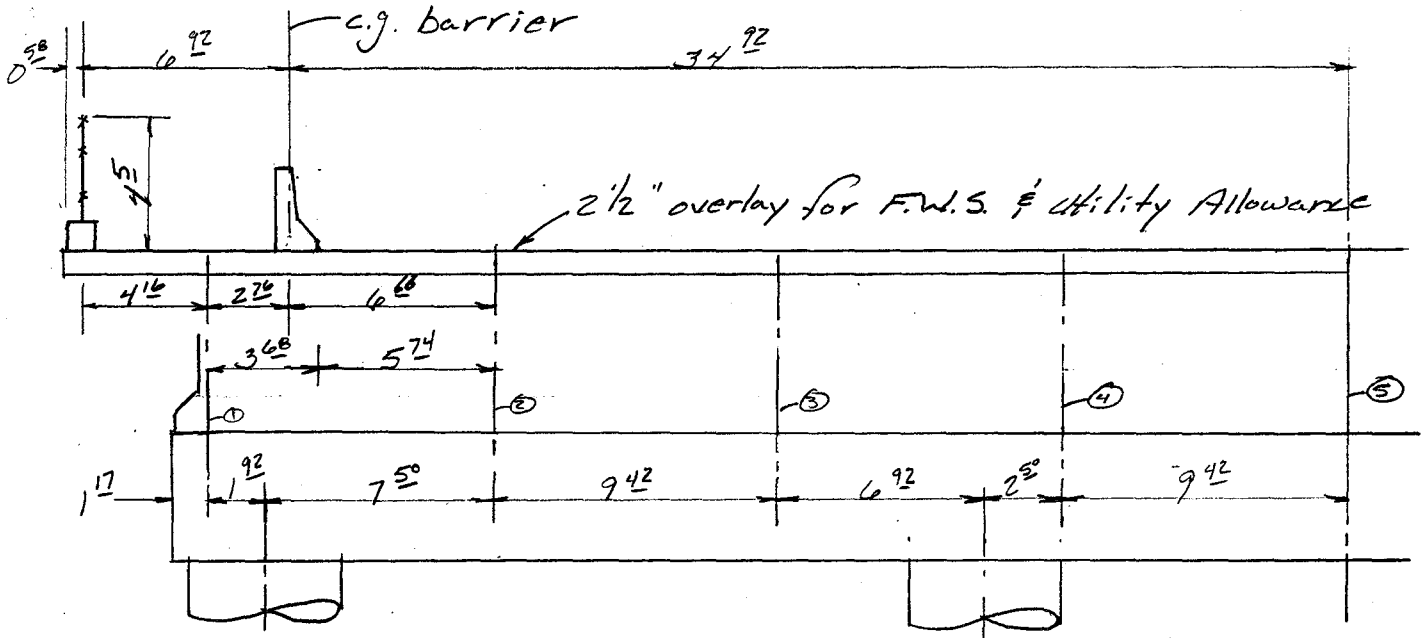


0	0				INVERT					
	7.42									
	14.8									
	22.5									
	29.7									
37.103	37.1									
	39.5									
	41.9									
	44.3									
	46.8									
	49.2									
	51.6									
53.988	54.0									
	66.7									
	79.4									
98.472	98.5				SCOUR		0.40		3.144	
	106.8	38	46	29	18	1.13		7.252		
	115.2	90	113	79	54	2.60		15.468		
	123.5	94	114	78	58	4.07	4.8	23.684	27.792	
128.46	131.8	98	115	79	62	5.49		30.759		
	140.1	102	116	80	66	6.86		36.694		
	147.5	42	47	30	22	8.23		42.629		
156.828	156.8	106	117	81	70	9.95	9.95	48.564	48.564	
	177.7	46	48	31	26	11.67		53.658		
198.6	198.6	110	118	84	74	13.38	13.38	58.752	58.752	
	206.0	50	49	32	30	14.76		59.628		
	213.4	114	119	84	78	15.87		60.504		
220.812	220.8	118	120	84	82	16.97	16.97	61.38	61.38	
	236.1	122	121	84	86	18.23		68.43		
251.412	251.4			33	34	BASE 19.48	19.48	75.48	75.48	

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. P3D
MADE BY T Stones DATE 1-12-90 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 109
CALCULATIONS FOR First Bent Frame Analysis

Dead Load:



Bridge Barrier: Type B weight/ft $= \left(\frac{0.0918\ cy}{ft}\right)\left(\frac{27\ ft^3}{cy}\right)\left(\frac{.150\ k}{ft^3}\right) = 0.372$ kpf

fraction to girder ① $= \frac{6\ 6/6}{9\ 4/2} = 0.707$

fraction to girder ② $= \frac{2\ 7/6}{9\ 4/2} = 0.293$

Pedestrian Rail & Curb: Applied to girder ①

Allow 125 p/ft for chain link fence $= 0.125$ k/ft

Curb dimensions are 12" x 10" : $w = 1\left(\frac{10}{12}\right)(.15) = \underline{0.125}$ k/ft

Total $= 0.25$ k/ft

From PSGDES program output: DL end reaction/girder is 102.7 k includes girder and slab.

Diaphragm Load varies due to 0.02% cross-slope

$DL_{Dia\,①} = \left(\frac{7}{12}\right)(3.42\ ft)(.150) = 3.1$ k/ft $\quad DL_{Dia\,⑤} = \left(\frac{7\ 9/25}{12}\right)(3.42)(.150) = 3.4$ k/ft

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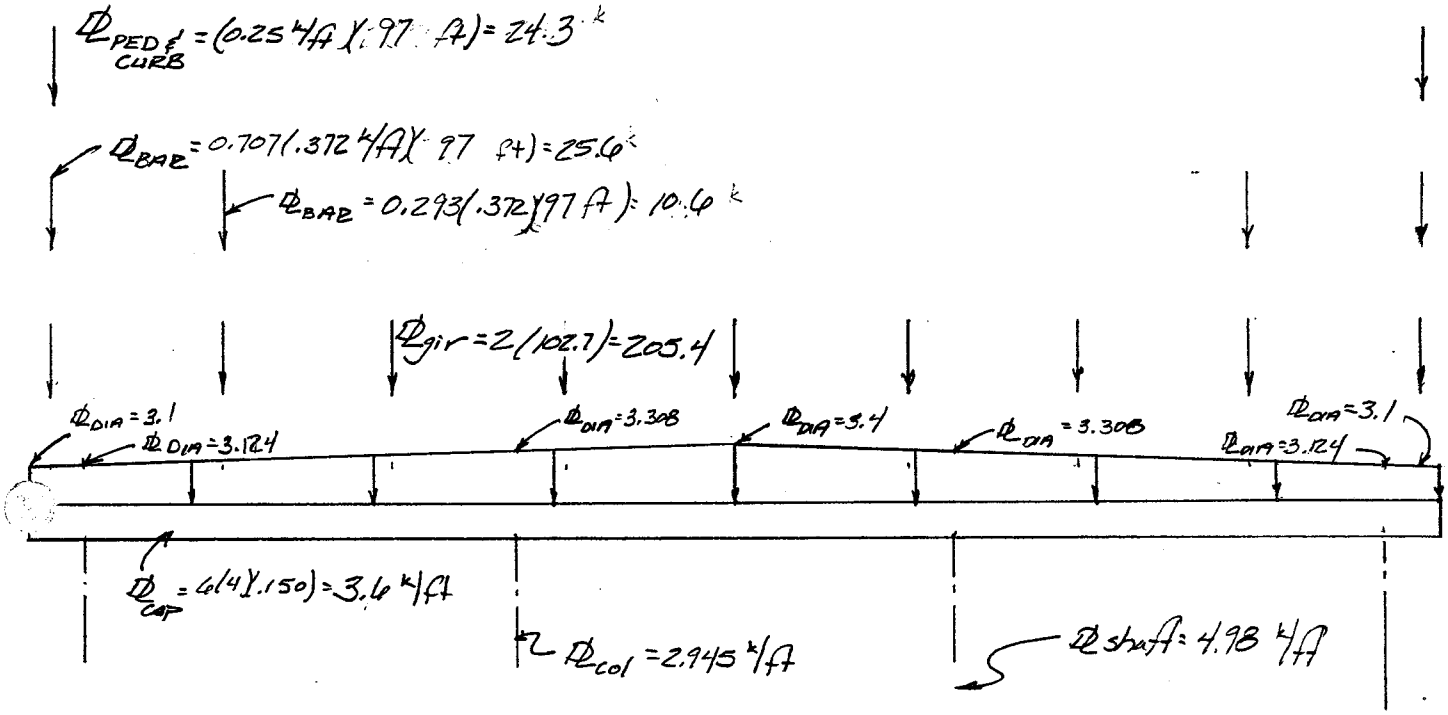
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. P4
MADE BY K Stones DATE 12-22-89 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 110
CALCULATIONS FOR Transverse Frame Analysis

DL of superstructure will be applied as point loads @ each girder location assuming simple span of 95 ft for each girder. Diaphragm and Slab @ Piers will be applied as a Distributed load.



Sidewalk Load: 60 lb/ft^2 applied to exterior girders only

From Beam Diagrams in AISC Manual pg. 2-127 for 4 equal spans all loaded, the maximum reaction @ the 1st interior support is $1.143\,wl$ $\quad w = 6(60) = 360\ p/ft$

$l = 97\ ft$

$R_{SDWK} = 1.143(97)(360/1000) = 39.9^k$ applied to exterior girder locations.

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. P5
MADE BY T Stones DATE 12-22-89 JOB NO. 33-89102
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. III
CALCULATIONS FOR Transverse Frame Analysis

Traffic Live Load: For distribution of 4 reactions applied to each beam for various loading conditions refer to sht. P6B

AASHTO Distribution Factor $g = \frac{S}{5.5} = \frac{9.42}{5.5} = 1.713$

Impact Factor for Reaction @ 1st interior pier

$I = \frac{50}{(97)+125} = 0.225$

From Continuous Highway Bridge Tables:

$R_{95ft} = 100.4^k$

$R_{100ft} = 104.3^k$

$R_{97ft} = 100.4 + \frac{2}{5}(3.9) = 102.0^k$

$I = .225(102) = 23.0$

w/ Impact $T = 125.0^k$/lane

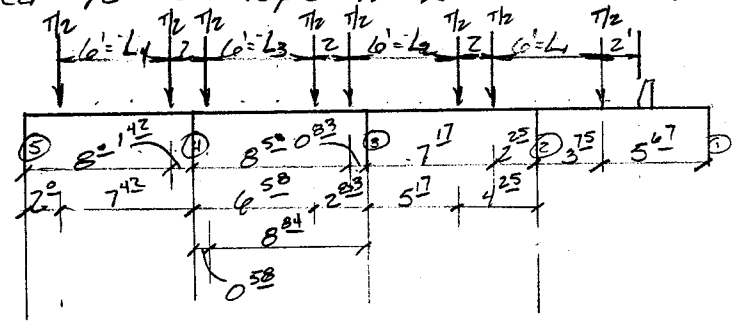
# of Traffic Lanes $= \frac{68}{12} = 5.67$ use 5 lanes

Reduce for improbability of coincidence: 5 (.75) = 3.75 lan

for simplicity I'll load no more than 4 lanes at a time: this will yield conservative design.

Note: Impact is not required below scour line for design of shafts.

In addition to the load patterns shown on sht P6B, the following will be applied to develope max moms in shafts



JOB Northern Ave @ New River SHEET NO. P 6

MADE BY T Stones DATE 12-22-89 JOB NO.

CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 112

CALCULATIONS FOR Transverse Frame Analysis

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Live Load Reactions for shaft design:

① $\mathcal{L} = 3.75/9.42 \left(\frac{T}{2}\right) = 0.398\ T/2$

② $\mathcal{L} = \frac{1}{9.42}(5.67 + 7.17 + 5.17)(T/2) = 1.912\ T/2$

③ $\mathcal{L} = \frac{1}{9.42}(2.25 + 4.25 + 8.58 + 6.58 + 0.58)(T/2) = 2.361\ T/2$

④ $\mathcal{L} = \frac{1}{9.42}(0.83 + 2.83 + 8.84 + 8.0 + 2.0)(T/2) = 2.389\ T/2$

⑤ $\mathcal{L} = \frac{1}{9.42}(1.42 + 7.42)(T/2) = 0.938\ T/2$

✓ $7.998\ T/2$

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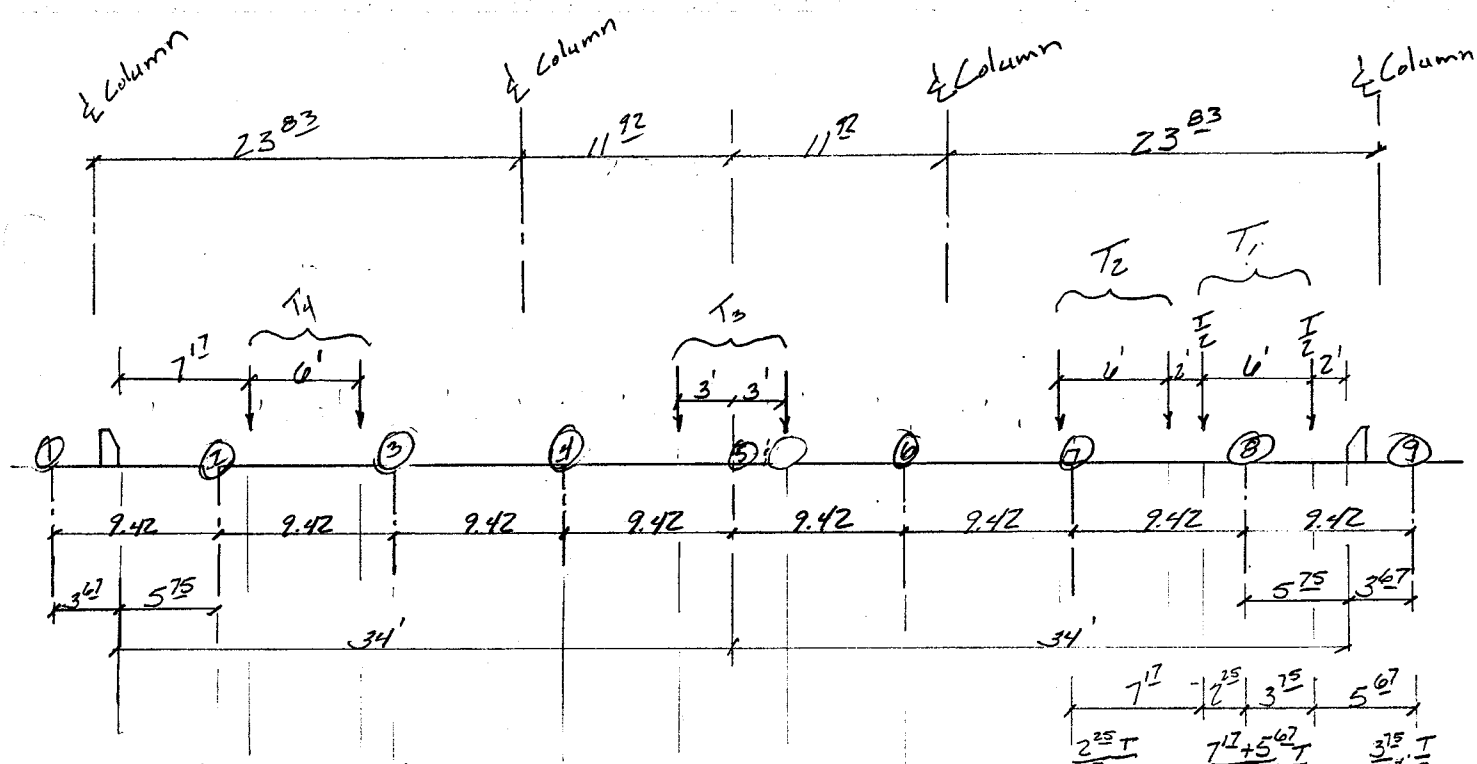
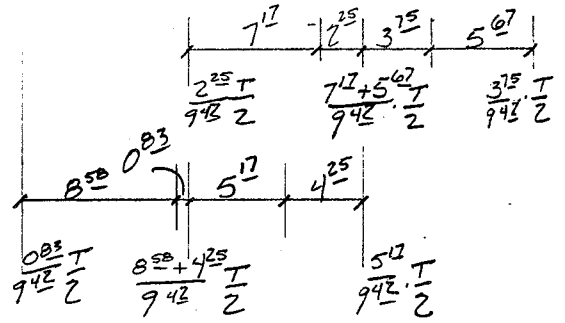
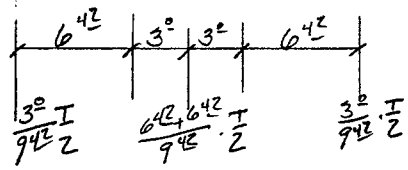
JOB Northern Ave @ New River SHEET NO. P 6 A
MADE BY T. Stones DATE 12-27-89 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 113
CALCULATIONS FOR Transverse Frame Analysis

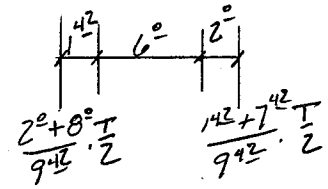


Loading w/ T_1 & T_2 will give max. positive moments in exterior cap spans & combined w/ same loading on other side will give max neg. moments in center cap span



$\checkmark 4.0\left(\frac{T}{2}\right) = 0.09\left(\frac{T}{2}\right) \quad 1.60\left(\frac{T}{2}\right) \quad 1.91\left(\frac{T}{2}\right) \quad 0.40\left(\frac{T}{2}\right)$





Loading w/ T_3 & T_4 will give max neg mom. over column

$1.06 \cdot \frac{T}{2} \quad 0.94 \frac{T}{2} \quad 0.32 \frac{T}{2} \quad 1.36 \cdot \frac{T}{2} \quad 0.32 \frac{T}{2} = 4.0 \frac{T}{2} \checkmark$

$T = 125^k$ includes Impact

$T = 102^k$ w/o Impact

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JOB Northern Ave @ New River SHEET NO. P 6B
MADE BY T Jones DATE 12-20-89 JOB NO. 338962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 114
CALCULATIONS FOR Frame Analysis: Live Load

LATERAL LOADS

Wind Load:

Force From Superstructure: AASHTO 3.15.2.1.3

WI: Wind Load on Structure
50 psf transverse } both forces acting simult.
12 psf longitudinal

Effective Area: Height: Structure 6' 1½"
Barrier 2' 8"
8' 9½" (8.792)

Length: 97 ft.

$W_T = 50(8.792)(97)/1000 = 42.64^k$ acting @ 6.9 ft Above the c.g. of Cap.

$W_L = 12(8.792)(97)/1000 = 10.24^k$

WL: Wind on Live Load
100 plf transverse } Both acting simultaneous
40 plf longitudinal

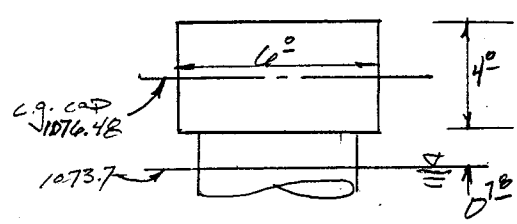
$WL_T = 100(97)/1000 = 9.7^k$ acting @ 14.625 ft above the c.g. of cap

$WL_L = 40(97)/1000 = 3.9^k$

Force From Substructure: AASHTO 3.15.2.2

W = 40 psf acting on exposed area. Maximum combination will be with stream flow forces during SPF (Standard Project Flood)

Water surface Elev @ SPF = 1073.7



Area = (4)(6) + 5(0.78) = 27.9 ☐ft

$W_T = 40(27.9)/1000 = 1.12^k$ acting at the c.g. of cap

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JOB Northern Ave @ New River SHEET NO. P7
MADE BY T. Stores DATE 10-19-89 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 115
CALCULATIONS FOR Center Bent Frame Analysis

Overturning Force: AASHTO 3.15.3

OT 20 psf for Group II and V
OT 6 psf for Group III and VI

Contributing Area = (97 ft)(84.84 ft) = 8228.8 #/ft

Group II & V $OT = \frac{8228.8(20)}{1000} = 164.6$ k

Group III & VI $OT = \frac{8228.8(6)}{1000} = 49.4$ k

These forces act at 1/4 pt

X Coordinate = 42.42 (1/2) = 21.21 ft

Forces From Stream Current: AASHTO 3.18.1

V = 14.7 fps K = 1.375

Use twice column diameter and square pier shape to account for drift

$P_{@} = 1.375 (14.7)^2 = 297$ psf

use a uniform load on member 5 of

$W_{5F_{@}} = .297 (10) = 2.970$ klf

Assume little drift on middle and downstream columns K = 0.667

$P_{7,8,9} = 0.667 (14.7)^2 = 144$ psf

$W_{5F_{7,8,9}} = 0.144 (5) = 0.72$ klf

Thermal Force: M-Strudl Program will calculate forces from temperature rise of 30°F and temperature fall of 40°F

Differential Settlement: M-Strudl Program will calculate forces from a differential settlement between columns of 3/8" per Thomas-Hartig Soil Analysis.

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JOB Northern Ave @ New River SHEET NO. P8
MADE BY T. Stones DATE 10-20-89 JOB NO. 33-89102
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 116
CALCULATIONS FOR Center Bent Frame Analysis

2/90

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*************************************************************************
*                                                                       *
* TYPE OF THE STRUCTURE                    : PLANE FRAME                *
* RESTART STATUS                           :    NONE GIVEN              *
* DESIGNATED UNITS                         : KIPS     FEET   DEGREES    *
* NO. OF JOINTS DEFINED                    :     0                      *
* NO. OF MEMBERS DEFINED                   :     0                      *
* NO. OF PLATE ELEMENTS DEFINED            :     0                      *
* NO. OF DIFF. MATERIAL DEFINED            :     0                      *
* NO. OF DIFF. SECTION PROPERTIES DEFINED:      0                      *
* NO. OF DIFF. TYPES OF SPRINGS DEFINED    :     0                      *
*                                                                       *
*************************************************************************
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****************************
* LISTING OF THE INPUT FILE *
****************************

  1) $ THE FOLLOWING DATA IS THE INPUT FILE FOR THE ANALYSIS OF THE
  2) $ NORTHERN AVENUE BRIDGE , COLUMN BENTS 1 AND 3 WITH SCOUR
  3) $
  4) TITLE ANALYSIS OF RIVER PIER BENTS WITH SCOUR
  5) TYPE PLANE FRAME
  6) UNIT KIP FEET DEGREE
  7) REPORT DEV A:NAOUT3
  8) REPORT PAG 9
  9) $ FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
 10) $ NORTHERN AVENUE BRIDGE OVER THE NEW RIVER
 11) $ BRW, INC.; PHOENIX, ARIZONA
 12) $ JOB NUMBER 33-8962; JANUARY, 1990
 13) REPORT LOGO
 14) PRINT INPUT
 15) OUTPUT DEC 2
 16) OUTPUT BY MEMBER
 17) JOINT COORDINATES
 18) 1 -38.83 1076.48 F
 19) 2 -35.75 1076.48
 20) 3 -11.92 1076.48
 21) 4 11.92 1076.48
 22) 5 35.75 1076.48
 23) 6 38.83 1076.48 F
 24) 7 -35.75 1058.3
 25) 8 -11.92 1058.3
 26) 9 11.92 1058.3
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 33) 16 -11.92 1025.3
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2/90

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 72) 51 TO 63 BY 4
 73) 66 35.75 1017.3
 74) 54 TO 66 BY 4
 75) 51 TO 54
 76) 55 TO 58
 77) 59 TO 62
 78) 63 TO 66
 79) 67 -35.75 1012.8
 80) 70 35.75 1012.8
 81) 67 TO 70
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✓ 2/90

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* LISTING OF THE INPUT FILE *
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 82) 71 -35.75 1007.8
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 84) 71 TO 74
 85) 75 -35.75 1003.3
 86) 78 35.75 1003.3
 87) 83 -35.75 999.3
 88) 86 35.75 999.3
 89) 75 TO 83 BY 4
 90) 78 TO 86 BY 4
 91) 75 TO 78
 92) 79 TO 82
 93) 83 TO 86
 94) 87 -35.67 1023.3
 95) 90 35.83 1023.3
 96) 99 -35.67 1017.3
 97) 102 35.83 1017.3
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 99) 90 TO 102 BY 4
100) 87 TO 90
101) 91 TO 94
102) 95 TO 98
103) 99 TO 102
104) 103 -35.67 1012.8
105) 106 35.83 1012.8
106) 103 TO 106
107) 107 -35.67 1007.8
108) 110 35.83 1007.8
109) 107 TO 110
110) 111 -35.67 1003.3
111) 114 35.83 1003.3
112) 119 -35.67 999.3
113) 122 35.83 999.3
114) 111 TO 119 BY 4
115) 114 TO 122 BY 4
116) 111 TO 114
117) 115 TO 118
118) 119 TO 122
119) $
120) EDIT $ INCREASE COLUMN SPACING TO 24 FEET ON CENTERS
121) MOVE 2 7 TO 119 BY 4 X -0.25
122) MOVE 3 8 TO 120 BY 4 X -0.08
123) MOVE 4 9 TO 121 BY 4 X 0.08
124) MOVE 5 10 TO 122 BY 4 X 0.25
125) $
126) JOINT RELEASE
127) 87 TO 122 AS SUPPORT
128) 31 TO 34 MOM Z KFX 75.48
129) 47 TO 50 KFX 59.628
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119

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* LISTING OF THE INPUT FILE *
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132) 35 TO 38 KFX 7.252
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134) 91 TO 94 KFX 23.684
135) 95 TO 98 KFX 30.759
136) 99 TO 102 KFX 36.694
137) 103 TO 106 KFX 48.564
138) 107 TO 110 KFX 58.752
139) 111 TO 114 KFX 60.504
140) 115 TO 118 KFX 61.38
141) 119 TO 122 KFX 68.43
142) $
143) MEMBER INCIDENCES
144) 1 1 2
145) 2 2 3
146) 3 3 4
147) 4 4 5
148) 5 5 6
149) 6 7 2
150) 7 8 3
151) 8 9 4
152) 9 10 5
153) 10 11 7
154) 11 15 11
155) 12 19 63
156) 13 23 67
157) 14 27 71
158) 15 31 83
159) 16 12 8
160) 17 16 12
161) 18 20 64
162) 19 24 68
163) 20 28 72
164) 21 32 84
165) 22 13 9
166) 23 17 13
167) 24 21 65
168) 25 25 69
169) 26 29 73
170) 27 33 85
171) 28 14 10
172) 29 18 14
173) 30 22 66
174) 31 26 70
175) 32 30 74
176) 33 34 86
177) 34 15 35
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* LISTING OF THE INPUT FILE *
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178) 35 19 39
179) 36 23 43
180) 37 27 47
181) 38 16 36
182) 39 20 40
183) 40 24 44
184) 41 28 48
185) 42 17 37
186) 43 21 41
187) 44 25 45
188) 45 29 49
189) 46 18 38
190) 47 22 42
191) 48 26 46
192) 49 30 50
193) 50 51 15
194) 51 55 51
195) 52 59 55
196) 53 63 59
197) 54 67 19
198) 55 71 23
199) 56 75 27
200) 57 79 75
201) 58 83 79
202) 59 52 16
203) 60 56 52
204) 61 60 56
205) 62 64 60
206) 63 68 20
207) 64 72 24
208) 65 76 28
209) 66 80 76
210) 67 84 80
211) 68 53 17
212) 69 57 53
213) 70 61 57
214) 71 65 61
215) 72 69 21
216) 73 73 25
217) 74 77 29
218) 75 81 77
219) 76 85 81
220) 77 54 18
221) 78 58 54
222) 79 62 58
223) 80 66 62
224) 81 70 22
225) 82 74 26
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* LISTING OF THE INPUT FILE *
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226) 83 78 30
227) 84 82 78
228) 85 86 82
229) 86 51 87
230) 87 55 91
231) 88 59 95
232) 89 63 99
233) 90 67 103
234) 91 71 107
235) 92 75 111
236) 93 79 113
237) 94 83 119
238) 95 52 88
239) 96 56 92
240) 97 60 96
241) 98 64 100
242) 99 68 104
243) 100 72 108
244) 101 76 112
245) 102 80 116
246) 103 84 120
247) 104 53 89
248) 105 57 93
249) 106 61 97
250) 107 65 101
251) 108 69 105
252) 109 73 109
253) 110 77 113
254) 111 81 117
255) 112 85 121
256) 113 54 90
257) 114 58 94
258) 115 62 98
259) 116 66 102
260) 117 70 106
261) 118 74 110
262) 119 78 114
263) 120 82 118
264) 121 86 122
265) $
266) MEMBER RELEASE
267) 34 TO 49 START FORCE Y MOM Z
268) 86 TO 121 START FORCE Y MOM Z
269) $
270) MATERIAL PROPERTY
271) CONCR E 519120. DEN .150 CT 0.000006 1 TO 121
272) $
273) MEMBER PROPERTY
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* LISTING OF THE INPUT FILE *
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274) CAP AX 24.0 IZ 320.0 1 2 3 4 5 $ COLUMN CAP MEMBERS
275) COLUMN AX 19.632 IZ 30.68 6 7 8 9 $ COLUMNS
276) SHAFT AX 33.183 IZ 87.624 10 TO 33 $ SHAFTS
277) CONN AX 33.183 IZ 87.624 34 TO 49 $ FALSE CONNECTORS TO SPRINGS
278) SHAFTI AX 33.183 IZ 87.624 50 TO 85 $ SHAFT MEMBER FOR SPRING CON
     .
279) CONNI AX 33.183 IZ 87.624 86 TO 121 $ FALSE CONNECTORS TO SPRINGS

280) $
281) STIFFNESS ANALYSIS
282) $
283) LOAD DL
284) MEMBER LOADS
285) 1 FORCE Y CONC P -305.0 L 1.17 $ GIRDER 1 D.L. PLUS BARRIER & PED
      RAIL
286) 2 FORCE Y CONC P -226.6 L 7.75 $ GIRDER 2 D.L. PLUS BARRIER LOAD
287) 2 FORCE Y CONC P -205.4 L 17.17 $ GIRDER 3 D.L.
288) 3 FORCE Y CONC P -205.4 L 2.58 $ GIRDER 4 D.L.
289) 3 FORCE Y CONC P -205.4 L 12.0 $ GIRDER 5 D.L.
290) 3 FORCE Y CONC P -205.4 L 21.42 $ GIRDER 6 D.L.
291) 4 FORCE Y CONC P -205.4 L 6.84 $ GIRDER 7 D.L.
292) 4 FORCE Y CONC P -226.6 L 16.26 $ GIRDER 8 D.L. PLUS BARRIER LOAD

293) 5 FORCE Y CONC P -305.0 L 1.68 $ GIRDER 9 D.L. PLUS BARRIER & PED
      RAIL
294) 1 FORCE Y LIN WA -3.1 WB -3.124 $ END DIAPHRAGM PLUS SLAB D.L.
295) 2 FORCE Y LIN WA -3.124 WB -3.308 $ END DIAPHRAGM PLUS SLAB D.L.
296) 3 FORCE Y LIN WA -3.308 WB -3.4 LB 11.92 $ END DIA. PLUS SLAB D.L
     .
297) 3 FORCE Y LIN WA -3.4 WB -3.308 LA 11.92 $ END DIA. PLUS SLAB D.L
     .
298) 4 FORCE Y LIN WA -3.308 WB -3.124 $ END DIAPHRAGM PLUS SLAB D.L.
299) 5 FORCE Y LIN WA -3.124 WB -3.1 $ END DIAPHRAGM PLUS SLAB D.L.
300) 1 TO 5 FORCE Y UNI W -3.6 $ CAP D.L.
301) 6 TO 9 FORCE Y GLOBAL UNI W -2.945 $ COLUMN D.L.
302) 10 TO 33 FORCE Y GLOBAL UNI W -4.977 $ SHAFT D.L.
303) 50 TO 85 FORCE Y GLOBAL UNI W -4.977 $ SHAFT D.L.
304) $
305) LOAD FWS
306) MEMBER LOADS
307) 2 FORCE Y CONC P -15.0 L 7.75 $ GIRDER 2 FWS+UTIL
308) 2 FORCE Y CONC P -28.5 L 17.17 $ GIRDER 3 FWS+UTIL
309) 3 FORCE Y CONC P -28.5 L 2.58 $ GIRDER 4 FWS+UTIL
310) 3 FORCE Y CONC P -28.5 L 12.0 $ GIRDER 5 FWS+UTIL
311) 3 FORCE Y CONC P -28.5 L 21.42 $ GIRDER 6 FWS+UTIL
312) 4 FORCE Y CONC P -28.5 L 6.84 $ GIRDER 7 FWS+UTIL
313) 4 FORCE Y CONC P -15.0 L 16.26 $ GIRDER 8 FWS+UTIL
314) $
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* LISTING OF THE INPUT FILE *
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315) LOAD SDWK_LL
316) MEMBER LOADS
317) 5 FORCE Y CONC P -39.9 L 1.68 $ SIDEWALK L.L. ON ONE SIDE OF BRID
     GE
318) $
319) LOAD LL_I_1 $ MAX. TRANS. MOMENTS IN SHAFTS AND COLUMNS
320) MEMBER LOADS
321) 3 FORCE Y CONC P -55.4 L 12.0 $ GIRDER 5 L.L.
322) 3 FORCE Y CONC P -142.0 L 21.42 $ GIRDER 6 L.L.
323) 4 FORCE Y CONC P -139.3 L 6.84 $ GIRDER 7 L.L.
324) 4 FORCE Y CONC P -112.8 L 16.26 $ GIRDER 8 L.L.
325) 5 FORCE Y CONC P -23.5 L 1.68 $ GIRDER 9 L.L.
326) $
327) LOAD LL_I_2 $ MAX. POS. MOM. IN SPANS 2 & 4 AND MAX. NEG. MOM. SP
     AN 3
328) MEMBER LOADS
329) 1 FORCE Y CONC P -23.6 L 1.17 $ GIRDER 1 L.L.
330) 2 FORCE Y CONC P -112.7 L 7.75 $ GIRDER 2 L.L.
331) 2 FORCE Y CONC P -94.4 L 17.17 $ GIRDER 3 L.L.
332) 3 FORCE Y CONC P -5.3 L 2.58 $ GIRDER 4 L.L.
333) 3 FORCE Y CONC P -5.3 L 21.42 $ GIRDER 6 L.L.
334) 4 FORCE Y CONC P -94.4 L 6.84 $ GIRDER 7 L.L.
335) 4 FORCE Y CONC P -112.7 L 16.26 $ GIRDER 8 L.L.
336) 5 FORCE Y CONC P -23.6 L 1.68 $ GIRDER 9 L.L.
337) $
338) LOAD LL_I_3 $ MAX. NEG. MOM. OVER COLUMNS
339) MEMBER LOADS
340) 2 FORCE Y CONC P -62.6 L 7.75 $ GIRDER 2 L.L.
341) 2 FORCE Y CONC P -55.5 L 17.17 $ GIRDER 3 L.L.
342) 3 FORCE Y CONC P -18.9 L 2.58 $ GIRDER 4 L.L.
343) 3 FORCE Y CONC P -80.3 L 12.0 $ GIRDER 5 L.L.
344) 3 FORCE Y CONC P -18.9 L 21.42 $ GIRDER 6 L.L.
345) $
346) LOAD THER_EXP
347) MEMBER TEMPERATURE
348) 1 TO 10 16 22 28 AXI 30 $ TEMPERATURE RISE OF 30 DEGREES
349) $
350) LOAD WIND
351) JOINT LOAD
352) $ 1 FORCE X 46.38 $ WIND LOAD FROM SUPERSTRUCTURE
353) 1 FORCE X 1.12 $ WIND LOAD FROM SUBSTRUCTURE
354) $
355) $ LOAD WIND_LL
356) $ JOINT LOAD
357) $ 1 FORCE X 11.5 $ WIND LOAD ON MOVING LIVE LOAD
358) $
359) LOAD OT_GRP_2
360) MEMBER LOAD
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* LISTING OF THE INPUT FILE *
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361) 2 FORCE Y CONC P 164.6 L 16.08 $ OVERTURNING FORCE FOR GROUPS II
     AND V
362) $
363) LOAD STREAM
364) MEMBER LOAD
365) 6 FORCE Y UNI W -2.97 $ STREAM FLOW ON UPSTREAM COLUMN
366) 7 8 9 FORCE Y UNI W -0.72 $ STREAM FLOW ON CENTRAL AND DOWNSTREAM
      COLUMNS
367) $
368) LOAD DIF_SET1
369) JOINT DISPLACEMENT
370) 34 DIS Y -0.031 $ 3/8" SETTLEMENT ON THE DOWNSTREAM COLUMN
371) $
372) LOAD DIF_SET2
373) JOINT DISPLACEMENT
374) 33 DIS Y -0.031 $ 3/8" SETTLEMENT ON THE SECOND INTERIOR COLUMN
375) $
376) $ LOAD COMBINATION DED_LOD $ COMBINE GIRDER, F.W.S. AND UTIL. INT
     O DEAD LOAD
377) $ COMBINE DL 1.0 FWS 1.0
378) $
379) $ LOAD COMBINATION LIV_LOD $ COMBINE TRAFFIC AND SIDEWALK L.L.
380) $ COMBINE SDWK_LL 1.0 LL_I_1 0.864
381) $
382) LOAD COMBINATION GROUP_1 $ AASHTO GROUP 1 LOADING
383) COMBINE DL 1.0 FWS 1.0 SDWK_LL 1.0 LL_I_1 0.864 DIF_SET2 1.0 STRE
     AM 1.0
384) $
385) LOAD COMBINATION GROUP_2 $ AASHTO GROUP 2 LOADING REDUCED FOR OVE
     RSTRESS
386) COMBINE DL 0.8 FWS 0.8 WIND 0.8 OT_GRP_2 0.8 DIF_SET2 0.8 STREAM
     0.8
387) $
388) LOAD COMBINATION GROUP_3 $ AASHTO GROUP 3 LOADING REDUCED FOR OVE
     RSTRESS
389) COM DED_LOD 0.8 LIV_LOD 0.8 WIND 0.8 OT_GRP_2 0.24 DIF_SET2 0.8 S
     TREAM 0.8
390) $
391) LOAD COMBINATION GROUP_4 $ AASHTO GROUP 4 LOADING REDUCED FOR OVE
     RSTRESS
392) COMBINE GROUP_1 0.8 THER_EXP 0.8
393) $
394) LOAD COMBINATION GROUP_5 $ AASHTO GROUP 5 LOADING REDUCED FOR OVE
     RSTRESS
395) COMBINE GROUP_2 0.893 THER_EXP 0.714
396) $
397) LOAD COMBINATION GROUP_6 $ AASHTO GROUP 6 LOADING REDUCED FOR OVE
     RSTRESS
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* LISTING OF THE INPUT FILE *
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398) COMBINE GROUP_3 0.893 THER_EXP 0.714
399) $
400) $ SELECT LOAD 7 TO 15
401) $ LIST MEM FOR 1 5 SECT FRA NS 1 .5
402) $ LIST MEM FOR 2 3 4 SECT FRA NS 3 .25 .5 .75
403) SELECT LOAD GROUP_1 GROUP_2 GROUP_3 GROUP_4 GROUP_5 GROUP_6
404) LIST MEM FOR 6 TO 9
405) $ LIST MEM FOR 10 TO 15
406) LIST MEM FOR 11 TO 29 BY 6
407) LIST MEM FOR 13 TO 31 BY 6
408) LIST MEM FOR 15 TO 33 BY 6
409) UNIT KIP INCH RADIAN
410) OUTPUT DECIMAL 5
411) LIST JOI DISP 2 TO 5 7 TO 34
412) FINISH
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* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
6 COLUMN	GROUP_1	7	296.55	20.40	-940.98
		2	-243.01	33.59	821.09
	GROUP_2	7	204.15	16.91	-782.41
		2	-161.32	26.29	697.12
	GROUP_3	7	-290.20	17.49	-813.70
		2	290.20	25.70	739.07
	GROUP_4	7	240.10	15.48	-710.81
		2	-197.27	27.72	599.51
	GROUP_5	7	184.86	14.34	-661.23
		2	-146.61	24.23	571.33
	GROUP_6	7	-256.59	14.87	-689.18
		2	256.59	23.71	608.79
7 COLUMN	GROUP_1	8	2182.81	15.14	-689.59
		3	-2129.27	-2.05	845.78
	GROUP_2	8	1610.42	12.39	-565.50
		3	-1567.58	-1.92	695.64
	GROUP_3	8	989.53	12.65	-577.95
		3	-989.53	-2.17	712.65
	GROUP_4	8	1743.29	11.84	-538.76
		3	-1700.45	-1.36	658.73
	GROUP_5	8	1435.46	10.82	-493.47
		3	-1397.21	-1.47	605.24
	GROUP_6	8	881.01	11.05	-504.58
		3	-881.01	-1.70	620.43
8 COLUMN	GROUP_1	9	-640.24	22.66	-1074.85
		4	693.78	-9.57	1367.75
	GROUP_2	9	-707.30	18.36	-870.76
		4	750.13	-7.89	1109.42
	GROUP_3	9	-1378.07	18.27	-865.04
		4	1378.07	-7.80	1101.97
	GROUP_4	9	-514.83	18.46	-877.18
		4	557.66	-7.99	1117.58
	GROUP_5	9	-633.97	16.70	-793.03
		4	672.22	-7.35	1011.58
	GROUP_6	9	-1232.97	16.61	-787.92
		4	1232.97	-7.26	1004.93

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	/--- AXIAL ---/ FORCE	--- LOCAL Y --/ SHEAR FORCE	-- LOCAL Z --/ MOMENT
9	GROUP_1	10	1618.37	35.07	-1699.15
COLUMN		5	-1564.83	-21.98	2217.70
	GROUP_2	10	1168.19	27.84	-1347.94
		5	-1125.35	-17.37	1758.93
	GROUP_3	10	639.23	27.10	-1310.36
		5	-639.23	-16.63	1707.87
	GROUP_4	10	1297.43	28.84	-1396.29
		5	-1254.60	-18.37	1825.42
	GROUP_5	10	1045.63	25.56	-1236.70
		5	-1007.38	-16.21	1616.47
	GROUP_6	10	573.28	24.90	-1203.15
		5	-573.28	-15.55	1570.87

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* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
11 SHAFT	GROUP_1	15	460.79	20.40	-267.67
		11	-450.83	-20.40	308.48
	GROUP_2	15	335.54	16.91	-224.50
		11	-327.58	-16.91	258.31
	GROUP_3	15	-290.20	17.49	-236.44
		11	290.20	-17.49	271.43
	GROUP_4	15	371.49	15.48	-200.11
		11	-363.53	-15.48	231.06
	GROUP_5	15	302.19	14.34	-187.96
		11	-295.08	-14.34	216.64
	GROUP_6	15	-256.59	14.87	-198.62
		11	256.59	-14.87	228.35
17 SHAFT	GROUP_1	16	2347.05	15.14	-190.09
		12	-2337.10	-15.14	220.37
	GROUP_2	16	1741.81	12.39	-156.48
		12	-1733.84	-12.39	181.27
	GROUP_3	16	989.53	12.65	-160.66
		12	-989.53	-12.65	185.95
	GROUP_4	16	1874.68	11.84	-148.20
		12	-1866.72	-11.84	171.87
	GROUP_5	16	1552.79	10.82	-136.28
		12	-1545.68	-10.82	157.93
	GROUP_6	16	881.01	11.05	-140.02
		12	-881.01	-11.05	162.11
23 SHAFT	GROUP_1	17	-476.00	22.66	-327.20
		13	485.95	-22.66	372.51
	GROUP_2	17	-575.90	18.36	-264.77
		13	583.87	-18.36	301.50
	GROUP_3	17	-1378.07	18.27	-262.20
		13	1378.07	-18.27	298.73
	GROUP_4	17	-383.44	18.46	-268.01
		13	391.40	-18.46	304.93
	GROUP_5	17	-516.64	16.70	-242.02
		13	523.75	-16.70	275.41
	GROUP_6	17	-1232.97	16.61	-239.72
		13	1232.97	-16.61	272.94

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
29	GROUP_1	18	1782.61	35.07	-541.90
SHAFT		14	-1772.66	-35.07	612.04
	GROUP_2	18	1299.58	27.84	-429.13
		14	-1291.61	-27.84	484.82
	GROUP_3	18	639.23	27.10	-416.02
		14	-639.23	-27.10	470.23
	GROUP_4	18	1428.83	28.84	-444.56
		14	-1420.86	-28.84	502.24
	GROUP_5	18	1162.97	25.56	-393.07
		14	-1155.86	-25.56	444.20
	GROUP_6	18	573.28	24.90	-381.36
		14	-573.28	-24.90	431.17

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* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
13	GROUP_1	23	535.44	9.18	-45.43
SHAFT		67	-523.00	-9.18	68.38
	GROUP_2	23	395.26	7.74	-39.29
		67	-385.31	-7.74	58.65
	GROUP_3	23	-290.20	8.22	-43.21
		67	290.20	-8.22	63.77
	GROUP_4	23	431.22	6.82	-32.56
		67	-421.26	-6.82	49.60
	GROUP_5	23	355.53	6.44	-31.71
		67	-346.64	-6.44	47.82
	GROUP_6	23	-256.59	6.87	-35.21
		67	256.59	-6.87	52.39
19	GROUP_1	24	2421.70	9.58	-29.78
SHAFT		68	-2409.25	-9.57	53.72
	GROUP_2	24	1801.52	7.65	-24.86
		68	-1791.57	-7.63	43.96
	GROUP_3	24	989.52	6.74	-25.98
		68	-989.52	-6.74	42.82
	GROUP_4	24	1934.40	7.53	-23.04
		68	-1924.44	-7.51	41.84
	GROUP_5	24	1606.12	6.70	-21.50
		68	-1597.23	-6.69	38.24
	GROUP_6	24	881.00	5.89	-22.50
		68	-881.00	-5.89	37.22
25	GROUP_1	25	-401.33	12.40	-68.60
SHAFT		69	413.77	-12.42	99.63
	GROUP_2	25	-516.17	10.27	-55.42
		69	526.12	-10.28	81.11
	GROUP_3	25	-1378.05	11.26	-54.54
		69	1378.05	-11.26	82.69
	GROUP_4	25	-323.70	10.21	-56.57
		69	333.66	-10.22	82.10
	GROUP_5	25	-463.29	9.42	-51.00
		69	472.18	-9.44	74.57
	GROUP_6	25	-1232.96	10.31	-50.21
		69	1232.96	-10.31	75.99

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
31	GROUP_1	26	1857.26	19.93	-127.35
SHAFT		70	-1844.82	-19.93	177.17
	GROUP_2	26	1359.30	15.77	-100.47
		70	-1349.35	-15.77	139.88
	GROUP_3	26	639.23	15.26	-96.83
		70	-639.23	-15.26	134.98
	GROUP_4	26	1488.55	16.33	-104.10
		70	-1478.60	-16.33	144.93
	GROUP_5	26	1216.30	14.43	-91.70
		70	-1207.41	-14.43	127.77
	GROUP_6	26	573.28	13.98	-88.45
		70	-573.28	-13.98	123.39

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* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
15	GROUP_1	31	600.08	1.25	-0.00
SHAFT		83	-590.13	-1.25	2.50
	GROUP_2	31	446.98	1.04	-0.00
		83	-439.02	-1.04	2.08
	GROUP_3	31	-290.23	1.07	0.00
		83	290.23	-1.07	2.13
	GROUP_4	31	482.91	1.01	-0.00
		83	-474.95	-1.01	2.02
	GROUP_5	31	401.70	0.94	0.00
		83	-394.59	-0.94	1.87
	GROUP_6	31	-256.63	0.96	0.00
		83	256.63	-0.96	1.92
21	GROUP_1	32	2486.40	4.37	0.00
SHAFT		84	-2476.45	-4.35	8.72
	GROUP_2	32	1853.29	3.29	0.00
		84	-1845.33	-3.27	6.56
	GROUP_3	32	989.53	1.88	-0.00
		84	-989.53	-1.88	3.76
	GROUP_4	32	1986.16	3.48	0.00
		84	-1978.20	-3.46	6.94
	GROUP_5	32	1652.34	2.92	0.00
		84	-1645.23	-2.91	5.83
	GROUP_6	32	881.00	1.66	0.00
		84	-881.00	-1.66	3.32
27	GROUP_1	33	-336.64	1.47	-0.00
SHAFT		85	346.59	-1.48	2.95
	GROUP_2	33	-464.42	1.54	0.00
		85	472.38	-1.55	3.09
	GROUP_3	33	-1378.06	3.09	0.00
		85	1378.06	-3.09	6.19
	GROUP_4	33	-271.95	1.15	0.00
		85	279.92	-1.16	2.31
	GROUP_5	33	-417.08	1.35	-0.00
		85	424.19	-1.36	2.71
	GROUP_6	33	-1232.97	2.74	0.00
		85	1232.97	-2.74	5.48

✓ 2/90

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
33	GROUP_1	34	1921.97	2.56	-0.00
SHAFT		86	-1912.01	-2.56	5.11
	GROUP_2	34	1411.06	2.01	-0.00
		86	-1403.10	-2.01	4.02
	GROUP_3	34	639.23	1.92	0.00
		86	-639.23	-1.92	3.84
	GROUP_4	34	1540.31	2.08	0.00
		86	-1532.35	-2.08	4.16
	GROUP_5	34	1262.52	1.82	0.00
		86	-1255.41	-1.82	3.65
	GROUP_6	34	573.28	1.75	0.00
		86	-573.28	-1.75	3.49

2/90

****
* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS INCHES RADIANS

*** LOAD INDEX : 13 LOAD TAG : GROUP_1 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	/--- GLOBAL DISPLACEMENTS ---/-- ROTATION --/ X DIRECTION	Y DIRECTION	Z DIRECTION
2	1.69078	-0.02483	-0.00032
3	1.69001	-0.14536	-0.00074
4	1.68928	-0.33696	-0.00004
5	1.68877	-0.10929	0.00118
7	1.50300	-0.01905	-0.00142
8	1.42723	-0.09921	-0.00164
9	1.51926	-0.35124	-0.00146
10	1.68852	-0.07522	-0.00108
11	0.88158	-0.01098	-0.00185
12	0.74988	-0.05040	-0.00195
13	0.86963	-0.36340	-0.00195
14	1.11696	-0.03861	-0.00187
15	0.83709	-0.01035	-0.00186
16	0.70299	-0.04714	-0.00196
17	0.82257	-0.36407	-0.00197
18	1.07181	-0.03613	-0.00189
19	0.61118	-0.00696	-0.00190
20	0.46603	-0.03062	-0.00199
21	0.58296	-0.36722	-0.00202
22	0.83886	-0.02354	-0.00198
23	0.49699	-0.00514	-0.00191
24	0.34664	-0.02222	-0.00199
25	0.46152	-0.36866	-0.00203
26	0.71957	-0.01711	-0.00200
27	0.38249	-0.00323	-0.00191
28	0.22703	-0.01375	-0.00199
29	0.33961	-0.37001	-0.00203
30	0.59941	-0.01060	-0.00201
31	0.19908	-0.00000	-0.00191
32	0.03545	-0.00000	-0.00200
33	0.14423	-0.37200	-0.00204
34	0.40654	-0.00000	-0.00201

✓ [illegible] 2/90

*** LOAD INDEX : 14 LOAD TAG : GROUP_2 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
2	1.36665	-0.01775	-0.00022
3	1.36602	-0.10761	-0.00058
4	1.36543	-0.25710	-0.00003
5	1.36503	-0.07935	0.00090
7	1.21977	-0.01384	-0.00114
8	1.15573	-0.07359	-0.00132
9	1.22947	-0.27270	-0.00117
10	1.35822	-0.05480	-0.00089
11	0.71830	-0.00809	-0.00150
12	0.60907	-0.03748	-0.00157
13	0.70616	-0.28664	-0.00157
14	0.89357	-0.02824	-0.00151
15	0.68227	-0.00763	-0.00151
16	0.57118	-0.03506	-0.00158
17	0.66823	-0.28745	-0.00159
18	0.85703	-0.02644	-0.00153
19	0.49926	-0.00516	-0.00154
20	0.37967	-0.02279	-0.00161
21	0.47509	-0.29133	-0.00163
22	0.66863	-0.01724	-0.00160
23	0.40671	-0.00381	-0.00155
24	0.28318	-0.01655	-0.00161
25	0.37719	-0.29316	-0.00164
26	0.57221	-0.01254	-0.00161
27	0.31389	-0.00240	-0.00155
28	0.18649	-0.01024	-0.00161
29	0.27890	-0.29492	-0.00164
30	0.47511	-0.00778	-0.00162
31	0.16519	-0.00000	-0.00155
32	0.03164	-0.00000	-0.00161
33	0.12139	-0.29760	-0.00164
34	0.31925	-0.00000	-0.00162

*** LOAD INDEX : 15 LOAD TAG : GROUP_3 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT /--- GLOBAL DISPLACEMENTS ---/-- ROTATION --/
NO. X DIRECTION Y DIRECTION Z DIRECTION

✓ 2/90

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	/--- GLOBAL DISPLACEMENTS ---/ X DIRECTION	Y DIRECTION	-- ROTATION --/ Z DIRECTION
2	1.36696	0.01854	-0.00017
3	1.36635	-0.06323	-0.00056
4	1.36578	-0.20954	-0.00003
5	1.36540	-0.04085	0.00083
7	1.22651	0.01233	-0.00113
8	1.15920	-0.04205	-0.00131
9	1.22995	-0.23904	-0.00117
10	1.34873	-0.02716	-0.00091
11	0.72540	0.00606	-0.00150
12	0.61368	-0.02068	-0.00157
13	0.70880	-0.26880	-0.00157
14	0.87999	-0.01336	-0.00152
15	0.68924	0.00566	-0.00151
16	0.57581	-0.01930	-0.00158
17	0.67104	-0.27072	-0.00158
18	0.84338	-0.01247	-0.00154
19	0.50536	0.00364	-0.00155
20	0.38433	-0.01241	-0.00161
21	0.47876	-0.28032	-0.00162
22	0.65479	-0.00802	-0.00160
23	0.41230	0.00263	-0.00155
24	0.28784	-0.00896	-0.00161
25	0.38129	-0.28512	-0.00163
26	0.55835	-0.00579	-0.00161
27	0.31895	0.00162	-0.00156
28	0.19117	-0.00551	-0.00161
29	0.28343	-0.28992	-0.00163
30	0.46125	-0.00356	-0.00162
31	0.16938	0.00000	-0.00156
32	0.03635	-0.00000	-0.00161
33	0.12658	-0.29760	-0.00163
34	0.30541	-0.00000	-0.00162

*** LOAD INDEX : 16 LOAD TAG : GROUP_4 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	/--- GLOBAL DISPLACEMENTS ---/ X DIRECTION	Y DIRECTION	-- ROTATION --/ Z DIRECTION
2	1.29015	0.06494	-0.00025
3	1.33099	-0.03112	-0.00059
4	1.37185	-0.18442	-0.00004
5	1.41290	-0.00263	0.00094

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	/--- GLOBAL DISPLACEMENTS ---/-- ROTATION --/ X DIRECTION	Y DIRECTION	Z DIRECTION
7	1.14726	0.03820	-0.00108
8	1.12288	-0.02567	-0.00129
9	1.23303	-0.22731	-0.00119
10	1.40618	-0.00673	-0.00092
11	0.67683	-0.00885	-0.00140
12	0.58900	-0.04026	-0.00154
13	0.70269	-0.29067	-0.00159
14	0.92532	-0.03094	-0.00157
15	0.64317	-0.00833	-0.00141
16	0.55206	-0.03765	-0.00154
17	0.66427	-0.29121	-0.00161
18	0.88749	-0.02896	-0.00159
19	0.47228	-0.00561	-0.00144
20	0.36542	-0.02446	-0.00156
21	0.46862	-0.29374	-0.00165
22	0.69249	-0.01886	-0.00165
23	0.38593	-0.00414	-0.00144
24	0.27140	-0.01775	-0.00157
25	0.36946	-0.29490	-0.00166
26	0.59268	-0.01371	-0.00167
27	0.29935	-0.00260	-0.00144
28	0.17720	-0.01098	-0.00157
29	0.26990	-0.29600	-0.00166
30	0.49217	-0.00850	-0.00168
31	0.16068	-0.00000	-0.00144
32	0.02633	-0.00000	-0.00157
33	0.11036	-0.29760	-0.00166
34	0.33083	-0.00000	-0.00168

*** LOAD INDEX : 17 LOAD TAG : GROUP_5 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	/--- GLOBAL DISPLACEMENTS ---/-- ROTATION --/ X DIRECTION	Y DIRECTION	Z DIRECTION
2	1.16466	0.05984	-0.00019
3	1.20109	-0.02008	-0.00052
4	1.23756	-0.15360	-0.00002
5	1.27420	0.00483	0.00080
7	1.04003	0.03535	-0.00096
8	1.01520	-0.01780	-0.00116
9	1.11364	-0.19561	-0.00107
10	1.26231	-0.00123	-0.00084

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RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
11	0.61606	-0.00728	-0.00126
12	0.53416	-0.03342	-0.00139
13	0.63684	-0.25592	-0.00143
14	0.82629	-0.02527	-0.00141
15	0.58562	-0.00687	-0.00127
16	0.50084	-0.03126	-0.00139
17	0.60228	-0.25665	-0.00145
18	0.79214	-0.02366	-0.00143
19	0.43097	-0.00464	-0.00130
20	0.33244	-0.02032	-0.00141
21	0.42627	-0.26013	-0.00148
22	0.61619	-0.01543	-0.00149
23	0.35279	-0.00343	-0.00131
24	0.24760	-0.01475	-0.00142
25	0.33704	-0.26177	-0.00149
26	0.52618	-0.01122	-0.00151
27	0.27438	-0.00216	-0.00131
28	0.16259	-0.00913	-0.00142
29	0.24747	-0.26335	-0.00149
30	0.43555	-0.00696	-0.00151
31	0.14878	-0.00000	-0.00131
32	0.02644	-0.00000	-0.00142
33	0.10391	-0.26576	-0.00150
34	0.29009	-0.00000	-0.00152

*** LOAD INDEX : 18 LOAD TAG : GROUP_6 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
2	1.16494	0.09224	-0.00015
3	1.20139	0.01955	-0.00050
4	1.23787	-0.11112	-0.00003
5	1.27453	0.03921	0.00074
7	1.04605	0.05871	-0.00096
8	1.01830	0.01037	-0.00116
9	1.11407	-0.16555	-0.00107
10	1.25383	0.02345	-0.00086
11	0.62240	0.00536	-0.00127
12	0.53828	-0.01841	-0.00138
13	0.63920	-0.23999	-0.00143
14	0.81417	-0.01198	-0.00142

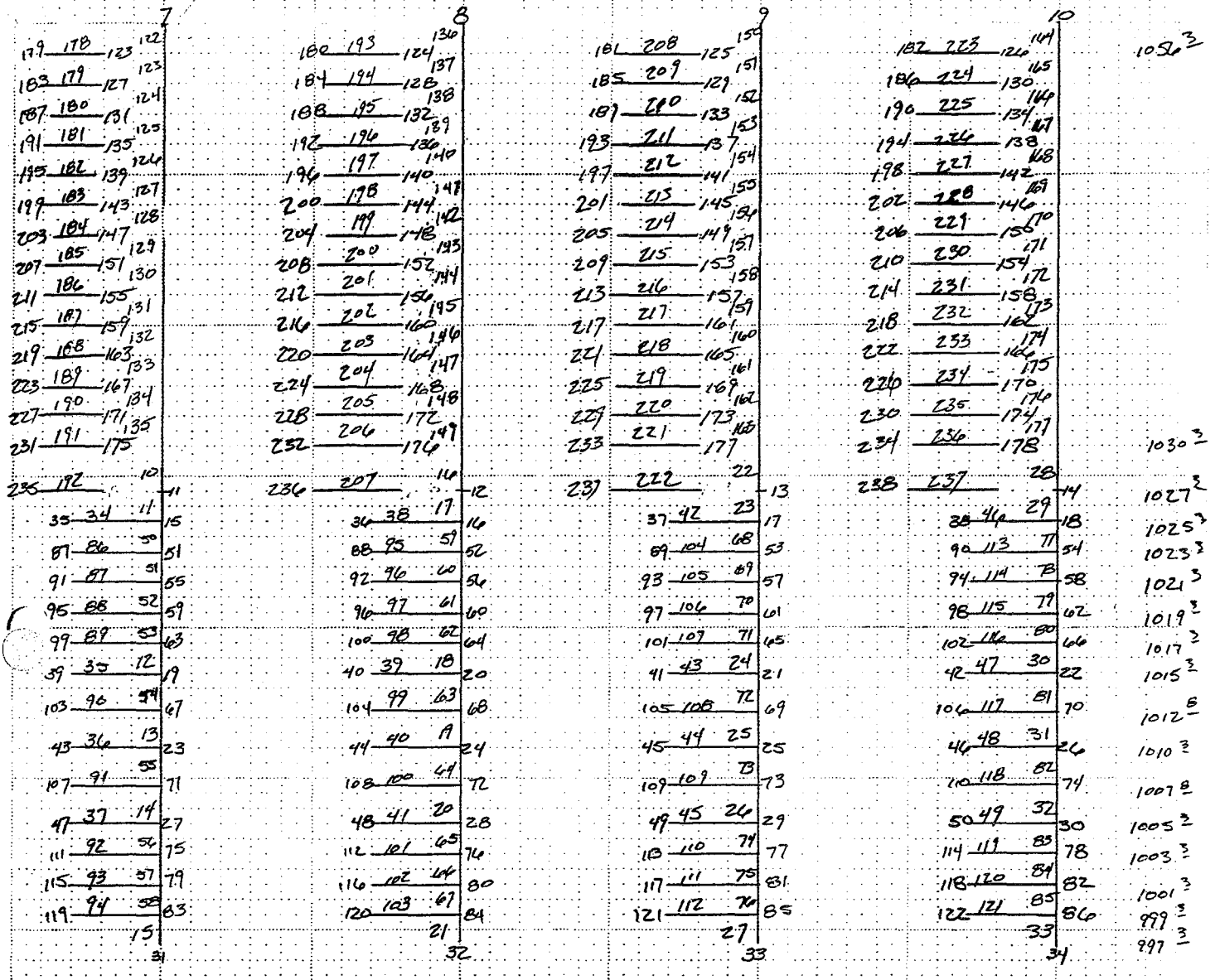
✓ 2/90

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	/--- GLOBAL DISPLACEMENTS ---/-- ROTATION --/ X DIRECTION	Y DIRECTION	Z DIRECTION
15	0.59183	0.00501	-0.00128
16	0.50497	-0.01719	-0.00139
17	0.60479	-0.24171	-0.00144
18	0.77995	-0.01118	-0.00143
19	0.43641	0.00322	-0.00131
20	0.33660	-0.01105	-0.00141
21	0.42955	-0.25030	-0.00148
22	0.60383	-0.00719	-0.00149
23	0.35778	0.00232	-0.00131
24	0.25177	-0.00798	-0.00142
25	0.34071	-0.25459	-0.00148
26	0.51380	-0.00519	-0.00151
27	0.27890	0.00143	-0.00132
28	0.16677	-0.00491	-0.00142
29	0.25151	-0.25889	-0.00149
30	0.42317	-0.00319	-0.00151
31	0.15253	0.00000	-0.00132
32	0.03065	-0.00000	-0.00142
33	0.10854	-0.26576	-0.00149
34	0.27773	-0.00000	-0.00152

Spring Connector & Shaft M-Strudl Joint & Members

Analysis of Bent w/o Scour



JOB Northern Ave @ New River SHEET NO. P 33

MADE BY T. Stones DATE 1-12-90 JOB NO. 33-89602

CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 141

CALCULATIONS FOR First Bent Frame Analysis

BRW

MINNEAPOLIS PHOENIX

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
120.00	78.00	36.0	.33D-01

Y IN	P LBS/IN
.000	.000
.108	331.500
.217	663.000
.325	994.500
.433	1326.000
.542	1657.500
.650	1989.000
.758	2320.500
.867	2652.000
.975	2983.500
1.083	3091.864
1.192	3184.544
1.300	3271.578
2.925	4538.838

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
288.00	78.00	34.0	.28D-01

Y IN	P LBS/IN
.000	.000
.108	795.600
.217	1591.200
.325	2386.800
.433	2852.557
.542	3187.293
.650	3489.748
.758	3767.756
.867	4026.416
.975	4269.272
1.083	4498.899
1.192	4717.245
1.300	4925.818
2.925	7987.464

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
366.00	78.00	38.0	.28D-01

Y IN	P LBS/IN
.000	.000
.108	1011.075
.217	2022.150
.325	3033.225
.433	4044.300
.542	5055.375
.650	6063.837
.758	6643.389
.867	7190.008
.975	7709.373
1.083	8205.672
1.192	8682.096
1.300	9141.152
2.925	15907.261

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
456.00	78.00	36.0	.30D-01

Y IN	P LBS/IN
.000	.000
.108	2173.600
.217	4023.624
.325	5148.501
.433	6132.585
.542	7023.683
.650	7847.062
.758	8618.084
.867	9346.948
.975	10040.852
1.083	10705.109
1.192	11343.782
1.300	11960.059
2.925	21049.703

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave. @ New River SHEET NO. P 34
MADE BY T. Stones DATE 1-15-90 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 142
CALCULATIONS FOR First Bent Frame Analysis

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
546.00	78.00	34.0	.31D-01

Y IN	P LBS/IN
.000	.000
.108	2602.600
.217	4912.154
.325	6285.436
.433	7486.834
.542	8574.711
.650	9579.915
.758	10521.201
.867	11411.019
.975	12258.156
1.083	13069.101
1.192	13848.810
1.300	14601.178
2.925	25698.074

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
606.00	78.00	34.5	.31D-01

Y IN	P LBS/IN
.000	.000
.108	2888.600
.217	5777.200
.325	7959.886
.433	9481.337
.542	10859.026
.650	12132.019
.758	13324.065
.867	14450.931
.975	15523.748
1.083	16550.729
1.192	17538.154
1.300	18490.954
2.925	32544.079

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
684.00	78.00	33.0	.31D-01

Y IN	P LBS/IN
.000	.000
.108	3260.400
.217	6520.800
.325	8849.619
.433	10541.133
.542	12072.816
.650	13488.100
.758	14813.389
.867	16066.214
.975	17258.946
1.083	18400.721
1.192	19498.517
1.300	20557.819
2.925	36181.760

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3
732.00	78.00	33.0	.31D-01

Y IN	P LBS/IN
.000	.000
.108	3489.200
.217	6978.400
.325	10076.310
.433	12002.295
.542	13746.292
.650	15357.756
.758	16866.750
.867	18293.235
.975	19651.298
1.083	20951.340
1.192	22201.308
1.300	23407.444
2.925	41197.102

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave c. New River SHEET NO. P-35
MADE BY T Stones DATE 1-15-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 143
CALCULATIONS FOR Bent Frame Analysis

w/o scour Δ=12"							w/ scour	
	0		INVERT					
	7.42							
	14.8							
	22.5							
	29.7							
37.103	37.1							
	39.5							
	41.9							
	44.3							
	46.8							
	49.2							
	51.6							
53.908	54.0							
	66.7							
	79.4							
98.472	98.5		14	SCOUR		0.40		3.144
	106.8	38 46 29	18	1.13			7.252	
	115.2	90 113	54	2.60			15.448	
	123.5	94 114 98	58	4.07		4.8	23.684	27.792
128.46	131.8	98 116 99	62	5.49			30.759	
	140.1	102 112 104	66	6.86			36.694	
	147.5	42 47 30	22	8.23			42.629	
156.828	150.8	106 117 81	70	9.95		9.95	48.564	48.564
	177.7	46 118 31	26	11.67			53.658	
198.6	198.6	110 118 84	74	13.38		13.38	58.752	58.752
	206.0	50 49 42	30	14.76			59.628	
	213.4	114 119 84	78	15.87			60.504	
220.812	220.8	118 110 80	82	16.97		16.97	61.38	61.38
	236.1	122 111 80	86	18.23			68.43	
251.472	251.4	33	38	BASE		19.48	75.48	75.48

Spring K's Trial 2

Trial 3 Use displ. of 12"

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. P34
MADE BY T Stones DATE 1-12-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 144
CALCULATIONS FOR First Bent Frame Analysis

2/90

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* RESULTS OF ANALYSIS *
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TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
6 COLUMN	GROUP_1	7	337.87	19.50	-326.62
		2	-284.33	34.50	190.25
	GROUP_2	7	237.27	16.37	-287.34
		2	-194.44	26.83	192.24
	GROUP_3	7	256.57	15.97	-270.95
		2	-213.74	27.23	168.57
	GROUP_4	7	274.48	13.25	-221.99
		2	-231.65	29.95	70.20
	GROUP_5	7	215.61	12.52	-221.51
		2	-177.36	26.05	98.49
	GROUP_6	7	232.85	12.16	-206.88
		2	-194.60	26.41	77.35
7 COLUMN	GROUP_1	8	2176.89	9.69	25.28
		3	-2123.35	3.40	31.96
	GROUP_2	8	1605.81	8.07	11.26
		3	-1562.97	2.40	40.32
	GROUP_3	8	1717.06	7.89	19.48
		3	-1674.23	2.59	28.69
	GROUP_4	8	1737.36	6.97	34.76
		3	-1694.53	3.50	-3.18
	GROUP_5	8	1430.28	6.51	23.03
		3	-1392.03	2.84	10.34
	GROUP_6	8	1529.63	6.34	30.36
		3	-1491.38	3.01	-0.04
8 COLUMN	GROUP_1	9	-648.20	21.45	-420.57
		4	701.74	-8.36	691.61
	GROUP_2	9	-713.23	17.46	-344.00
		4	756.06	-6.99	566.21
	GROUP_3	9	-521.90	17.34	-338.83
		4	564.73	-6.87	558.90
	GROUP_4	9	-522.81	17.98	-348.60
		4	565.64	-7.51	580.37
	GROUP_5	9	-640.71	16.32	-318.04
		4	678.95	-6.97	529.79
	GROUP_6	9	-469.85	16.22	-313.41
		4	508.10	-6.87	523.27

✓ 2/90

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
9	GROUP_1	10	1590.93	42.62	-1182.49
COLUMN		5	-1537.39	-29.53	1838.34
	GROUP_2	10	1145.61	33.61	-925.39
		5	-1102.77	-23.14	1441.21
	GROUP_3	10	1274.75	34.31	-949.92
		5	-1231.92	-23.84	1478.55
	GROUP_4	10	1276.96	36.41	-987.90
		5	-1234.13	-25.93	1554.56
	GROUP_5	10	1026.79	32.07	-863.78
		5	-988.54	-22.72	1361.87
	GROUP_6	10	1142.12	32.70	-885.68
		5	-1103.87	-23.35	1395.22

****
* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
11 SHAFT	GROUP_1	15	502.11	3.64	82.42
		11	-492.16	-3.64	-75.15
	GROUP_2	15	368.66	3.34	60.64
		11	-360.70	-3.34	-53.95
	GROUP_3	15	387.96	3.05	65.20
		11	-380.00	-3.05	-59.10
	GROUP_4	15	405.87	2.43	55.21
		11	-397.91	-2.43	-50.34
	GROUP_5	15	332.95	2.56	44.58
		11	-325.84	-2.56	-39.45
	GROUP_6	15	350.18	2.30	48.65
		11	-343.07	-2.30	-44.05
17 SHAFT	GROUP_1	16	2341.13	-2.64	156.98
		12	-2331.18	2.64	-162.27
	GROUP_2	16	1737.20	-1.97	124.59
		12	-1729.23	1.97	-128.54
	GROUP_3	16	1848.45	-2.12	127.02
		12	-1840.49	2.12	-131.27
	GROUP_4	16	1868.75	-2.29	123.32
		12	-1860.79	2.29	-127.89
	GROUP_5	16	1547.61	-1.92	109.24
		12	-1540.50	1.92	-113.07
	GROUP_6	16	1646.96	-2.05	111.41
		12	-1639.85	2.05	-115.51
23 SHAFT	GROUP_1	17	-483.96	5.12	45.12
		13	493.91	-5.12	-34.88
	GROUP_2	17	-581.84	4.21	35.79
		13	589.80	-4.21	-27.37
	GROUP_3	17	-390.50	4.11	37.23
		13	398.47	-4.11	-29.00
	GROUP_4	17	-391.42	4.24	41.31
		13	399.38	-4.24	-32.83
	GROUP_5	17	-523.37	3.89	36.62
		13	530.48	-3.89	-28.84
	GROUP_6	17	-352.51	3.80	37.90
		13	359.62	-3.80	-30.30

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	/--- AXIAL FORCE ---/	--- LOCAL Y SHEAR FORCE --/	-- LOCAL Z MOMENT --/
29 SHAFT	GROUP_1	18	1755.17	18.49	-121.46
		14	-1745.21	-18.49	158.43
	GROUP_2	18	1277.00	14.41	-91.34
		14	-1269.04	-14.41	120.16
	GROUP_3	18	1406.14	14.83	-96.46
		14	-1398.18	-14.83	126.13
	GROUP_4	18	1408.35	15.27	-89.85
		14	-1400.39	-15.27	120.40
	GROUP_5	18	1144.13	13.30	-75.04
		14	-1137.01	-13.30	101.64
	GROUP_6	18	1259.45	13.68	-79.61
		14	-1252.34	-13.68	106.97

✓ 2/90

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* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
13 SHAFT	GROUP_1	23	576.77	-4.69	55.81
		67	-564.33	4.69	-67.54
	GROUP_2	23	428.39	-3.57	43.56
		67	-418.43	3.57	-52.48
	GROUP_3	23	447.69	-3.74	44.80
		67	-437.73	3.74	-54.15
	GROUP_4	23	465.59	-3.14	37.17
		67	-455.64	3.14	-45.02
	GROUP_5	23	386.28	-2.64	32.22
		67	-377.39	2.64	-38.82
	GROUP_6	23	403.52	-2.80	33.33
		67	-394.63	2.80	-40.32
19 SHAFT	GROUP_1	24	2415.79	-4.02	68.98
		68	-2403.35	4.03	-79.04
	GROUP_2	24	1796.93	-3.37	55.20
		68	-1786.97	3.38	-63.65
	GROUP_3	24	1908.18	-3.32	55.86
		68	-1898.23	3.33	-64.17
	GROUP_4	24	1928.48	-3.08	53.41
		68	-1918.53	3.09	-61.12
	GROUP_5	24	1600.95	-2.89	47.71
		68	-1592.06	2.90	-54.95
	GROUP_6	24	1700.30	-2.84	48.30
		68	-1691.41	2.85	-55.41
25 SHAFT	GROUP_1	25	-409.31	-2.33	42.40
		69	421.75	2.31	-48.20
	GROUP_2	25	-522.11	-1.62	34.18
		69	532.07	1.61	-38.21
	GROUP_3	25	-330.78	-1.92	34.55
		69	340.74	1.91	-39.33
	GROUP_4	25	-331.70	-2.20	36.85
		69	341.65	2.18	-42.33
	GROUP_5	25	-470.04	-1.74	33.13
		69	478.93	1.73	-37.48
	GROUP_6	25	-299.18	-2.01	33.47
		69	308.07	2.00	-38.48

✓ [illegible] 2/90

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	/--- AXIAL FORCE ---/	--- LOCAL Y --/ SHEAR FORCE	-- LOCAL Z --/ MOMENT
31 SHAFT	GROUP_1	26	1829.82	2.66	7.53
		70	-1817.38	-2.66	-0.88
	GROUP_2	26	1336.72	1.91	7.56
		70	-1326.77	-1.91	-2.78
	GROUP_3	26	1465.87	2.09	6.55
		70	-1455.91	-2.09	-1.33
	GROUP_4	26	1468.08	1.68	11.50
		70	-1458.12	-1.68	-7.29
	GROUP_5	26	1197.46	1.31	11.63
		70	-1188.57	-1.31	-8.36
	GROUP_6	26	1312.79	1.46	10.73
		70	-1303.90	-1.46	-7.07

✓ Lee Ehrnull 2/90

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* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
15 SHAFT	GROUP_1	31	641.37	-1.53	0.00
		83	-631.42	1.53	-3.06
	GROUP_2	31	480.07	-1.23	-0.00
		83	-472.11	1.23	-2.45
	GROUP_3	31	499.37	-1.24	0.00
		83	-491.41	1.24	-2.48
	GROUP_4	31	517.30	-1.10	0.00
		83	-509.34	1.10	-2.21
	GROUP_5	31	432.46	-0.99	0.00
		83	-425.35	0.99	-1.97
	GROUP_6	31	449.69	-1.00	0.00
		83	-442.58	1.00	-2.00
21 SHAFT	GROUP_1	32	2480.49	2.03	-0.00
		84	-2470.53	-2.01	4.04
	GROUP_2	32	1848.68	1.39	-0.00
		84	-1840.72	-1.38	2.77
	GROUP_3	32	1959.94	1.56	0.00
		84	-1951.97	-1.55	3.11
	GROUP_4	32	1980.24	1.68	0.00
		84	-1972.27	-1.67	3.35
	GROUP_5	32	1647.17	1.29	0.00
		84	-1640.06	-1.28	2.57
	GROUP_6	32	1746.52	1.44	-0.00
		84	-1739.41	-1.43	2.88
27 SHAFT	GROUP_1	33	-344.61	-1.41	0.00
		85	354.56	1.40	-2.81
	GROUP_2	33	-470.35	-0.80	0.00
		85	478.32	0.79	-1.59
	GROUP_3	33	-279.02	-1.14	-0.00
		85	286.98	1.13	-2.27
	GROUP_4	33	-279.93	-1.11	-0.00
		85	287.90	1.10	-2.21
	GROUP_5	33	-423.82	-0.70	-0.00
		85	430.93	0.69	-1.39
	GROUP_6	33	-252.96	-1.01	-0.00
		85	260.07	0.99	-2.00

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	/--- AXIAL ---/ FORCE	--- LOCAL Y --/ SHEAR FORCE	-- LOCAL Z --/ MOMENT
33 SHAFT	GROUP_1	34	1894.52	-0.98	-0.00
		86	-1884.57	0.98	-1.97
	GROUP_2	34	1388.48	-0.82	-0.00
		86	-1380.52	0.82	-1.64
	GROUP_3	34	1517.63	-0.80	-0.00
		86	-1509.67	0.80	-1.61
	GROUP_4	34	1519.84	-0.98	-0.00
		86	-1511.88	0.98	-1.96
	GROUP_5	34	1243.68	-0.91	0.00
		86	-1236.57	0.91	-1.81
	GROUP_6	34	1359.01	-0.89	-0.00
		86	-1351.90	0.89	-1.78

✓ Lee Ellwood 2/90

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* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS INCHES RADIANS

*** LOAD INDEX : 15 LOAD TAG : GROUP_1 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
2	0.55773	-0.02747	-0.00029
3	0.55693	-0.14498	-0.00073
4	0.55606	-0.33646	-0.00003
5	0.55537	-0.10754	0.00119
7	0.45483	-0.02081	-0.00068
8	0.39414	-0.09895	-0.00075
9	0.47120	-0.35091	-0.00069
10	0.61152	-0.07406	-0.00055
11	0.18319	-0.01185	-0.00074
12	0.12688	-0.05028	-0.00067
13	0.18525	-0.36324	-0.00079
14	0.30684	-0.03803	-0.00097
15	0.16558	-0.01115	-0.00073
16	0.11095	-0.04702	-0.00066
17	0.16623	-0.36392	-0.00079
18	0.28355	-0.03559	-0.00097
19	0.07887	-0.00748	-0.00071
20	0.03354	-0.03054	-0.00063
21	0.07194	-0.36712	-0.00078
22	0.16579	-0.02319	-0.00099
23	0.03631	-0.00552	-0.00071
24	-0.00409	-0.02217	-0.00062
25	0.02532	-0.36859	-0.00077
26	0.10664	-0.01686	-0.00099
27	-0.00589	-0.00346	-0.00070
28	-0.04129	-0.01371	-0.00062
29	-0.02102	-0.36997	-0.00077
30	0.04754	-0.01045	-0.00098
31	-0.07308	-0.00000	-0.00070
32	-0.10048	-0.00000	-0.00062
33	-0.09486	-0.37200	-0.00077
34	-0.04693	-0.00000	-0.00098

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*** LOAD INDEX : 16 LOAD TAG : GROUP_2 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
2	0.45015	-0.01986	-0.00020
3	0.44951	-0.10731	-0.00057
4	0.44882	-0.25673	-0.00002
5	0.44828	-0.07791	0.00092
7	0.37134	-0.01524	-0.00054
8	0.31979	-0.07340	-0.00061
9	0.38130	-0.27245	-0.00056
10	0.48835	-0.05384	-0.00045
11	0.15112	-0.00879	-0.00060
12	0.10377	-0.03739	-0.00054
13	0.15057	-0.28652	-0.00064
14	0.24333	-0.02777	-0.00077
15	0.13675	-0.00828	-0.00060
16	0.09086	-0.03497	-0.00054
17	0.13520	-0.28734	-0.00064
18	0.22471	-0.02600	-0.00078
19	0.06587	-0.00557	-0.00058
20	0.02807	-0.02273	-0.00051
21	0.05903	-0.29125	-0.00063
22	0.13063	-0.01696	-0.00079
23	0.03104	-0.00411	-0.00058
24	-0.00246	-0.01651	-0.00051
25	0.02136	-0.29311	-0.00063
26	0.08341	-0.01234	-0.00079
27	-0.00350	-0.00259	-0.00057
28	-0.03265	-0.01021	-0.00050
29	-0.01608	-0.29489	-0.00062
30	0.03623	-0.00765	-0.00079
31	-0.05851	-0.00000	-0.00057
32	-0.08068	-0.00000	-0.00050
33	-0.07576	-0.29760	-0.00062
34	-0.03916	-0.00000	-0.00079

*** LOAD INDEX : 17 LOAD TAG : GROUP_3 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION

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RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
2	0.45210	-0.02110	-0.00022
3	0.45145	-0.11442	-0.00059
4	0.45074	-0.26895	-0.00003
5	0.45019	-0.08616	0.00095
7	0.36984	-0.01606	-0.00055
8	0.31970	-0.07812	-0.00061
9	0.38166	-0.28058	-0.00056
10	0.49416	-0.05933	-0.00045
11	0.14931	-0.00919	-0.00060
12	0.10300	-0.03971	-0.00054
13	0.15000	-0.29052	-0.00064
14	0.24745	-0.03047	-0.00078
15	0.13498	-0.00865	-0.00060
16	0.09009	-0.03714	-0.00054
17	0.13461	-0.29107	-0.00064
18	0.22862	-0.02851	-0.00079
19	0.06443	-0.00581	-0.00058
20	0.02730	-0.02413	-0.00051
21	0.05828	-0.29365	-0.00063
22	0.13344	-0.01858	-0.00080
23	0.02979	-0.00429	-0.00057
24	-0.00322	-0.01751	-0.00051
25	0.02055	-0.29484	-0.00063
26	0.08565	-0.01351	-0.00080
27	-0.00456	-0.00269	-0.00057
28	-0.03339	-0.01083	-0.00050
29	-0.01696	-0.29596	-0.00062
30	0.03790	-0.00837	-0.00080
31	-0.05925	-0.00000	-0.00057
32	-0.08140	-0.00000	-0.00050
33	-0.07673	-0.29760	-0.00062
34	-0.03842	-0.00000	-0.00079

*** LOAD INDEX : 18 LOAD TAG : GROUP_4 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
2	0.38418	0.00917	-0.00023
3	0.42496	-0.08430	-0.00058
4	0.46566	-0.23748	-0.00003
5	0.50653	-0.05489	0.00095

✓ 2/90

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
7	0.31147	-0.01682	-0.00047
8	0.29788	-0.07899	-0.00058
9	0.39487	-0.28054	-0.00058
10	0.54154	-0.05942	-0.00052
11	0.12419	-0.00956	-0.00051
12	0.09449	-0.04014	-0.00051
13	0.15692	-0.29050	-0.00066
14	0.26638	-0.03051	-0.00086
15	0.11204	-0.00900	-0.00050
16	0.08243	-0.03754	-0.00050
17	0.14113	-0.29105	-0.00066
18	0.24569	-0.02856	-0.00086
19	0.05222	-0.00604	-0.00049
20	0.02390	-0.02438	-0.00048
21	0.06287	-0.29364	-0.00065
22	0.14129	-0.01861	-0.00087
23	0.02284	-0.00445	-0.00049
24	-0.00453	-0.01770	-0.00047
25	0.02421	-0.29483	-0.00064
26	0.08895	-0.01353	-0.00087
27	-0.00630	-0.00279	-0.00048
28	-0.03263	-0.01095	-0.00047
29	-0.01421	-0.29595	-0.00064
30	0.03669	-0.00838	-0.00087
31	-0.05270	-0.00000	-0.00048
32	-0.07734	-0.00000	-0.00047
33	-0.07543	-0.29760	-0.00064
34	-0.04682	-0.00000	-0.00087

*** LOAD INDEX : 19 LOAD TAG : GROUP_5 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
2	0.34665	0.01006	-0.00017
3	0.38304	-0.06756	-0.00051
4	0.41937	-0.20098	-0.00002
5	0.45586	-0.04177	0.00081
7	0.28484	-0.01377	-0.00042
8	0.27001	-0.06539	-0.00052
9	0.35648	-0.24314	-0.00052
10	0.48280	-0.04824	-0.00047

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RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
11	0.11499	-0.00792	-0.00046
12	0.08640	-0.03331	-0.00046
13	0.14224	-0.25579	-0.00059
14	0.23596	-0.02488	-0.00077
15	0.10389	-0.00747	-0.00046
16	0.07548	-0.03116	-0.00045
17	0.12800	-0.25652	-0.00059
18	0.21749	-0.02329	-0.00077
19	0.04912	-0.00502	-0.00045
20	0.02244	-0.02025	-0.00043
21	0.05746	-0.26004	-0.00058
22	0.12438	-0.01519	-0.00078
23	0.02218	-0.00371	-0.00045
24	-0.00333	-0.01471	-0.00043
25	0.02261	-0.26171	-0.00058
26	0.07773	-0.01105	-0.00078
27	-0.00454	-0.00233	-0.00044
28	-0.02881	-0.00910	-0.00042
29	-0.01203	-0.26332	-0.00058
30	0.03116	-0.00685	-0.00078
31	-0.04711	-0.00000	-0.00044
32	-0.06934	-0.00000	-0.00042
33	-0.06724	-0.26576	-0.00057
34	-0.04325	-0.00000	-0.00077

*** LOAD INDEX : 20 LOAD TAG : GROUP_6 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
2	0.34839	0.00896	-0.00020
3	0.38477	-0.07390	-0.00052
4	0.42109	-0.21189	-0.00003
5	0.45756	-0.04914	0.00085
7	0.28350	-0.01450	-0.00042
8	0.26993	-0.06961	-0.00052
9	0.35680	-0.25040	-0.00052
10	0.48799	-0.05314	-0.00047
11	0.11337	-0.00828	-0.00046
12	0.08572	-0.03538	-0.00046
13	0.14174	-0.25936	-0.00060
14	0.23963	-0.02729	-0.00077

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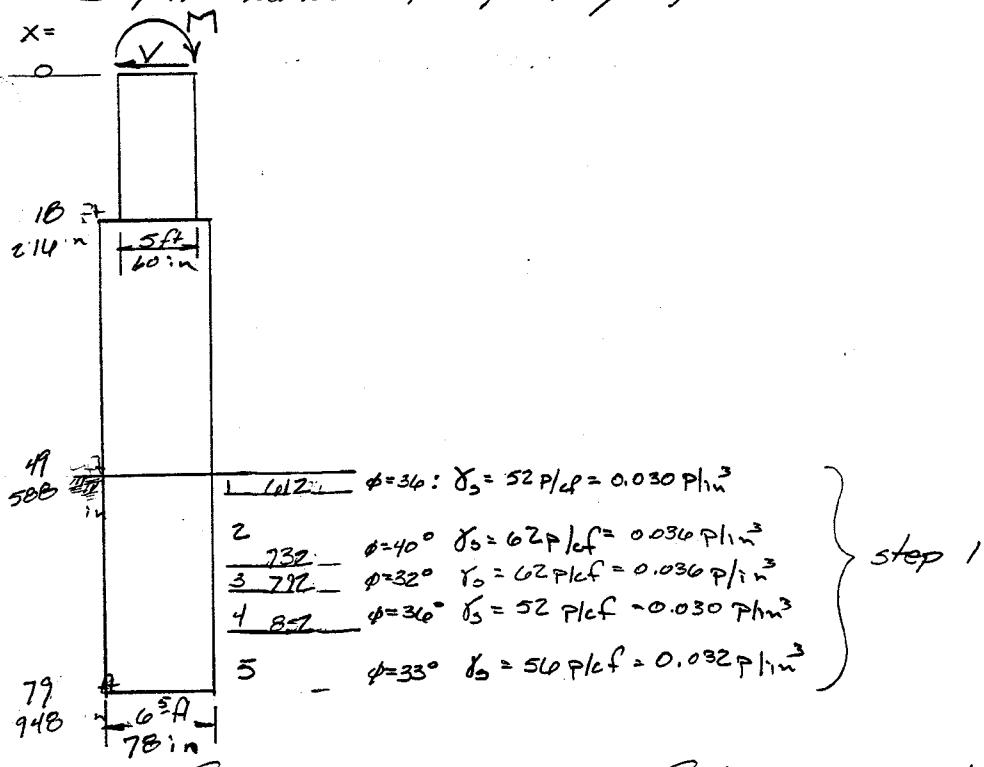
RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
15	0.10231	-0.00780	-0.00046
16	0.07480	-0.03309	-0.00045
17	0.12747	-0.25985	-0.00059
18	0.22098	-0.02554	-0.00078
19	0.04783	-0.00524	-0.00045
20	0.02176	-0.02150	-0.00043
21	0.05679	-0.26218	-0.00058
22	0.12689	-0.01664	-0.00079
23	0.02106	-0.00386	-0.00044
24	-0.00401	-0.01561	-0.00043
25	0.02188	-0.26326	-0.00058
26	0.07974	-0.01210	-0.00079
27	-0.00549	-0.00243	-0.00044
28	-0.02947	-0.00965	-0.00042
29	-0.01282	-0.26427	-0.00058
30	0.03264	-0.00749	-0.00078
31	-0.04777	-0.00000	-0.00044
32	-0.06998	-0.00000	-0.00042
33	-0.06811	-0.26576	-0.00058
34	-0.04259	-0.00000	-0.00078

Input values of p & y for various depths



The following calculations follow procedure outline in COM624G users manual pages 78 thru 85 for p-y curve at x = 402 in 1st soil level

2. $\alpha = \frac{\phi}{2} = 18°$ $\beta = 45 + \phi/2 = 63°$ $K_o = 0.4$ $K_a = \tan^2(45 - \phi/2) = 0.260$

note: x = 12 in. in following eqn.

3. Ultimate soil resistance per unit length — x is depth below top of soil

$$P_{st} = \gamma x \left[\frac{K_o x \tan\phi \sin\beta}{\tan(\beta-\phi)\cos\alpha} + \frac{\tan\beta}{\tan(\beta-\phi)}(b + x\tan\beta\tan\alpha) + K_o x \tan\beta(\tan\phi\sin\beta - \tan\alpha) - K_a b \right]$$

$$= .03x\,[0.534x + 277.332 + 2.456x + 0.508x - 0.255x - 18.72$$

$P_{st} = 107.04$ p/in $\qquad P_{st} = 0.097x^2 + 7.758x$

$$P_{sd} = K_a b \gamma x (\tan^8\beta - 1) + K_o b \gamma x \tan\phi \tan^4\beta \qquad P_{sd} = 132.375x$$

$P_{sd} = 1588.5$ p/in

use $P_{st} = 107.04$ p/in

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JOB Northern Ave @ New River SHEET NO. COM 1
MADE BY T Stones DATE 11-6-89 JOB NO. 33-8902
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 159
CALCULATIONS FOR COM624G Input

4. Find depth x_t where $P_{st} = P_{sd}$

$$0.097x^2 + 7.758x = 132.375x$$

$$0.097x^2 - 124.617x = 0$$

$$x = \frac{+124.617 \pm \sqrt{(124.617)^2 - \cancel{4(.097)(0)}^{\,0}}}{2(0.097)}$$

$$x = \frac{124.617 \mp 124.617}{0.19}$$ solutions @ $x = 1284.711$ in ; 0

this is deeper than bottom of shaft so P_{st} should control. However, ϕ and γ change with soil stratum

5. determine p-y curve at $x = 60$ in or 24 in below scour

6. $y_u = 3b/80 = 3(78)/80 = 2.925$ in

$p_u = \bar{A}_s P_s$ $\quad \frac{x}{b} = \frac{12}{78} = 0.154$ $\quad \bar{A}_s = 2.78$ from Fig 3.22

$p_u = 2.78\,(107.04) = 297.6$ p/in

See Sht P3A

7. $y_m = b/60 = 78/60 = 1.3$ in

$p_m = B_s P_s$ $\quad B_s = 2.15$ for $\frac{x}{b} = 0.154$ from Fig 3.23

$p_m = 2.15\,(107.04) = 230.2$ p/in

8. $p = (kx)y$ $\quad k = 25.5$ p/in³ see sht 5-9

$P_k = (25.5\ \text{lb/in}^3)(12\ \text{in})\,y = 306\ y$ lb/in³ $\quad y$ in inches

9. parabolic section

$$p = \bar{C}\,y^{1/n}$$

a. $m = \frac{p_u - p_m}{y_u - y_m} = \frac{297.6 - 230.2}{2.925 - 1.3} = 41.47$

b. $n = \frac{p_m}{m y_m} = \frac{230.2}{41.47(1.3)} = 4.269$

c. $\bar{C} = \frac{p_m}{y_m^{1/n}} = \frac{230.2}{(1.3)^{1/4.269}} = 216.48$

JOB Northern Ave @ New River SHEET NO. COM 2

MADE BY T Stones DATE 11-6-89 JOB NO. 33-89062

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 1160

CALCULATIONS FOR COM6246 Input

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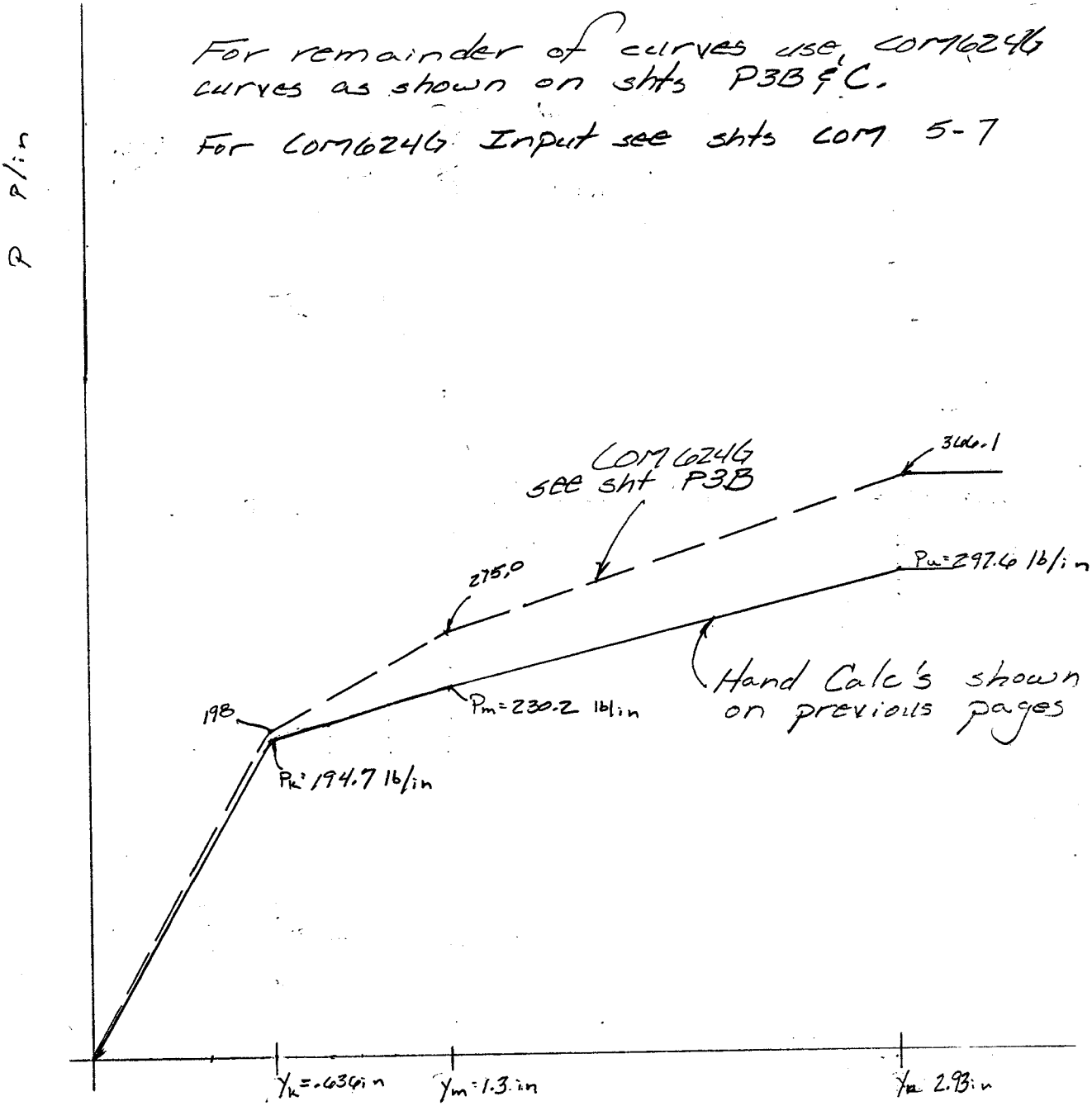
9d. $Y_k = \left(\frac{\bar{\tau}}{kx}\right)^{n/n-1} = \left[\frac{216.48}{25.5(12)}\right]^{4.269/3.269} = 0.636$ in

$P_k = 306 \cdot (0.636) = 194.7$

For remainder of curves use COM624G curves as shown on shts P3B & C.

For COM624G Input see shts COM 5-7



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JOB Northern Ave @ New River SHEET NO. COM 3
MADE BY T Stones DATE 11-6-89 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 161
CALCULATIONS FOR COM624G Input

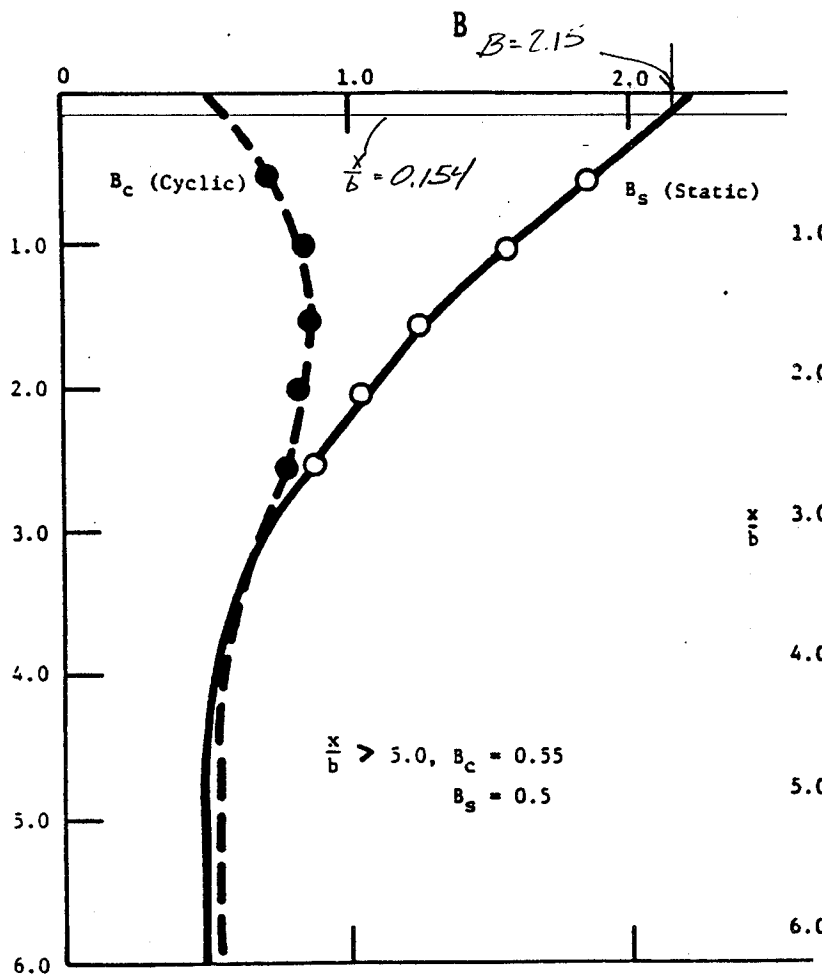

Fig. 3.23. Non dimensional Coefficient B for Soil Resista versus Depth

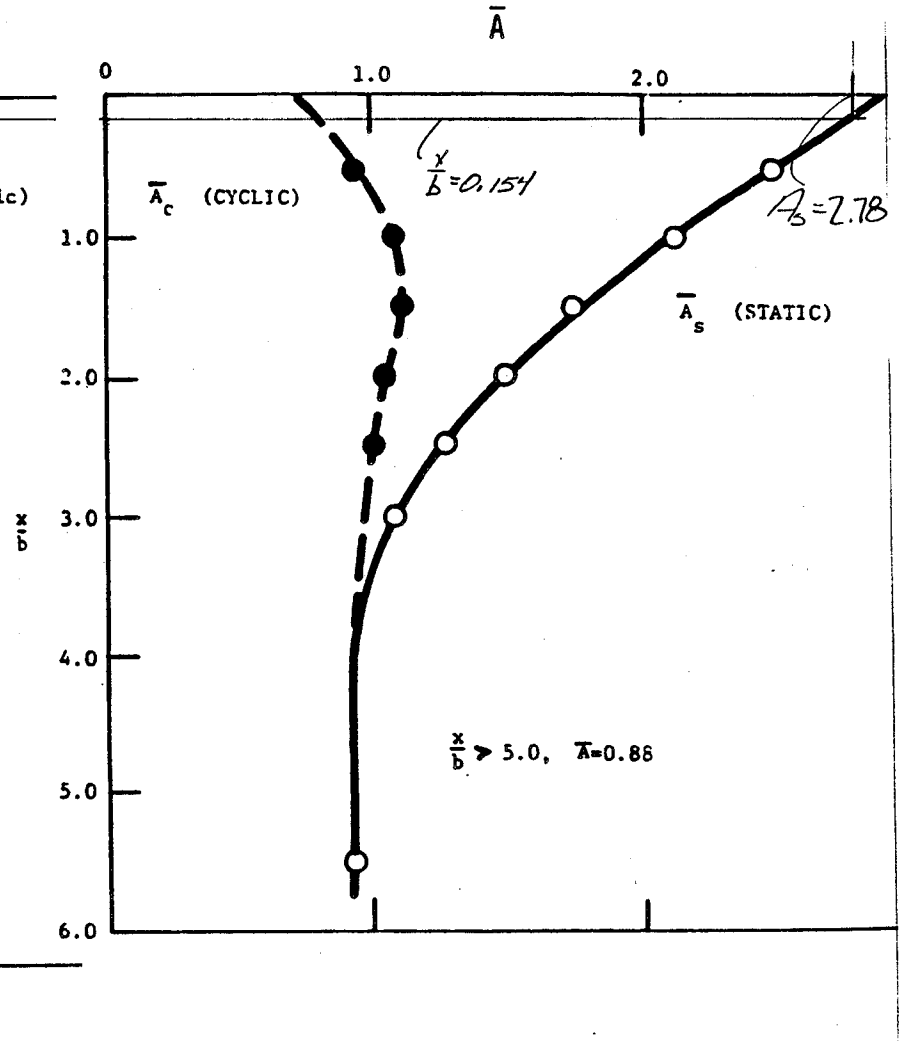

Fig. 3.22. Values of Coefficients $\bar{A}_c$ and $\bar{A}_s$

MINNEAPOLIS PHOENIX

JOB Northern Avenue @ New River SHEET NO. COM 4

MADE BY T Stokes DATE 1-16-90 JOB NO. 33-8962

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 162

CALCULATIONS FOR COM 6246

1 Title Card TITLE

NORTHERN AVE. TRANS: W/ SCOUR

2 Units Card ISYSTM

ENGL

3 Input Control Card NI, NL, NDIAM, NW

79, 5, 2, 00

4 Input Control Card NGI, NSTR, NPY

6, 6, 0

5 Geometry Card LENGTH, EPILE, XGS

948, 3605000, 588

6. Output Control Card KPYUP, ING

1, 1

7. Analysis Control Card KBC, KOUTPT, KCYCL, RCYCL
note: S_y = slope at top of shaft

3, 2, 1, 0

8. Run Control Card MAXIT, YLOL, EXDEFL

0, 0, 12

9. Distributed Loads NW = 0 so this card omitted

10. Pile Properties Card XDIAM(I), DIAM(I), MINERT(I), AREA(I)

0.0, 60, 636173, 2827.4
216, 78, 1816972, 4778.4

11. Soil Profile Card LAYER, KSOIL, XTOP, XBOT, K, AE, FR

1, 4, 588 612, 25.5, 0, 0
2, 4, 612 732, 44.0, 0, 0
3, 4, 732 792, 44.0 0, 0
4, 4, 792, 852, 44.0, 0, 0
5, 4, 852 948. 44.0, 0, 0

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MINNEAPOLIS PHOENIX

JOB Northern Ave c New River SHEET NO. COM 5
MADE BY T. Stones DATE 11-6-89 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 163
CALCULATIONS FOR COM 10246 : Input

12. Unit Weight Card XG1 , GAM1

588, 0.03
612, 0.036
732, 0.036
792, 0.030
852, 0.032
948, 0.032

13. Strength Parameter Card XSTR(I), C1(I), PHI(I), EES0(I)

588, 0, 36, 0
612, 0, 40, 0
732, 0, 32, 0
792, 0, 36, 0
852, 0, 33, 0
948, 0, 33, 0

14. Control Card for Input of p-y curve NPY=0 so this card omitted

15. Card for Depth of p-y curve NPY=0 so this card omitted

16. p-y Curve Data Card NPY=0 so this card omitted

17. Control Card for Output of Internally Generated p-y Curves NN

6

18. Control Card for Depths at which Internally Generated P-Y Curves are to be printed XN(I)

600 822

672 900

762 948

19. Card to Establish Loads on Pile Head KOP, PT, BCZ, PX

1, -2.20 E4, 2.256 E10, +1.773 E6
1, -1.84 E4, 2.331 E10, 1.963 E6

-1, 0, 0, 0

20 END

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. Com 6
MADE BY T Stones DATE 11-6-89 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 164
CALCULATIONS FOR COM624G: Input

For Line no. 19 : Loads at top of Column head refer to M-Strudl out put

Axial Load @ Top of Column will be input as M-Strudl Load plus ℄ to scour.

$$P_X = A_X + (.150)\left(\frac{\pi}{4}\right)\left[18^2(5)^2 + 31(6^2)^2\right] = A_X + 208^k$$

Rotational Restraint @ Column Top $R_T = \frac{M_T}{S_T}$

M_T = Moment @ Top of pile

S_T = Slope @ Top of pile in Radians

Units in M-Strudl are kips · feet · radians

Units in COM624G are pounds · inch · radians

M-Strudl Output for Member 9, Joint 5

AASHTO GROUP	M-Strudl Results					COM624G Input		
	S_5	M_5	R_5 (Restraint)	P_5 (Shear)	A_5 (Axial)	R_T Restraint	P_T Shear	P_X Axial M-Strudl + 208^k
With Scour								
1 *	0.00118	2218	1.880E6	-22.0	-1565	2.256E10	-2.20E4	1.773E6
2		1759		-17.4	-1126			
3		1708		-16.6	-640			
4 *	0.00094	1826	1.943E6	-18.4	-1255	2.331E10	-1.84E4	1.463E6
5		1615		-16.2	-1008			
6		1571		-15.6	-574			
Without Scour								
1 *	0.00119	1840	1.546E6	-29.5	-1540	1.855E10	-2.95E4	1.748E6
2		1441		-23.2	-1103			
3		1479		-23.8	-1232			
4 *	0.00095	1555	1.637E6	-25.9	-1235	1.964E10	-2.59E4	1.443E6
5		1362		-22.7	-989			
6		1395		-23.4	-1104			

* Loading will be input into COM624G Analysis

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. COM 7
MADE BY T Stones DATE 1-16-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 165
CALCULATIONS FOR COM624G Input

```
*************************************************************
*                                                           *
*    LATERALLY LOADED PILE ANALYSIS   -  COM624G            *
*                                                           *
*    PC ADAPTATION OF COM624                                *
*                                                           *
*    ORIGINALLY WRITTEN BY REESE AND SULLIVAN AT            *
*    UNIVERSITY OF TEXAS AT AUSTIN                          *
*                                                           *
*    ADAPTED FOR PC USE BY                                  *
*                                                           *
*                                                           *
*    GEOSOFT                                                *
*    1442 LINCOLN AVE., SUITE 146                           *
*    ORANGE, CA 92665  (714) 998-4030                       *
*                                                           *
*                                                           *
*    PORTIONS (C) COPYRIGHT 1984 GEOSOFT                    *
*                                                           *
*                                                           *
*************************************************************
```

✓ Lee Sullivan 2/90

1

NORTHERN AVE. SHAFTS WITH SCOUR

UNITS--ENGL

INPUT INFORMATION

****

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 948.00 IN
MODULUS OF ELASTICITY OF PILE = .361D+07 LBS/IN**2
2 SECTION(S)

X IN	DIAMETER IN	MOMENT OF INERTIA IN**4	AREA IN**2
.00	60.000	.636D+06	.283D+04
216.00			

166

78.000 .182D+07 .478D+04

948.00

✓ Lee Arnold 2/90

SOILS INFORMATION

X AT THE GROUND SURFACE = 588.00 IN

5 LAYER(S) OF SOIL

LAYER 1
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 588.00 IN
X AT THE BOTTOM OF THE LAYER = 612.00 IN
MODULUS OF SUBGRADE REACTION = .255D+02 LBS/IN**3

LAYER 2
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 612.00 IN
X AT THE BOTTOM OF THE LAYER = 732.00 IN
MODULUS OF SUBGRADE REACTION = .440D+02 LBS/IN**3

LAYER 3
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 732.00 IN
X AT THE BOTTOM OF THE LAYER = 792.00 IN
MODULUS OF SUBGRADE REACTION = .440D+02 LBS/IN**3

LAYER 4
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 792.00 IN
X AT THE BOTTOM OF THE LAYER = 852.00 IN
MODULUS OF SUBGRADE REACTION = .440D+02 LBS/IN**3

LAYER 5
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 852.00 IN
X AT THE BOTTOM OF THE LAYER = 948.00 IN
MODULUS OF SUBGRADE REACTION = .440D+02 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
6 POINTS

X,IN	WEIGHT,LBS/IN**3
588.00	.30D-01
612.00	.36D-01
732.00	.36D-01
792.00	.30D-01
852.00	.32D-01
948.00	.32D-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
6 POINTS

X,IN	C,LBS/IN**2	PHI,DEGREES	E50
588.00	.000D+00	36.000	*****
612.00	.000D+00	40.000	*****
732.00	.000D+00	32.000	*****
792.00	.000D+00	36.000	*****
852.00	.000D+00	33.000	*****
948.00	.000D+00	33.000	*****

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FINITE DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS = 79
TOLERANCE ON DETERMINATION OF DEFLECTIONS = .100D-04 IN

MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100
MAXIMUM ALLOWABLE DEFLECTION = .12D+02 IN

INPUT CODES
OUTPT = 2
KCYCL = 1
KBC = 3
KPYOP = 1
INC = 1

1 NORTHERN AVE. SHAFTS WITH SCOUR

UNITS--ENGL

OUTPUT INFORMATION
****

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 6
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
12.00	78.00	38.0	.32D-01	2.72	2.04	.13D+03	.23D+04

Y IN	P LBS/IN
.000	.000
.108	33.150
.217	66.300
.325	99.450
.433	132.600
.542	165.750
.650	198.900
.758	232.050
.867	246.941
.975	254.776
1.083	261.994
1.192	268.699
1.300	274.971
2.925	366.111
80.925	366.111
158.925	366.111
236.925	366.111

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
84.00	78.00	36.0	.35D-01	2.06	1.50	.16D+04	.14D+05

Y IN	P LBS/IN
.000	.000
.108	400.400
.217	800.800

168

✓ Lee Rhinold 2/10

.325	1201.200
.433	1601.600
.542	1883.749
.650	1988.834
.758	2082.242
.867	2166.696
.975	2244.027
1.083	2315.538
1.192	2382.190
1.300	2444.711
2.925	3354.575
80.925	3354.575
158.925	3354.575
236.925	3354.575

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
174.00	78.00	34.0	.35D-01	1.36	.96	.45D+04	.23D+0

Y IN	P LBS/IN
.000	.000
.108	829.400
.217	1658.800
.325	2488.200
.433	2978.860
.542	3209.682
.650	3411.492
.758	3591.988
.867	3756.042
.975	3906.956
1.083	4047.084
1.192	4178.170
1.300	4301.546
2.925	6099.874
80.925	6099.874
158.925	6099.874
236.925	6099.874

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
234.00	78.00	34.5	.34D-01	1.06	.71	.74D+04	.32D+05

Y IN	P LBS/IN
.000	.000
.108	1115.400
.217	2230.800
.325	3045.178
.433	3411.020
.542	3724.795
.650	4002.476
.758	4253.342
.867	4483.326
.975	4696.487
1.083	4895.740
1.192	5083.259
1.300	5260.715
2.925	7854.025
80.925	7854.025
158.925	7854.025
236.925	7854.025

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
312.00	78.00	33.0	.34D-01	.90	.53	.11D+05	.34D+0

Y IN	P LBS/IN
.000	.000
.108	1413.777
.217	2082.108
.325	2611.251
.433	3066.378
.542	3473.354
.650	3845.661
.758	4191.411
.867	4515.940
.975	4822.990
1.083	5115.305
1.192	5394.969
1.300	5663.612
2.925	9617.454
80.925	9617.454
158.925	9617.454
236.925	9617.454

- Lee Plunkett 2/90

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
360.00	78.00	33.0	.33D-01	.89	.51	.14D+05	.39D+0

Y IN	P LBS/IN
.000	.000
.108	1623.295
.217	2440.532
.325	3097.946
.433	3669.202
.542	4183.897
.650	4657.587
.758	5099.691
.867	5516.439
.975	5912.217
1.083	6290.253
1.192	6653.011
1.300	7002.420
2.925	12151.567
80.925	12151.567
158.925	12151.567
236.925	12151.567

PILE LOADING CONDITION

ROTATIONAL RESTRAINT = .226D+11 LBS-IN
LATERAL LOAD AT PILE HEAD = -.220D+05 LBS
AXIAL LOAD AT PILE HEAD = .177D+07 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********

170

.00	-.617D+00	.788D+07	.999D+03	.000D+00	.000D+00	.229D+13
12.00	-.613D+00	.761D+07	.986D+03	.000D+00	.000D+00	.229D+13
24.00	-.608D+00	.733D+07	.973D+03	.000D+00	.000D+00	.229D+13
36.00	-.603D+00	.706D+07	.960D+03	.000D+00	.000D+00	.229D+13
48.00	-.597D+00	.679D+07	.947D+03	.000D+00	.000D+00	.229D+13
60.00	-.591D+00	.651D+07	.934D+03	.000D+00	.000D+00	.229D+13
72.00	-.584D+00	.623D+07	.921D+03	.000D+00	.000D+00	.229D+13
84.00	-.577D+00	.596D+07	.908D+03	.000D+00	.000D+00	.229D+13
96.00	-.569D+00	.568D+07	.895D+03	.000D+00	.000D+00	.229D+13
108.00	-.562D+00	.540D+07	.882D+03	.000D+00	.000D+00	.229D+13
120.00	-.554D+00	.512D+07	.869D+03	.000D+00	.000D+00	.229D+13
132.00	-.545D+00	.485D+07	.856D+03	.000D+00	.000D+00	.229D+13
144.00	-.536D+00	.457D+07	.842D+03	.000D+00	.000D+00	.229D+13
156.00	-.527D+00	.429D+07	.829D+03	.000D+00	.000D+00	.229D+13
168.00	-.518D+00	.401D+07	.816D+03	.000D+00	.000D+00	.229D+13
180.00	-.508D+00	.372D+07	.803D+03	.000D+00	.000D+00	.229D+13
192.00	-.499D+00	.344D+07	.789D+03	.000D+00	.000D+00	.229D+13
204.00	-.489D+00	.316D+07	.776D+03	.000D+00	.000D+00	.229D+13
216.00	-.478D+00	.288D+07	.433D+03	.000D+00	.000D+00	.442D+13
228.00	-.468D+00	.260D+07	.427D+03	.000D+00	.000D+00	.655D+13
240.00	-.458D+00	.231D+07	.421D+03	.000D+00	.000D+00	.655D+13
252.00	-.447D+00	.203D+07	.415D+03	.000D+00	.000D+00	.655D+13
264.00	-.437D+00	.175D+07	.409D+03	.000D+00	.000D+00	.655D+13
276.00	-.427D+00	.147D+07	.403D+03	.000D+00	.000D+00	.655D+13
288.00	-.416D+00	.118D+07	.396D+03	.000D+00	.000D+00	.655D+13
300.00	-.405D+00	.902D+06	.390D+03	.000D+00	.000D+00	.655D+13
312.00	-.395D+00	.619D+06	.384D+03	.000D+00	.000D+00	.655D+13
324.00	-.384D+00	.336D+06	.378D+03	.000D+00	.000D+00	.655D+13
336.00	-.374D+00	.537D+05	.372D+03	.000D+00	.000D+00	.655D+13
348.00	-.363D+00	-.229D+06	.376D+03	.000D+00	.000D+00	.655D+13
360.00	-.353D+00	-.512D+06	.382D+03	.000D+00	.000D+00	.655D+13
372.00	-.342D+00	-.795D+06	.388D+03	.000D+00	.000D+00	.655D+13
384.00	-.331D+00	-.108D+07	.394D+03	.000D+00	.000D+00	.655D+13
396.00	-.321D+00	-.136D+07	.400D+03	.000D+00	.000D+00	.655D+13
408.00	-.310D+00	-.164D+07	.406D+03	.000D+00	.000D+00	.655D+13
420.00	-.300D+00	-.193D+07	.412D+03	.000D+00	.000D+00	.655D+13
432.00	-.290D+00	-.221D+07	.418D+03	.000D+00	.000D+00	.655D+13
444.00	-.279D+00	-.249D+07	.424D+03	.000D+00	.000D+00	.655D+13
456.00	-.269D+00	-.277D+07	.431D+03	.000D+00	.000D+00	.655D+13
468.00	-.259D+00	-.305D+07	.437D+03	.000D+00	.000D+00	.655D+13
480.00	-.248D+00	-.334D+07	.443D+03	.000D+00	.000D+00	.655D+13
492.00	-.238D+00	-.362D+07	.449D+03	.000D+00	.000D+00	.655D+13
504.00	-.228D+00	-.390D+07	.455D+03	.000D+00	.000D+00	.655D+13
516.00	-.218D+00	-.418D+07	.461D+03	.000D+00	.000D+00	.655D+13
528.00	-.209D+00	-.446D+07	.467D+03	.000D+00	.000D+00	.655D+13
540.00	-.199D+00	-.474D+07	.473D+03	.000D+00	.000D+00	.655D+13
552.00	-.189D+00	-.503D+07	.479D+03	.000D+00	.000D+00	.655D+13
564.00	-.180D+00	-.531D+07	.485D+03	.000D+00	.000D+00	.655D+13
576.00	-.170D+00	-.559D+07	.491D+03	.000D+00	.000D+00	.655D+13
588.00	-.161D+00	-.587D+07	.497D+03	.000D+00	.000D+00	.655D+13
600.00	-.152D+00	-.615D+07	.503D+03	.000D+00	.306D+03	.655D+13
612.00	-.143D+00	-.642D+07	.509D+03	.000D+00	.106D+04	.655D+13
624.00	-.134D+00	-.667D+07	.514D+03	.000D+00	.158D+04	.655D+13
636.00	-.125D+00	-.689D+07	.519D+03	.000D+00	.211D+04	.655D+13
648.00	-.116D+00	-.708D+07	.523D+03	.000D+00	.264D+04	.655D+13
660.00	-.108D+00	-.721D+07	.526D+03	.000D+00	.317D+04	.655D+13
672.00	-.997D-01	-.730D+07	.528D+03	.000D+00	.370D+04	.655D+13
684.00	-.915D-01	-.734D+07	.529D+03	.000D+00	.422D+04	.655D+13
696.00	-.836D-01	-.732D+07	.528D+03	.000D+00	.475D+04	.655D+13
708.00	-.758D-01	-.724D+07	.526D+03	.000D+00	.528D+04	.655D+13
720.00	-.681D-01	-.710D+07	.523D+03	.000D+00	.581D+04	.655D+13
732.00	-.607D-01	-.691D+07	.519D+03	.000D+00	.634D+04	.655D+13
744.00	-.533D-01	-.666D+07	.514D+03	.000D+00	.686D+04	.655D+13
756.00	-.461D-01	-.636D+07	.507D+03	.000D+00	.739D+04	.655D+13
768.00	-.391D-01	-.600D+07	.500D+03	.000D+00	.792D+04	.655D+13
780.00	-.322D-01	-.561D+07	.491D+03	.000D+00	.845D+04	.655D+13

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792.00	-.254D-01	-.517D+07	.482D+03	.000D+00	.898D+04	.655D+13
804.00	-.187D-01	-.470D+07	.472D+03	.000D+00	.950D+04	.655D+13
816.00	-.121D-01	-.421D+07	.461D+03	.000D+00	.100D+05	.655D+13
828.00	-.564D-02	-.370D+07	.450D+03	.000D+00	.106D+05	.655D+13
840.00	.765D-03	-.318D+07	.439D+03	.000D+00	.111D+05	.655D+13
852.00	.710D-02	-.266D+07	.428D+03	.000D+00	.116D+05	.655D+13
864.00	.134D-01	-.215D+07	.417D+03	.000D+00	.121D+05	.655D+13
876.00	.196D-01	-.166D+07	.407D+03	.000D+00	.127D+05	.655D+13
888.00	.258D-01	-.121D+07	.397D+03	.000D+00	.132D+05	.655D+13
900.00	.320D-01	-.814D+06	.389D+03	.000D+00	.137D+05	.655D+13
912.00	.381D-01	-.477D+06	.381D+03	.000D+00	.143D+05	.655D+13
924.00	.442D-01	-.218D+06	.376D+03	.000D+00	.148D+05	.655D+13
936.00	.504D-01	-.536D+05	.372D+03	.000D+00	.153D+05	.655D+13
948.00	.565D-01	.000D+00	.371D+03	.000D+00	.158D+05	.655D+13

✓ Lee Chenault 2/90

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.319D-05 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.312D-06 LBS

COMPUTED LATERAL FORCE AT PILE HEAD = -.22000D+05 LBS
COMPUTED ROTATIONAL STIFFNESS AT PILE HEAD = .22560D+11 IN-LBS
COMPUTED SLOPE AT PILE HEAD = .34918D-03

THE OVERALL MOMENT IMBALANCE = -.256D-03 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE = .668D-06 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = -.617D+00 IN
MAXIMUM BENDING MOMENT = .788D+07 IN-LBS
MAXIMUM TOTAL STRESS = .999D+03 LBS/IN**2
MAXIMUM SHEAR FORCE = .434D+05 LBS
NO. OF ITERATIONS = 2
MAXIMUM DEFLECTION ERROR = .000D+00 IN

PILE LOADING CONDITION

ROTATIONAL RESTRAINT = .233D+11 LBS-IN
LATERAL LOAD AT PILE HEAD = -.184D+05 LBS
AXIAL LOAD AT PILE HEAD = .146D+07 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	-.507D+00	.655D+07	.826D+03	.000D+00	.000D+00	.229D+13
12.00	-.504D+00	.632D+07	.816D+03	.000D+00	.000D+00	.229D+13
24.00	-.500D+00	.610D+07	.805D+03	.000D+00	.000D+00	.229D+13
36.00	-.495D+00	.587D+07	.794D+03	.000D+00	.000D+00	.229D+13
48.00	-.491D+00	.564D+07	.784D+03	.000D+00	.000D+00	.229D+13
60.00	-.485D+00	.541D+07	.773D+03	.000D+00	.000D+00	.229D+13
72.00	-.480D+00	.519D+07	.762D+03	.000D+00	.000D+00	.229D+13
84.00	-.474D+00	.496D+07	.751D+03	.000D+00	.000D+00	.229D+13

96.00	-.468D+00	.473D+07	.740D+03	.000D+00	.000D+00	.229D+13
108.00	-.462D+00	.450D+07	.730D+03	.000D+00	.000D+00	.229D+13
120.00	-.455D+00	.427D+07	.719D+03	.000D+00	.000D+00	.229D+13
132.00	-.448D+00	.404D+07	.708D+03	.000D+00	.000D+00	.229D+13
144.00	-.441D+00	.380D+07	.697D+03	.000D+00	.000D+00	.229D+13
156.00	-.434D+00	.357D+07	.686D+03	.000D+00	.000D+00	.229D+13
168.00	-.426D+00	.334D+07	.675D+03	.000D+00	.000D+00	.229D+13
180.00	-.418D+00	.311D+07	.664D+03	.000D+00	.000D+00	.229D+13
192.00	-.410D+00	.288D+07	.653D+03	.000D+00	.000D+00	.229D+13
204.00	-.402D+00	.264D+07	.642D+03	.000D+00	.000D+00	.229D+13
216.00	-.394D+00	.241D+07	.358D+03	.000D+00	.000D+00	.442D+13
228.00	-.385D+00	.218D+07	.353D+03	.000D+00	.000D+00	.655D+13
240.00	-.377D+00	.194D+07	.348D+03	.000D+00	.000D+00	.655D+13
252.00	-.368D+00	.171D+07	.343D+03	.000D+00	.000D+00	.655D+13
264.00	-.360D+00	.148D+07	.338D+03	.000D+00	.000D+00	.655D+13
276.00	-.351D+00	.124D+07	.333D+03	.000D+00	.000D+00	.655D+13
288.00	-.342D+00	.101D+07	.328D+03	.000D+00	.000D+00	.655D+13
300.00	-.334D+00	.777D+06	.323D+03	.000D+00	.000D+00	.655D+13
312.00	-.325D+00	.544D+06	.318D+03	.000D+00	.000D+00	.655D+13
324.00	-.316D+00	.310D+06	.313D+03	.000D+00	.000D+00	.655D+13
336.00	-.308D+00	.765D+05	.308D+03	.000D+00	.000D+00	.655D+13
348.00	-.299D+00	-.157D+06	.310D+03	.000D+00	.000D+00	.655D+13
360.00	-.290D+00	-.391D+06	.315D+03	.000D+00	.000D+00	.655D+13
372.00	-.282D+00	-.624D+06	.320D+03	.000D+00	.000D+00	.655D+13
384.00	-.273D+00	-.858D+06	.325D+03	.000D+00	.000D+00	.655D+13
396.00	-.264D+00	-.109D+07	.330D+03	.000D+00	.000D+00	.655D+13
408.00	-.256D+00	-.132D+07	.335D+03	.000D+00	.000D+00	.655D+13
420.00	-.247D+00	-.156D+07	.340D+03	.000D+00	.000D+00	.655D+13
432.00	-.238D+00	-.179D+07	.345D+03	.000D+00	.000D+00	.655D+13
444.00	-.230D+00	-.202D+07	.350D+03	.000D+00	.000D+00	.655D+13
456.00	-.221D+00	-.226D+07	.355D+03	.000D+00	.000D+00	.655D+13
468.00	-.213D+00	-.249D+07	.360D+03	.000D+00	.000D+00	.655D+13
480.00	-.205D+00	-.272D+07	.365D+03	.000D+00	.000D+00	.655D+13
492.00	-.196D+00	-.296D+07	.370D+03	.000D+00	.000D+00	.655D+13
504.00	-.188D+00	-.319D+07	.375D+03	.000D+00	.000D+00	.655D+13
516.00	-.180D+00	-.342D+07	.380D+03	.000D+00	.000D+00	.655D+13
528.00	-.172D+00	-.366D+07	.385D+03	.000D+00	.000D+00	.655D+13
540.00	-.164D+00	-.389D+07	.390D+03	.000D+00	.000D+00	.655D+13
552.00	-.156D+00	-.412D+07	.395D+03	.000D+00	.000D+00	.655D+13
564.00	-.148D+00	-.435D+07	.400D+03	.000D+00	.000D+00	.655D+13
576.00	-.140D+00	-.458D+07	.405D+03	.000D+00	.000D+00	.655D+13
588.00	-.133D+00	-.482D+07	.410D+03	.000D+00	.000D+00	.655D+13
600.00	-.125D+00	-.505D+07	.415D+03	.000D+00	.306D+03	.655D+13
612.00	-.118D+00	-.527D+07	.419D+03	.000D+00	.106D+04	.655D+13
624.00	-.110D+00	-.548D+07	.424D+03	.000D+00	.158D+04	.655D+13
636.00	-.103D+00	-.567D+07	.428D+03	.000D+00	.211D+04	.655D+13
648.00	-.959D-01	-.582D+07	.431D+03	.000D+00	.264D+04	.655D+13
660.00	-.890D-01	-.593D+07	.433D+03	.000D+00	.317D+04	.655D+13
672.00	-.822D-01	-.601D+07	.435D+03	.000D+00	.370D+04	.655D+13
684.00	-.755D-01	-.604D+07	.436D+03	.000D+00	.422D+04	.655D+13
696.00	-.689D-01	-.602D+07	.435D+03	.000D+00	.475D+04	.655D+13
708.00	-.625D-01	-.596D+07	.434D+03	.000D+00	.528D+04	.655D+13
720.00	-.562D-01	-.584D+07	.432D+03	.000D+00	.581D+04	.655D+13
732.00	-.500D-01	-.569D+07	.428D+03	.000D+00	.634D+04	.655D+13
744.00	-.440D-01	-.548D+07	.424D+03	.000D+00	.686D+04	.655D+13
756.00	-.381D-01	-.523D+07	.419D+03	.000D+00	.739D+04	.655D+13
768.00	-.323D-01	-.495D+07	.412D+03	.000D+00	.792D+04	.655D+13
780.00	-.266D-01	-.462D+07	.405D+03	.000D+00	.845D+04	.655D+13
792.00	-.210D-01	-.426D+07	.398D+03	.000D+00	.898D+04	.655D+13
804.00	-.155D-01	-.388D+07	.389D+03	.000D+00	.950D+04	.655D+13
816.00	-.101D-01	-.347D+07	.381D+03	.000D+00	.100D+05	.655D+13
828.00	-.474D-02	-.305D+07	.372D+03	.000D+00	.106D+05	.655D+13
840.00	.530D-03	-.262D+07	.362D+03	.000D+00	.111D+05	.655D+13
852.00	.574D-02	-.219D+07	.353D+03	.000D+00	.116D+05	.655D+13
864.00	.109D-01	-.177D+07	.344D+03	.000D+00	.121D+05	.655D+13
876.00	.160D-01	-.137D+07	.336D+03	.000D+00	.127D+05	.655D+13

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888.00	.211D-01	-.100D+07	.328D+03	.000D+00	.132D+05	.655D+13
900.00	.262D-01	-.674D+06	.321D+03	.000D+00	.137D+05	.655D+13
912.00	.313D-01	-.396D+06	.315D+03	.000D+00	.143D+05	.655D+13
924.00	.363D-01	-.182D+06	.310D+03	.000D+00	.148D+05	.655D+13
936.00	.414D-01	-.455D+05	.307D+03	.000D+00	.153D+05	.655D+13
948.00	.464D-01	.000D+00	.306D+03	.000D+00	.158D+05	.655D+13

✓ Lee Ulmell

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.341D-05 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = -.341D-06 LBS

COMPUTED LATERAL FORCE AT PILE HEAD = -.18400D+05 LBS
COMPUTED ROTATIONAL STIFFNESS AT PILE HEAD = .23310D+11 IN-LBS
COMPUTED SLOPE AT PILE HEAD = .28103D-03

THE OVERALL MOMENT IMBALANCE = .627D-03 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE = -.157D-05 LBS

OUTPUT SUMMARY

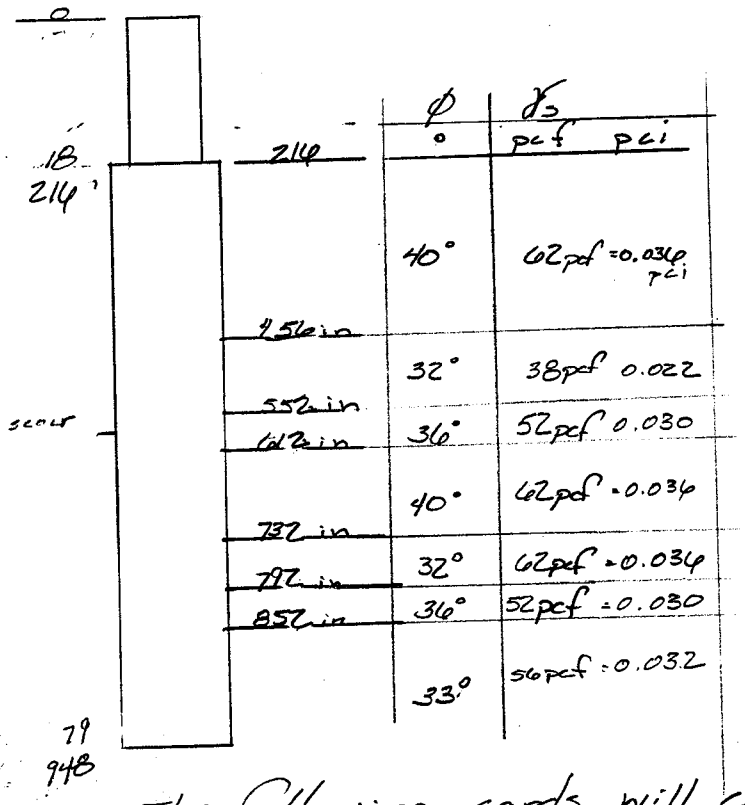
PILE HEAD DEFLECTION = -.507D+00 IN
MAXIMUM BENDING MOMENT = .655D+07 IN-LBS
MAXIMUM TOTAL STRESS = .826D+03 LBS/IN**2
MAXIMUM SHEAR FORCE = .357D+05 LBS
NO. OF ITERATIONS = 2
MAXIMUM DEFLECTION ERROR = .000D+00 IN

1 NORTHERN AVE. SHAFTS WITH SCOUR

S U M M A R Y T A B L E
****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2)
-.220D+05	.226D+11	.177D+07	-.617D+00	.349D-03	.788D+07	.999D+03
-.184D+05	.233D+11	.146D+07	-.507D+00	.281D-03	.655D+07	.826D+03

The following soil parameters will be used to determine p-y curves for the column w/o scour



ϕ °	γ_s pcf	pci
40°	62pcf	0.036 pci
32°	38pcf	0.022
36°	52pcf	0.030
40°	62pcf	0.036
32°	62pcf	0.036
36°	52pcf	0.030
33°	56pcf	0.032

The following cards will change on original COM624

① NORTHERN AVE. TRANS W/O SCOUR

③ 79, 7, 2, 0

④ 8, 8, 0

⑤ 948, 3605000, 216

⑪

1	4	216	456	25.5	0	0
2	4	456	552	25.5	0	0
3	4	552	612	25.5	0	0
4	4	612	732	25.5	0	0
5	4	732	792	44.0	0	0
6	4	792	852	44.0	0	0
7	4	852	948	44.0	0	0

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. COM 17
MADE BY T Stones DATE 11-13-89 JOB NO. 33-89602
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 175
CALCULATIONS FOR COM624 Input w/o Scour

(12)

216	0.036
456	0.022
552	0.030
612	0.036
732	0.036
792	0.030
852	0.032
948	0.032

(13)

216	0	40
456	0	32
552	0	36
612	0	40
732	0	32
792	0	36
852	0	33
948	0	33

(17) 8

(18) 336, 504, 582, 672, 762, 822, 900, 948

(19)

-2.95 E4	1.855 E10	1.748 E6
-2.59 E4	1.964 E10	1.443 E6

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. COM 18
MADE BY T Stones DATE 11-13-89 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 176
CALCULATIONS FOR COM624 Input w/o Scour

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*************************************************************
*                                                           *
*     LATERALLY LOADED PILE ANALYSIS   -  COM624G           *
*                                                           *
*     PC ADAPTATION OF COM624                               *
*                                                           *
*     ORIGINALLY WRITTEN BY REESE AND SULLIVAN AT           *
*     UNIVERSITY OF TEXAS AT AUSTIN                         *
*                                                           *
*     ADAPTED FOR PC USE BY                                 *
*                                                           *
*     GEOSOFT                                               *
*     1442 LINCOIN AVE., SUITE 146                          *
*     ORANGE, CA 92665  (714) 998-4030                      *
*                                                           *
*     PORTIONS (C) COPYRIGHT 1984 GEOSOFT                   *
*     ✓ [illegible] 2/90                                    *
*************************************************************
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1 NORTHERN AVE. SHAFTS WITHOUT SCOUR

UNITS--ENGL

I N P U T I N F O R M A T I O N

****

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 948.00 IN
MODULUS OF ELASTICITY OF PILE = .361D+07 LBS/IN**2
2 SECTION(S)

X IN	DIAMETER IN	MOMENT OF INERTIA IN**4	AREA IN**2
.00	60.000	.636D+06	.283D+04
216.00			

177

78.000 .182D+07 .478D+04

948.00

✓ Lee Arnold 2/90

SOILS INFORMATION

X AT THE GROUND SURFACE = 216.00 IN

7 LAYER(S) OF SOIL

LAYER 1
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 216.00 IN
X AT THE BOTTOM OF THE LAYER = 456.00 IN
MODULUS OF SUBGRADE REACTION = .255D+02 LBS/IN**3

LAYER 2
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 456.00 IN
X AT THE BOTTOM OF THE LAYER = 552.00 IN
MODULUS OF SUBGRADE REACTION = .255D+02 LBS/IN**3

LAYER 3
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 552.00 IN
X AT THE BOTTOM OF THE LAYER = 612.00 IN
MODULUS OF SUBGRADE REACTION = .255D+02 LBS/IN**3

LAYER 4
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 612.00 IN
X AT THE BOTTOM OF THE LAYER = 732.00 IN
MODULUS OF SUBGRADE REACTION = .440D+02 LBS/IN**3

LAYER 5
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 732.00 IN
X AT THE BOTTOM OF THE LAYER = 792.00 IN
MODULUS OF SUBGRADE REACTION = .440D+02 LBS/IN**3

LAYER 6
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 792.00 IN
X AT THE BOTTOM OF THE LAYER = 852.00 IN
MODULUS OF SUBGRADE REACTION = .440D+02 LBS/IN**3

LAYER 7
THE SOIL IS A SAND
X AT THE TOP OF THE LAYER = 852.00 IN
X AT THE BOTTOM OF THE LAYER = 948.00 IN
MODULUS OF SUBGRADE REACTION = .440D+02 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
8 POINTS

X,IN	WEIGHT,LBS/IN**3
216.00	.36D-01
456.00	.22D-01
552.00	.30D-01
612.00	.36D-01
732.00	.36D-01
792.00	.30D-01
852.00	.32D-01
948.00	.32D-01

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✓ [illegible] 2/90

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
8 POINTS

X, IN	C, LBS/IN**2	PHI, DEGREES	E50
216.00	.000D+00	40.000	*****
456.00	.000D+00	32.000	*****
552.00	.000D+00	36.000	*****
612.00	.000D+00	40.000	*****
732.00	.000D+00	32.000	*****
792.00	.000D+00	36.000	*****
852.00	.000D+00	33.000	*****
948.00	.000D+00	33.000	*****

FINITE DIFFERENCE PARAMETERS

NUMBER OF PILE INCREMENTS	=	79
TOLERANCE ON DETERMINATION OF DEFLECTIONS	=	.100D-04 IN
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS	=	100
MAXIMUM ALLOWABLE DEFLECTION	=	.12D+02 IN

INPUT CODES

OUTPT = 2
KCYCL = 1
KBC = 3
KPYOP = 1
INC = 1

1 NORTHERN AVE. SHAFTS WITHOUT SCOUR

UNITS--ENGL

O U T P U T I N F O R M A T I O N
****

GENERATED P-Y CURVES

THE NUMBER OF CURVES = 8
THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
120.00	78.00	36.0	.33D-01	1.74	1.25	.26D+04	.19D+0[illegible]

Y IN	P LBS/IN
.000	.000
.108	331.500
.217	663.000
.325	994.500
.433	1326.000
.542	1657.500
.650	1989.000
.758	2320.500
.867	2652.000
.975	2983.500
1.083	3091.864
1.192	3184.544

1.300	3271.578
2.925	4538.838
80.925	4538.838
158.925	4538.838
236.925	4538.838

✓ Lee Arnold 2/90

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
288.00	78.00	34.0	.28D-01	.95	.59	.84D+04	.30D+0

Y IN	P LBS/IN
.000	.000
.108	795.600
.217	1591.200
.325	2386.800
.433	2852.557
.542	3187.293
.650	3489.748
.758	3767.756
.867	4026.416
.975	4269.272
1.083	4498.899
1.192	4717.245
1.300	4925.818
2.925	7987.464
80.925	7987.464
158.925	7987.464
236.925	7987.464

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
366.00	78.00	38.0	.28D-01	.89	.51	.18D+05	.65D+0

Y IN	P LBS/IN
.000	.000
.108	1011.075
.217	2022.150
.325	3033.225
.433	4044.300
.542	5055.375
.650	6063.837
.758	6643.389
.867	7190.008
.975	7709.373
1.083	8205.672
1.192	8682.096
1.300	9141.152
2.925	15907.261
80.925	15907.261
158.925	15907.261
236.925	15907.261

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
456.00	78.00	36.0	.30D-01	.88	.50	.24D+05	.65D+0

Y IN	P LBS/IN
.000	.000
.108	2173.600
.217	4023.624

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.325	5148.501
.433	6132.585
.542	7023.683
.650	7847.062
.758	8618.084
.867	9346.948
.975	10040.852
1.083	10705.109
1.192	11343.782
1.300	11960.059
2.925	21049.703
80.925	21049.703
158.925	21049.703
236.925	21049.703

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
546.00	78.00	34.0	.31D-01	.88	.50	.29D+05	.62D+0

Y IN	P LBS/IN
.000	.000
.108	2602.600
.217	4912.154
.325	6285.436
.433	7486.834
.542	8574.711
.650	9579.915
.758	10521.201
.867	11411.019
.975	12258.156
1.083	13069.101
1.192	13848.810
1.300	14601.178
2.925	25698.074
80.925	25698.074
158.925	25698.074
236.925	25698.074

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
606.00	78.00	34.5	.31D-01	.88	.50	.37D+05	.73D+0

Y IN	P LBS/IN
.000	.000
.108	2888.600
.217	5777.200
.325	7959.886
.433	9481.337
.542	10859.026
.650	12132.019
.758	13324.065
.867	14450.931
.975	15523.748
1.083	16550.729
1.192	17538.154
1.300	18490.954
2.925	32544.079
80.925	32544.079
158.925	32544.079
236.925	32544.079

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
684.00	78.00	33.0	.31D-01	.88	.50	.41D+05	.69D+0

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Y IN	P LBS/IN
.000	.000
.108	3260.400
.217	6520.800
.325	8849.619
.433	10541.133
.542	12072.816
.650	13488.100
.758	14813.389
.867	16066.214
.975	17258.946
1.083	18400.721
1.192	19498.517
1.300	20557.819
2.925	36181.760
80.925	36181.760
158.925	36181.760
236.925	36181.760

DEPTH IN	DIAM IN	PHI	GAMMA LBS/IN**3	A	B	PCT	PCD
732.00	78.00	33.0	.31D-01	.88	.50	.47D+05	.74D+0

Y IN	P LBS/IN
.000	.000
.108	3489.200
.217	6978.400
.325	10076.310
.433	12002.295
.542	13746.292
.650	15357.756
.758	16866.750
.867	18293.235
.975	19651.298
1.083	20951.340
1.192	22201.308
1.300	23407.444
2.925	41197.102
80.925	41197.102
158.925	41197.102
236.925	41197.102

PILE LOADING CONDITION

ROTATIONAL RESTRAINT = .186D+11 LBS-IN
LATERAL LOAD AT PILE HEAD = -.295D+05 LBS
AXIAL LOAD AT PILE HEAD = .175D+07 LBS

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X IN	DEFLECTION IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********

.00	-.173D+00	.455D+07	.833D+03	.000D+00	.000D+00	.229D+13
12.00	-.170D+00	.419D+07	.816D+03	.000D+00	.000D+00	.229D+13
24.00	-.166D+00	.383D+07	.799D+03	.000D+00	.000D+00	.229D+13
36.00	-.163D+00	.347D+07	.782D+03	.000D+00	.000D+00	.229D+13
48.00	-.159D+00	.311D+07	.765D+03	.000D+00	.000D+00	.229D+13
60.00	-.155D+00	.275D+07	.748D+03	.000D+00	.000D+00	.229D+13
72.00	-.151D+00	.239D+07	.731D+03	.000D+00	.000D+00	.229D+13
84.00	-.146D+00	.203D+07	.714D+03	.000D+00	.000D+00	.229D+13
96.00	-.142D+00	.167D+07	.697D+03	.000D+00	.000D+00	.229D+13
108.00	-.137D+00	.131D+07	.680D+03	.000D+00	.000D+00	.229D+13
120.00	-.133D+00	.943D+06	.663D+03	.000D+00	.000D+00	.229D+13
132.00	-.128D+00	.581D+06	.646D+03	.000D+00	.000D+00	.229D+13
144.00	-.123D+00	.219D+06	.629D+03	.000D+00	.000D+00	.229D+13
156.00	-.119D+00	-.144D+06	.625D+03	.000D+00	.000D+00	.229D+13
168.00	-.114D+00	-.506D+06	.642D+03	.000D+00	.000D+00	.229D+13
180.00	-.109D+00	-.868D+06	.659D+03	.000D+00	.000D+00	.229D+13
192.00	-.104D+00	-.123D+07	.676D+03	.000D+00	.000D+00	.229D+13
204.00	-.999D-01	-.159D+07	.693D+03	.000D+00	.000D+00	.229D+13
216.00	-.954D-01	-.195D+07	.408D+03	.000D+00	.000D+00	.442D+13
228.00	-.910D-01	-.232D+07	.416D+03	.000D+00	.306D+03	.655D+13
240.00	-.866D-01	-.267D+07	.423D+03	.000D+00	.612D+03	.655D+13
252.00	-.823D-01	-.302D+07	.431D+03	.000D+00	.918D+03	.655D+13
264.00	-.781D-01	-.336D+07	.438D+03	.000D+00	.122D+04	.655D+13
276.00	-.739D-01	-.369D+07	.445D+03	.000D+00	.153D+04	.655D+13
288.00	-.698D-01	-.400D+07	.452D+03	.000D+00	.184D+04	.655D+13
300.00	-.658D-01	-.429D+07	.458D+03	.000D+00	.214D+04	.655D+13
312.00	-.619D-01	-.456D+07	.464D+03	.000D+00	.245D+04	.655D+13
324.00	-.581D-01	-.480D+07	.469D+03	.000D+00	.275D+04	.655D+13
336.00	-.544D-01	-.503D+07	.474D+03	.000D+00	.306D+04	.655D+13
348.00	-.509D-01	-.523D+07	.478D+03	.000D+00	.337D+04	.655D+13
360.00	-.474D-01	-.540D+07	.482D+03	.000D+00	.367D+04	.655D+13
372.00	-.440D-01	-.555D+07	.485D+03	.000D+00	.398D+04	.655D+13
384.00	-.408D-01	-.568D+07	.488D+03	.000D+00	.428D+04	.655D+13
396.00	-.377D-01	-.578D+07	.490D+03	.000D+00	.459D+04	.655D+13
408.00	-.347D-01	-.585D+07	.491D+03	.000D+00	.490D+04	.655D+13
420.00	-.319D-01	-.590D+07	.492D+03	.000D+00	.520D+04	.655D+13
432.00	-.291D-01	-.592D+07	.493D+03	.000D+00	.551D+04	.655D+13
444.00	-.266D-01	-.593D+07	.493D+03	.000D+00	.581D+04	.655D+13
456.00	-.241D-01	-.590D+07	.493D+03	.000D+00	.612D+04	.655D+13
468.00	-.218D-01	-.586D+07	.492D+03	.000D+00	.643D+04	.655D+13
480.00	-.196D-01	-.580D+07	.490D+03	.000D+00	.673D+04	.655D+13
492.00	-.175D-01	-.572D+07	.489D+03	.000D+00	.704D+04	.655D+13
504.00	-.155D-01	-.562D+07	.486D+03	.000D+00	.734D+04	.655D+13
516.00	-.137D-01	-.550D+07	.484D+03	.000D+00	.765D+04	.655D+13
528.00	-.120D-01	-.537D+07	.481D+03	.000D+00	.796D+04	.655D+13
540.00	-.104D-01	-.523D+07	.478D+03	.000D+00	.826D+04	.655D+13
552.00	-.895D-02	-.507D+07	.475D+03	.000D+00	.857D+04	.655D+13
564.00	-.759D-02	-.490D+07	.471D+03	.000D+00	.887D+04	.655D+13
576.00	-.634D-02	-.472D+07	.467D+03	.000D+00	.918D+04	.655D+13
588.00	-.519D-02	-.453D+07	.463D+03	.000D+00	.949D+04	.655D+13
600.00	-.414D-02	-.434D+07	.459D+03	.000D+00	.979D+04	.655D+13
612.00	-.319D-02	-.414D+07	.455D+03	.000D+00	.174D+05	.655D+13
624.00	-.232D-02	-.393D+07	.450D+03	.000D+00	.180D+05	.655D+13
636.00	-.155D-02	-.371D+07	.445D+03	.000D+00	.185D+05	.655D+13
648.00	-.854D-03	-.349D+07	.441D+03	.000D+00	.190D+05	.655D+13
660.00	-.236D-03	-.327D+07	.436D+03	.000D+00	.195D+05	.655D+13
672.00	.310D-03	-.305D+07	.431D+03	.000D+00	.201D+05	.655D+13
684.00	.789D-03	-.282D+07	.426D+03	.000D+00	.206D+05	.655D+13
696.00	.121D-02	-.260D+07	.422D+03	.000D+00	.211D+05	.655D+13
708.00	.157D-02	-.239D+07	.417D+03	.000D+00	.216D+05	.655D+13
720.00	.187D-02	-.218D+07	.413D+03	.000D+00	.222D+05	.655D+13
732.00	.213D-02	-.197D+07	.408D+03	.000D+00	.227D+05	.655D+13
744.00	.235D-02	-.177D+07	.404D+03	.000D+00	.232D+05	.655D+13
756.00	.253D-02	-.158D+07	.400D+03	.000D+00	.238D+05	.655D+13
768.00	.267D-02	-.140D+07	.396D+03	.000D+00	.243D+05	.655D+13
780.00	.278D-02	-.123D+07	.392D+03	.000D+00	.248D+05	.655D+13

✓ Lee Ramsey 2/90

792.00	.286D-02	-.106D+07	.389D+03	.000D+00	.253D+05	.655D+13
804.00	.293D-02	-.909D+06	.385D+03	.000D+00	.259D+05	.655D+13
816.00	.297D-02	-.767D+06	.382D+03	.000D+00	.264D+05	.655D+13
828.00	.299D-02	-.636D+06	.379D+03	.000D+00	.269D+05	.655D+13
840.00	.300D-02	-.517D+06	.377D+03	.000D+00	.275D+05	.655D+13
852.00	.300D-02	-.410D+06	.375D+03	.000D+00	.280D+05	.655D+13
864.00	.299D-02	-.315D+06	.373D+03	.000D+00	.285D+05	.655D+13
876.00	.298D-02	-.232D+06	.371D+03	.000D+00	.290D+05	.655D+13
888.00	.295D-02	-.162D+06	.369D+03	.000D+00	.296D+05	.655D+13
900.00	.293D-02	-.104D+06	.368D+03	.000D+00	.301D+05	.655D+13
912.00	.290D-02	-.585D+05	.367D+03	.000D+00	.306D+05	.655D+13
924.00	.287D-02	-.261D+05	.366D+03	.000D+00	.312D+05	.655D+13
936.00	.284D-02	-.657D+04	.366D+03	.000D+00	.317D+05	.655D+13
948.00	.281D-02	.000D+00	.366D+03	.000D+00	.322D+05	.655D+13

✓ [illegible] 12/90

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.500D-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .443D-07 LBS

COMPUTED LATERAL FORCE AT PILE HEAD = -.29500D+05 LBS
COMPUTED ROTATIONAL STIFFNESS AT PILE HEAD = .18550D+11 IN-LBS
COMPUTED SLOPE AT PILE HEAD = .24546D-03

THE OVERALL MOMENT IMBALANCE = -.288D-04 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE = .123D-06 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = -.173D+00 IN
MAXIMUM BENDING MOMENT = -.593D+07 IN-LBS
MAXIMUM TOTAL STRESS = .833D+03 LBS/IN**2
MAXIMUM SHEAR FORCE = .185D+05 LBS
NO. OF ITERATIONS = 2
MAXIMUM DEFLECTION ERROR = .000D+00 IN

PILE LOADING CONDITION

ROTATIONAL RESTRAINT = .196D+11 LBS-IN
LATERAL LOAD AT PILE HEAD = -.257D+05 LBS
AXIAL LOAD AT PILE HEAD = .144D+07 LBS

X IN	DEFLECTION IN	MOMENT LBS-IN	TOTAL STRESS LBS/IN**2	DISTR. LOAD LBS/IN	SOIL MODULUS LBS/IN**2	FLEXURAL RIGIDITY LBS-IN**2
*****	**********	**********	**********	**********	**********	**********
.00	-.148D+00	.401D+07	.700D+03	.000D+00	.000D+00	.229D+13
12.00	-.146D+00	.370D+07	.685D+03	.000D+00	.000D+00	.229D+13
24.00	-.143D+00	.339D+07	.670D+03	.000D+00	.000D+00	.229D+13
36.00	-.140D+00	.308D+07	.655D+03	.000D+00	.000D+00	.229D+13
48.00	-.137D+00	.276D+07	.641D+03	.000D+00	.000D+00	.229D+13
60.00	-.133D+00	.245D+07	.626D+03	.000D+00	.000D+00	.229D+13
72.00	-.130D+00	.214D+07	.611D+03	.000D+00	.000D+00	.229D+13
84.00	-.126D+00	.182D+07	.596D+03	.000D+00	.000D+00	.229D+13

96.00	-.122D+00	.151D+07	.581D+03	.000D+00	.000D+00	.229D+13
108.00	-.118D+00	.119D+07	.567D+03	.000D+00	.000D+00	.229D+13
120.00	-.114D+00	.880D+06	.552D+03	.000D+00	.000D+00	.229D+13
132.00	-.110D+00	.566D+06	.537D+03	.000D+00	.000D+00	.229D+13
144.00	-.106D+00	.251D+06	.522D+03	.000D+00	.000D+00	.229D+13
156.00	-.102D+00	-.630D+05	.513D+03	.000D+00	.000D+00	.229D+13
168.00	-.981D-01	-.377D+06	.528D+03	.000D+00	.000D+00	.229D+13
180.00	-.941D-01	-.691D+06	.543D+03	.000D+00	.000D+00	.229D+13
192.00	-.901D-01	-.101D+07	.558D+03	.000D+00	.000D+00	.229D+13
204.00	-.862D-01	-.132D+07	.573D+03	.000D+00	.000D+00	.229D+13
216.00	-.823D-01	-.163D+07	.337D+03	.000D+00	.000D+00	.442D+13
228.00	-.785D-01	-.195D+07	.344D+03	.000D+00	.306D+03	.655D+13
240.00	-.748D-01	-.226D+07	.350D+03	.000D+00	.612D+03	.655D+13
252.00	-.711D-01	-.256D+07	.357D+03	.000D+00	.918D+03	.655D+13
264.00	-.674D-01	-.286D+07	.363D+03	.000D+00	.122D+04	.655D+13
276.00	-.638D-01	-.314D+07	.369D+03	.000D+00	.153D+04	.655D+13
288.00	-.603D-01	-.341D+07	.375D+03	.000D+00	.184D+04	.655D+13
300.00	-.569D-01	-.366D+07	.381D+03	.000D+00	.214D+04	.655D+13
312.00	-.535D-01	-.389D+07	.386D+03	.000D+00	.245D+04	.655D+13
324.00	-.502D-01	-.411D+07	.390D+03	.000D+00	.275D+04	.655D+13
336.00	-.471D-01	-.430D+07	.394D+03	.000D+00	.306D+04	.655D+13
348.00	-.440D-01	-.448D+07	.398D+03	.000D+00	.337D+04	.655D+13
360.00	-.410D-01	-.463D+07	.401D+03	.000D+00	.367D+04	.655D+13
372.00	-.381D-01	-.476D+07	.404D+03	.000D+00	.398D+04	.655D+13
384.00	-.353D-01	-.487D+07	.407D+03	.000D+00	.428D+04	.655D+13
396.00	-.326D-01	-.496D+07	.408D+03	.000D+00	.459D+04	.655D+13
408.00	-.300D-01	-.502D+07	.410D+03	.000D+00	.490D+04	.655D+13
420.00	-.276D-01	-.507D+07	.411D+03	.000D+00	.520D+04	.655D+13
432.00	-.252D-01	-.509D+07	.411D+03	.000D+00	.551D+04	.655D+13
444.00	-.230D-01	-.509D+07	.411D+03	.000D+00	.581D+04	.655D+13
456.00	-.209D-01	-.508D+07	.411D+03	.000D+00	.612D+04	.655D+13
468.00	-.189D-01	-.504D+07	.410D+03	.000D+00	.643D+04	.655D+13
480.00	-.170D-01	-.499D+07	.409D+03	.000D+00	.673D+04	.655D+13
492.00	-.152D-01	-.492D+07	.408D+03	.000D+00	.704D+04	.655D+13
504.00	-.135D-01	-.484D+07	.406D+03	.000D+00	.734D+04	.655D+13
516.00	-.119D-01	-.474D+07	.404D+03	.000D+00	.765D+04	.655D+13
528.00	-.104D-01	-.463D+07	.401D+03	.000D+00	.796D+04	.655D+13
540.00	-.907D-02	-.450D+07	.399D+03	.000D+00	.826D+04	.655D+13
552.00	-.779D-02	-.437D+07	.396D+03	.000D+00	.857D+04	.655D+13
564.00	-.662D-02	-.422D+07	.393D+03	.000D+00	.887D+04	.655D+13
576.00	-.553D-02	-.407D+07	.389D+03	.000D+00	.918D+04	.655D+13
588.00	-.454D-02	-.391D+07	.386D+03	.000D+00	.949D+04	.655D+13
600.00	-.363D-02	-.374D+07	.382D+03	.000D+00	.979D+04	.655D+13
612.00	-.280D-02	-.357D+07	.379D+03	.000D+00	.174D+05	.655D+13
624.00	-.206D-02	-.339D+07	.375D+03	.000D+00	.180D+05	.655D+13
636.00	-.138D-02	-.320D+07	.371D+03	.000D+00	.185D+05	.655D+13
648.00	-.781D-03	-.301D+07	.367D+03	.000D+00	.190D+05	.655D+13
660.00	-.244D-03	-.282D+07	.363D+03	.000D+00	.195D+05	.655D+13
672.00	.230D-03	-.263D+07	.358D+03	.000D+00	.201D+05	.655D+13
684.00	.646D-03	-.244D+07	.354D+03	.000D+00	.206D+05	.655D+13
696.00	.101D-02	-.225D+07	.350D+03	.000D+00	.211D+05	.655D+13
708.00	.132D-02	-.206D+07	.346D+03	.000D+00	.216D+05	.655D+13
720.00	.159D-02	-.188D+07	.342D+03	.000D+00	.222D+05	.655D+13
732.00	.182D-02	-.170D+07	.339D+03	.000D+00	.227D+05	.655D+13
744.00	.201D-02	-.153D+07	.335D+03	.000D+00	.232D+05	.655D+13
756.00	.216D-02	-.137D+07	.331D+03	.000D+00	.238D+05	.655D+13
768.00	.229D-02	-.121D+07	.328D+03	.000D+00	.243D+05	.655D+13
780.00	.239D-02	-.106D+07	.325D+03	.000D+00	.248D+05	.655D+13
792.00	.246D-02	-.919D+06	.322D+03	.000D+00	.253D+05	.655D+13
804.00	.252D-02	-.787D+06	.319D+03	.000D+00	.259D+05	.655D+13
816.00	.256D-02	-.665D+06	.316D+03	.000D+00	.264D+05	.655D+13
828.00	.258D-02	-.551D+06	.314D+03	.000D+00	.269D+05	.655D+13
840.00	.259D-02	-.448D+06	.312D+03	.000D+00	.275D+05	.655D+13
852.00	.259D-02	-.356D+06	.310D+03	.000D+00	.280D+05	.655D+13
864.00	.258D-02	-.273D+06	.308D+03	.000D+00	.285D+05	.655D+13
876.00	.257D-02	-.201D+06	.306D+03	.000D+00	.290D+05	.655D+13

888.00	.256D-02	-.140D+06	.305D+03	.000D+00	.296D+05	.655D+13
900.00	.254D-02	-.900D+05	.304D+03	.000D+00	.301D+05	.655D+13
912.00	.251D-02	-.508D+05	.303D+03	.000D+00	.306D+05	.655D+13
924.00	.249D-02	-.227D+05	.302D+03	.000D+00	.312D+05	.655D+13
936.00	.247D-02	-.570D+04	.302D+03	.000D+00	.317D+05	.655D+13
948.00	.244D-02	.000D+00	.302D+03	.000D+00	.322D+05	.655D+13

✓ [illegible] 2/90

OUTPUT VERIFICATION

THE MAXIMUM MOMENT IMBALANCE FOR ANY ELEMENT = -.537D-06 IN-LBS
THE MAX. LATERAL FORCE IMBALANCE FOR ANY ELEMENT = .715D-07 LBS

COMPUTED LATERAL FORCE AT PILE HEAD = -.25700D+05 LBS
COMPUTED ROTATIONAL STIFFNESS AT PILE HEAD = .19640D+11 IN-LBS
COMPUTED SLOPE AT PILE HEAD = .20431D-03

THE OVERALL MOMENT IMBALANCE = -.230D-05 IN-LBS
THE OVERALL LATERAL FORCE IMBALANCE = .192D-07 LBS

OUTPUT SUMMARY

PILE HEAD DEFLECTION = -.148D+00 IN
MAXIMUM BENDING MOMENT = -.509D+07 IN-LBS
MAXIMUM TOTAL STRESS = .700D+03 LBS/IN**2
MAXIMUM SHEAR FORCE = .160D+05 LBS
NO. OF ITERATIONS = 2
MAXIMUM DEFLECTION ERROR = .000D+00 IN

1 NORTHERN AVE. SHAFTS WITHOUT SCOUR

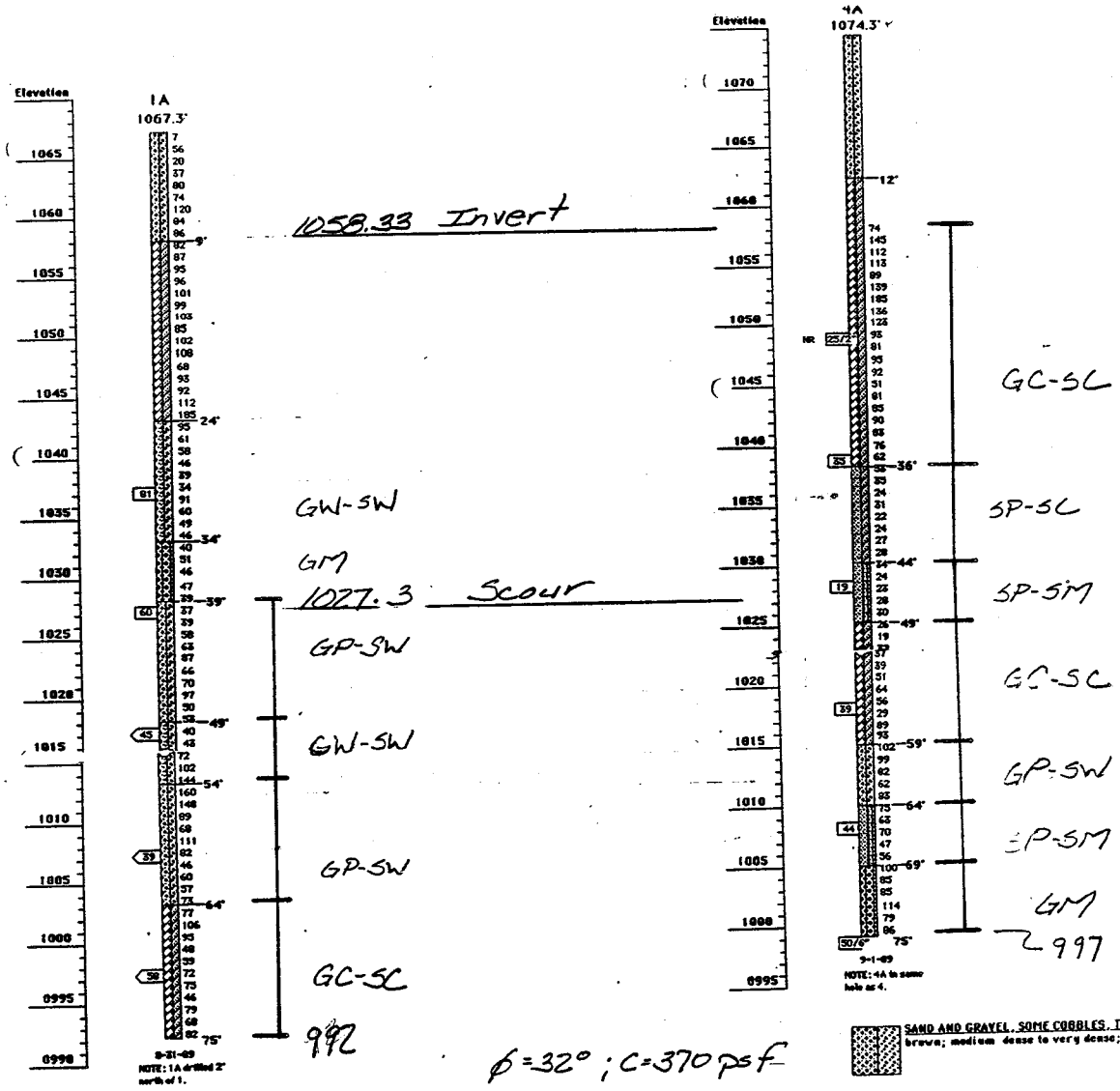
S U M M A R Y T A B L E
****

LATERAL LOAD (LBS)	BOUNDARY CONDITION BC2	AXIAL LOAD (LBS)	YT (IN)	ST (IN/IN)	MAX. MOMENT (IN-LBS)	MAX. STRESS (LBS/IN**2)
-.295D+05	.186D+11	.175D+07	-.173D+00	.245D-03	-.593D+07	.833D+03
-.257D+05	.196D+11	.144D+07	-.148D+00	.204D-03	-.509D+07	.700D+03

From Soils Report

Sta. 48+08 ℄ Abut 2

Sta 45+17 ℄ Pier 1



$\phi = 32°$; $C = 370$ psf

$\phi = 40°$; $C = 480$ psf

$\phi = 34°$; $C = 563$ psf

$\phi \approx 40$

$\phi \approx 34$

$\phi = 36°$; $C = 0$ psf

ϕ: internal angle of friction and C: Cohesion taken from soils analysis

JOB Northern Ave @ Nor River SHEET NO. S-1

MADE BY T. Stones DATE 11-2-89 JOB NO. 33-8962

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 187

CALCULATIONS FOR Shaft Design

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MINNEAPOLIS PHOENIX

From NAVFAC 7.1 Fig 22

TABLE 6
Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids(1)					Unit Weight(2) (lb./cu.ft.)							
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight		
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense	
GRANULAR MATERIALS																	
Uniform Materials																	
a. Equal spheres (theoretical values)	-	-	-	1.0	0.92	-	0.35	47.6	26	-	-	-	-	-	-	-	
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	-	110	93	131	57	69	
c. Clean, uniform SAND (fine or medium)	-	-	-	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73	SP-SM
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	-	0.40	52	29	80	-	118	81	136	51	73	
Well-graded Materials																	
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	-	0.30	47	23	87	122	127	88	142	54	79	GW-SW
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86	
c. Micaceous SAND	-	-	-	-	1.2	-	0.40	55	29	76	-	120	77	138	48	76	
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	-	0.14	46	12	89	-	146(3)	90	155(3)	56	92	GM
MIXED SOILS																	
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	-	0.25	64	20	60	130	135	100	147	38	85	SP-SC
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	-	-	1.0	-	0.20	50	17	84	-	140	115	151	53	89	
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	-	0.13	41	11	100	140	148(4)	125	156(4)	62	94	GP-SW GC-SC
CLAY SOILS																	
CLAY (30%-50% clay sizes)	0.05	0.5μ	0.001	-	2.4	-	0.50	71	33	50	105	112	94	133	31	71	
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	-	-	12	-	0.60	92	37	13	90	106	71	128	8	66	
ORGANIC SOILS																	
Organic SILT	-	-	-	-	3.0	-	0.55	75	35	40	-	110	87	131	25	69	
Organic CLAY (30% - 50% clay sizes)	-	-	-	-	4.4	-	0.70	81	41	30		100	81	125	18	62	

Layer 1

Layer 2 PICT

MINNEAPOLIS PHOENIX

JOB Northern Ave C New River SHEET NO. 5-2

MADE BY T Stokes DATE 11-2-89 JOB NO. 33-89-62

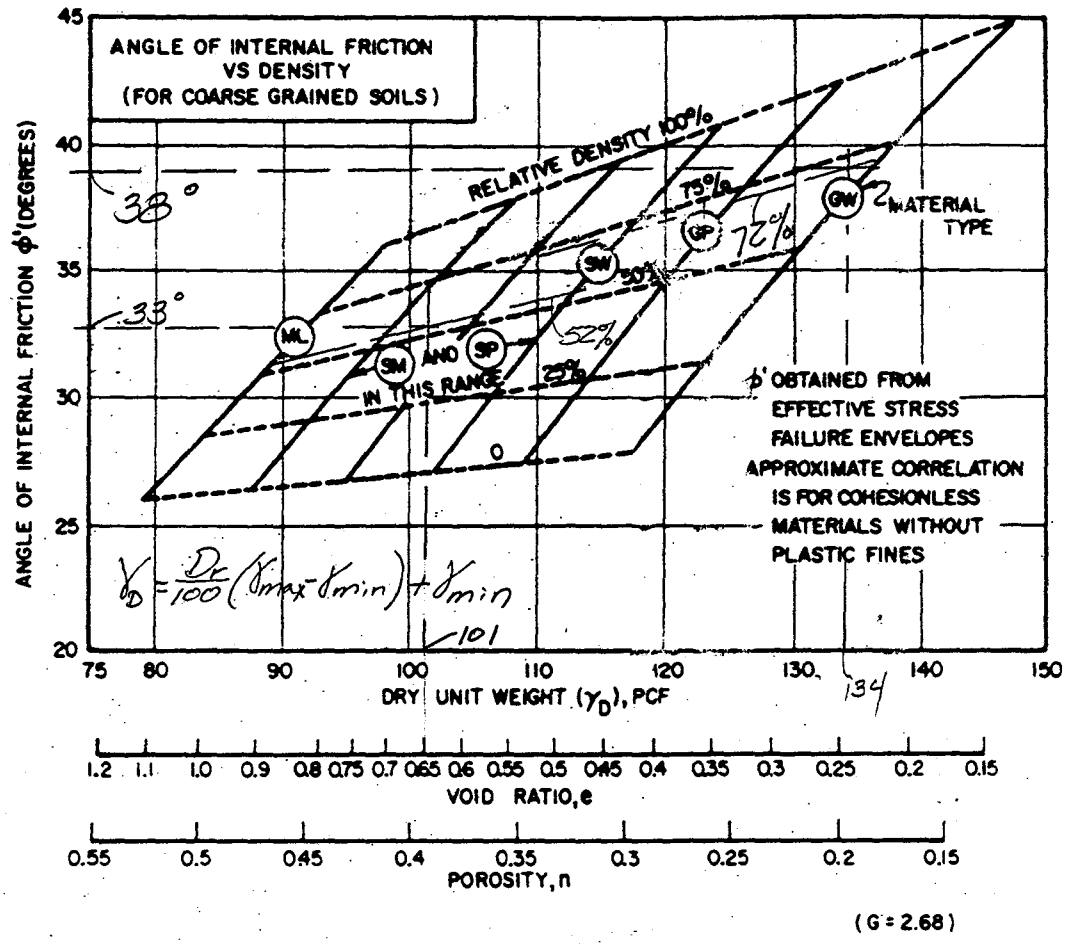
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 188

CALCULATIONS FOR Shaft Design

Using Boring 4A for pier loading the following values are from NAVFAC 7.1-22 Table 6 on Sht 5-2

	Relative Dens. from soil test	Unit Weight Submerg. pcf		Unit Weight Dry pcf		γ_D
	Dr	Min.	Max	Min	Max	
Layer 1 from 0 to 2 ft below scour SP-SM	52%	52	73	83	118	101
Layer 2 below 2 ft below scour GP-SW, GC-SC; GM, GW-SW	72%	62	94	100	148	134

From NAVFAC 7.1-149 Based on Soil Analysis



$$\gamma_D = \frac{D_r}{100}(\gamma_{max} - \gamma_{min}) + \gamma_{min}$$

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. 5-3
MADE BY T Stones DATE 11-2-89 JOB NO. 338962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 189
CALCULATIONS FOR Shaft Design

Results of NAVFAC DM 7.1 $\gamma_{sat} = \frac{D_r}{100}(\gamma_{sat\,max} - \gamma_{sat\,min}) + \gamma_{sat\,min}$

$\gamma_{sub} = \gamma_D$ RD

Layer 1: top 2 ft below scour : $\gamma_{sat} = 63$ pcf $\phi = 33°$

Layer 2: remainder of shaft below scour : $\gamma_{sat} = 85$ pcf $\phi = 38°$

From NAVFAC DM 7.2 pg 192 to 194

For Bearing Capacity: $Q_{ult} = P_T N_q A_T$ no side friction allowed

Q_{ult} = Ultimate Load Capacity in Compression
P_T = Effective Vertical stress at pile tip
N_q = Bearing Capacity Factor at pile tip
A_T = Area of Pile tip

From NAVFAC 7.2-194 Figure 1 For Drilled Piers

$\phi = 33°$ $N_q = 17$

$\phi = 38°$ $N_q = 43$

$A_T = \frac{\pi(6.5\,ft)^2}{4} = 33.2\ ft^2$

Check P_T & Q_{ULT} @ Base of Shaft: 35 ft below Scour

$P_{T_{7.5}} = 2.0(63) + 33(85) = 2931$ psf

$Q_{ULT_{7.5}} = (2.931\ ksf)(43)(33.2) = 4185^k$

Grp I $Q_{req} = 1773^k$ see sht com7 This Axial Load is
Grp IV $Q_{req} = 1463^k$
From the following loading

Grp I = Dead Load + Live Load w/o impact + Differential Settlement + Stream Flow.
Grp IV = Grp I + .3(Wind) + Thermal and reduced by 1.25 factor

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JOB Northern Ave @ New River SHEET NO. S4
MADE BY T Stones DATE 11-2-89 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 190
CALCULATIONS FOR Shaft Design

For Bearing capacity I will also include the effects of buoyancy associated with the stream flow as follows.

Water Surface Elev = 1073.7 } 15.4 L.F. of 5'ϕ column
Invert Elev = 1058.3 } 31.0 L.F. of 6⅝'ϕ shaft
Scour Elev 1027.3

$$\text{Volume} = \frac{\pi}{4}\left[(5)^2(15.4) + (6\tfrac{5}{8})^2(31)\right] = 1331\ ft^3$$

$$\text{Buoyancy} = 1331\ ft\left(\frac{0.0624^k}{ft}\right) = 83.0\ k$$

Due to long term action of Differential Settlement, the effects of this will be included in Group IV Loading:

The 1463^k Axial Load shown on Sht S4 includes the M-Strudl Load Case Dif_Set2 and has been reduced by 125 % factor.

Group IV $Q_{req} = 1463^k - 83/1.25^k = 1397\ k$

$FS = \frac{4185}{1397} = 3.0$ controls shaft Depth

Group I Differential Settlement is included in the 1773^k Axial Load Shown on sht S-4. On Sht S-4B, the M-Strudl anaylis for Differential Settlement Loads at scour is shown and will be deducted from the Group I Load Case

$Q_{req} = 1773 - 714 - 83 = 976^k$

$FS = \frac{4185}{976} = 4.3$

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JOB Northern Ave & New River SHEET NO. S 4A
MADE BY T Stones DATE 1-16-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 191
CALCULATIONS FOR Shaft Design

****
* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES ------

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
11 SHAFT	DIF_SET1	15	-55.09	5.82	-105.68
		11	55.09	-5.82	117.32
	DIF_SET2	15	-276.38	-2.55	46.69
		11	276.38	2.55	-51.78
17 SHAFT	DIF_SET1	16	-273.51	4.87	-69.61
		12	273.51	-4.87	79.34
	DIF_SET2	16	1267.13	-7.68	133.39
		12	-1267.13	7.68	-148.75
23 SHAFT	DIF_SET1	17	714.72	-2.63	75.01
		13	-714.72	2.63	-80.27
	DIF_SET2	17	-1705.48	-0.68	6.73
		13	1705.48	0.68	-8.09
29 SHAFT	DIF_SET1	18	-386.12	-8.06	158.67
		14	386.12	8.06	-174.78
	DIF_SET2	18	714.72	10.91	-194.97
		14	-714.72	-10.91	216.78

Axial Load Due to Differential Settlement included in Qreq shown on sht S-4

Northern Ave @ New River
T Stones 1-16-90
L Arnold 2/90
Shaft Design

S-4B
33-8962
192

Columns and Shafts will be Design by allowable Stress Methods per AASHTO §8.15.4

Slenderness Effects in Accordance with 8.16.5.

8.16.5.2.1 Unsupported Length

$E = 57\sqrt{4000} = 3605$ ksi

$l_{u\,cap} = 24 - 5 = 19$ ft

$I_{cap} = 6(4)^3/12 = 32$ ft^4

$l_{u\,column} = 18^{\underline{2}}$ ft

$l_{u\,shaft} = 31^{\underline{0}}$ ft $\Big\}\ 49^{\underline{2}}$

$I_{col} = \frac{\pi}{64}(5)^4 = 30.68$ ft^4

$I_{shaft} = \frac{\pi}{64}(6.5)^4 = 87.62$ ft^4

8.16.5.2.2 Radius of Gyration $r = 0.25(5\text{ ft}) = 1.25$ ft

8.16.5.2.3 Determination of k

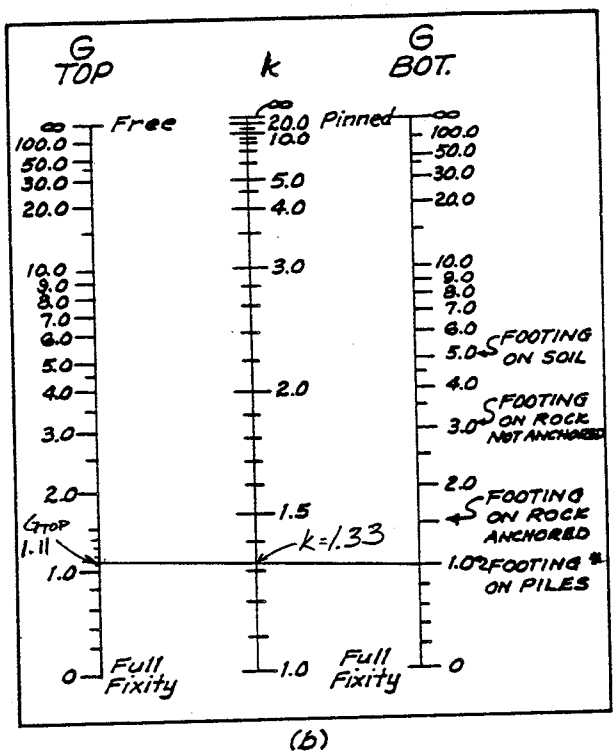
$G_{cap} = EI/l_u = 3605(32)/19 = 6071.6$ ksi

$G_{col+shaft} = 3605\left[\frac{30.68(18^2) + 87.62(31^2)}{79^3}\right]\Big/49^2 = 5465$ ksi

$I_{comp} = 74.6$ ft^4

$G_{TOP} = \frac{6071.6}{5465} = 1.11$

$G_{BOT} = 1.0$



(b)
Unbraced Frame

8.16.5.2.5 Determine kl_u^*/r

use $l_u^* = (l_u + 10)$

$kl_u^*/r = 1.33\,(49^{\underline{2}} + 10)/1.25 = 63$ ft

$22 \leq 63 < 100$

∴ Design as long Column

* For column analysis only, when a pile supported footing is used add ten feet to the column length. ie: effective length of column = $k(l_u + 10)$. Do not use $(l + 10)$ in solving for k.

JOB Northern Ave @ New River SHEET NO. S 5

MADE BY T Stones DATE 1-16-90 JOB NO. 33-8962

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 193

CALCULATIONS FOR Shaft Design

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8.16.5.2.7 Moment Magnification Factors

$$\delta_s = \frac{1}{1 - \frac{\Sigma P_u}{\phi \Sigma P_c}}$$

As Per 8.15.4 $\phi = 1.0$ $P_u = 2.5$ times Axial Load

$$P_c = \frac{\pi E I}{k(l_u)^2} = \frac{\pi (3605\ ksi)(74.6\ ft^4)}{1.33\ (49.2\ ft)^2} \cdot \frac{144\ in^2}{ft^2} = 37{,}790\ k$$

$$\delta = \frac{1}{1 - \frac{2.5(P_x)}{37{,}790}}$$

M-Strudl Analysis with scour is critical for column design.

AASHTO GROUP	M_T ft·k	P_x	$2.5\ P_x$	δ	Magnified Moment	Axial Load
1	2206	1516	3790	1.111	2492	1516
2	1749	1086	2715	1.077	1885	1086
3	1708	640	1600	1.044	1784	640
4	1816	1215	3038	1.087	1975	1215
5	1608	972	2430	1.069	1719	972
6	1571	574	1435	1.039	1633	574

Moment Interaction diagram will be made using results from RCBIAX4 Computer program with strength reduction factors input as 1.0.

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JOB Northern Ave @ New River SHEET NO. S 6
MADE BY T Stones DATE 1-16-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 194
CALCULATIONS FOR Shaft Design

RCBIAXU Program Input

Card 1

1. % Interaction Diagram: Working Stress: 5 ft dia.

Card 2 f'_c, f_y, E_c, E_s, e_u, k_1

f'_c = 4000 psi

f_y = 60,000 psi

E_c = 3605000 psi

E_s = 29,000,000 psi

e_u = 0.003

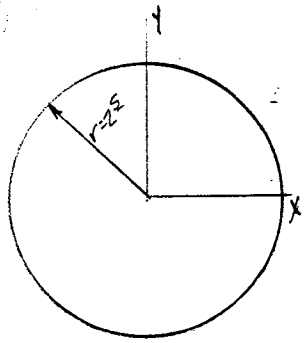
k_1 = 0.85

Card 3: ϕ_p, ϕ_c, ϕ_t All ϕ's = 1.0 for Working Stress

Card 4: Units, Iref

Units = 12 Iref = 1

Card 5: NP = 89



$x^2+y^2=30^2$
(900)

Card 6 →

PT #	X	Y	Neg X PT #	Pt #	X	Y	Neg X PT #	Pt #	X	Y	Neg X Pt #
1	0	30.0	89	11	20	22.89	79	21	29.73	4	69
45	0	-30.0		35	20	-22.89	55	25	29.73	-4	65
2	2	29.93	88	12	20.98	22	78	22	29.93	2	68
44	2	-29.93	46	34	20.98	-22	56	24	29.93	-2	66
3	4	29.73	87	13	22.89	20	77	23	30.0	0	67
43	4	-29.73	47	33	22.89	-20	57				
4	6	29.39	86	14	24.0	18	76				
42	6	-29.39	48	32	24.0	-18	58				
5	8	28.91	85	15	25.38	16	75				
41	8	-28.91	49	31	25.38	-16	59				
6	10	28.28	84	16	26.53	14	74				
40	10	-28.28	50	30	26.53	-14	60				
7	12	27.50	83	17	27.50	12	73				
39	12	-27.50	51	29	27.50	-12	61				
8	14	26.53	82	18	28.28	10	72				
38	14	-26.53	52	28	28.28	-10	62				
9	16	25.38	81	19	28.91	8	71				
37	16	-25.38	53	27	28.91	-8	63				
10	18	24.0	80	20	29.39	6	70				
36	18	-24.0	54	26	29.39	-6	64				

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JOB Northern Ave @ New River SHEET NO. S7

MADE BY T Stones DATE 1-17-90 JOB NO. 33-8962

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 195

CALCULATIONS FOR Shaft Design

Card 7 NR = 19

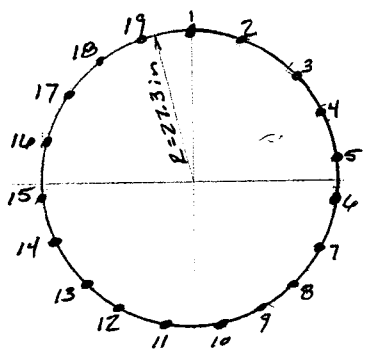
Try 1% steel $A_{col} = 2827.4\ in^2$ $A_{1\%} = 28.27\ in^2$

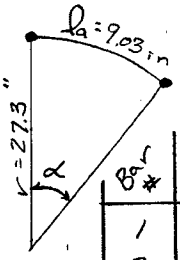
Use #11 bars 19 bars req'd

Examine easy pattern to discontinue bars from shaft.

$A_{shaft} = 4778\ in^2$ $A_{1\%} = 47.78\ in^2$

requires 31 # 11 bars use 38 # 11 bars in shaft and discontinue half @ 6'-0" above column end.





$\frac{\alpha}{360°} = \frac{9.03}{2\pi r}$ ∴ $\alpha = 18.94°$

Card 8

Bar #	X	Y	Bar #	X	Y
1	0	0	11	-4.48	-26.93
2	8.86	25.82	12	-12.98	-24.02
3	16.77	21.55	13	-20.07	-18.50
4	22.85	14.94	14	-24.99	-10.99
5	26.46	6.71	15	-27.20	-2.28
6	27.21	-2.25	16	-26.47	6.68
7	25.00	-10.96	17	-22.87	14.91
8	20.09	-18.48	18	-16.79	21.53
9	13.0	-24.0	19	-8.89	25.91
10	4.51	-26.93			

Card 9 NL = 6

Card 10 P, M_x, M_y, XA, YA

Refer to M-Strudl page 20 for Load Cases

-1545,	2218,	0.0,	0.0,	0.0
-1126,	1759,			
-640,	1708,			
-1255,	1824,			
-1008,	1617,			
-574,	1571,			

Card 11 0

BRW MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. S8

MADE BY T Stones DATE 1-17-90 JOB NO. 33-8962

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 196

CALCULATIONS FOR Shaft Design

RCBIAXU Reinforced Concrete Biaxial Bending Analysis
NORTH. AVE @ NEW RIVER; WORK. STRESS COL DES 1%

✓ [illegible] 2/90

SUMMARY OF INPUT DATA

Design Parameters

f'c = 4000.0 psi fy = 60000.0 psi
Ultimate concrete strain = .00300
Ratio of max. concrete stress to f'c = .850
Ec = 3605.0 ksi Es = 29000.0 ksi
Undercapacity factor in compression = 1.000
Undercapacity factor in tension = 1.000
Maximum axial force = 1.000*P0

Concrete Outline

Point	x (inches)	y (inches)
1	.0000	30.0000
2	2.0000	29.9300
3	4.0000	29.7300
4	6.0000	29.3900
5	8.0000	28.9100
6	10.0000	28.2800
7	12.0000	27.5000
8	14.0000	26.5300
9	16.0000	25.3800
10	18.0000	24.0000
11	20.0000	22.8900
12	20.9800	22.0000
13	22.8900	20.0000
14	24.0000	18.0000
15	25.3800	16.0000
16	26.5300	14.0000
17	27.5000	12.0000
18	28.2800	10.0000
19	28.9100	8.0000
20	29.3900	6.0000
21	29.7300	4.0000
22	29.9300	2.0000
23	30.0000	.0000
24	29.9300	-2.0000
25	29.7300	-4.0000
26	29.3900	-6.0000
27	28.9100	-8.0000
28	28.2800	-10.0000
29	27.5000	-12.0000
30	26.5300	-14.0000
31	25.3800	-16.0000
32	24.0000	-18.0000
33	22.8900	-20.0000
34	20.9800	-22.0000
35	20.0000	-22.8900
36	18.0000	-24.0000
37	16.0000	-25.3800
38	14.0000	-26.5300
39	12.0000	-27.5000
40	10.0000	-28.2800
41	8.0000	-28.9100
42	6.0000	-29.3900

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43	4.0000	-29.7300
44	2.0000	-29.9300
45	.0000	-30.0000
46	-2.0000	-29.9300
47	-4.0000	-29.7300
48	-6.0000	-29.3900
49	-8.0000	-28.9100
50	-10.0000	-28.2800
51	-12.0000	-27.5000
52	-14.0000	-26.5300
53	-16.0000	-25.3800
54	-18.0000	-24.0000
55	-20.0000	-22.8900
56	-20.9800	-22.0000
57	-22.8900	-20.0000
58	-24.0000	-18.0000
59	-25.3800	-16.0000
60	-26.5300	-14.0000
61	-27.5000	-12.0000
62	-28.2800	-10.0000
63	-28.9100	-8.0000
64	-29.3900	-6.0000
65	-29.7300	-4.0000
66	-29.9300	-2.0000
67	-30.0000	.0000
68	-29.9300	2.0000
69	-29.7300	4.0000
70	-29.3900	6.0000
71	-28.9100	8.0000
72	-28.2800	10.0000
73	-27.5000	12.0000
74	-26.5300	14.0000
75	-25.3800	16.0000
76	-24.0000	18.0000
77	-22.8900	20.0000
78	-20.9800	22.0000
79	-20.0000	22.8900
80	-18.0000	24.0000
81	-16.0000	25.3800
82	-14.0000	26.5300
83	-12.0000	27.5000
84	-10.0000	28.2800
85	-8.0000	28.9100
86	-6.0000	29.3900
87	-4.0000	29.7300
88	-2.0000	29.9300
89	.0000	30.0000

Concrete outline properties:
Area = 2834.744 in2
I(x) = 639515.439 in4 I(y) = 639507.845 in4
centroid : x = .0000 in. , y = .0000 in.

Reinforcement

Row	No. of Bars	Area/Bar (in2)	x1 (inches)	y1 (inches)	x2 (inches)	y2 (inches)
1	1	1.000	.0000	.0000	.0000	.0000
2	1	1.000	8.8600	25.8200	.0000	.0000
3	1	1.000	16.7700	21.5500	.0000	.0000
4	1	1.000	22.8500	14.9400	.0000	.0000
5	1	1.000	26.4600	6.7100	.0000	.0000
6	1	1.000	27.2100	-2.2500	.0000	.0000
7	1	1.000	25.0000	-10.9600	.0000	.0000
8	1	1.000	20.0900	-18.4800	.0000	.0000
9	1	1.000	13.0000	-24.0000	.0000	.0000
10	1	1.000	4.5100	-26.9300	.0000	.0000
11	1	1.000	-4.4800	-26.9300	.0000	.0000
12	1	1.000	-12.9800	-24.0200	.0000	.0000
13	1	1.000	-20.0700	-18.5000	.0000	.0000
14	1	1.000	-24.9900	-10.9900	.0000	.0000
15	1	1.000	-27.2000	-2.2800	.0000	.0000
16	1	1.000	-26.4700	6.6800	.0000	.0000
17	1	1.000	-22.8700	14.9100	.0000	.0000
18	1	1.000	-16.7900	21.5300	.0000	.0000
19	1	1.000	-8.8900	25.9100	.0000	.0000

% Reinforcement = 1.046

Reference point (x,y) for moments = .0000, .0000

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INTERACTION DIAGRAM POINTS (Nominal Values)
* indicates balanced point M(x),M(y) = 1.000, .000

Point	Axial Force (kips)	X-Axis Moment (kip-ft)	Y-Axis Moment (kip-ft)	Strength Reduction Factor	No. of Iterations
1	-10713.531	128.718	.047	1.0000	1
2	-10476.460	516.707	.002	1.0000	2
3	-10239.390	964.664	-.001	1.0000	2
4	-10002.319	1394.576	-.001	1.0000	2
5	-9765.248	1805.922	-.006	1.0000	1
6	-9528.178	2201.715	-.021	1.0000	1
7	-9291.107	2581.482	.004	1.0000	2
8	-9054.036	2945.500	.004	1.0000	1
9	-8816.966	3290.151	.005	1.0000	1
10	-8579.896	3614.279	.018	1.0000	1
11	-8342.825	3917.805	.032	1.0000	1
12	-8105.754	4200.773	.036	1.0000	1
13	-7868.684	4463.274	.032	1.0000	1
14	-7631.613	4705.632	.035	1.0000	1
15	-7394.542	4928.205	.038	1.0000	1
16	-7157.472	5131.551	.041	1.0000	1
17	-6920.401	5316.135	.045	1.0000	1
18	-6683.330	5482.695	.035	1.0000	1
19	-6446.260	5631.702	.039	1.0000	1
20	-6209.189	5763.452	.043	1.0000	1
21	-5972.119	5878.476	.000	1.0000	2
22	-5735.048	5977.441	.005	1.0000	1
23	-5497.977	6060.645	.010	1.0000	1
24	-5260.907	6127.468	.002	1.0000	2
25	-5023.836	6178.128	.005	1.0000	1
26	-4786.765	6215.018	.008	1.0000	1
27	-4549.695	6238.840	.011	1.0000	1
28*	-4525.023	6240.598	.022	1.0000	1
29	-4312.624	6230.476	.004	1.0000	2
30	-4075.554	6205.341	-.014	1.0000	1
31	-3838.483	6151.488	.039	1.0000	1
32	-3601.412	6074.861	.010	1.0000	1
33	-3364.342	5983.833	-.023	1.0000	1
34	-3127.271	5871.244	.000	1.0000	2
35	-2890.200	5728.084	-.041	1.0000	1
36	-2653.130	5569.478	.002	1.0000	2
37	-2416.059	5396.431	-.049	1.0000	1
38	-2178.988	5201.766	.006	1.0000	2
39	-1941.918	4976.058	-.049	1.0000	1
40	-1704.847	4733.485	.011	1.0000	2
41	-1467.777	4472.981	.007	1.0000	2
42	-1230.706	4185.309	.009	1.0000	2
43	-993.636	3858.525	.000	1.0000	2
44	-756.565	3513.089	.006	1.0000	2
45	-519.494	3151.350	.018	1.0000	2
46	-282.424	2762.816	.001	1.0000	2
47	-45.353	2338.493	.012	1.0000	2
48	191.717	1896.440	.000	1.0000	3
49	428.788	1429.533	.000	1.0000	3
50	665.859	929.675	.000	1.0000	3
51	902.930	412.781	.000	1.0000	3
52	1140.000	-136.450	-.050	1.0000	1

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RESULTS OF LOAD ANALYSIS

Factored Loads : Force = -1565.000 kips
M(x) = 2218.000 kip-ft
M(y) = .000 kip-ft

Nominal Loads : Force = -1565.000 kips
M(x) = 2218.000 kip-ft
M(y) = .000 kip-ft
Strength Reduction Factor = 1.0000

Max. concrete strain = -.00056 (.188 of ultimate)
at (x,y) = (.0000, -30.0000)

Max. steel strain = .00033 (.161 of yield)
at (x,y) = (-8.8900, 25.9100)

Strain constants (C1,C2,C3) = .00000000 .00001604 -.00008348
where, strain = C1*x + C2*y + C3
No. of iterations = 5

Portion of balanced reinforcement = .059

✓ Lee Rhuall 2/90

RESULTS OF LOAD ANALYSIS

Factored Loads : Force = -1126.000 kips
M(x) = 1759.000 kip-ft
M(y) = .000 kip-ft

Nominal Loads : Force = -1126.000 kips
M(x) = 1759.000 kip-ft
M(y) = .000 kip-ft
Strength Reduction Factor = 1.0000

Max. concrete strain = -.00046 (.154 of ultimate)
at (x,y) = (.0000, -30.0000)

Max. steel strain = .00034 (.165 of yield)
at (x,y) = (-8.8900, 25.9100)

Strain constants (C1,C2,C3) = .00000000 .00001438 -.00003167
where, strain = C1*x + C2*y + C3
No. of iterations = 5

Portion of balanced reinforcement = .059

✓ 2/90

RESULTS OF LOAD ANALYSIS

Factored Loads : Force = -640.000 kips
M(x) = 1708.000 kip-ft
M(y) = .000 kip-ft

Nominal Loads : Force = -640.000 kips
M(x) = 1708.000 kip-ft
M(y) = .000 kip-ft
Strength Reduction Factor = 1.0000

Max. concrete strain = -.00056 (.188 of ultimate)
at (x,y) = (.0000, -30.0000)

Max. steel strain = .00092 (.445 of yield)
at (x,y) = (-8.8900, 25.9100)

Strain constants (C1,C2,C3) = .00000000 .00002655 .00023384
where, strain = C1*x + C2*y + C3
No. of iterations = 6

Portion of balanced reinforcement = .059

✓ 2/90

RESULTS OF LOAD ANALYSIS

```
Factored Loads : Force =    -1255.000 kips
                 M(x)  =     1826.000 kip-ft
                 M(y)  =         .000 kip-ft

Nominal Loads  : Force =    -1255.000 kips
                 M(x)  =     1826.000 kip-ft
                 M(y)  =         .000 kip-ft
                 Strength Reduction Factor = 1.0000


Max. concrete strain = -.00047 (  .156 of ultimate)
at (x,y) = (       .0000,     -30.0000)

Max. steel strain    =  .00029 (  .142 of yield)
at (x,y) = (     -8.8900,      25.9100)

Strain constants (C1,C2,C3) =   .00000000  .00001364 -.00005965
where, strain = C1*x + C2*y + C3
No. of iterations =   5

Portion of balanced reinforcement =  .059
```

✓ Lee Clunell 2/90

RESULTS OF LOAD ANALYSIS

```
Factored Loads : Force =    -1008.000 kips
                 M(x)  =     1617.000 kip-ft
                 M(y)  =         .000 kip-ft

Nominal Loads  : Force =    -1008.000 kips
                 M(x)  =     1617.000 kip-ft
                 M(y)  =         .000 kip-ft
                 Strength Reduction Factor = 1.0000


Max. concrete strain = -.00043 (  .143 of ultimate)
at (x,y) = (       .0000,     -30.0000)

Max. steel strain    =  .00034 (  .162 of yield)
at (x,y) = (     -8.8900,      25.9100)

Strain constants (C1,C2,C3) =   .00000000  .00001370 -.00001950
where, strain = C1*x + C2*y + C3
No. of iterations =   5

Portion of balanced reinforcement =  .059
```

✓ 2/90

RESULTS OF LOAD ANALYSIS

Factored Loads : Force = -574.000 kips
M(x) = 1571.000 kip-ft
M(y) = .000 kip-ft

Nominal Loads : Force = -574.000 kips
M(x) = 1571.000 kip-ft
M(y) = .000 kip-ft
Strength Reduction Factor = 1.0000

Max. concrete strain = -.00052 (.174 of ultimate)
at (x,y) = (.0000, -30.0000)

Max. steel strain = .00087 (.423 of yield)
at (x,y) = (-8.8900, 25.9100)

Strain constants (C1,C2,C3) = .00000000 .00002497 .00022751
where, strain = C1*x + C2*y + C3
No. of iterations = 6

Portion of balanced reinforcement = .059

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Max Moments Occur @ Top of Columns. 1% steel is adequate here so 1% steel will be adequate in larger shaft section as well.

Spirals: use 1/2" (#4) spirals $a_s = 0.2 \text{ in}^2$

AASHTO 8.18.2.2.2

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y}$$

$$A_g = \frac{\pi}{4}(60^2) = 2827.4 \text{ in}^2$$

$$A_c = \frac{\pi}{4}(60-1.5)^2 = 2687.8 \text{ in}^2$$

$$\rho_s = 0.45\left[\frac{2827.4}{2687.8} - 1\right]\frac{4000}{60{,}000} = 0.00156$$

$$\rho_s = \frac{a_s \pi (D_c - d_b)}{(\pi D_c^2/4)\, s}$$

$$s_{max} = \frac{0.2 \text{ in}^2 \cancel{\pi}(58.5 \text{ in} - 0.5 \text{ in})4}{\cancel{\pi}(58.5 \text{ in})^2(0.00156)} = 8.7 \text{ in} \quad (4\tfrac{3}{4})'' \text{ for } \tfrac{3}{8}''\phi$$

AASHTO 8.18.2.2.3 $s_{max} = 3$ in will control

Provide 1 1/2 turns of spiral into column Cap.

Examine: l_d for #11 bars:

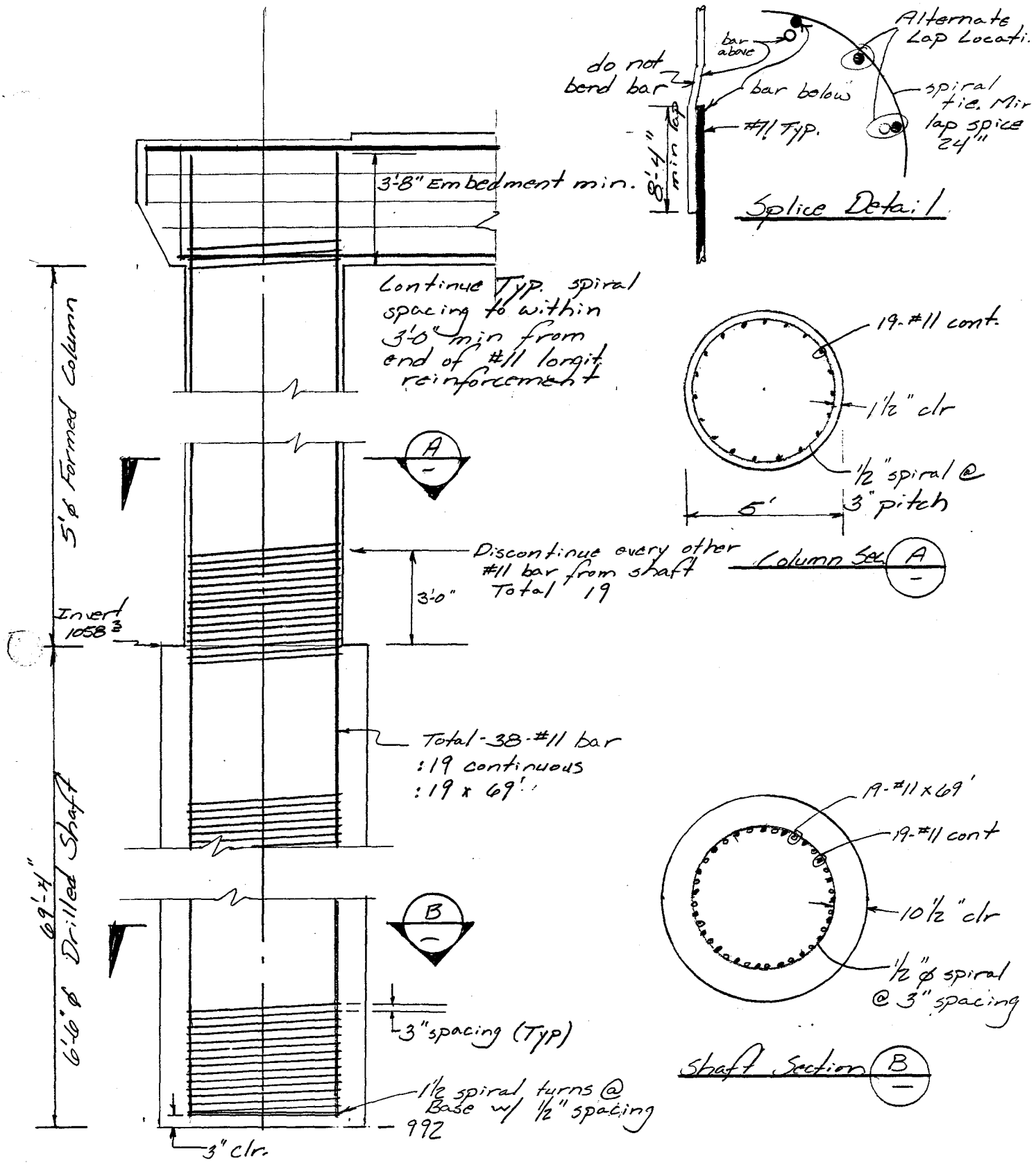
#11 $f_s = 60{,}000$ psi $f'_c = 4000$ psi $l_d = 59.2$ in

Reduction factor 8.25.3.1 applies $l_d = 0.8(59.2) = 47\tfrac{3}{8}$ in

8.25.3.2 and 8.25.3.3 can also apply therefore with 4" min. cover on ends of bars (provide for Lap reinf.) $l_d = 44''$ will be adequate. $\frac{44}{47.375} = .93$ additional reduction

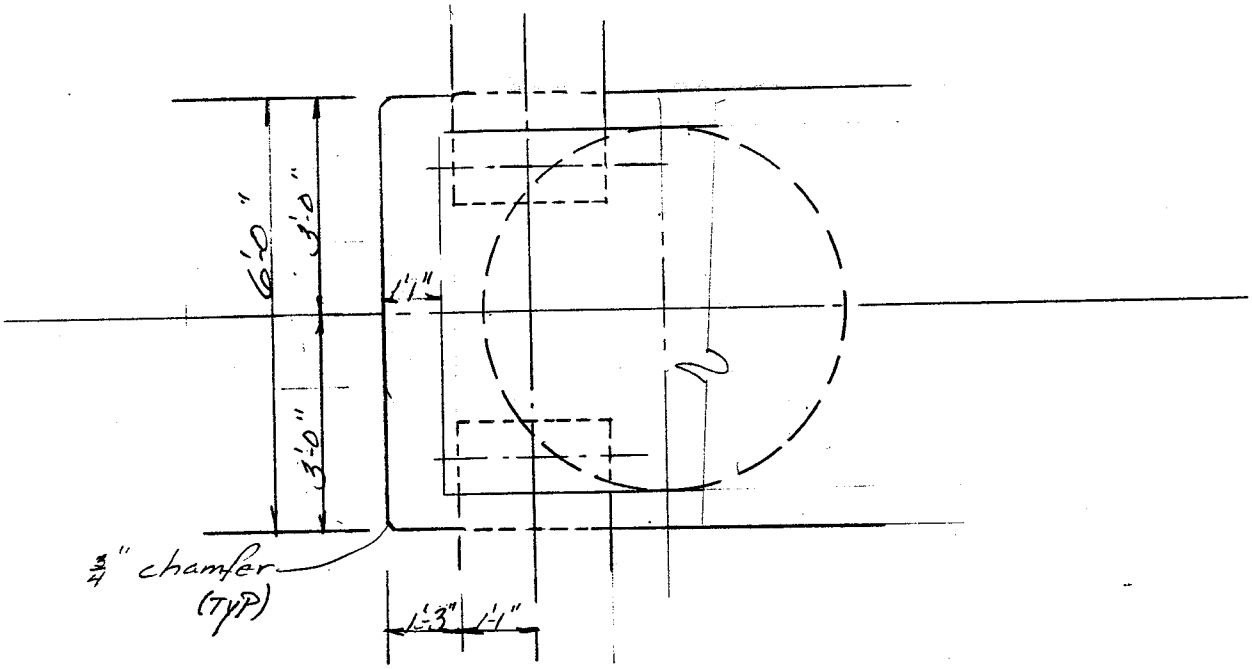
BRW
MINNEAPOLIS PHOENIX

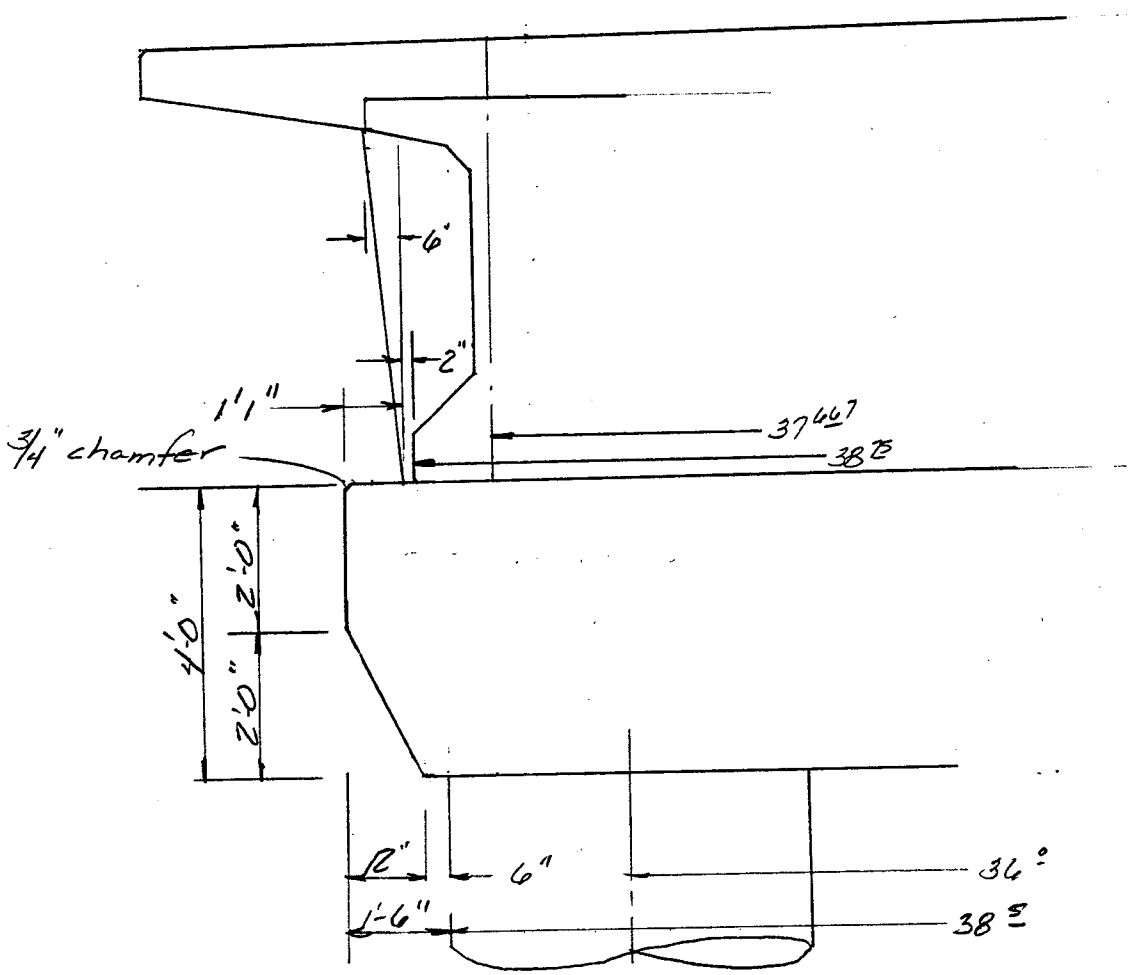
JOB Northern Ave @ New River SHEET NO. S 19
MADE BY T Stones DATE 1-17-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 207
CALCULATIONS FOR Shaft Design



BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. S 20
MADE BY T Stones DATE 1-18-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 208
CALCULATIONS FOR Shaft Design





BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. Cap 1
MADE BY T Stones DATE 1-4-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 209
CALCULATIONS FOR CAP DESIGN

✓ Lee Clemell 2/90

```
*********************************************************************
*                                                                   *
* TYPE OF THE STRUCTURE                    : PLANE FRAME            *
* RESTART STATUS                           :    NONE GIVEN          *
* DESIGNATED UNITS                         : KIPS     FEET   DEGREES *
* NO. OF JOINTS DEFINED                    :     0                  *
* NO. OF MEMBERS DEFINED                   :     0                  *
* NO. OF PLATE ELEMENTS DEFINED            :     0                  *
* NO. OF DIFF. MATERIAL DEFINED            :     0                  *
* NO. OF DIFF. SECTION PROPERTIES DEFINED:      0                  *
* NO. OF DIFF. TYPES OF SPRINGS DEFINED    :     0                  *
*                                                                   *
*********************************************************************
```

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*****************************
* LISTING OF THE INPUT FILE *
*****************************

  1) $ THE FOLLOWING DATA IS THE INPUT FILE FOR THE ANALYSIS OF THE
  2) $ NORTHERN AVENUE BRIDGE , COLUMN BENT CAPS
  3) $
  4) TITLE ANALYSIS OF RIVER PIER BENT CAPS
  5) TYPE PLANE FRAME
  6) UNIT KIP FEET DEGREE
  7) REPORT DEV A:NAOUT20
  8) REPORT PAG 2
  9) $ FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
 10) $ NORTHERN AVENUE BRIDGE OVER THE NEW RIVER
 11) $ BRW, INC.; PHOENIX, ARIZONA
 12) $ JOB NUMBER 33-8962; JANUARY, 1990
 13) REPORT LOGO
 14) PRINT INPUT
 15) OUTPUT DEC 2
 16) OUTPUT BY MEMBER
 17) JOINT COORDINATES
 18) 1 -40.0 1076.48 F
 19) 2 -35.75 1076.48
 20) 3 -11.92 1076.48
 21) 4 11.92 1076.48
 22) 5 35.75 1076.48
 23) 6 40.0 1076.48 F
 24) 7 -35.75 1058.3
 25) 8 -11.92 1058.3
 26) 9 11.92 1058.3
 27) 10 35.75 1058.3
 28) 11 -35.75 1027.3
 29) 12 -11.92 1027.3
 30) 13 11.92 1027.3
 31) 14 35.75 1027.3
 32) 15 -35.75 1025.3
 33) 16 -11.92 1025.3
```

✓ Lee Ahrendt 2/90

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****************************
* LISTING OF THE INPUT FILE *
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 34) 17 11.92 1025.3
 35) 18 35.75 1025.3
 36) 19 -35.75 1015.3
 37) 20 -11.92 1015.3
 38) 21 11.92 1015.3
 39) 22 35.75 1015.3
 40) 23 -35.75 1010.3
 41) 24 -11.92 1010.3
 42) 25 11.92 1010.3
 43) 26 35.75 1010.3
 44) 27 -35.75 1005.3
 45) 28 -11.92 1005.3
 46) 29 11.92 1005.3
 47) 30 35.75 1005.3
 48) 31 -35.75 997.3 S
 49) 32 -11.92 997.3 S
 50) 33 11.92 997.3 S
 51) 34 35.75 997.3 S
 52) 35 -35.67 1025.3 S
 53) 36 -11.84 1025.3 S
 54) 37 12.0 1025.3 S
 55) 38 35.83 1025.3 S
 56) 39 -35.67 1015.3 S
 57) 40 -11.84 1015.3 S
 58) 41 12.0 1015.3 S
 59) 42 35.83 1015.3 S
 60) 43 -35.67 1010.3 S
 61) 44 -11.84 1010.3 S
 62) 45 12.0 1010.3 S
 63) 46 35.83 1010.3 S
 64) 47 -35.67 1005.3 S
 65) 48 -11.84 1005.3 S
 66) 49 12.0 1005.3 S
 67) 50 35.83 1005.3 S
 68) 51 -35.75 1023.3
 69) 54 35.75 1023.3
 70) 51 TO 54
 71) 63 -35.75 1017.3
 72) 51 TO 63 BY 4
 73) 66 35.75 1017.3
 74) 54 TO 66 BY 4
 75) 51 TO 54
 76) 55 TO 58
 77) 59 TO 62
 78) 63 TO 66
 79) 67 -35.75 1012.8
 80) 70 35.75 1012.8
 81) 67 TO 70
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2/90

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* LISTING OF THE INPUT FILE *
****************************

 82) 71 -35.75 1007.8
 83) 74 35.75 1007.8
 84) 71 TO 74
 85) 75 -35.75 1003.3
 86) 78 35.75 1003.3
 87) 83 -35.75 999.3
 88) 86 35.75 999.3
 89) 75 TO 83 BY 4
 90) 78 TO 86 BY 4
 91) 75 TO 78
 92) 79 TO 82
 93) 83 TO 86
 94) 87 -35.67 1023.3
 95) 90 35.83 1023.3
 96) 99 -35.67 1017.3
 97) 102 35.83 1017.3
 98) 87 TO 99 BY 4
 99) 90 TO 102 BY 4
100) 87 TO 90
101) 91 TO 94
102) 95 TO 98
103) 99 TO 102
104) 103 -35.67 1012.8
105) 106 35.83 1012.8
106) 103 TO 106
107) 107 -35.67 1007.8
108) 110 35.83 1007.8
109) 107 TO 110
110) 111 -35.67 1003.3
111) 114 35.83 1003.3
112) 119 -35.67 999.3
113) 122 35.83 999.3
114) 111 TO 119 BY 4
115) 114 TO 122 BY 4
116) 111 TO 114
117) 115 TO 118
118) 119 TO 122
119) $
120) EDIT $ INCREASE COLUMN SPACING TO 24 FEET ON CENTERS
121) MOVE 2 7 TO 119 BY 4 X -0.25
122) MOVE 3 8 TO 120 BY 4 X -0.08
123) MOVE 4 9 TO 121 BY 4 X 0.08
124) MOVE 5 10 TO 122 BY 4 X 0.25
125) $
126) JOINT RELEASE
127) 87 TO 122 AS SUPPORT
128) 31 TO 34 MOM Z KFX 75.48
129) 47 TO 50 KFX 59.628
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*****************************
* LISTING OF THE INPUT FILE *
*****************************

130) 43 TO 46 KFX 53.658
131) 39 TO 42 KFX 42.629
132) 35 TO 38 KFX 7.252
133) 87 TO 90 KFX 15.468
134) 91 TO 94 KFX 23.684
135) 95 TO 98 KFX 30.759
136) 99 TO 102 KFX 36.694
137) 103 TO 106 KFX 48.564
138) 107 TO 110 KFX 58.752
139) 111 TO 114 KFX 60.504
140) 115 TO 118 KFX 61.38
141) 119 TO 122 KFX 68.43
142) $
143) MEMBER INCIDENCES
144) 1 1 2
145) 2 2 3
146) 3 3 4
147) 4 4 5
148) 5 5 6
149) 6 7 2
150) 7 8 3
151) 8 9 4
152) 9 10 5
153) 10 11 7
154) 11 15 11
155) 12 19 63
156) 13 23 67
157) 14 27 71
158) 15 31 83
159) 16 12 8
160) 17 16 12
161) 18 20 64
162) 19 24 68
163) 20 28 72
164) 21 32 84
165) 22 13 9
166) 23 17 13
167) 24 21 65
168) 25 25 69
169) 26 29 73
170) 27 33 85
171) 28 14 10
172) 29 18 14
173) 30 22 66
174) 31 26 70
175) 32 30 74
176) 33 34 86
177) 34 15 35
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* LISTING OF THE INPUT FILE *
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178) 35 19 39
179) 36 23 43
180) 37 27 47
181) 38 16 36
182) 39 20 40
183) 40 24 44
184) 41 28 48
185) 42 17 37
186) 43 21 41
187) 44 25 45
188) 45 29 49
189) 46 18 38
190) 47 22 42
191) 48 26 46
192) 49 30 50
193) 50 51 15
194) 51 55 51
195) 52 59 55
196) 53 63 59
197) 54 67 19
198) 55 71 23
199) 56 75 27
200) 57 79 75
201) 58 83 79
202) 59 52 16
203) 60 56 52
204) 61 60 56
205) 62 64 60
206) 63 68 20
207) 64 72 24
208) 65 76 28
209) 66 80 76
210) 67 84 80
211) 68 53 17
212) 69 57 53
213) 70 61 57
214) 71 65 61
215) 72 69 21
216) 73 73 25
217) 74 77 29
218) 75 81 77
219) 76 85 81
220) 77 54 18
221) 78 58 54
222) 79 62 58
223) 80 66 62
224) 81 70 22
225) 82 74 26
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* LISTING OF THE INPUT FILE *
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226) 83 78 30
227) 84 82 78
228) 85 86 82
229) 86 51 87
230) 87 55 91
231) 88 59 95
232) 89 63 99
233) 90 67 103
234) 91 71 107
235) 92 75 111
236) 93 79 113
237) 94 83 119
238) 95 52 88
239) 96 56 92
240) 97 60 96
241) 98 64 100
242) 99 68 104
243) 100 72 108
244) 101 76 112
245) 102 80 116
246) 103 84 120
247) 104 53 89
248) 105 57 93
249) 106 61 97
250) 107 65 101
251) 108 69 105
252) 109 73 109
253) 110 77 113
254) 111 81 117
255) 112 85 121
256) 113 54 90
257) 114 58 94
258) 115 62 98
259) 116 66 102
260) 117 70 106
261) 118 74 110
262) 119 78 114
263) 120 82 118
264) 121 86 122
265) $
266) MEMBER RELEASE
267) 34 TO 49 START FORCE Y MOM Z
268) 86 TO 121 START FORCE Y MOM Z
269) $
270) MATERIAL PROPERTY
271) CONCR E 519120. DEN .150 CT 0.000006 1 TO 121
272) $
273) MEMBER PROPERTY
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* LISTING OF THE INPUT FILE *
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274) CAP AX 24.0 IZ 32.0 1 2 3 4 5 $ COLUMN CAP MEMBERS
275) COLUMN AX 19.632 IZ 30.68 6 7 8 9 $ COLUMNS
276) SHAFT AX 33.183 IZ 87.624 10 TO 33 $ SHAFTS
277) CONN AX 33.183 IZ 87.624 34 TO 49 $ FALSE CONNECTORS TO SPRINGS
278) SHAFTI AX 33.183 IZ 87.624 50 TO 85 $ SHAFT MEMBER FOR SPRING CON
     .
279) CONNI AX 33.183 IZ 87.624 86 TO 121 $ FALSE CONNECTORS TO SPRINGS

280) $
281) STIFFNESS ANALYSIS
282) $
283) LOAD DL
284) MEMBER LOADS
285) 1 FORCE Y CONC P -305.0 L 2.33 $ GIRDER 1 D.L. PLUS BARRIER & PED
      RAIL
286) 2 FORCE Y CONC P -226.6 L 7.75 $ GIRDER 2 D.L. PLUS BARRIER LOAD
287) 2 FORCE Y CONC P -205.4 L 17.17 $ GIRDER 3 D.L.
288) 3 FORCE Y CONC P -205.4 L 2.58 $ GIRDER 4 D.L.
289) 3 FORCE Y CONC P -205.4 L 12.0 $ GIRDER 5 D.L.
290) 3 FORCE Y CONC P -205.4 L 21.42 $ GIRDER 6 D.L.
291) 4 FORCE Y CONC P -205.4 L 6.84 $ GIRDER 7 D.L.
292) 4 FORCE Y CONC P -226.6 L 16.26 $ GIRDER 8 D.L. PLUS BARRIER LOAD

293) 5 FORCE Y CONC P -305.0 L 1.68 $ GIRDER 9 D.L. PLUS BARRIER & PED
      RAIL
294) 1 FORCE Y LIN WA -3.1 WB -3.124 $ END DIAPHRAGM PLUS SLAB D.L.
295) 2 FORCE Y LIN WA -3.124 WB -3.308 $ END DIAPHRAGM PLUS SLAB D.L.
296) 3 FORCE Y LIN WA -3.308 WB -3.4 LB 11.92 $ END DIA. PLUS SLAB D.L
     .
297) 3 FORCE Y LIN WA -3.4 WB -3.308 LA 11.92 $ END DIA. PLUS SLAB D.L
     .
298) 4 FORCE Y LIN WA -3.308 WB -3.124 $ END DIAPHRAGM PLUS SLAB D.L.
299) 5 FORCE Y LIN WA -3.124 WB -3.1 $ END DIAPHRAGM PLUS SLAB D.L.
300) 1 TO 5 FORCE Y UNI W -3.6 $ CAP D.L.
301) 6 TO 9 FORCE Y GLOBAL UNI W -2.945 $ COLUMN D.L.
302) 10 TO 33 FORCE Y GLOBAL UNI W -4.977 $ SHAFT D.L.
303) 50 TO 85 FORCE Y GLOBAL UNI W -4.977 $ SHAFT D.L.
304) $
305) LOAD FWS
306) MEMBER LOADS
307) 2 FORCE Y CONC P -15.0 L 7.75 $ GIRDER 2 FWS+UTIL
308) 2 FORCE Y CONC P -28.5 L 17.17 $ GIRDER 3 FWS+UTIL
309) 3 FORCE Y CONC P -28.5 L 2.58 $ GIRDER 4 FWS+UTIL
310) 3 FORCE Y CONC P -28.5 L 12.0 $ GIRDER 5 FWS+UTIL
311) 3 FORCE Y CONC P -28.5 L 21.42 $ GIRDER 6 FWS+UTIL
312) 4 FORCE Y CONC P -28.5 L 6.84 $ GIRDER 7 FWS+UTIL
313) 4 FORCE Y CONC P -15.0 L 16.26 $ GIRDER 8 FWS+UTIL
314) $
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* LISTING OF THE INPUT FILE *
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315) LOAD SDWK_LL
316) MEMBER LOADS
317) 5 FORCE Y CONC P -39.9 L 1.68 $ SIDEWALK L.L. ON ONE SIDE OF BRID
     GE
318) $
319) LOAD LL_I_1 $ MAX. TRANS. MOMENTS IN SHAFTS AND COLUMNS
320) MEMBER LOADS
321) 3 FORCE Y CONC P -55.4 L 12.0 $ GIRDER 5 L.L.
322) 3 FORCE Y CONC P -142.0 L 21.42 $ GIRDER 6 L.L.
323) 4 FORCE Y CONC P -139.3 L 6.84 $ GIRDER 7 L.L.
324) 4 FORCE Y CONC P -112.8 L 16.26 $ GIRDER 8 L.L.
325) 5 FORCE Y CONC P -23.5 L 1.68 $ GIRDER 9 L.L.
326) $
327) LOAD LL_I_2 $ MAX. POS. MOM. IN SPANS 2 & 4 AND MAX. NEG. MOM. SP
     AN 3
328) MEMBER LOADS
329) 1 FORCE Y CONC P -23.6 L 1.17 $ GIRDER 1 L.L.
330) 2 FORCE Y CONC P -112.7 L 7.75 $ GIRDER 2 L.L.
331) 2 FORCE Y CONC P -94.4 L 17.17 $ GIRDER 3 L.L.
332) 3 FORCE Y CONC P -5.3 L 2.58 $ GIRDER 4 L.L.
333) 3 FORCE Y CONC P -5.3 L 21.42 $ GIRDER 6 L.L.
334) 4 FORCE Y CONC P -94.4 L 6.84 $ GIRDER 7 L.L.
335) 4 FORCE Y CONC P -112.7 L 16.26 $ GIRDER 8 L.L.
336) 5 FORCE Y CONC P -23.6 L 1.68 $ GIRDER 9 L.L.
337) $
338) LOAD LL_I_3 $ MAX. NEG. MOM. OVER COLUMNS
339) MEMBER LOADS
340) 2 FORCE Y CONC P -62.6 L 7.75 $ GIRDER 2 L.L.
341) 2 FORCE Y CONC P -55.5 L 17.17 $ GIRDER 3 L.L.
342) 3 FORCE Y CONC P -18.9 L 2.58 $ GIRDER 4 L.L.
343) 3 FORCE Y CONC P -80.3 L 12.0 $ GIRDER 5 L.L.
344) 3 FORCE Y CONC P -18.9 L 21.42 $ GIRDER 6 L.L.
345) $
346) LOAD THER_EXP
347) MEMBER TEMPERATURE
348) 1 TO 10 16 22 28 AXI 30 $ TEMPERATURE RISE OF 30 DEGREES
349) $
350) LOAD WIND
351) JOINT LOAD
352) $ 1 FORCE X 46.38 $ WIND LOAD FROM SUPERSTRUCTURE
353) 1 FORCE X 1.12 $ WIND LOAD FROM SUBSTRUCTURE
354) $
355) LOAD OT_GRP_2
356) MEMBER LOAD
357) 2 FORCE Y CONC P 164.6 L 16.08 $ OVERTURNING FORCE FOR GROUPS II
     AND V
358) $
359) LOAD STREAM
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* LISTING OF THE INPUT FILE *
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360) MEMBER LOAD
361) 6 FORCE Y UNI W -2.97 $ STREAM FLOW ON UPSTREAM COLUMN
362) 7 8 9 FORCE Y UNI W -0.72 $ STREAM FLOW ON CENTRAL AND DOWNSTREAM
      COLUMNS
363) $
364) LOAD DIF_SET1
365) JOINT DISPLACEMENT
366) 34 DIS Y -0.031 $ 3/8" SETTLEMENT ON THE DOWNSTREAM COLUMN
367) $
368) LOAD DIF_SET2
369) JOINT DISPLACEMENT
370) 33 DIS Y -0.031 $ 3/8" SETTLEMENT ON THE SECOND INTERIOR COLUMN
371) $
372) LOAD COMBINATION DED_LOD $ COMBINE GIRDER, F.W.S. AND UTIL. INTO
     DEAD LOAD
373) COMBINE DL 1.0 FWS 1.0
374) $
375) SELECT LOAD 3 TO 13
376) LIST MEM FOR 1 5 SECT FRA NS 1 .5
377) LIST MEM FOR 2 3 4 SECT FRA NS 3 .25 .5 .75
378) $ LIST MEM FOR 6 TO 9
379) $ LIST MEM FOR 10 TO 15
380) $ LIST MEM FOR 11 TO 29 BY 6
381) $ LIST MEM FOR 13 TO 31 BY 6
382) $ LIST MEM FOR 15 TO 33 BY 6
383) UNIT KIP INCH DEGREE
384) LIST JOI DISP 1 TO 6
385) FINISH
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****
* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
1	SDWK_LL	1	0.00	0.00	-0.00
		2.00	0.00	0.00	-0.00
CAP		2	0.00	0.00	-0.00
	LL_I_1	1	0.00	-0.00	-0.00
		2.00	0.00	-0.00	0.00
		2	0.00	-0.00	0.00
	LL_I_2	1	-0.00	0.00	-0.00
		2.00	-0.00	23.60	-19.59
		2	-0.00	23.60	-66.79
	LL_I_3	1	-0.00	0.00	-0.00
		2.00	-0.00	0.00	-0.00
		2	-0.00	0.00	-0.00
	THER_EXP	1	0.00	0.00	0.00
		2.00	0.00	0.00	-0.00
		2	0.00	0.00	-0.00
	WIND	1	-1.12	0.00	-0.00
		2.00	-1.12	0.00	-0.00
		2	-1.12	0.00	-0.00
	OT_GRP_2	1	0.00	-0.00	0.00
		2.00	0.00	-0.00	0.00
		2	0.00	-0.00	0.00
	STREAM	1	0.00	-0.00	-0.00
		2.00	0.00	-0.00	-0.00
		2	0.00	-0.00	-0.00
	DIF_SET1	1	-0.00	0.00	-0.00
		2.00	-0.00	0.00	-0.00
		2	-0.00	0.00	-0.00
	DIF_SET2	1	0.00	-0.00	0.00
		2.00	0.00	-0.00	0.00

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
		2	0.00	-0.00	0.00
	DED_LOD	1	0.00	-0.00	0.00
		2.00	0.00	13.41	-13.41
		2	0.00	331.85	-563.01
5	SDWK_LL	5	0.00	-39.90	-67.03
		2.00	0.00	0.00	-0.00
CAP		6	0.00	0.00	0.00
	LL_I_1	5	-0.00	-23.50	-39.48
		2.00	-0.00	0.00	0.00
		6	-0.00	0.00	0.00
	LL_I_2	5	-0.00	-23.60	-39.65
		2.00	-0.00	-0.00	0.00
		6	-0.00	0.00	0.00
	LL_I_3	5	0.00	0.00	0.00
		2.00	0.00	0.00	0.00
		6	0.00	0.00	-0.00
	THER_EXP	5	0.00	0.00	-0.00
		2.00	0.00	0.00	-0.00
		6	0.00	0.00	-0.00
	WIND	5	-0.00	0.00	0.00
		2.00	-0.00	0.00	0.00
		6	-0.00	0.00	0.00
	OT_GRP_2	5	0.00	-0.00	-0.00
		2.00	0.00	-0.00	-0.00
		6	0.00	-0.00	0.00
	STREAM	5	-0.00	-0.00	-0.00
		2.00	-0.00	-0.00	-0.00
		6	-0.00	-0.00	0.00
	DIF_SET1	5	-0.00	0.00	0.00
		2.00	-0.00	0.00	-0.00
		6	-0.00	0.00	0.00
	DIF_SET2	5	-0.00	-0.00	-0.00
		2.00	-0.00	-0.00	0.00
		6	-0.00	-0.00	0.00

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
	DED_LOD	5	-0.00	-331.85	-566.06
		2.00	-0.00	-13.41	-13.41
		6	-0.00	0.00	0.00

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* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
2	SDWK_LL	2	0.07	0.30	4.45
		6.00	0.07	0.30	2.64
		12.00	0.07	0.30	0.82
		18.00	0.07	0.30	-0.99
CAP		3	0.07	0.30	-2.81
	LL_I_1	2	-0.03	4.75	2.09
		6.00	-0.03	4.75	-26.41
		12.00	-0.03	4.75	-54.90
		18.00	-0.03	4.75	-83.40
		3	-0.03	4.75	-111.90
	LL_I_2	2	-2.07	-97.39	-209.89
		6.00	-2.07	-97.39	374.44
		12.00	-2.07	15.31	479.81
		18.00	-2.07	109.71	309.59
		3	-2.07	109.71	-348.67
	LL_I_3	2	-1.19	-47.55	-87.75
		6.00	-1.19	-47.55	197.55
		12.00	-1.19	15.05	216.80
		18.00	-1.19	70.55	80.43
		3	-1.19	70.55	-342.87
	THER_EXP	2	-0.85	-3.41	-57.06
		6.00	-0.85	-3.41	-36.63
		12.00	-0.85	-3.41	-16.19
		18.00	-0.85	-3.41	4.24
		3	-0.85	-3.41	24.67
	WIND	2	-0.87	1.14	16.18
		6.00	-0.87	1.14	9.35
		12.00	-0.87	1.14	2.52
		18.00	-0.87	1.14	-4.30
		3	-0.87	1.14	-11.13
	OT_GRP_2	2	1.69	45.20	120.96
		6.00	1.69	45.20	-150.26

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
		12.00	1.69	45.20	-421.48
		18.00	1.69	-119.40	-376.66
		3	1.69	-119.40	339.72
	STREAM	2	-30.64	75.11	1015.59
		6.00	-30.64	75.11	564.93
		12.00	-30.64	75.11	114.27
		18.00	-30.64	75.11	-336.40
		3	-30.64	75.11	-787.06
	DIF_SET1	2	4.24	11.38	287.04
		6.00	4.24	11.38	218.75
		12.00	4.24	11.38	150.47
		18.00	4.24	11.38	82.18
		3	4.24	11.38	13.90
	DIF_SET2	2	0.94	69.28	83.26
		6.00	0.94	69.28	-332.43
		12.00	0.94	69.28	-748.13
		18.00	0.94	69.28	-1163.82
		3	0.94	69.28	-1579.52
	DED_LOD	2	-3.77	-275.81	-830.13
		6.00	-3.77	-235.33	703.45
		12.00	-3.77	47.03	966.51
		18.00	-3.77	321.96	367.26
		3	-3.77	363.27	-1688.29
3	SDWK_LL	3	0.16	0.20	2.79
		6.00	0.16	0.20	1.57
		12.00	0.16	0.20	0.36
		18.00	0.16	0.20	-0.85
CAP		4	0.16	0.20	-2.07
	LL_I_1	3	-0.99	-28.86	-176.80
		6.00	-0.99	-28.86	-3.65
		12.00	-0.99	26.54	169.50
		18.00	-0.99	26.54	10.26
		4	-0.99	168.54	-515.35
	LL_I_2	3	-0.42	-5.08	-234.96
		6.00	-0.42	0.22	-222.63
		12.00	-0.42	0.22	-223.97
		18.00	-0.42	0.22	-225.31
		4	-0.42	5.52	-240.32

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
	LL_I_3	3	-0.96	-65.60	-327.32
		6.00	-0.96	-46.70	1.66
		12.00	-0.96	33.60	281.87
		18.00	-0.96	33.60	80.29
		4	-0.96	52.50	-170.06
	THER_EXP	3	-1.21	0.16	1.57
		6.00	-1.21	0.16	0.60
		12.00	-1.21	0.16	-0.38
		18.00	-1.21	0.16	-1.36
		4	-1.21	0.16	-2.33
	WIND	3	-0.56	0.71	8.43
		6.00	-0.56	0.71	4.20
		12.00	-0.56	0.71	-0.04
		18.00	-0.56	0.71	-4.27
		4	-0.56	0.71	-8.51
	OT_GRP_2	3	0.16	10.63	234.74
		6.00	0.16	10.63	170.98
		12.00	0.16	10.63	107.21
		18.00	0.16	10.63	43.45
		4	0.16	10.63	-20.31
	STREAM	3	-19.62	54.24	666.31
		6.00	-19.62	54.24	340.89
		12.00	-19.62	54.24	15.47
		18.00	-19.62	54.24	-309.96
		4	-19.62	54.24	-635.38
	DIF_SET1	3	10.34	76.01	415.91
		6.00	10.34	76.01	-40.17
		12.00	10.34	76.01	-496.24
		18.00	10.34	76.01	-952.32
		4	10.34	76.01	-1408.39
	DIF_SET2	3	-9.70	-234.09	-2304.97
		6.00	-9.70	-234.09	-900.43
		12.00	-9.70	-234.09	504.11
		18.00	-9.70	-234.09	1908.65
		4	-9.70	-234.09	3313.18
	DED_LOD	3	-3.21	-433.60	-1649.30
		6.00	-3.21	-158.12	27.75
		12.00	-3.21	117.65	850.99

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
		18.00	-3.21	159.51	19.37
		4	-3.21	434.99	-1666.03
4	SDWK_LL	4	0.25	2.23	4.10
		6.00	0.25	2.23	-9.30
		12.00	0.25	2.23	-22.71
		18.00	0.25	2.23	-36.11
CAP		5	0.25	2.23	-49.52
	LL_I_1	4	-2.30	-152.95	-606.44
		6.00	-2.30	-152.95	311.25
		12.00	-2.30	-13.65	510.14
		18.00	-2.30	99.15	395.75
		5	-2.30	99.15	-199.17
	LL_I_2	4	-2.11	-110.94	-355.40
		6.00	-2.11	-110.94	310.24
		12.00	-2.11	-16.54	488.78
		18.00	-2.11	96.16	391.92
		5	-2.11	96.16	-185.04
	LL_I_3	4	-0.01	-4.22	-101.42
		6.00	-0.01	-4.22	-76.07
		12.00	-0.01	-4.22	-50.72
		18.00	-0.01	-4.22	-25.38
		5	-0.01	-4.22	-0.03
	THER_EXP	4	-0.81	3.22	25.01
		6.00	-0.81	3.22	5.70
		12.00	-0.81	3.22	-13.62
		18.00	-0.81	3.22	-32.93
		5	-0.81	3.22	-52.24
	WIND	4	-0.26	1.18	11.50
		6.00	-0.26	1.18	4.43
		12.00	-0.26	1.18	-2.64
		18.00	-0.26	1.18	-9.71
		5	-0.26	1.18	-16.78
	OT_GRP_2	4	0.09	-1.42	-28.27
		6.00	0.09	-1.42	-19.77
		12.00	0.09	-1.42	-11.26
		18.00	0.09	-1.42	-2.76
		5	0.09	-1.42	5.74

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	AXIAL FORCE	LOCAL Y SHEAR FORCE	LOCAL Z MOMENT
	STREAM	4	-8.18	87.83	850.09
		6.00	-8.18	87.83	323.09
		12.00	-8.18	87.83	-203.91
		18.00	-8.18	87.83	-730.91
		5	-8.18	87.83	-1257.90
	DIF_SET1	4	8.52	-90.18	-1559.58
		6.00	8.52	-90.18	-1018.50
		12.00	8.52	-90.18	-477.43
		18.00	8.52	-90.18	63.65
		5	8.52	-90.18	604.73
	DIF_SET2	4	-13.16	166.19	3075.12
		6.00	-13.16	166.19	2077.97
		12.00	-13.16	166.19	1080.82
		18.00	-13.16	166.19	83.67
		5	-13.16	166.19	-913.49
	DED_LOD	4	-3.58	-363.71	-1685.87
		6.00	-3.58	-322.40	372.33
		12.00	-3.58	-47.47	976.58
		18.00	-3.58	234.89	718.59
		5	-3.58	275.37	-812.34

2/90

****
* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS INCHES DEGREES

*** LOAD INDEX : 3 LOAD TAG : SDWK_LL ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	/--- GLOBAL DISPLACEMENTS ---/-- ROTATION --/ X DIRECTION	Y DIRECTION	Z DIRECTION
1	0.01	0.00	-0.00
2	0.01	0.00	-0.00
3	0.01	-0.00	-0.00
4	0.01	0.00	0.00
5	0.01	-0.00	-0.00
6	0.01	-0.00	-0.00

*** LOAD INDEX : 4 LOAD TAG : LL_I_1 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	/--- GLOBAL DISPLACEMENTS ---/-- ROTATION --/ X DIRECTION	Y DIRECTION	Z DIRECTION
1	-0.02	-0.00	0.00
2	-0.02	0.00	0.00
3	-0.02	-0.00	-0.00
4	-0.02	-0.02	-0.00
5	-0.02	-0.01	0.01
6	-0.02	0.00	0.01

*** LOAD INDEX : 5 LOAD TAG : LL_I_2 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	/--- GLOBAL DISPLACEMENTS ---/-- ROTATION --/ X DIRECTION	Y DIRECTION	Z DIRECTION
1	-0.00	0.00	-0.01
2	-0.00	-0.01	-0.01

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
3	-0.00	-0.01	0.01
4	-0.00	-0.01	-0.01
5	-0.00	-0.01	0.01
6	-0.00	0.00	0.01

*** LOAD INDEX : 6 LOAD TAG : LL_I_3 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
1	0.01	0.00	-0.01
2	0.01	-0.00	-0.01
3	0.01	-0.01	0.00
4	0.01	-0.00	0.00
5	0.01	0.00	-0.00
6	0.01	-0.00	-0.00

*** LOAD INDEX : 7 LOAD TAG : THER_EXP ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
1	-0.09	0.10	0.00
2	-0.08	0.11	0.00
3	-0.03	0.11	-0.00
4	0.03	0.11	-0.00
5	0.08	0.11	-0.00
6	0.09	0.11	-0.00

*** LOAD INDEX : 8 LOAD TAG : WIND ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
1	0.02	0.00	-0.00
2	0.02	0.00	-0.00
3	0.02	-0.00	-0.00
4	0.02	0.00	-0.00
5	0.02	-0.00	-0.00
6	0.02	-0.00	-0.00

*** LOAD INDEX : 9 LOAD TAG : OT_GRP_2 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
1	-0.01	-0.00	0.01
2	-0.01	0.00	0.01
3	-0.01	0.01	-0.01
4	-0.01	-0.00	0.00
5	-0.01	0.00	-0.00
6	-0.01	0.00	-0.00

*** LOAD INDEX : 10 LOAD TAG : STREAM ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
1	1.91	0.02	-0.02
2	1.91	0.00	-0.02
3	1.91	-0.00	-0.01
4	1.91	0.00	-0.01
5	1.91	-0.01	-0.02
6	1.91	-0.03	-0.02

*** LOAD INDEX : 11 LOAD TAG : DIF_SET1 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
1	0.39	0.01	-0.01
2	0.39	0.00	-0.01
3	0.39	0.00	0.01
4	0.39	-0.01	-0.04
5	0.39	-0.37	-0.08
6	0.39	-0.43	-0.08

*** LOAD INDEX : 12 LOAD TAG : DIF_SET2 ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
1	-0.16	-0.01	0.01
2	-0.16	0.00	0.01
3	-0.16	-0.02	-0.05
4	-0.16	-0.35	-0.01
5	-0.16	-0.01	0.08
6	-0.16	0.06	0.08

*** LOAD INDEX : 13 LOAD TAG : DED_LOD ***

RESULTANT JOINT DISPLACEMENTS -----

JOINT NO.	GLOBAL DISPLACEMENTS X DIRECTION	Y DIRECTION	ROTATION Z DIRECTION
1	-0.00	-0.03	-0.02
2	-0.00	-0.05	-0.02
3	-0.00	-0.06	0.00
4	-0.00	-0.06	-0.00
5	-0.00	-0.05	0.02
6	-0.00	-0.03	0.02

Summary of M-Strudl Moments

	MEMBER 2					MEMBER 3					MEMBER 4				
	$M_{2.0}$	$M_{2.25}$	$M_{2.5}$	$M_{2.75}$	$M_{2.99}$	$M_{3.0}$	$M_{3.25}$	$M_{3.5}$	$M_{3.75}$	$M_{3.99}$	$M_{4.0}$	$M_{4.25}$	$M_{4.5}$	$M_{4.75}$	$M_{4.99}$
DED_LOD	-830	731	967	368	-1689	-1659	28	851	20	-1667	-1686	373	977	719	-813
DIF_SET 1	287	219	151	83	14	416	-41	-497	-953	-1409	-1560	-1019	-478	64	605
DIF_SET 2	84	-333	-749	-1164	-1580	-2305	-901	505	1909	3314	3076	2078	1081	84	-914
SDWK_LL	5	3	1	-1	-3	3	2	1	-1	-2	5	-10	-23	-37	-50
LL_I_1	2	-27	-55	-84	-112	-177	-4	170	11	-516	-607	312	510	396	-200
LL_I_2	-210	375	480	310	-349	-235	-223	-224	-226	-241	-356	311	489	392	-185
LL_I_3	-88	198	217	81	-343	-328	2	282	81	-170	-102	-76	-51	-24	0
WIND	17	10	3	-5	-12	9	5	0	-5	-9	12	5	-3	-10	-17
OT_GRP_2	121	-151	-422	-377	340	235	171	108	44	-21	-29	-20	-12	-3	6
STREAM	1016	565	115	-337	-787	667	341	16	-310	-636	851	323	-204	-731	-1258
THER_EXP	-57	-37	-17	5	25	2	1	-1	-2	-3	25	6	-14	-32	-53

Values in Line DIF_SET 1 will be reversed if J31 settles instead of J34
" " " DIF_SET 2 " " " " J32 " " " J33
" " " SDWK_LL " " " " Member 1 is loaded instead of 5
" " " LL_I_1 " " " " " 1,2,3 are " " " 3,4,5
" " " LL_I_3 " " " " " 3,4 " " " " 2,3
" " " WIND " " " " Wind blows from other direction
" " " OT_GRP_2 " " " " " " " " "

IBRW MINNEAPOLIS PHOENIX

JOB Northern Pacific New River SHEET NO. CAP 23
MADE BY TJhones DATE 1-18-90 JOB NO. 33-89-62
CHECKED BY LFrankel DATE 2/90 SEC. SHT. NO. 231
CALCULATIONS FOR CAP DESIGN

Assume Group I Loading Will Control

Exterier Spans:

Max. Neg. Mom Over Ext. Support

-813 -914 -50 -200 -1258 = -3235 ft·k

Max. Neg. Mom Over Int. Support

-1689 -1580 -3 -607 -787 = -4666 ft·k

Max Neg. Mom. Mid Span

977 -749 -23 -55 -204 = -54 ft·k

Max Pos. Mom Mid Span

967 + 1081 + 1 + 489 + 115 = 2653 ft·k

Max Pos Mom. over Ext. Support

-830 + 605 + 5 + 2 + 1016 = 798 ft·k

Max Pos Mom Over Int Support

-1686 +3076 +5 — + 851 = 2246 ft·k

Interier Span:

Max Neg Mom Over Support

-1667 -2305 -2 -516 -636 = -5126 ft·k

Max Pos Mom Over Support

-1650 + 3314 +3 — + 667 = 2334 ft·k

Max Pos Mom Mid-Span

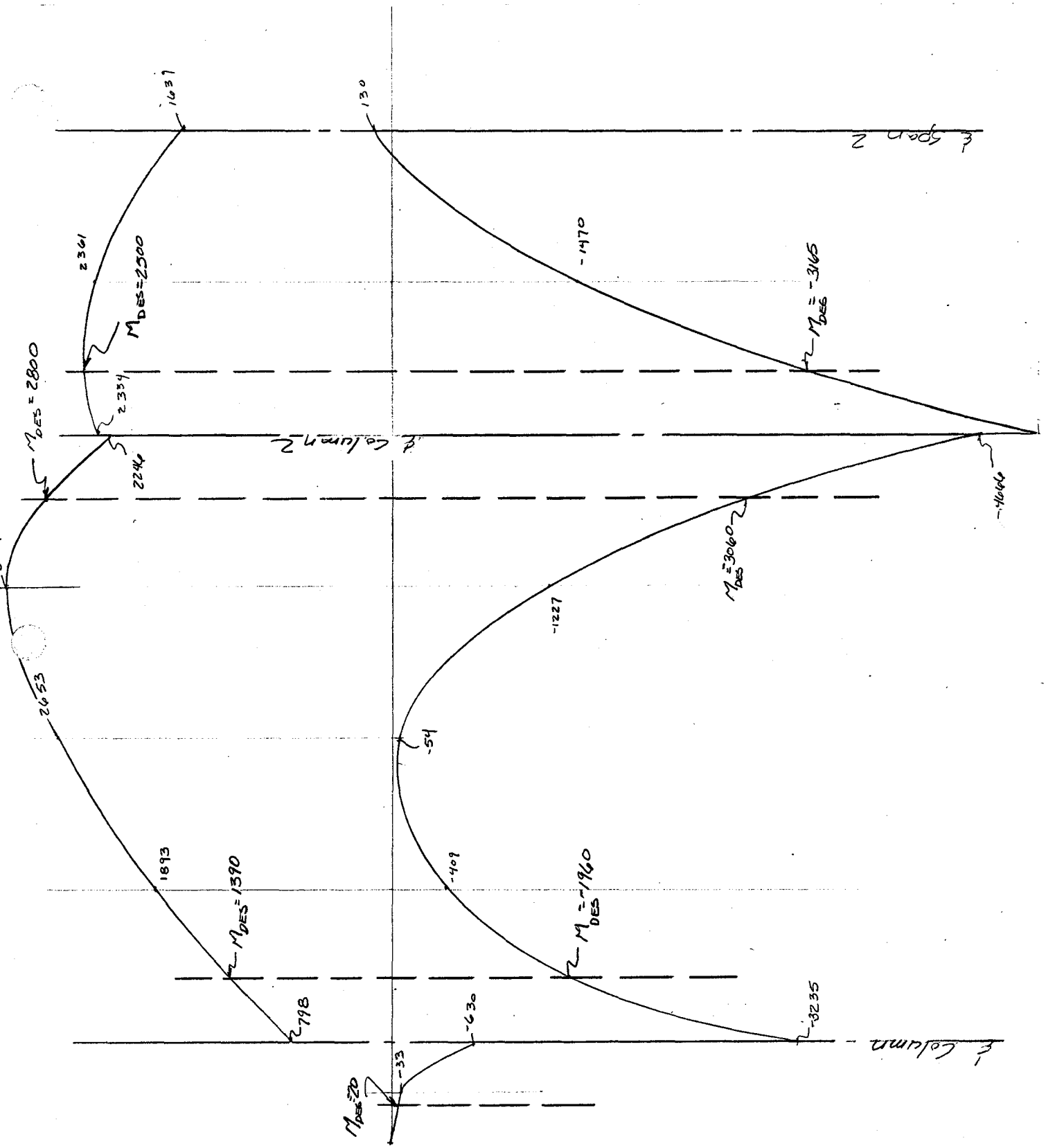
851 + 505 + 1 + 282 + 16 = 1639 ft·k

Max Neg Mom Mid-Span

851 -497 — -224 — = + 130 ft·k

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. CAP 24
MADE BY T Stones DATE 1-16-90 JOB NO. 33-89162
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 232
CALCULATIONS FOR Cap Design



BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. CAP 25
MADE BY T Stones DATE 1-19-90 JOB NO. 33-89102
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 233
CALCULATIONS FOR Cap Design

Cap Design by Allowable stress:

Cap Dimension 6 ft x 4 ft

$d_{\#11} = 48 - 2 - 0.625 - 1.41/2 = 44.67$ in $\qquad b = 72$ in

$n = \frac{E_s}{E_c} = \frac{29{,}000}{57\sqrt{4000}} = 8.04 \qquad f_c = 0.4 f'_c = .4(4000) = 1600$ psi

$k_b = \frac{n}{n + f_s/f_c} = \frac{8}{8 + 24/1.6} = 0.348$

$j = 1 - \frac{k_b}{3} = 0.884$

check section w/ $M = 3165$ ft-k

$bd^2 = \frac{2M}{f_c j k} = \frac{2(3165 \text{ ft-k})}{1.6 \text{ ksi}(.884)(.348)} \frac{12 \text{ in}}{\text{ft}} = 154{,}314 \text{ in}^3$

w/ $b = 72$ in $\quad d_{req} = 46^3$ in $\quad$ must increase section

$h = 50.33$

Try a Section 6 ft x 4^5 ft

$I = \frac{6(4.5)^3}{12} = 42.54 \text{ ft}^4 \qquad A = 6(4.5) = 27.0 \text{ ft}^2$

$w = 27(.150) = 4.05$ k/ft

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. CAP 26
MADE BY T Stones DATE 1-19-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 234
CALCULATIONS FOR Cap Design

****
* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES ------

MEMBER SECTION	FILE/LOAD	JOINT NO.	/--- AXIAL FORCE	---/--- LOCAL Y SHEAR FORCE	--/-- LOCAL Z MOMENT --/
1	SDWK_LL	1	0.00	0.00	0.00
		2.00	0.00	0.00	-0.00
CAP		2	0.00	0.00	0.00
	LL_I_1	1	0.00	-0.00	-0.00
		2.00	0.00	-0.00	-0.00
		2	0.00	-0.00	0.00
	LL_I_2	1	-0.00	0.00	-0.00
		2.00	-0.00	23.60	-19.59
		2	-0.00	23.60	-66.79
	LL_I_3	1	0.00	0.00	0.00
		2.00	0.00	0.00	-0.00
		2	0.00	0.00	-0.00
	THER_EXP	1	0.00	-0.00	-0.00
		2.00	0.00	-0.00	0.00
		2	0.00	-0.00	0.00
	WIND	1	-1.12	-0.00	0.00
		2.00	-1.12	-0.00	0.00
		2	-1.12	-0.00	0.00
	OT_GRP_2	1	-0.00	0.00	0.00
		2.00	-0.00	0.00	0.00
		2	-0.00	0.00	-0.00
	STREAM	1	0.00	-0.00	-0.00
		2.00	0.00	-0.00	0.00
		2	0.00	-0.00	0.00
	DIF_SET1	1	-0.00	0.00	0.00
		2.00	-0.00	0.00	-0.00
		2	-0.00	0.00	-0.00
	DIF_SET2	1	-0.00	0.00	0.00
		2.00	-0.00	0.00	0.00

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	/--- AXIAL ---/ FORCE	/--- LOCAL Y --/ SHEAR FORCE	/-- LOCAL Z --/ MOMENT
		2	-0.00	0.00	0.00
	DED_LOD	1	0.00	-0.00	0.00
		2.00	0.00	14.31	-14.31
		2	0.00	333.65	-566.61
5	SDWK_LL	5	-0.00	-39.90	-67.03
		2.00	-0.00	0.00	-0.00
CAP		6	-0.00	0.00	0.00
	LL_I_1	5	0.00	-23.50	-39.48
		2.00	0.00	0.00	0.00
		6	0.00	0.00	0.00
	LL_I_2	5	0.00	-23.60	-39.65
		2.00	0.00	-0.00	0.00
		6	0.00	0.00	0.00
	LL_I_3	5	-0.00	-0.00	-0.00
		2.00	-0.00	-0.00	-0.00
		6	-0.00	-0.00	-0.00
	THER_EXP	5	0.00	-0.00	0.00
		2.00	0.00	-0.00	0.00
		6	0.00	-0.00	-0.00
	WIND	5	0.00	0.00	0.00
		2.00	0.00	0.00	0.00
		6	0.00	0.00	0.00
	OT_GRP_2	5	0.00	-0.00	-0.00
		2.00	0.00	-0.00	-0.00
		6	0.00	-0.00	-0.00
	STREAM	5	0.00	0.00	0.00
		2.00	0.00	0.00	-0.00
		6	0.00	0.00	0.00
	DIF_SET1	5	-0.00	0.00	0.00
		2.00	-0.00	0.00	0.00
		6	-0.00	0.00	-0.00
	DIF_SET2	5	0.00	0.00	0.00
		2.00	0.00	0.00	0.00
		6	0.00	0.00	0.00

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	/--- AXIAL FORCE	---/--- LOCAL Y SHEAR FORCE	--/-- LOCAL Z MOMENT --/
	DED_LOD	5	-0.00	-333.65	-569.66
		2.00	-0.00	-14.31	-14.31
		6	-0.00	-0.00	0.00

2/90

****
* RESULTS OF ANALYSIS *
****

TYPE OF THE PROBLEM : PLANE FRAME RESTART STATUS : NONE GIVEN
ACTIVE UNITS : KIPS FEET DEGREES

RESULTANT MEMBER FORCES -----

MEMBER SECTION	FILE/LOAD	JOINT NO.	/--- AXIAL FORCE ---/	--- LOCAL Y SHEAR FORCE --/	-- LOCAL Z MOMENT --/
2	SDWK_LL	2	0.06	0.24	4.04
		6.00	0.06	0.24	2.61
		12.00	0.06	0.24	1.18
		18.00	0.06	0.24	-0.25
CAP		3	0.06	0.24	-1.69
	LL_I_1	2	-0.01	6.13	2.61
		6.00	-0.01	6.13	-34.20
		12.00	-0.01	6.13	-71.01
		18.00	-0.01	6.13	-107.81
		3	-0.01	6.13	-144.62
	LL_I_2	2	-1.59	-96.42	-176.87
		6.00	-1.59	-96.42	401.67
		12.00	-1.59	16.28	501.22
		18.00	-1.59	110.68	325.21
		3	-1.59	110.68	-338.86
	LL_I_3	2	-0.99	-47.47	-72.66
		6.00	-0.99	-47.47	212.19
		12.00	-0.99	15.13	230.99
		18.00	-0.99	70.63	94.17
		3	-0.99	70.63	-329.59
	THER_EXP	2	-0.91	-3.58	-61.08
		6.00	-0.91	-3.58	-39.60
		12.00	-0.91	-3.58	-18.12
		18.00	-0.91	-3.58	3.36
		3	-0.91	-3.58	24.85
	WIND	2	-0.86	1.14	16.54
		6.00	-0.86	1.14	9.70
		12.00	-0.86	1.14	2.85
		18.00	-0.86	1.14	-4.00
		3	-0.86	1.14	-10.85
	OT_GRP_2	2	1.38	45.58	98.73
		6.00	1.38	45.58	-174.73

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RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	/--- AXIAL ---/ FORCE	/--- LOCAL Y --/ SHEAR FORCE	/-- LOCAL Z --/ MOMENT
		12.00	1.38	45.58	-449.19
		18.00	1.38	-119.02	-405.61
		3	1.38	-119.02	308.53
	STREAM	2	-30.48	75.01	1030.42
		6.00	-30.48	75.01	580.39
		12.00	-30.48	75.01	130.35
		18.00	-30.48	75.01	-319.69
		3	-30.48	75.01	-769.72
	DIF_SET1	2	4.58	10.05	311.44
		6.00	4.58	10.05	251.17
		12.00	4.58	10.05	190.89
		18.00	4.58	10.05	130.62
		3	4.58	10.05	70.35
	DIF_SET2	2	0.73	94.13	68.90
		6.00	0.73	94.13	-495.88
		12.00	0.73	94.13	-1060.66
		18.00	0.73	94.13	-1625.44
		3	0.73	94.13	-2190.22
	DED_LOD	2	-3.08	-278.87	-795.31
		6.00	-3.08	-235.69	758.48
		12.00	-3.08	49.37	1015.56
		18.00	-3.08	327.01	394.12
		3	-3.08	371.02	-1699.81
3	SDWK_LL	3	0.14	0.35	3.36
		6.00	0.14	0.35	1.27
		12.00	0.14	0.35	-0.81
		18.00	0.14	0.35	-2.89
CAP		4	0.14	0.35	-4.97
	LL_I_1	3	-0.86	-31.33	-201.90
		6.00	-0.86	-31.33	-13.91
		12.00	-0.86	24.07	174.07
		18.00	-0.86	24.07	29.66
		4	-0.86	166.07	-481.11
	LL_I_2	3	-0.34	-5.09	-252.75
		6.00	-0.34	0.21	-240.36
		12.00	-0.34	0.21	-241.65
		18.00	-0.34	0.21	-242.94
		4	-0.34	5.51	-257.90

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	/--- AXIAL FORCE ---/	--- LOCAL Y SHEAR FORCE --/	-- LOCAL Z MOMENT --/
	LL_I_3	3	-0.81	-65.32	-317.49
		6.00	-0.81	-46.42	9.79
		12.00	-0.81	33.88	288.32
		18.00	-0.81	33.88	85.04
		4	-0.81	52.78	-167.00
	THER_EXP	3	-1.26	0.18	1.77
		6.00	-1.26	0.18	0.70
		12.00	-1.26	0.18	-0.37
		18.00	-1.26	0.18	-1.45
		4	-1.26	0.18	-2.52
	WIND	3	-0.56	0.71	8.47
		6.00	-0.56	0.71	4.22
		12.00	-0.56	0.71	-0.03
		18.00	-0.56	0.71	-4.28
		4	-0.56	0.71	-8.52
	OT_GRP_2	3	0.19	9.58	227.18
		6.00	0.19	9.58	169.71
		12.00	0.19	9.58	112.24
		18.00	0.19	9.58	54.77
		4	0.19	9.58	-2.70
	STREAM	3	-19.68	54.48	671.01
		6.00	-19.68	54.48	344.14
		12.00	-19.68	54.48	17.28
		18.00	-19.68	54.48	-309.59
		4	-19.68	54.48	-636.45
	DIF_SET1	3	10.75	99.23	477.10
		6.00	10.75	99.23	-118.25
		12.00	10.75	99.23	-713.61
		18.00	10.75	99.23	-1308.96
		4	10.75	99.23	-1904.32
	DIF_SET2	3	-10.06	-304.02	-2926.08
		6.00	-10.06	-304.02	-1101.93
		12.00	-10.06	-304.02	722.21
		18.00	-10.06	-304.02	2546.35
		4	-10.06	-304.02	4370.50
	DED_LOD	3	-2.69	-439.10	-1672.36
		6.00	-2.69	-160.91	29.59

240

12.00	-2.69	117.55	861.51

2/90

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	/--- AXIAL FORCE	---/--- LOCAL Y SHEAR FORCE	--/-- LOCAL Z MOMENT --/
		18.00	-2.69	162.11	22.39
		4	-2.69	440.30	-1686.72
4	SDWK_LL	4	0.21	2.16	-0.44
		6.00	0.21	2.16	-13.41
		12.00	0.21	2.16	-26.38
		18.00	0.21	2.16	-39.36
CAP		5	0.21	2.16	-52.33
	LL_I_1	4	-1.91	-151.91	-554.49
		6.00	-1.91	-151.91	357.00
		12.00	-1.91	-12.61	549.69
		18.00	-1.91	100.19	429.11
		5	-1.91	100.19	-172.00
	LL_I_2	4	-1.62	-111.91	-344.88
		6.00	-1.62	-111.91	326.58
		12.00	-1.62	-17.51	510.93
		18.00	-1.62	95.19	419.88
		5	-1.62	95.19	-151.27
	LL_I_3	4	-0.03	-4.55	-110.98
		6.00	-0.03	-4.55	-83.68
		12.00	-0.03	-4.55	-56.38
		18.00	-0.03	-4.55	-29.08
		5	-0.03	-4.55	-1.78
	THER_EXP	4	-0.86	3.38	25.46
		6.00	-0.86	3.38	5.21
		12.00	-0.86	3.38	-15.04
		18.00	-0.86	3.38	-35.29
		5	-0.86	3.38	-55.54
	WIND	4	-0.26	1.18	11.17
		6.00	-0.26	1.18	4.10
		12.00	-0.26	1.18	-2.97
		18.00	-0.26	1.18	-10.05
		5	-0.26	1.18	-17.12
	DT_GRP_2	4	0.06	-0.75	-14.33
		6.00	0.06	-0.75	-9.83
		12.00	0.06	-0.75	-5.32
		18.00	0.06	-0.75	-0.82
		5	0.06	-0.75	3.68

2/90

RESULTANT MEMBER FORCES -----

MEMBER NO.	SECTION TAG.	JOINT NO.	/--- AXIAL ---/ FORCE	/--- LOCAL Y --/ SHEAR FORCE	/-- LOCAL Z --/ MOMENT
	STREAM	4	-8.56	88.13	828.70
		6.00	-8.56	88.13	299.90
		12.00	-8.56	88.13	-228.90
		18.00	-8.56	88.13	-757.70
		5	-8.56	88.13	-1286.49
	DIF_SET1	4	8.89	-112.05	-2058.86
		6.00	8.89	-112.05	-1386.58
		12.00	8.89	-112.05	-714.30
		18.00	8.89	-112.05	-42.01
		5	8.89	-112.05	630.27
	DIF_SET2	4	-13.61	211.27	4126.27
		6.00	-13.61	211.27	2858.63
		12.00	-13.61	211.27	1590.99
		18.00	-13.61	211.27	323.35
		5	-13.61	211.27	-944.29
	DED_LOD	4	-2.90	-371.35	-1697.00
		6.00	-2.90	-327.34	398.95
		12.00	-2.90	-49.71	1024.74
		18.00	-2.90	235.35	772.09
		5	-2.90	278.53	-769.69

Summary of Group I Moments w/ Cap @ 6ft x 4 5/8 ft

	Member 2					Member 3					Member 4				
	0.0	0.25	0.5	0.75	1.0	0.0	0.25	0.5	0.75	1.0	0.0	0.25	0.5	0.75	1.00
DED_LOD	-785	758	1016	394	-1700	-1672	30	862	22	-1685	-1697	399	1025	772	-770
DIF_SET 1	311	251	191	131	70	477	-118	-714	-1309	-1904	-2059	-1387	-714	-42	630
DIF_SET 2	69	-496	-1061	-1625	-2190	-2926	-1102	722	2546	4371	4126	2859	1591	323	-944
SDWK_LL	4	3	1	-1	-2	3	1	-1	-3	-5	-1	-13	-27	-39	-52
LL_I_1	3	-34	-71	-108	-145	-202	-14	174	30	-481	-554	357	550	429	-172
LL_I_2	-177	402	501	325	-339	-253	-240	-242	-243	-258	-345	327	511	420	-151
LL_I_3	-73	212	231	94	-330	-317	10	288	85	-167	-111	-84	-56	-29	-2
STREAM	1031	580	130	-320	-770	671	344	17	-310	-636	829	300	-229	-758	-1286

	-785	758	1016	394	-1700	-1672	30	862	22	-1685	-1697	399	1025	772	-770
Max Pos Mom	630	323	1591	2859	4126	4371	2546	722	2546	4371	4126	2859	1591	323	630
	4	3	1			3	1		1	3			1	3	4
	3	402	550	357			85	288	85			357	550	420	3
	1031	580	130			671	344	17			829	300			
	883	2064	3288	3610	2426	3373	3006	1889	2664	2689	3258	3915	3167	1518	-133
	-785	758	1016	394	-1700	-1670	30	862	22	-1685	-1697	399	1025	772	-770
Max Neg Mom	-944	-496	-1061	-1625	-2190	-2926	-1309	-714	-1309	-2926	-2190	-1625	-1061	-496	-944
	-52	-39	-27	-13	-2	-5	-3	-1	-3	-5	-2	-13	-27	-39	-52
	-177	-34	-71	-108	-554	-481	-240	-242	-243	-481	-554	-108	-71	-34	-151
				-320	-770				-310	-636			-229	-758	-1286
	-1958	189	-143	-1672	-5216	-5082	-1522	-95	-1843	-5733	-4443	-1347	-363	-555	-3203

* For Moment Reversal Possibilities see sht. CAP 23

MINNEAPOLIS PHOENIX

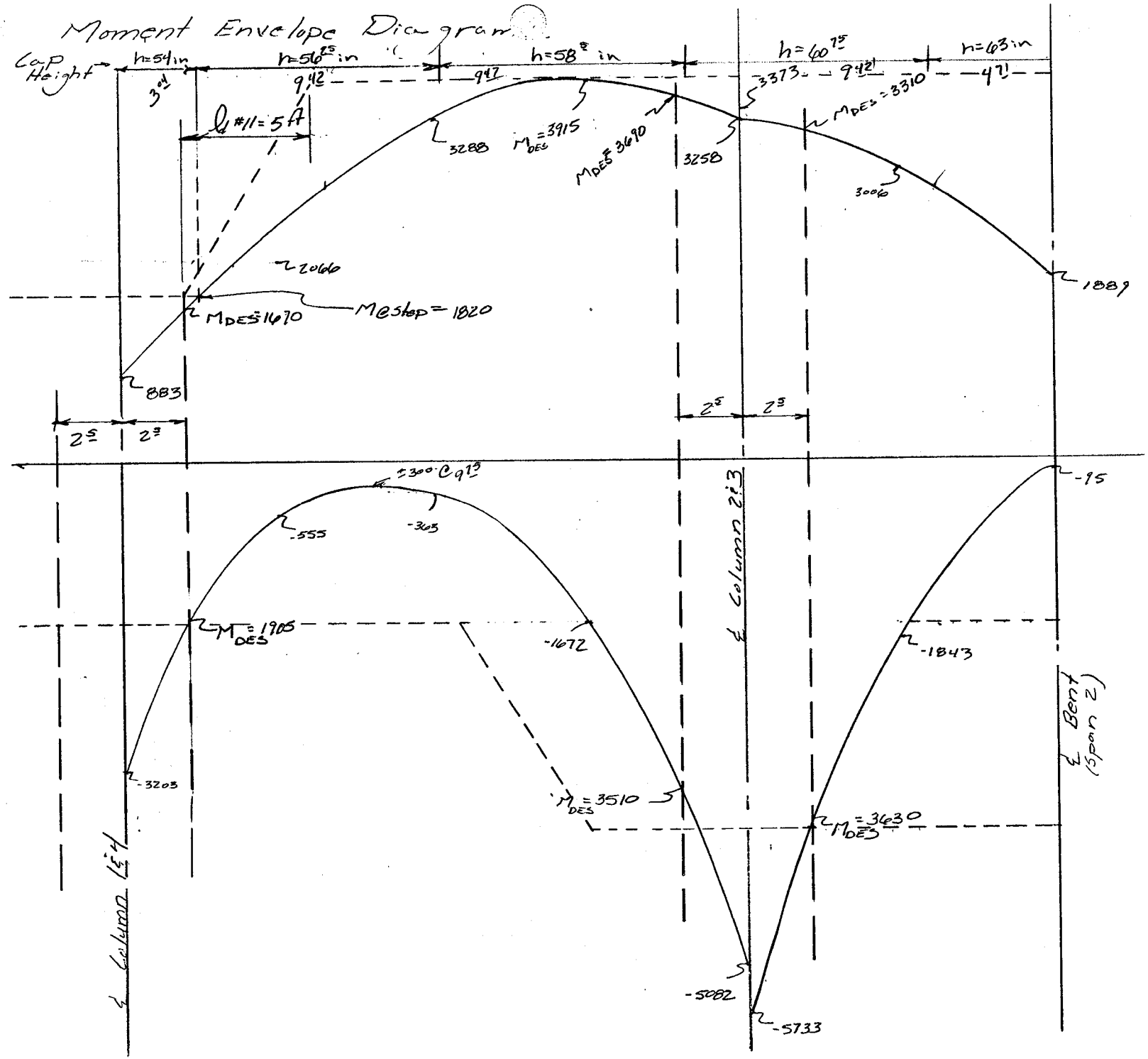
JOB Northern Ave & Nevi River SHEET NO. CAP 35

MADE BY T Stokes DATE 1-19-90 JOB NO. 3308962

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 243

CALCULATIONS FOR Cap Design

Moment Envelope Diagram



JOB Northern Pac. & New River SHEET NO. CAP 36

MADE BY T Stones DATE 1-22-90 JOB NO. 33-9962

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 244

CALCULATIONS FOR Cap Design

IBRW MINNEAPOLIS PHOENIX

In Max Moment Regions increased cap depth due to girder step will be utilized to reduce reinforcement requirements. Place Main, Continuous steel in region where $h = 56^{25}$ in. Assume #11 bars bundle by two's vertically.

$d_{avail} = 56^{25} - 2 - .625 - 1.41 = 52.22$ in

Check Section:

$n = 8$ $f_c = 1600$ psi $f_s = 24{,}000$ psi $k_b = 0.348$ $j_b = 0.884$

Max. Pos. Mom.

$$bd^2 = \frac{2M}{f_c jk} = \frac{2(3915)(12)}{1.6(.884)(.348)} = 190{,}894 \text{ in}^3 \Rightarrow d_{req} = 51.5 \text{ in} < 52.2 \checkmark$$

for this section $d_{avail} = 58.5 - 2 - 0.625 - 1.41 = 54.47$

$$A_{s_{req}} = \frac{M}{f_s jd} = \frac{3915\,(12)}{24\,(.884)(54.47)} = 40.66 \text{ in}^2$$ use 26 #11 $A_{s_{prov}} = 40.56 \checkmark$ close enough

Postive Mom. Rein. Req'd Over Ext. Columns. use $M = 1820$ ft-k

$d_{avail} = 54 - 2 - .625 - 1.41/2 = 50.67$ in

$$A_s = \frac{1820\,(12)}{24(.884)(50.67)} = 20.32 \text{ in}^2$$ use 13 #11 $A_{s_{prov}} = 20.28$ close enough

Neg Mom. Rein. Over Int Col:

$M = 3630$ $d = 52.22$ in

$$A_{s_{req}} = \frac{3630\,(12)}{24(.884)(52.22)} = 39.32 \text{ in}^2$$ use 26 #11 $= 40.56$ in^2

Neg. Mom. Rein. Over Ext. Column $d = 54 - 2 - .625 - 1.41 = 50$ in

$$A_{s_{req}} = \frac{1905\,(12)}{24(.884)(50)} = 21.55 \text{ in}^2$$ use 14 #11 $A_{s_{prov}} = 21.84$ in^2

* Check Cracking Moment $f_r = 7.5\sqrt{f'_c} = 474$ psi

$M_{cr} = f_r I_g / y_t$

Section	I_g in^4	y_t in	M_{cr}
1st Step	944,784	27 in	= 1.66 E7 in-p = 1382 ft-k ✓
3rd Step	1,201,210	29.25 in	1.95 E7 = 1622 ft-k ✓
4th Step	1,345,210	30.375	2.10 E7 = 1750 ft-k ✓

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. CAP 37
MADE BY T Stones DATE 1-22-90 JOB NO. 33-8962
CHECKED BY Lee Arnold DATE 2/90 SEC. SHT. NO. 245
CALCULATIONS FOR Cap Design

Shear

	Member 1		Member 2					Member 3					Member 4					Mem 5	
	0.5	1.0	0.0	0.25	0.5	0.75	1.0	0.0	0.25	0.5	0.75	1.0	0.0	0.25	0.5	0.75	1.0	0.0	0.5
DED_LOD	14	334	-279	-236	49	327	371	-439	-161	118	162	440	-371	-327	-50	235	279	-334	-14
DIF_SET 1			10	10	10	10	10	99	99	99	99	99	-112	-112	-112	-112	-112		
DIF_SET 2			94	94	94	94	94	-304	-304	-304	-304	-304	211	211	211	211	211		
$DWL_LL			0	0	0	0	0	0	0	0	0	0	2	2	2	2	2	-40	0
LL_I_1			6	6	6	6	6	-31	-31	24	24	166	-152	-152	-13	100	100	-24	
LL_I_2	24	24	-96	-96	16	111	111	-5	0	0	0	6	-112	-112	-18	95	95	-24	0
LL_I_3			-47	-47	15	71	71	-65	-46	34	34	53	-5	-5	-5	-5	-5		
WIND			1	1	1	1	1	1	1	1	1	1	1	1	1	1	1		
OT_GRP_2			46	46	46	-119	-119	10	10	10	10	10	-1	-1	-1	-1	-1		
STREAM			75	75	75	75	75	54	54	54	54	54	88	88	88	88	88		
THER_EXP			-4	-4	-4	-4	-4	0	0	0	0	0	3	3	3	3	3		

Group I Will Control Shear: The only significant shear not included in Group I is Member 1, OT_GRP_2. This can be included in Group II However there would be no live load and there would be a reduction for overstress.

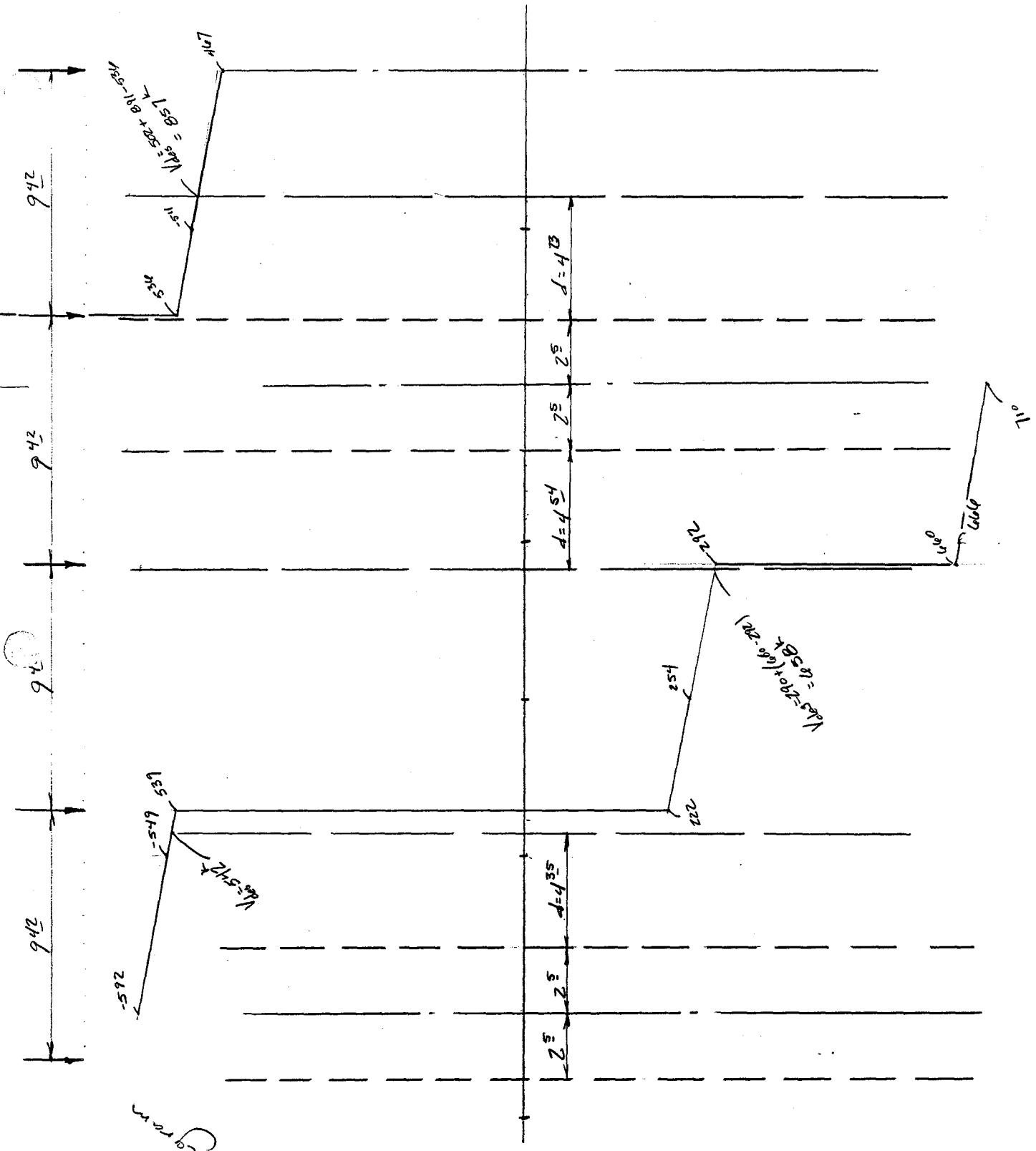
JOB Northern Ave Culvert River SHEET NO. CAP 38

MADE BY T. Stones DATE 1-22-80 JOB NO. 33-082

CHECKED BY L. Arnold DATE 2/80 SEC. SHT. NO. 246

CALCULATIONS FOR Cap Design

BRW MINNEAPOLIS PHOENIX



JOB Northern Ave @ New River SHEET NO. CAP 39

MADE BY T Stones DATE 1-22-90 JOB NO. 33-8962

CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 247

CALCULATIONS FOR Cap Design

BRW

MINNEAPOLIS PHOENIX

Middle Spans: $V_{des} = 857^k$ $d = 4^{73} ft$ $b = 72''$

$v = \frac{V}{b_w d} = \frac{857}{72(4^{73})(12)} = 0.210$ ksi

$V_c = 0.95\sqrt{f'_c} = 0.95\frac{\sqrt{4000}}{1000} = 0.060$ ksi

$V_s = V - V_c = 0.210 - 0.06 = 0.15$ ksi

if spacing is 6" $A_v = \frac{0.15 \text{ ksi}(72 \text{ in})(6 \text{ in})}{24 \text{ ksi}} = 2.7 \text{ in}^2$

this will req. 8.7 legs of #5 Tie use 6 #5

$A_v = 6(0.31) = 1.86 \text{ in}^2$

$s = \frac{1.86(24)}{0.15(72)} = 4.133$ in use $s = 4$ in

@ d from face $V = 502^k$ $v = \frac{502}{72(4^{73})(12)} = 0.123$ ksi

$s = \frac{1.86(24)}{(0.123-0.06)(72)} = 9.9$ in use $s = 9$ in

@ Mid span $V = 467^k$ $v = \frac{467}{72(4^{73})(12)} = 0.114$

$s = \frac{1.86(24)}{(0.114-0.06)(72)} = 11.4$ keep 9 in spacing

Outer Spans: Int. Col. $V = 658^k$ $d = 4^{54}(12) = 54.5$ in

$v = \frac{658}{72(54.5)} = 0.168$ $s = \frac{1.86(24)}{(0.168-0.06)72} = 5.75$ in

@ d from face $v = \frac{290}{72(54.5)} = 0.074$

$s = \frac{1.86(24)}{(0.074-0.06)(72)} = 44$ in 24 in. max may Apply

Ext. Col. $V = 542^k$ $d = 4^{35}(12) = 52.2$ in

$v = \frac{542}{72(522)} = 0.144$ $s = \frac{1.86(24)}{(.144-0.06)(72)} = 7.36$ use $s = 6$ in

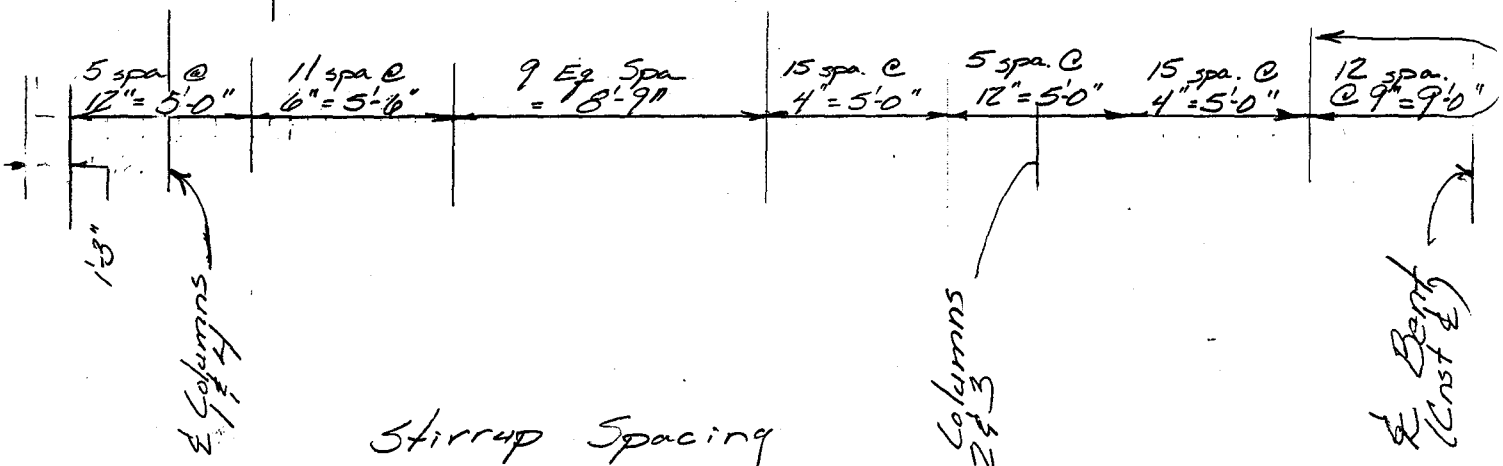
BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave a New River SHEET NO. CAP 40
MADE BY T Stones DATE 1-22-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 248
CALCULATIONS FOR Cap Design

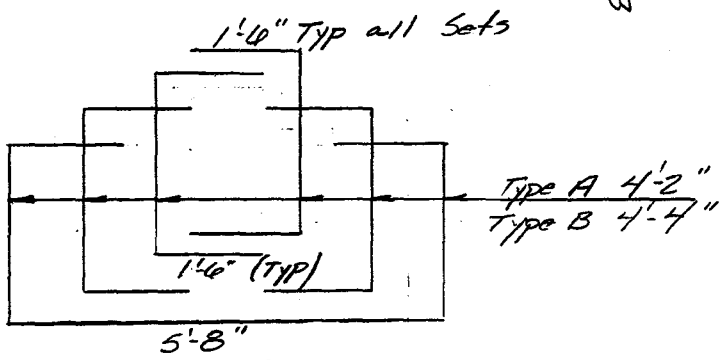
Outer Spans 7.75 from ℄ Ext Column

$v = \frac{222}{72(52.2)} = 0.059 < v_c$ 24 in max spacing may apply

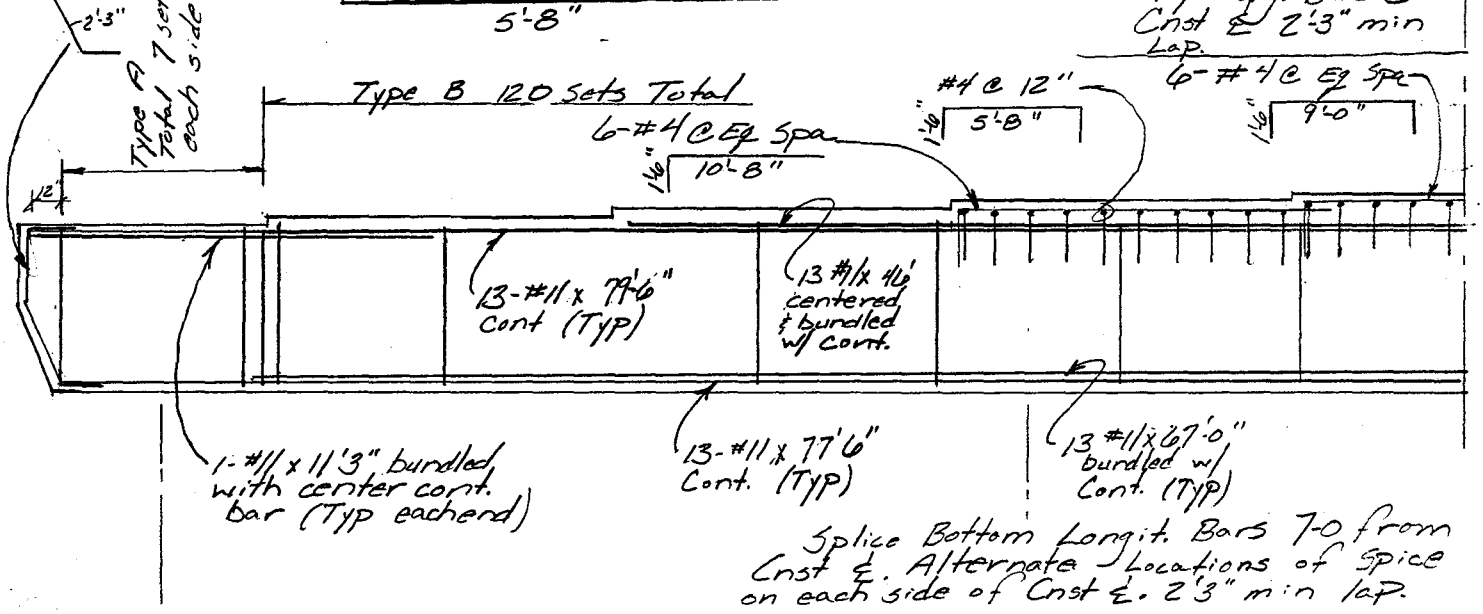
14 sets Type A | 120 Sets Type B } Total # of Stirrup Sets/Bent



Stirrup Spacing



Symetrical About Cnst ℄



Splice Bottom Longit. Bars 7-0 from Cnst ℄. Alternate Locations of Spice on each side of Cnst ℄. 2'3" min lap.

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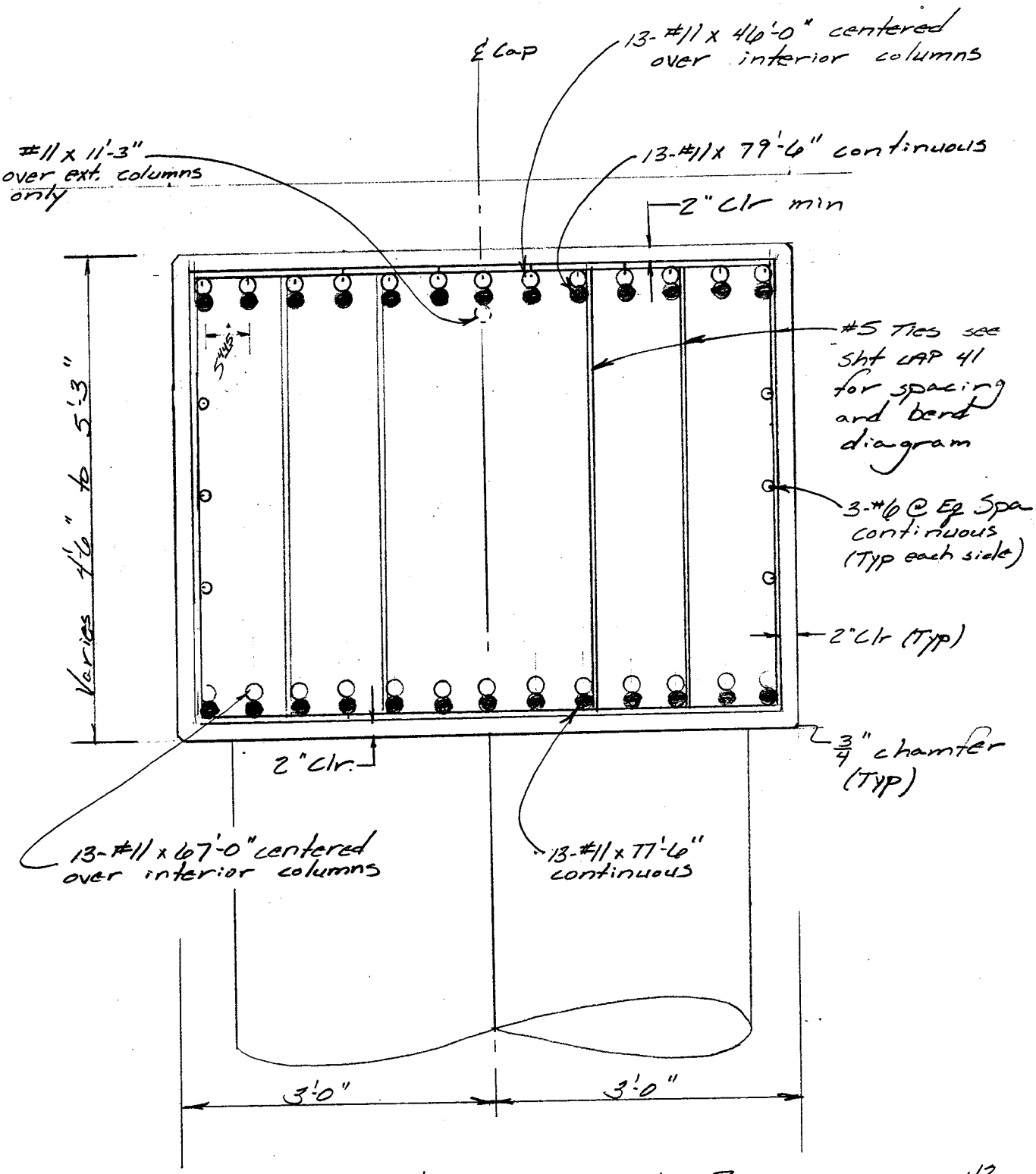
JOB Northern Ave @ New River SHEET NO. CAP 4/1
MADE BY J Stones DATE 1-22-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 249
CALCULATIONS FOR Cap Design

For additional Reinforcement in Girdes Steps in middle span see sht CAP-41



BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. CAP 42
MADE BY T Stones DATE 1-22-90 JOB NO. 33-8962
CHECKED BY L Arnold DATE 2/90 SEC. SHT. NO. 250
CALCULATIONS FOR Cap Design

QUANTITIES

DRILLED SHAFTS, f'_c = 4000 PSI

PIER 1-3 W/ 4 SHAFTS EACH
DIAMETER 6'-6" R: 3.25'

TOP OF SHAFT 1058.3
TIP OF SHAFT 992.0
h 66.3'

VOL: $\pi (3.25')^2 \, 66.3'/27$ = 81.5 CY / SHAFT

TOTAL 12 (81.5) = 977.8 CY

ABUTMENTS 1:2 f'_c = 4000 PSI

DIAMETER = 6.5'

	TOP DECK			T/SHAFT = BTM ABUT
ABUT 1	1084.90	- [~~45.25~~ 37.667' (.02 ft/ft) + 6.33' + 4.0']		= 1073.81
ABUT 2	1084.08	- 11.09		= 1072.99

	T/SHAFT	- TIP OF SHAFT		h
ABUT 1	1072.81	- 997.0	=	75.81
ABUT 2	1072.99	- 997.0	=	75.99

$VOL_{ABUT\,1}$ / SHAFT = $\pi (3.25 \text{ ft})^2 (75.81 \text{ ft})/27$ = 93.2 CY

ABUT 1 3 SHAFTS 3(93.2 CY) = 279.6 CY

$VOL_{ABUT\,2}$ / SHAFT = $\pi (3.25 \text{ ft})^2 (75.99)/27$ = 93.4 CY

ABUT 2 3 SHAFTS 3(93.4 CY) 280.2 CY

VOL DRILL SHAFTS = 559.8 CY f'_c = 4000 PSI

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MINNEAPOLIS PHOENIX

JOB NORTHGAN AVE & NEW RIVER SHEET NO. C-1
MADE BY MED DATE 2 FEB 90 JOB NO. 33-8962
CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 251
CALCULATIONS FOR QUANTITIES

COLUMN CONCRETE

f'_c = 4000 PSI

DIAM = 5'-0" RAD = 2.5'

	T/DECK ℄	- [8" + 2½" + 5'-3" + 1½" + 5'-3"]	= T/COL
PIER 1	1085.32	- 11.50	= 1073.82
PIER 2	1085.34	- 11.50	1073.84
PIER 3	1084.93	- 11.50	1073.43

	T/COL	- T/SHAFT	= h
PIER 1	1073.82	- 1058.3	15.52
PIER 2	1073.84	- 1058.3	15.54
PIER 3	1073.43	- 1058.3	15.13

VOL COL/PIER1 $\pi(2.5')^2\ 15.52'/27$ = 11.3 CY
4 COL/PIER : 4(11.3) = 45.20 CY

VOL COL/PIER2 $\pi(2.5)^2\ 15.54'/27$ = 11.3 CY
4 COL/PIER 4(11.3) = 45.2 CY

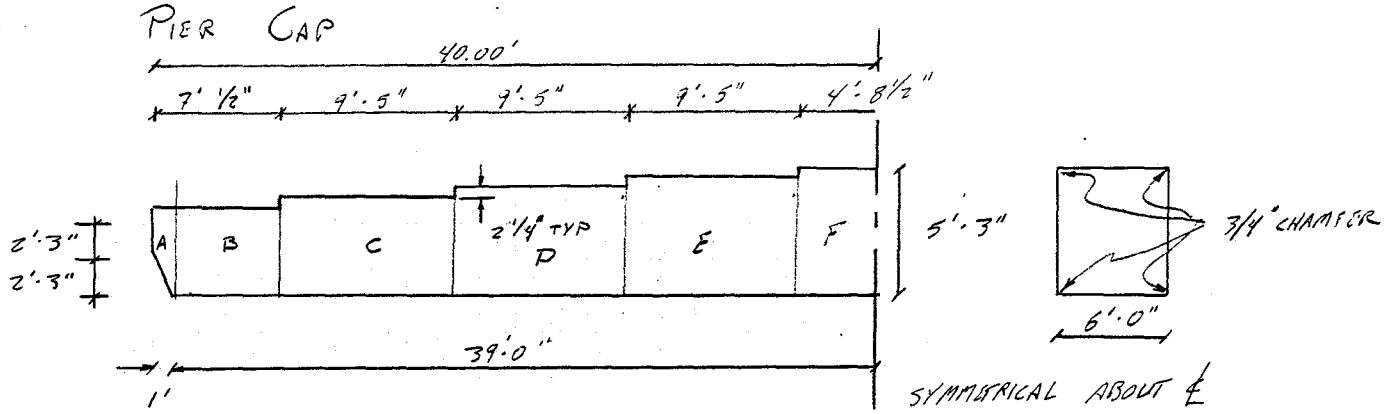
VOL COL/PIER3 $\pi(2.5)^2\ 15.13'/27$ = 11.0 CY
4 COL/PIER 4(11.0) = 44.0 CY

TOTAL VOL f'_c = 4000 PSI CONC 134.4 CY

BRW
MINNEAPOLIS PHOENIX

JOB NORTHERN AVE @ NEW RIVER SHEET NO. Q-2
MADE BY MED DATE 2 FEB 90 JOB NO. 33-8962
CHECKED BY J Stones DATE 2/90 SEC. SHT. NO. 252
CALCULATIONS FOR QUANTITIES

PIER CAP



VOL SECT A

$[(2.25')(1.0')(6.0') + (2.25')(1.0')(6.0')/2]$ = 12.0 ft³

SECT B

$(6.0417')(4.5')(6.0')$ = 163.125 ft³

SECT C

$(9.4167)(4.6875')(6.0')$ = 264.844 ft³

SECT D

$(9.4167)(4.875')(6.0')$ = 275.4375 ft³

SECT E

$(9.4167)(5.0625)(6.0')$ 286.0313 ft³

SECT F

$(4.7083)(5.25')(6.0')$ 148.3125 ft³

1149.75 ft³ ≐ 42.6 CY

(3/4" CHAMFER = $2[(.0625)^2/2\,(40')] + 2[.0625^2/2\,(39')]/27$ = .01 CY)

2 (42.6 CY) = 85.2 CY/PIER

VOL PIER CAPS = 255.6 CY f'_c = 4000 PSI

JOB NORTHERN AVE @ NEW RIVER SHEET NO. 8-3

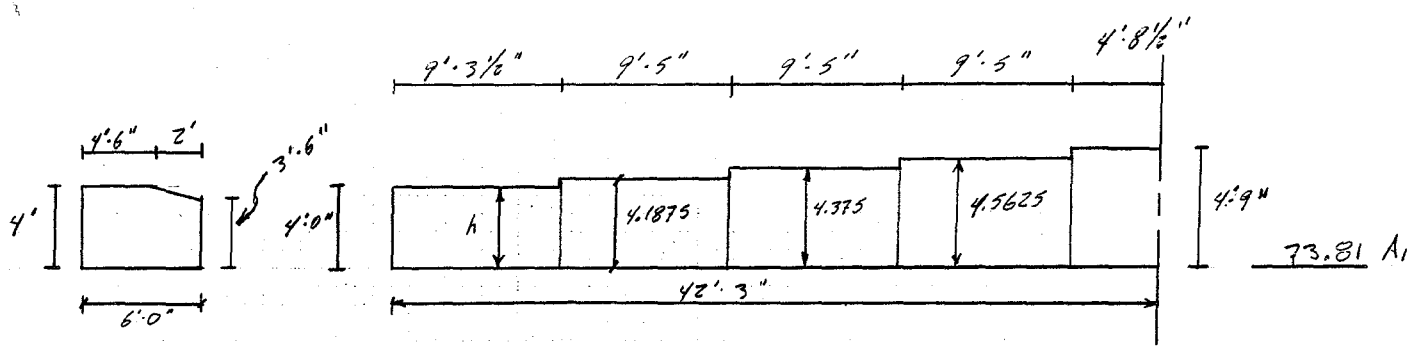
MADE BY MED DATE 2 FEB 90 JOB NO. 33-8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 253

CALCULATIONS FOR QUANTITIES

BRW MINNEAPOLIS PHOENIX

ABUTMENT CAP



X-SECT AREA $6'(h) - 2'(.5')/2 = 6h' - .5ft^2$

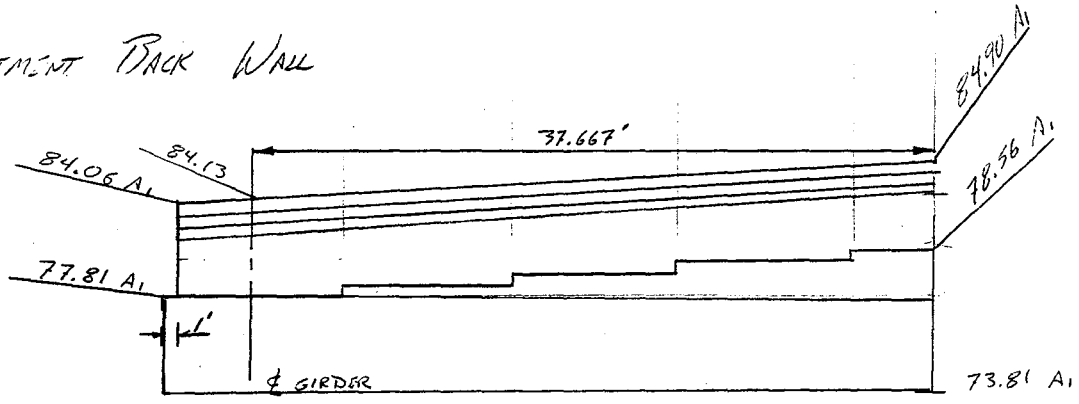
VOL ABUT CAP $2[(6'(4') - .5ft^2)9.2917) + (6'(4.1875') - .5ft^2)9.4167 + (6'(4.375') - .5ft^2)9.4167$
$+ (6'(4.5625') - .5ft^2)9.4167 + (6'(4.75') - .5ft^2)4.7083]$

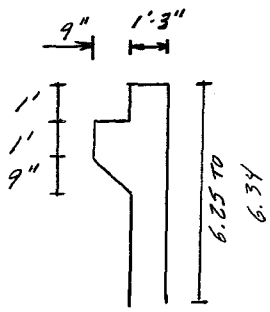
$= 2[218.35ft^2 + 231.89ft^2 + 242.48ft^2 + 253.07ft^2 + 151.83]$

$= 2155.25 ft^3 = 79.8$ CY

2 ABUTMENTS $f'_c = 4000$ PSI **159.6 CY**

ABUTMENT BACK WALL



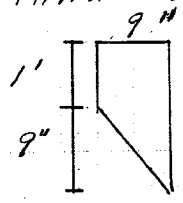


BRW
MINNEAPOLIS PHOENIX

JOB NORTHERN AVE @ NEW RIVER SHEET NO. C-4
MADE BY MED DATE 2 FEB 90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 754
CALCULATIONS FOR QUANTITIES

ABUTMENT BACK WALL (CONT)

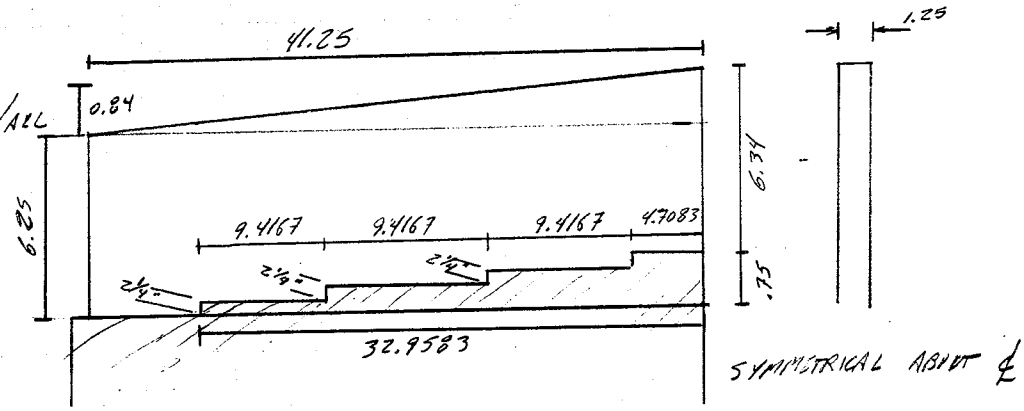
VOL APPROACH SLAB KEY



X SECT AREA = .75'(1') + .75'(1')/2 = 1.125 ft²

VOL SLAB KEY 2(41.25')(1.125 ft²)/27 = 3.44 CY

VOL BK WALL



$$Vol/2 = 1.25'\left[6.25'(41.25) + \tfrac{1}{2}(.84')(41.25) - \left(9.4167(2.25" + 4.5" + 6.75") + 4.7083(9")\right)\right]/27$$

$$= 1.25'[275.14\ ft^2 - 14.125\ ft^2]/27 = 12.1\ CY$$

VOL ABUT 1 f'c = 4000 PSI = 27.64 CY

ABUT 2 " = 27.64 CY

55.3 CY f'c = 4000 PSI IN ABUT BACK WALL

+ 159.6 CY f'c = 4000 PSI IN ABUT CAPS

214.9 CY f'c 4000 PSI IN ABUTMENTS

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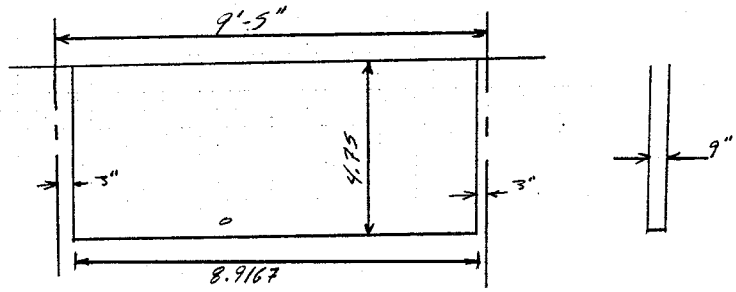
MINNEAPOLIS PHOENIX

JOB Northern Arc Neu River SHEET NO. Q-5
MADE BY MED DATE 2 FEB 90 JOB NO. 53-8962
CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 255
CALCULATIONS FOR Quantities

INTERMEDIATE DIAPHRAGM

8 SPA PER / DIAPHRAGM , 8 INTERIOR DIAPHRAGMS TOTAL



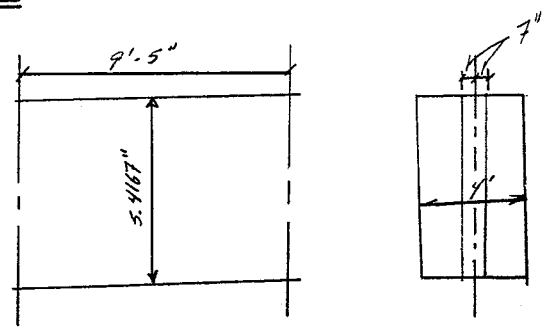
X SECT. AREA OF AASHTO TYPE V MOD GIRDER
= 887 sq in = 6.16 ft²

VOLUME 1 SPA = 9.4167(4.75)(.75) - 6.16 ft²(.75')/27
= 1.07 CY

8 SPA/DIAPHRAGM : 8 DIAPHRAGMS

64 SPA = 68.6 CU YDS

PIER DIAPHRAGM



8 SPA PER PIER 3 PIER

VOL 1 SPA = (5.4167'(9.4167')(4.0') - 6.16 ft²(2.833')/27 = 6.9 CY

24 (6.9 CY) = 165.8 CY

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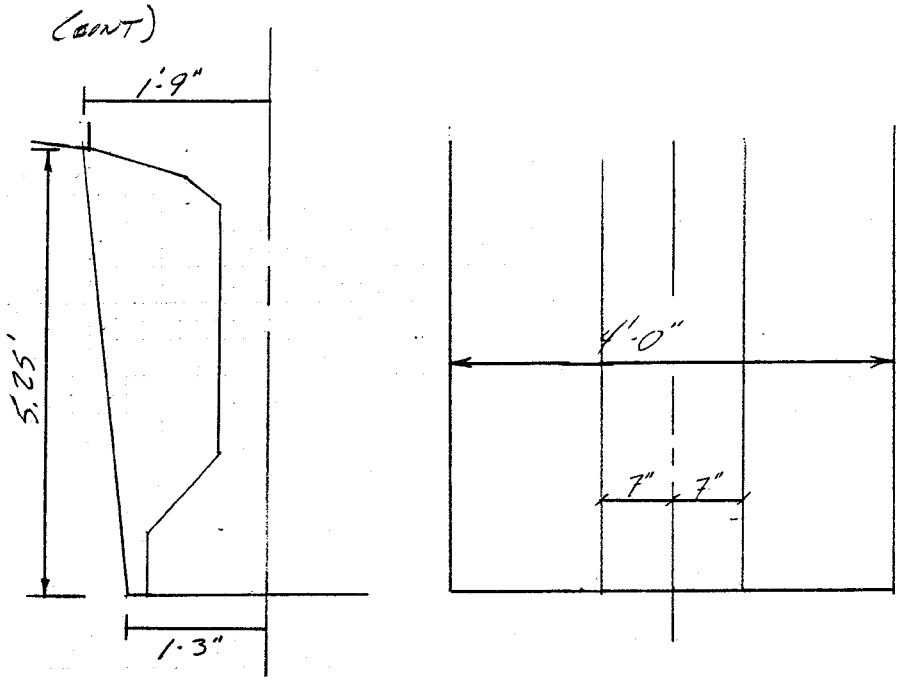
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. CP-6
MADE BY MO DATE 2 Feb 90 JOB NO. 50-9962
CHECKED BY TStores DATE 2/90 SEC. SHT. NO. 2560
CALCULATIONS FOR Quantities

PIER DIAPHRAGM (CONT)



END TREATMENT 2/PIER = 6

$$\frac{\frac{1.75'+1.25'}{2}(5.25')(4.0') - \frac{6.16ft^2}{2}(2.833')}{27} \quad .872 \text{ CY}$$

$6(.872) = \underline{5.2 \text{ CY}}$

VOL $f'_c = 4000$ PSI IN PIER DIAPHRAGMS

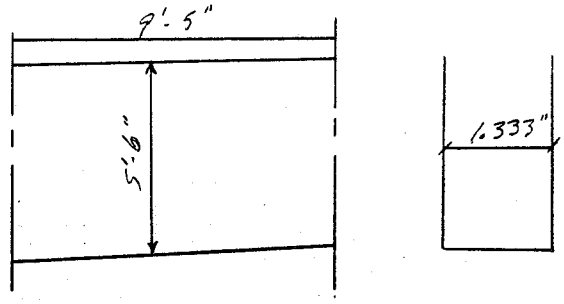
165.8 CY.
5.2 CY

171.0 CY

JOB Northern Ave @ New River SHEET NO. A-7
MADE BY MJD DATE 2 Feb 90 JOB NO. 32-8962
CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 257
CALCULATIONS FOR Quantities

BRW
MINNEAPOLIS PHOENIX

ABUTMENT DIAPHRAGM



TYPE V MOD GIRDER

VOL 1 SPA

$$\left[(9.4167)(5.5') - 6.16\,ft^2\right] 1.333/27 = 2.25\ CY$$

8 SPA PER ABUT

$$16\ (2.25\ CY) = \underline{\underline{36.0\ CY}}$$

ABUTMENT DIAPHRAGMS	36.0 CY	
PIER DIAPHRAGMS	163.2 CY	171
INTERIOR DIAPHRAGMS	68.6 CY	

267.8 CY f'c = 4000 PSI IN DIAPHRAGMS

2756

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. Q-8
MADE BY MED DATE 2 Feb 90 JOB NO. 32-88-32
CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 258
CALCULATIONS FOR QUANTITIES

ROADWAY SLAB

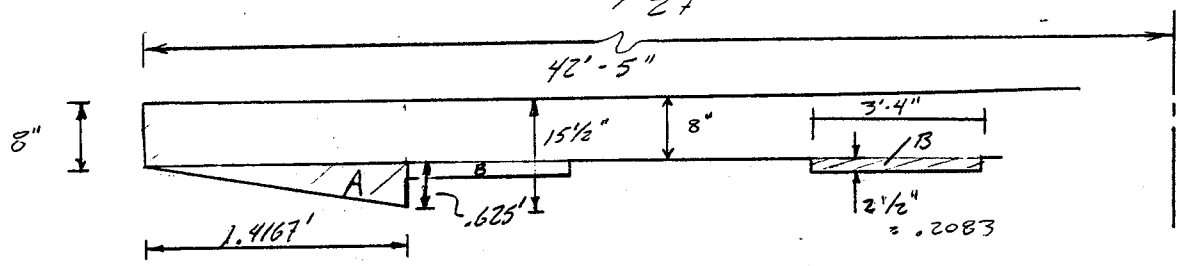
BEG BRIDGE 44+18.20

END BRIDGE 48+11.20

393.0'

SLAB THICKNESS 8" DECK WIDTH 84'-10"

$393'(84.833')(.667')/27 =$ 823.2 CY



SLAB THICKENING

VOL A $393.0\left(\frac{.625'(1.4167)}{2}\right)/27 = 14.6$ CY

ONE EACH SIDE 2(14.6 CY) 29.2 CY

VOL B $393(.2083)(3.333')/27 = 10.1$ CY

ONE PER GIRDER / TOTAL 9 =

9(10.1 CY) 90.9 CY

f'_c 4000 PSI CONC IN DECK SLAB 943.3 CY

JOB NORTHERN AVE @ NEW RIVER SHEET NO. A-9

MADE BY MTD DATE 2 FEB 90 JOB NO. 32-8962

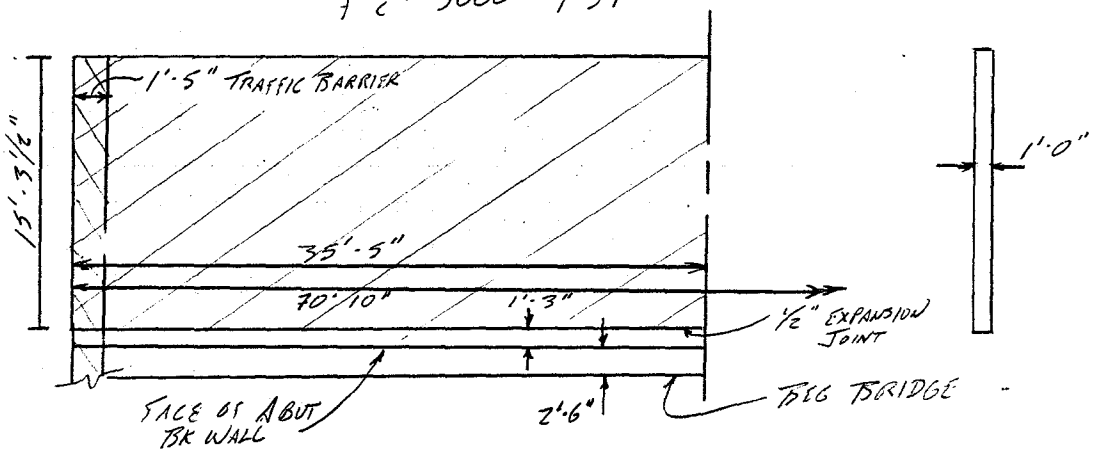
CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 259

CALCULATIONS FOR

BRW MINNEAPOLIS PHOENIX

APPROACH SLAB

f'_c = 3000 PSI

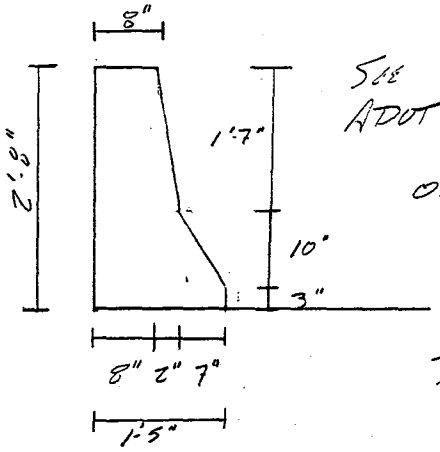


$VOL_{APP\ SLAB}$ = 70.833'(15.2917')(1.0')/27 = 40.1 CY

ONE SLAB EACH END

f'_c = 3000 PSI 2(40.1) = 80.2 CY

TRAFFIC BARRIER



SEE ADOT STANDARD DWG B 21.18

0.0918 CY/ft

BEG BRDG TO BK FACE ABUT WALL TOP ABUT BK WALL APPROACH SLAB & JOINT FILLER

393.00 + 2(2.5' + 1.25' + 15.333')

= 431.1667' BARRIER EACH SIDE

ADOT STND DRW B-21.20 { 4 BARRIER TRANSITIONS = 4(16.5833') = W/ 1.516 CY f'_c = 3000 PSI CONC

TOTAL f'_c = 3000 PSI CONC IN BARRIER =

2[(431.1667' − 2(16.5833')) 0.0918 CY] + 4(1.516 CY) = 79.14 CY

BRW MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. Q-10

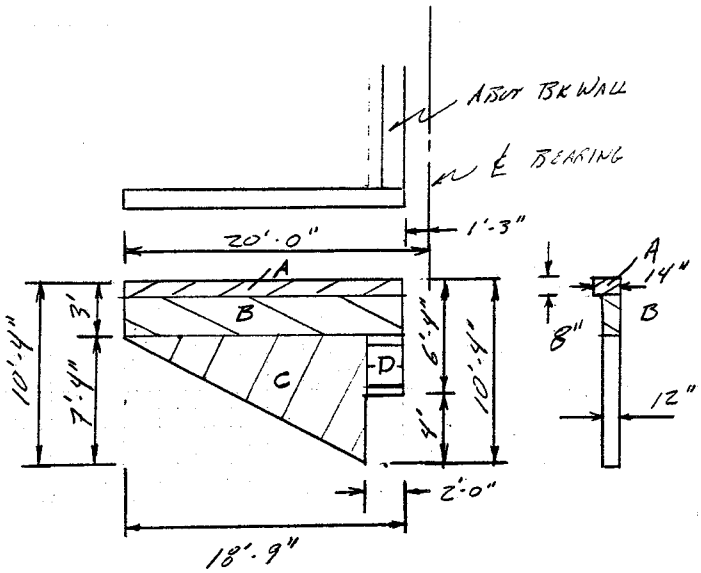
MADE BY MED DATE 2 FEB 90 JOB NO. 55 8962

CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 260

CALCULATIONS FOR

WING WALLS

4 EACH



$$Vol_A = 18.75'(.667')(1.1667')/27 = 0.54\ CY$$

$$Vol_B = 18.75'(2.333')(1.0')/27 = 1.62\ CY$$

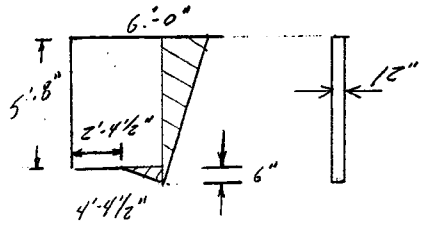
$$Vol_C = \tfrac{1}{2}\ 16.75(7.333)(1.0)/27 = 2.27\ CY$$

$$Vol_D = 2.0'(3.333')(1.0)/27 = \underline{0.25\ CY}$$

4.7 CY ABISCE

Vol f'_c = 4000 PSI IN 4 WINGWALLS = 4(4.7CY) = 18.7 CY

CURTAIN WALL



$$Vol = 5.667'(4.375')(1.0') + (\tfrac{1}{2}(2.0')(.5')(1.0')) + \tfrac{1}{2}(6.1667)(1.625')(1.0')/27$$

$$= 1.12\ CY$$

4 TOTAL = 4.5 CY f_c = 4000 PSI

JOB NORTHVIEW AVE @ NEW RIVER SHEET NO. Q-11

MADE BY MFD DATE 2 FEB 90 JOB NO. 33 3962

CHECKED BY TJ Jones DATE 2/90 SEC. SHT. NO. 261

CALCULATIONS FOR QUANTITIES

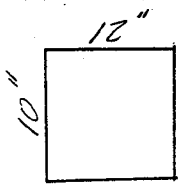
BRW MINNEAPOLIS PHOENIX

PEDESTRIAN CURB

BEG WINGWALL$_{1,2}$ 44+00.70

END WINGWALL$_{3,4}$ 48+28.70

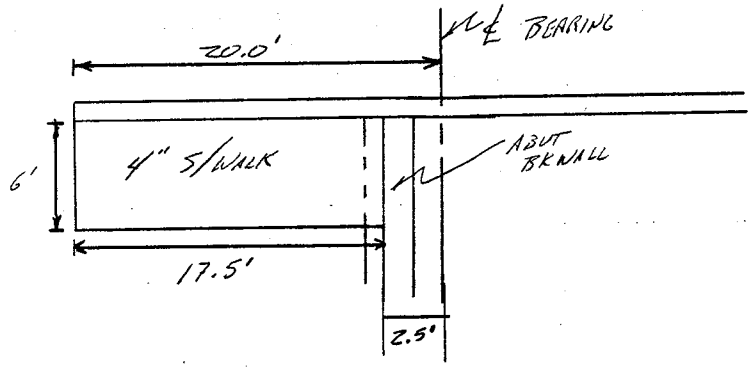
428.0 ft CURB EACH SIDE



VOL f'_c = 3000 PSI CONC IN PED CURB

$= 2(428.0')(1.0')(0.8333')/27 = \underline{26.4 \text{ CY}}$

SIDE WALK



4 EACH $17.5'(6.0')(0.3333')/27 = 1.3$ CY

VOL f'_c = 3000PSI CONC IN SIDEWALKS = $4(1.3 \text{CY}) = \underline{5.2 \text{ CY}}$

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. Q-11A
MADE BY MED DATE 2 Feb 90 JOB NO. 358962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 262
CALCULATIONS FOR

f'_c = 4000 PSI CONCRETE

DRILLED SHAFTS IN COLS (12 TOT)	977.8 CY	PS Q-1
DRILLED SHAFTS IN ABUT (6 TOT)	559.8 CY	PS Q-1
COLUMNS (12 TOT)	134.4 CY	PS Q-2
PIER CAPS (3 TOT)	255.6 CY	PS Q-3
ABUTMENT CAP : BK WALL (2 TOT)	214.9 CY	PS Q-5
INTERMEDIATE DIAPHRAGM (8 TOT)	68.6 CY	PS Q-6
PIER DIAPHRAGM (3 TOT)	171.0 CY	PS Q-7
ABUTMENT DIAPHRAGM (2 TOT)	36.0	PS Q-8
ROAD WAY SLAB	943.3 CY	PS Q9
WING WALLS (4 TOT)	18.7 CY	PS Q-11
CURTAIN WALL (4 TOT)	4.5	PS Q-11
TOTAL f'_c = 4000 PSI CONC	3385 CY	

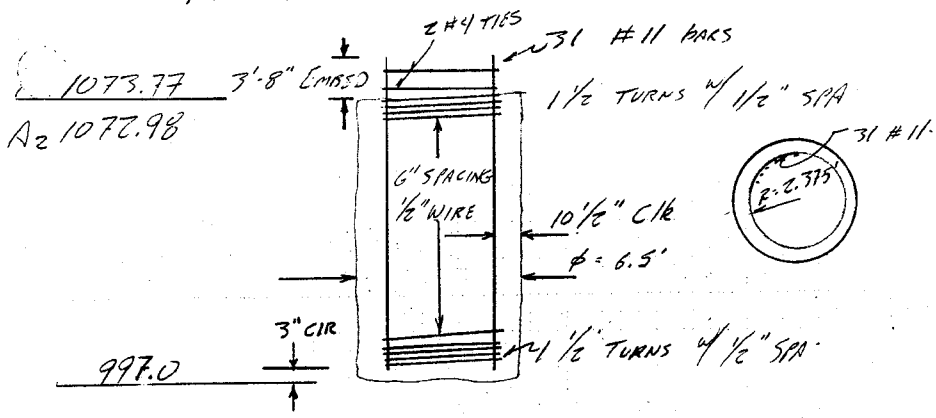
f'_c = 3000 PSI CONCRETE

APPROACH SLAB	80.2 CY	PS Q-10
TRAFFIC BARRIER	79.1 CY	PS Q-10
PEDESTRIAN CURB	26.4 CY	PS Q-11A
SIDE WALK	5.2 CY	PS Q-11A
TOTAL f'_c = 3000 PSI CONC	191 CY	

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave & Kern River SHEET NO. Q-11B
MADE BY MJD DATE 6 Feb 90 JOB NO. 55 8962
CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 263
CALCULATIONS FOR Quantities

REINFORCING IN DRILLED SHAFTS



$C = 2\pi(2.375')$
$= 14.92'$

1 1/2 TURNS @ TOP & BTM

BTM FT $12" - 3/4" = 11 1/4"$ W/ 6" SPA
$= 1.875$ TURNS

$+ 1.5$ TURNS @ 1/2" SPA
3.375 TURNS

$W/ft = 31\#11 + 14.92(2\text{ turns})$ (1/2" COLD DRAWN WIRE)

$31(5.313\ lb/ft) + 14.92'(2\text{ turn})(.668\ lb/ft) = 184.636\ lb/ft$

$W_{SHAFT\ A1} = 74.52'(184.636\ lb/ft) + 31(5.313\ lb/ft)(3.667')$ EMBED

$+ 2(31(5.313\ lb/ft) + 3.375\text{ turns}(14.92')(.668\ lb/ft))$ TOP & BTM FT OF SPIRAL

$= \underline{14759.7\ lb}$ / DRILL SHAFT ABUT 1

$W_{SHAFT\ A2} = 73.73'(184.636\ lb/ft) + 31(5.313\ lb/ft)(3.667')$

$+ 2(31(5.313\ lb/ft) + 3.375(14.92')(.668\ lb/ft)$

$= \underline{14613.8\ lb}$

LAP SPLICES $(8.333')(5.313\ lb/ft)(31) = 1372.5\ lb$ / SHAFT

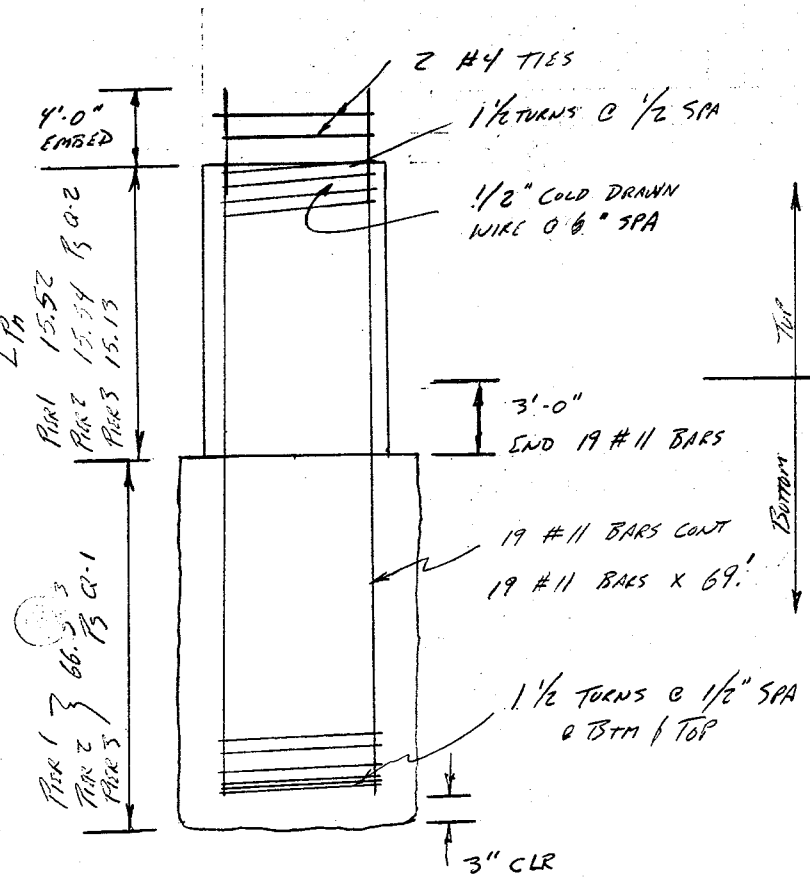
TOTAL STEEL IN DRILL SHAFTS

$6(2(14.92')(.668\ lb/ft)) + 3(14{,}759.7 + 1372.5) + 3(14613.8\ lb + 1372.5\ lb) = \underline{96{,}475.1\ lb}$

BRW MINNEAPOLIS PHOENIX

JOB NORTHERN AVE NEW RIVER SHEET NO. Q-12
MADE BY MED DATE 5 FEB 90 JOB NO. 55 8962
CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 264
CALCULATIONS FOR QUANTITIES

DRILLED SHAFT / COLUMN REINFORCING



OF TURNS 1ft

CIRCUMFERENCE

3/8 WIRE

$W/ft = 19\ \#11\ bars + 2(14.92')(.668\ lb/ft)$

$19(5.313) + 2(14.92')(.668\ lb/ft) = 120.88\ lb/ft$

TOP

$W/ft = 38(5.313\ lb) + 2(14.92')(.668\ lb/ft)$

$= 221.827\ lb/ft$

BOTTOM

15306.0630

EXTRA TURNS

$W/COL = 69.0'(221.827\ lb/ft) + 2(1.5(14.92')(.668\ lb/ft))$

15759.68

$(L_{P_n} - 3.0')(120.88\ lb/ft) + 19(4.0')(5.313\ lb/ft)$ EMBED $+ 2(14.92')(.668\ lb/ft) =$

1634.2992

$W_{COL\ 1}$	17283.1 lb	x 4	= 69092.4 lb
$W_{COL\ 2}$	17275.5 lb	x 4	= 69102.1 lb
$W_{COL\ 3}$	17226.0 lb	x 4	= 68903.8 lb

STEEL IN PIER COLUMNS W/OUT SPLICES: 207,098 lb

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MINNEAPOLIS PHOENIX

JOB Northern Ave @ Nicollet Creek SHEET NO. 0-13

MADE BY MKD DATE 2 FEB 90 JOB NO. 55 8962

CHECKED BY T Stokes DATE 2/90 SEC. SHT. NO. 265

CALCULATIONS FOR

PIER SHAFTS & COLUMNS (CONT)

1 SPLICE / #11 BAR

38 #11 PER COLUMN, 12 COLUMNS

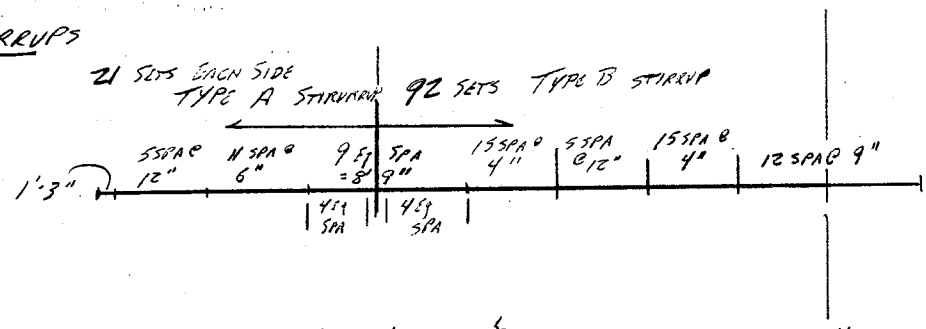
38 (8.3333') (5.313 lb/ft) (12) = 20189.4 lb

2 #4 TIES IN EMBED / PER COLUMN

TOTAL STEEL IN PIER SHAFTS / COLUMNS 227,287.7 lb

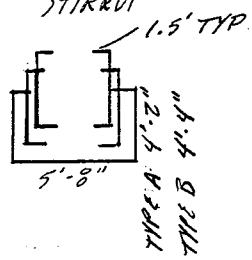
PIER CAP STEEL

#5 STIRRUPS



TYPE A STIRRUP

#5 BARS



4 [1.5' + 1.5' + 4.1667'] 1.043 lb/ft = 29.9 lb

[2(1.5') + 2(4.1667') + 5.6667'] 1.043 lb/ft = 17.7 lb

STIRRUP A WEIGHT PER SET = 47.6 lb

42 NEEDED 1999.2 lbs

TYPE B STIRRUP

#5 BARS

4 (1.5' + 1.5' + 4.333') 1.043 lb/ft = 30.6 lb

[2(1.5') + 2(4.333') + 5.6667'] 1.043 lb/ft = 18.1 lb

STIRRUP B WEIGHT PER SET 48.7 lb

92 NEEDED 4480 lb

JOB Northern Ave @ New River SHEET NO. Q-14

MADE BY MED DATE 2 Feb 90 JOB NO. 338962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 266

CALCULATIONS FOR Quantities

BRW

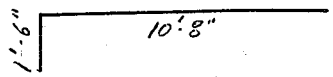
MINNEAPOLIS PHOENIX

PIER CAP STEEL (CONT)

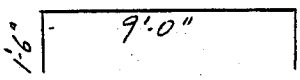
TOTAL STEEL IN #5 TIES PER PIER CAP = 6480 lb

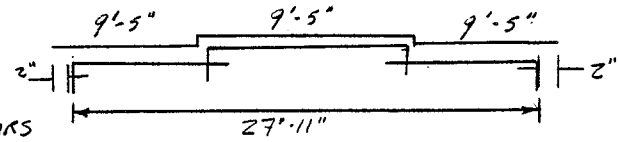
PIER CAP STEP REINFORCING

2(6 #4 @ EQ SPA) = 2(6[(.668)(10.667'+1.5')]) = 97.5 lb

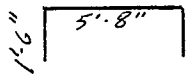


6 #4 @ EQ SPA = 6[(.668)((9'-0" + 2(1'-6"))] = 48.1 lb





#4 @ 12" FOR 27'-11" .. 29 BARS

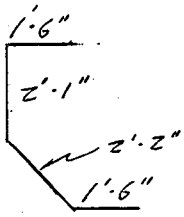


29[(.668)((5'-8") + 2(1'-6"))] = 167.9 lb

TOTAL 313.5 lb

END REINFORCING

6 #6 @ EQ SPA EACH END



12(1.502)(2(1'-6") + 2'-1" + 2'-2")

= 130.7 lb

TOTAL REIN IN PIER CAP STEPS & END REIN 444.2 lb

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MINNEAPOLIS PHOENIX

JOB NORTHERN AVE @ NW RIVER SHEET NO. Q-15
MADE BY MED DATE 5 FEB 90 JOB NO. 338962
CHECKED BY T Stokes DATE 2/90 SEC. SHT. NO. 267
CALCULATIONS FOR QUANTITIES

PIER CAP REIN (CONT)

LONGITUDINAL STEEL

TOP

13 #11 x 46'-9"

13 (5.313 lb/ft) (46.75') = 3229.0 lb

13 #11 x 77'-6" CONT

SPLICE

13 (5.313 lb/ft) (77.5' + 2.25') = 5508.3 lb

1 #11 x 11'-3" CONT

1 (5.313 lb/ft) (11.25") = 59.8 lb

TOTAL LONGIT REIN IN TOP PIER CAP 8797.1 lb

BOTTOM

13 #11 x 67'-0"

SPLICE

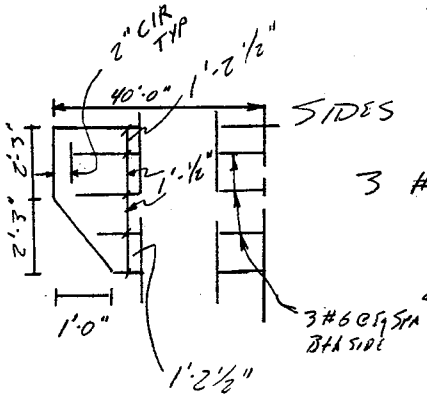
13 (5.313 lb/ft) (67.0' + 2.25') = 4783.0 lb

13 #11 x 77'-6" CONT

SPLICE

13 (5.313 lb/ft) (77.5' + 2.25') = 5508.3 lb

TOTAL LONGIT REIN IN BTM PIER CAP 10291.3 lb



3 #6 @ Eq SPA CONT BOTH SIDES

2 (1.502 lb/ft) (2 (79'-8") + 78'-8") = 715.0 lb

TOTAL LONGITUDINAL REIN IN PIER CAP 19803.4 lb

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. Q-16
MADE BY MSD DATE 5 Feb 90 JOB NO. 33-8962
CHECKED BY TJones DATE 2/90 SEC. SHT. NO. 26B
CALCULATIONS FOR Quantities

PIER CAP REIN (CONT.)

VERTICAL BARS 6 #11 x 7'-6" ONE SET PER GIRDER

9 [6 (5.313 lb/ft) (7.5') = 2151.8 lb

STIRRUPS	6480 lb	P3 Q-15
STEP & END REIN	444.2 lb	P3 Q-15
LONGITUDINAL REIN	19803.4 lb	P3 Q-16
VERTICAL BARS	2151.4 lb	

28,879 lb / PIER CAP

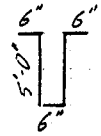
TOTAL REIN IN 3 PIER CAPS 86,637 lb

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave c New River SHEET NO. Q-17
MADE BY MED DATE 5 FEB 90 JOB NO. 33-8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 269
CALCULATIONS FOR Quantities

INTERMEDIATE DIAPHRAGM REINFORCING

STIRRUPS : 6 #5 @ 1'-1"



PER SECTION, 8 SECTIONS / DIAPHRAGM

$= 8\left[6(1.043\ lb/ft)(2(5'\text{-}0") + 3(0'\text{-}6"))\right]$

$= 575.7$ lb PER DIAPHRAGM

LONGITUDINAL REIN

5 - #5 x 7'-0" PER SECTION 8 SECTIONS PER DIAPHRAGM
EACH SIDE

$= 8\left[2(5)(1.043\ lb/ft)(7'\text{-}0")\right]$

$= 584.1$ lb PER DIAPHRAGM

OR 3/4" ϕ x 2'6" ROD

INSERTS 4 - #6 x 2'-6" 2 SETS PER SECTION 8 SECTIONS PER DIAPHRAGM

$= 8\left[2(4)(1.502\ lb/ft)(2.5')\right]$

$= 240.3$ lb PER DIAPHRAGM

TOTAL REIN IN 1 INTERMEDIATE DIAPHRAGM 1400.1 lb

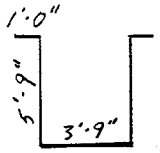
8 INTERMEDIATE DIAPHRAGMS $= 8(1400.1\ lb) =$ 11200.8 lb

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. Q-18
MADE BY MED DATE 5 Feb 90 JOB NO. 53 8962
CHECKED BY T Jones DATE 2-90 SEC. SHT. NO. 270
CALCULATIONS FOR QUANTITIES

PIER DIAPHRAGM REIN

STIRRUPS 6 #5 @ 1'-1"



PER SECTION, 8 SECTS/DIAPHRAGM

$= 8[6(1.043\ lb/ft)(2(1'-0") + 2(5'-9") + (3'-9"))]$

$= 863.6$ lb PER DIAPHRAGM

LONG REIN 5 - #5 x 7'-0" BOTH SIDES PER SECTION, 8 SECTIONS / DIAPHRAGM

$8[2(5)(1.043\ lb/ft)(7'-0")]$

$= 584.1$ lb PER DIAPHRAGM

INSERTS 5 - #6 x 2'-6" [OR 3/4" ϕ x 2'-6" RODS] 2 SETS EACH SECTION, 8 SECTIONS / DIAPHRAGM

$= 8[2(5)(1.502\ lb/ft)(2'-6")]$

$= 300.4$ lb PER DIAPHRAGM

END REINFORCING

4 #4 x 5'-0" PER END

$2(4(.668\ lb/ft)(5'-0") =$

26.7 lb / DIAPHRAGM

4 #6 x 2'-6" INSERTS PER END

$2(4(1.502\ lb/ft)(2'-6")$

$= 30.04$ lb PER DIAPHRAGM

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. Q-19
MADE BY MED DATE 5 FEB 90 JOB NO. 33 8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 271
CALCULATIONS FOR QUANTIES

PIER DIAPHRAGM REIN

END REINFORCING (CONT)

#4 SPACED W/ #6 INSERTS PER EACH END

3'-9" 1'-0"

$2(4(.668\ lb/ft)(2(1'-0") + (3'-9")))$

$= 30.7\ lb$

TOTAL END REINFORCING PER PIER

$26.7\ lb + 30.0\ lb + 30.7\ lb = 87.4\ lb$ PER DIAPHRAGM

TOTAL REIN PER PIER DIAPHRAGM

STIRRUPS	863.6 lb
LONGIT REIN	584.1 lb
INSERTS	500.4 lb
END REIN	87.4 lb
	1835.5 lb PER PIER DIAPHRAGM

3 PIER DIAPHRAGMS : 3(1835.5 lb) = 5506.5 lb

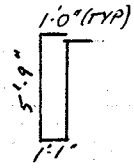
NOTE: 6 #11 x 7'-0" VERT BARS @ EACH GIRDER COUNTED WITH PIER CAP REINFORCING

BRW
MINNEAPOLIS PHOENIX

JOB NORTHERN AVE @ NEW RIVER SHEET NO. Q-20
MADE BY MED DATE 5 FEB 90 JOB NO. 33-8962
CHECKED BY TSTORES DATE 2/90 SEC. SHT. NO. 272
CALCULATIONS FOR

ABUTMENT DIAPHRAGMS

STIRRUPS 5 #5 x 7'-0" PER SECTION 8 SECTIONS / DIAPHRAGM



$= 8(5(1.043\ lb/ft)(2(5'\text{-}9") + 2(1'\text{-}0") + (1'\text{-}1")))$

$= 608.4$ lb PER DIAPHRAGM

LONGIT REIN 5 #5 x 7'-0" PER SECTION 8 SECTIONS / DIAPHRAGM
BOTH SIDES

$= 2[8(5(1.043\ lb/ft)(7'\text{-}0"))]$

$= 584.1$ lb PER DIAPHRAGM

INSERTS 5 #6 x 2'-6" (OR 3/4" ϕ x 2'-6" ROD) BOTH SIDES PER SET

2 SETS PER SECTION 8 SECTIONS PER DIAPHRAGM

$8(2(2(5(1.502\ lb/ft)(2'\text{-}6"))))$

$= 600.8$ lb PER DIAPHRAGM

TOTAL REIN IN ABUTMENT DIAPHRAGM

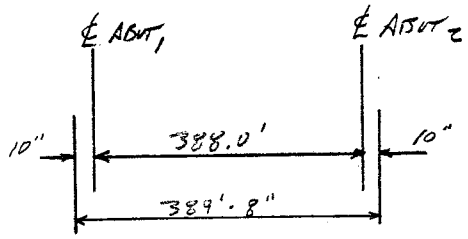
608.4 lb + 584.1 lb + 600.8 lb = 1793.3 lb PER DIAPHRAGM

2 ABUTMENT DIAPHRAGMS 3586.6 lb

BRW
MINNEAPOLIS PHOENIX

JOB NORTHERN AVE @ NEW RIVER SHEET NO. Q-21
MADE BY MED DATE 5 FEB 90 JOB NO. 33 8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 273
CALCULATIONS FOR QUANTITIES

DECK REINFORCEMENT



TOP LONGIT REINF

5 #5 x 23'-10" & 5 #5 x 25'-10" EACH GIRDER EACH END

$2(9)[5(23.833' + 25.833')(1.043\ lb/ft)] = \underline{4662.2\ lb}$

10 #5 x 60'-0" EACH GIRDER EACH SPAN

$4(9)[10(60')(1.043\ lb/ft)] = \underline{22,528.8\ lb}$

5 #10 x 38'-0" & 5 #10 x 42'-0" EACH GIRDER 2 OUTSIDE PIERS

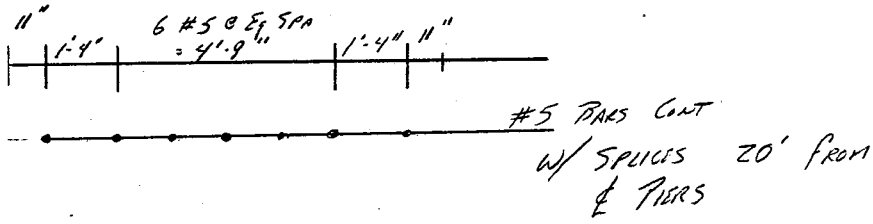
$2(9)[5(38.0' + 42.0')(4.303\ lb/ft)] = \underline{30981.6\ lb}$

5 #10 x 32'-0" & 5 #10 x 36'-0" EACH GIRDER CENTER PIER

$9[5(32.0' + 42.0')(4.303\ lb/ft)] = \underline{14,329.0\ lb}$

TOTAL LONGIT REIN TOP DECK = $\underline{\underline{72501.6\ lb}}$

BOTTOM LONGIT REIN



$9(7(389.667' + 6(1.5'\ \text{SPLICE}))(1.043\ lb/ft) = \underline{26196.0\ lb}$

BRW
MINNEAPOLIS PHOENIX

JOB NORTHERN AVE @ NEW RIVER SHEET NO. Q-22
MADE BY MED DATE 5 FEB 90 JOB NO. 33 8962
CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 274
CALCULATIONS FOR QUANTITIES

DECK REINFORCING

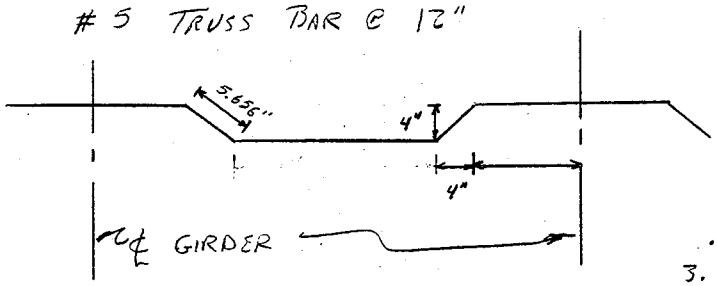
TRANSVERSE TOP

#5 STRAIGHT BAR @ 12" 84'-6"

389'-8" @ 12" = 390 SPA 391 BARS

391 (84'-6" + 1'-0" LAP) (1.043 lb/ft) = 34,868.0 lb

#5 TRUSS BAR @ 12"



.8284" PER BEND EXTRA
3.3137" 4 BENDS PER BTWN GIRDER
26.5097" 8 SPA BTWN GIRDERS
= 2.21' EXTRA PER TRUSS

LENGTH = 84'-6" + 2.21' = 86.71'

390 [(86.71') (1.043 lb/ft) + (1'-0" LAP) (1.043 lb/ft)]

= 35,677.8 lb

TRANSVERSE BOTTOM

#4 STRAIGHT BAR @ 12" 391 BARS

391 (84'-6" + 1'-0" LAP) (0.668 lb/ft)

= 22331.6 lb

TOTAL TRANSVERSE REIN IN DECK = 92877.4 lb

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MINNEAPOLIS PHOENIX

JOB NORTHERN AVE @ NEW RIVER SHEET NO. Q-23
MADE BY MED DATE 5 FEB 90 JOB NO. 338962
CHECKED BY TJones DATE 2/90 SEC. SHT. NO. 275
CALCULATIONS FOR QUANTITIES

DECK REINFORCING CONT

EXTRA REINFORCING @ DECK DRAINS

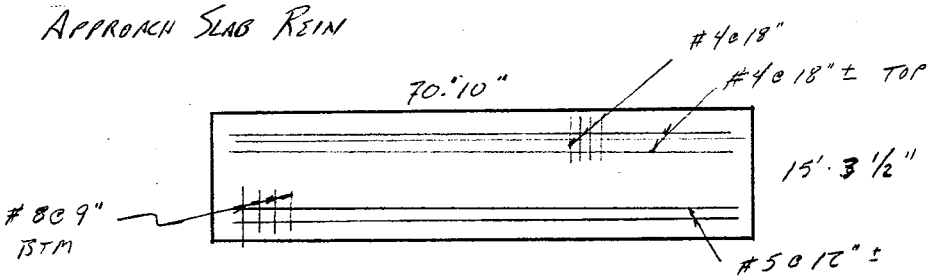
2 #4 x 2'-0" @ EACH DRAIN TOTAL 8 DRAINS

$8(2)(.668\ lb/ft)(2'\cdot0") = 21.4\ lb$

TOTAL REINFORCING STEEL IN DECK

TOP LONGIT	72 501.6	lb	PG Q-22
BTM LONGIT	26 196.0	lb	PG Q-22
TOP & BTM TRANSVERSE	92 877.4	lb	PG Q-23
EXTRA REIN @ DECK DRAIN	21.4	lb	FROM ABOVE
	191 596.4	lb	

APPROACH SLAB REIN



TOP STEEL #4 @ 18" TRANS & LONGIT

$$\frac{4(18")(.668\ \frac{lb}{ft})(\frac{1ft}{12"})}{[18"(\frac{1ft}{12"})]^2} = 1.7813\ lb/ft^2$$

BTM STEEL #5 @ 12" TRANS #8 @ 9" LONGIT

$$\frac{4(12")(1.043\ lb/ft)(1ft/12")}{[12"(\frac{1ft}{12"})]^2} + \frac{4(9")(2.670\ lb/ft^2)(1ft/12")}{[9"(\frac{1ft}{12"})]^2} =$$

JOB NORTHERN AVE @ NEW RIVER SHEET NO. Q-24

MADE BY MED DATE 5 FEB 90 JOB NO. 55 8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 276

CALCULATIONS FOR QUANTITIES

BRW MINNEAPOLIS PHOENIX

APPROACH SLAB REIN (CONT)

BTM STEEL (CONT.)

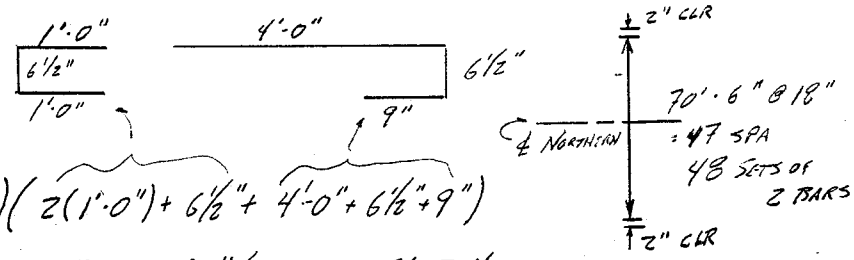
$= 4.172\ lb/ft^2 + 14.240\ lb/ft^2 = 18.412\ lb/ft^2$

$(15'\text{-}3\tfrac{1}{2}" - 2(2"))(70'\text{-}10" - 2(2"))(1.7813\ lb/ft^2 + 18.412\ lb/ft^2)$

$= 1054.56\ ft^2 (20.1933\ lb/ft^2)$

TRANS & LONGIT STEEL IN APP SLAB = 21295.1 lb

#4 @ 18"



$48(.668\ lb/ft)(2(1'\text{-}0") + 6\tfrac{1}{2}" + 4'\text{-}0" + 6\tfrac{1}{2}" + 9")$

$= 48(.668)(7.833)\ lb/ft = 251.2\ lb$

$= 251.2\ lb$

REIN IN APPROACH SLAB

TRANSVERSE & LONGIT	21295.1 lb
#4 TIES @ 18"	251.2 lb
	502.3 lb

BRW
MINNEAPOLIS PHOENIX

JOB NORTHERN AVE @ NEW RIVER SHEET NO. Q-25
MADE BY MED DATE 5 FEB 90 JOB NO. 33 8962
CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 277
CALCULATIONS FOR QUANTITIES

TRAFFIC BARRIER

431.1667' BARRIER BOTH SIDES Q-10

15.50' BARRIER TRANS 2 EACH END, TOTAL 4

2(431.1667') - 4(15.50') =

= 800.333' BARRIER (SECTION A)

1 #6 CONT	=	1.502 lb/ft
2 #5 CONT	= 2(1.043 lb/ft)	2.086 lb/ft
6 #4 CONT	6(.668 lb/ft)	2.672 lb/ft
#4 2'-4" @ 9"	= $\frac{1}{.75}$((2'-5" + 6" (STND HOOK))(.668 lb/ft)	= 2.5978 lb/ft
#4 x 1'-11" @ 18"	= $\frac{1}{1.5}$(1'-11")(.668 lb/ft)	= 0.8536 lb/ft
#4 x ⌐ @ 18"	= $\frac{1}{1.5}$((1'-3") + 9" + (1'-1") + (1'-7"))(.668 lb/ft)	= 2.0782 lb/ft
		11.79 lb/ft

BARRIER REINFORCING BOTH SIDES =

= 800.333'(11.79 lb/ft) = 9435.6 lb

BARRIER TRANSITION 2 EACH END 4 TOTAL

REINFORCING STEEL 141 lb FROM ADOT STND DRWG B-21.20

8 (141 lb) 1128 lb

TOTAL REIN IN TRAFFIC BARRIER 10564 lb

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ Nev River SHEET NO. Q-26
MADE BY MED DATE 5 FEB 90 JOB NO. 33 8962
CHECKED BY TStones DATE 2/90 SEC. SHT. NO. 278
CALCULATIONS FOR Quantities

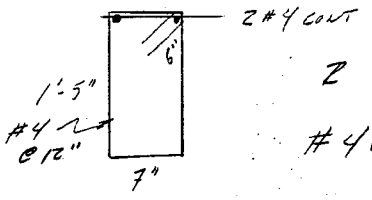
PEDESTRIAN CURBS

BEG WINGWALL, 44+00.70

END WINGWALL: 48+28.70

428.0' PED CURB BOTH SIDES

856.0' TOTAL



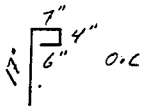
2 #4 CONT : .668 lb/ft

#4 @ 12" : $.668\ lb/ft \left[\frac{2(7") + 2(1'-5") + 2(6")}{12} \right]$

= 2.56 lb/ft

$856'(2.56\ lb/ft)$ = 2192 lb

8 DECK DRAINS

1 #4 

o.c. $.668\ lb/ft \left(\frac{17+7+4+6}{12} \right)'$ = 1.893 lb/DRAIN

8 DRAINS = 15.4 lb

REINFORCING STEEL IN PED CURBS : 2207.1 lb

41" CH LINK FENCE 853 l.f.

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ Nor River SHEET NO. Q-27
MADE BY MED DATE 5 Feb 90 JOB NO. ST 3962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 279
CALCULATIONS FOR Quantities

GIRDERS ; MODIFIED TYPE V

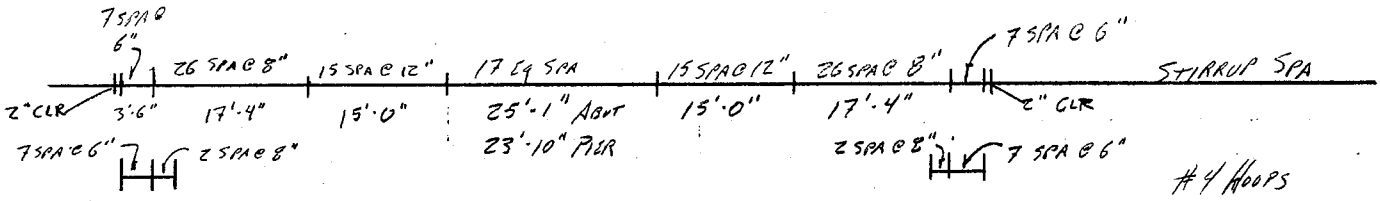
L_G = 18 x 97'-1" @ ABUTMENTS

18 x 95'-10" @ PIERS

ADD'L REINFORCING,

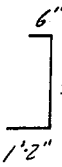
	ABUT	PIERS
2 #4 CONT, $= 2(L_G - 4")(.668\ lb/ft)$ =	129.26 lb	127.59 lb

8 bars + 26 bars + 15 bars + 17 bars + 15 bars + 26 bars + 7 bars = 114 SETS



8 SETS + 2 SETS = 10 SETS BOTH ENDS = 20 SETS TOTAL

STIRRUPS

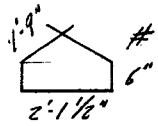
2 #5 

5'-7" PER SET @ 1 #4 x 2'-10" EACH SET

$$= 2\left(\frac{6" + 67" + 14"}{12"} ft\right)(1.043\ lb/ft) + \frac{34"}{12"} ft\,(.668\ lb/ft)$$

$$= 17.02\ lb/SET$$

114 SETS (17.02 lb/SET) = 1939.8 lb

HOOPS 

#5 STIRRUPS

$$= 1.043\ lb/ft\left(\frac{21" + 6" + 6" + 25\tfrac{1}{2}"}{12"} ft\right) = 5.085\ lb/SET$$

20 SETS (5.085 lb/SET) = 101.7 lb

ADD'L STEEL ABUT GIRDER	2170.8 lb ; 18 GIRDERS =	39074.4 lb
ADD'L STEEL PIER GIRDER	2169.1 lb ; 18 GIRDERS	39043.8 lb
ADD'L STEEL IN GIRDERS =		78118.2 lb

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. Q-28

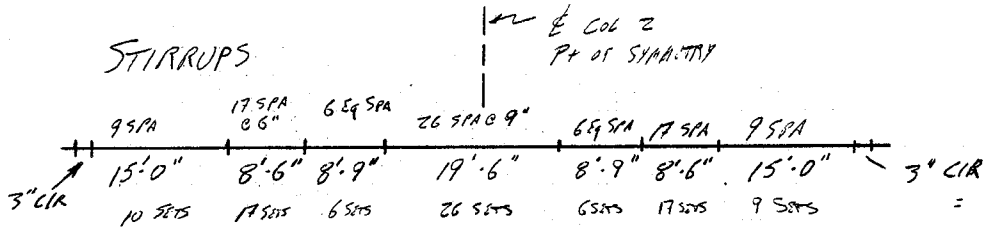
MADE BY MED DATE 8 Feb 90 JOB NO. 38 8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 280

CALCULATIONS FOR Quantities

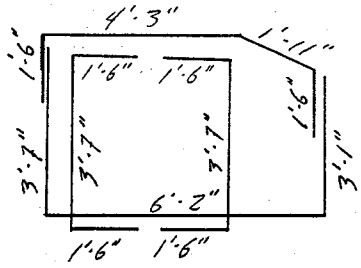
REINFORCING STEEL IN ABUTMENT CAP

STIRRUPS



= 91 SETS STIRRUP TIES

#5 TIES



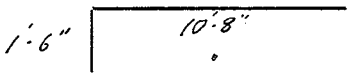
= 1 STIRRUP SET

$$W_{SET} = 1.043\ \text{lb/ft}\ [(\overset{9'-2''}{1.5' + 4.25' + 1.9167' + 1.5'})$$
$$+ (\overset{12'-10''}{3.5833' + 6.1667' + 3.0833'})$$
$$+ \overset{13'-2''}{2(2(1.5') + 3.5833')}]$$
$$= 1.043\ \text{lb/ft}\ (35.1667')$$
$$= 36.677\ \text{lb}$$

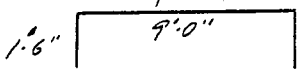
91 SETS / CAP = <u>3337.77 lb</u> / ABUT CAP

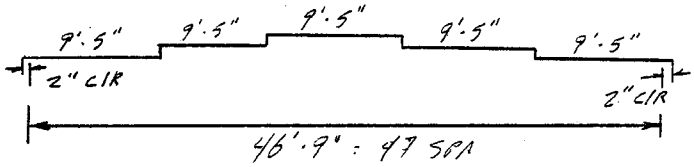
STEP REINFORCING

4 [6 #4 @ EQ SPA] = 4 [6 (.668 lb/ft) (10.667' + 1.5')] = 195.1 lb

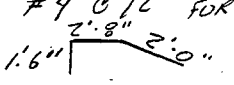


6 #4 @ EQ SPA = 6 (.668 lb/ft) (2(1.5') + 9.0') = 48.1 lb





#4 @ 12" FOR 46'-9" = 48 BARS



48 (1.5' + 2.667' + 2.0') (.668 lb/ft) = 197.7 lb

TOTAL STEEL IN STEPS = <u>440.9 lb</u>

JOB NORTHERN AVE @ NEW RIVER SHEET NO. Q-29

MADE BY MED DATE 6 FEB 90 JOB NO. 33 8962

CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 281

CALCULATIONS FOR QUANTITIES

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MINNEAPOLIS PHOENIX

ABUTMENT CAP (CONT)

LONGIT REIN

TOP ⊙ 14 #11 x 23'-0" CENTERED OVER CENTER SHAFT

$14(5.313\ lb/ft)(23'-0") = 1710.8\ lb$

⊙ 18 #11 x 20'-0" OVER OUTSIDE SHAFTS

$2[18(5.313\ lb/ft)(20'-0") = 3825.4\ lb$

⊙ 6 #11 CONT W/ STND HOOK EACH END

$6(5.313\ lb/ft)(84.0') + 6(2(5.313\ lb/ft)(\overset{HOOK}{2.0'})) + 6(5.313\ lb/ft)(\overset{LAP}{2.0'})$

$= 2869.0\ lb$

TOTAL TOP LONGIT REIN 8405.2 lb

BOTTOM

⊙ 12 #11 x 21'-0" OVER EXTERIOR SHAFTS

$2[12(5.313\ lb/ft)(21.0')] = 2677.8\ lb$

⊙ 6 #11 x 60'-0" OVER CENTER SHAFT

$6(5.313\ lb/ft)(60.0') = 1912.7\ lb$

⊙ 6 #11 CONT W/ STANDARD HOOK EACH END

$6(5.313\ lb/ft)(84.0' + 2(\overset{HOOK}{2.0'}) + \overset{LAP}{2.0}) = 2869.0\ lb$

TOTAL BTM LONGIT REIN 7459.5 lb

BRW
MINNEAPOLIS PHOENIX

JOB Northstar Ave @ New River SHEET NO. Q-30
MADE BY MED DATE 6 Feb 90 JOB NO. 55 8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 282
CALCULATIONS FOR QUANTITIES

ABUTMENT CAP

LONGIT REIN (CONT)

SIDES 2 #6 CONT W/STAND HOOK EACH SIDE

$$2[2(1.502\ lb/ft)(84.0' + 2(\overset{HOOK}{1.0'}) + \overset{LAP}{1.0'})$$

TOTAL SIDE LONGIT REIN = 522.7 lb

TOTAL REIN STEEL IN ABUTMENT CAP

STIRRUPS	3337.8 lb	Pg Q·29
STEP REIN	440.9 lb	Pg Q·29
LONGIT REIN		
TOP	8405.2 lb	Pg Q·30
BTM	7459.5 lb	Pg Q·30
SIDES	522.7 lb	FROM ABOVE
	20166.1 lb	PER ABUTMENT CAP

2 ABUTMENT CAPS TOTAL REIN : 40332. lb

VERTICAL RESTRAINERS

16 FIXED RESTRAINERS PER PIER

TOTAL 32 FIXED RESTRAINERS

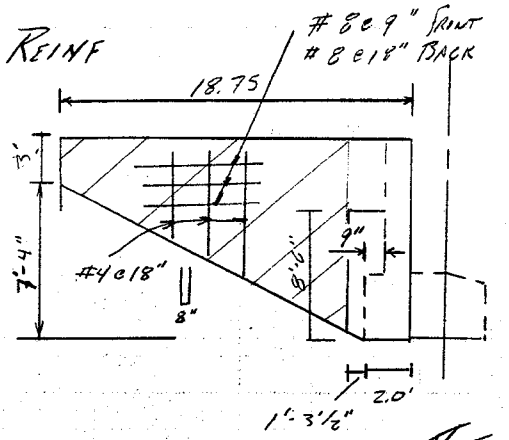
16 FIXED RESTRAINERS PER ABUT

TOTAL 32 EXPANSION RESTRAINERS

BRW
MINNEAPOLIS PHOENIX

JOB NORTHERN AVE @ MEN RIVER SHEET NO. Q·31
MADE BY MED DATE 6 FEB 90 JOB NO. 33 8962
CHECKED BY TSJones DATE 2/90 SEC. SHT. NO. 283
CALCULATIONS FOR QUANTITIES

WINGWALL REINF

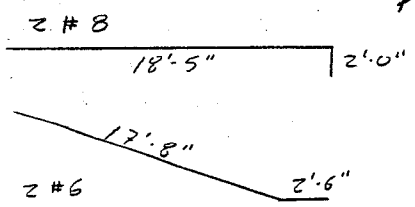


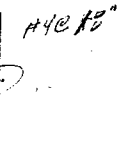
$3'(17.4583) + \frac{1}{2}(7.333')(17.4583')$

$= 81.47\ ft^2$

$W_{ft^2} = \frac{1}{.75'}(2.67\ lb/ft) + \frac{1}{1.5'}(2.67\ lb/ft) + \frac{1}{1.5'}(2(.668\ lb/ft))$

$= 6.23\ lb/ft^2$





98.345 60.5807

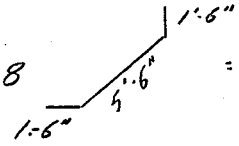
$W_{WINGWALL} = 2(18.4167 + 2.0')(2.67\ lb/ft) + 2(17.667' + 2.5')(1.502\ lb/ft) + (81.47\ ft^2)(6.23\ lb/ft^2) + 12(.667')(.668\ lb/ft)$

507.5581

TOTAL REIN WINGWALL = 671.8 lb

WINGWALL FILLET

12 #8 × 20'-0" = $12(20.0')(2.67\ lb/ft) = 640.8$ lb

12 #8  = $12(1.5' + 1.5' + 5.5')(2.67\ lb/ft) = 102$ lb

12 #8 × 3'-3 1/2" FRNT $12(2.67\ lb/ft)(3'-3\frac{1}{2}") = 105.5$ lb

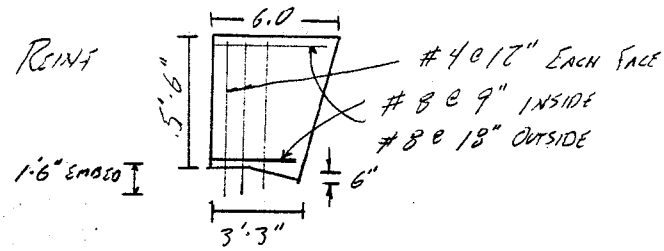
12 #8 × 3'-1 1/2" BK $12(2.67\ lb/ft)(3'-1\frac{1}{2}") = 100.1$ lb

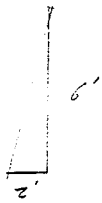
TOTAL REIN IN 2'×2' FILLET = 948.4 lb

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave & New River SHEET NO. C-32
MADE BY MED DATE 12 Feb 90 JOB NO. 33 8962
CHECKED BY T Stones DATE 2/90 SEC. SHT. NO. 284
CALCULATIONS FOR Quantities

CURTAIN WALL REINF





#8 @ 9" INSIDE 9 TOT

$9\left(\frac{5'\cdot10" + 3'\cdot1"}{2}\right) 2.67\ lb/ft = 107.1\ lb$

#8 @ 18" OUTSIDE 5 TOTAL

$5\left(\frac{5'\cdot10" + 3'\cdot1"}{2}\right) 2.67\ lb/ft = 59.5\ lb$

#4 @ 12" BOTH FACES 7 EACH SIDE 14 TOTAL

$14(6'\cdot10")\ 2.67\ lb/ft = 255.4\ lb$

2 #4 x 7'-6"

$2(7.5')(.668\ lb/ft) = 10\ lb$

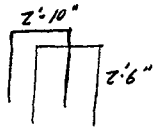
TOTAL REIN IN CURTAIN WALL 432.0 lb

4 TOTAL 1728.0 lb

LATERAL RESTRAINERS

4 #4 x 2'-6" = $4(2.5')(.668\ lb/ft) = 6.7\ lb$

5 SETS



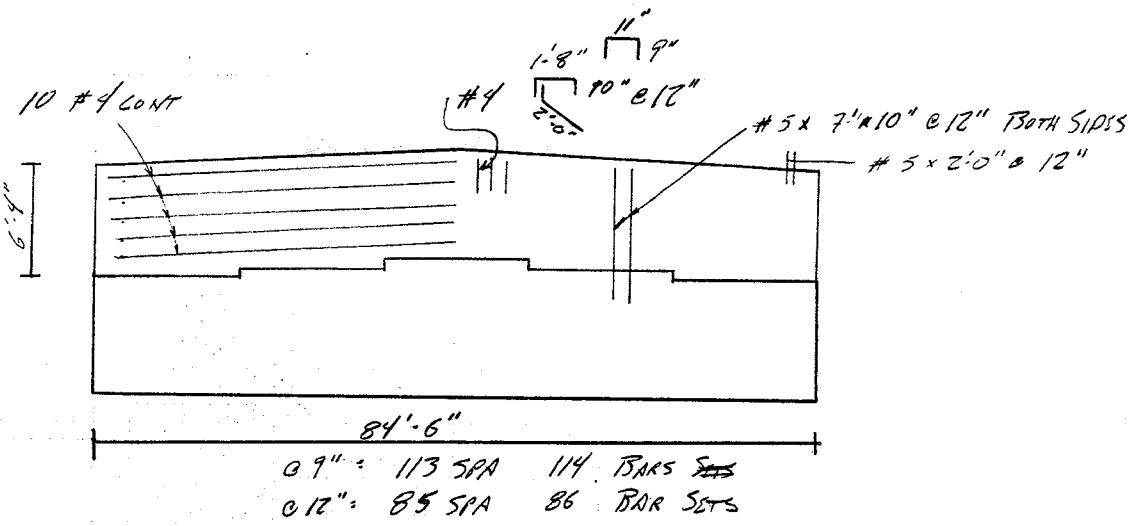
#10 = $5(2(2.5') + 2.83')(4.303\ lb/ft) = 168.5\ lb$

REIN IN LATERAL RESTRAINER 175.2 lb

4 TOTAL 700.8 lb

BRW
MINNEAPOLIS PHOENIX

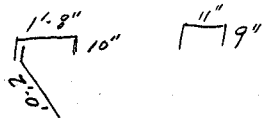
JOB Northern Ave @ New River SHEET NO. R-55
MADE BY MD DATE 12 Feb 90 JOB NO. 33 8962
CHECKED BY DATE SEC. SHT. NO. 285
CALCULATIONS FOR Quantities



10 #4 CONT

$$10(84'\text{-}2")(.668\ lb/ft) = 562\ lb$$

#4 @ 12"



$$86(3(10") + 2'\text{-}0" + 1'\text{-}8" + 2(9") + 11")(.668\ lb/ft) = 493.1\ lb$$

#5 x 7'-10" & 2'-0" @ 12"

$$86(7'\text{-}10" + 2'\text{-}0")(1.043\ lb/ft) = 882.0\ lb$$

TOTAL REIN IN ABUT BACK WALL 1937.1

Σ TOTAL 3874.2 lb

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. C-34
MADE BY MED DATE 12 Feb 90 JOB NO. 35 3962
CHECKED BY DATE SEC. SHT. NO. 286
CALCULATIONS FOR

REINFORCING STEEL SUMMARY

ABUTMENT DRILLED SHAFTS (6 TOTAL)	96,475.1 lb	PS Q-12
PIER SHAFTS / COL'S (12 TOTAL)	227,287.7 lb	PS Q-14
PIER CAP (3 TOTAL)	86,637 lb	PS Q-17
DIAPHRAGMS		
INTERMEDIATE (8 TOTAL)	11,200.8 lb	PS Q-18
PIER (3 TOTAL)	5,506.5 lb	PS Q-20
ABUTMENT (2 TOTAL)	3,586.6 lb	PS Q-21
DECK	191,596.4 lb	PS Q-24
APPROACH SLAB	502.3 lb	PS Q-25
TRAFFIC BARRIER	10,564.0 lb	PS Q-26
PEDESTRIAN CURB	2207.1 lb	PS Q-27
GIRDERS	78,118.2 lb	PS Q-28
ABUTMENT CAP	40332.0 lb	PS Q-31
ABUTMENT BACK WALL	3874.2 lb	PS Q-34
WING WALL (4 TOTAL)	2687.2 lb	PS Q-32
CURTAIN WALL (4 TOTAL)	1728.0 lb	PS Q-33
WINGWALL FILLET (4 TOTAL)	3793.6 lb	Q-32
LATERAL RESTRAINER (1 TOTAL)	700.8 lb	PS Q-53

TOTAL REINFORCING 766,798 lb

BRW
MINNEAPOLIS PHOENIX

JOB Northern Ave @ New River SHEET NO. Q-~~34~~ 35
MADE BY MED DATE 12 FEB 90 JOB NO. 53 2962
CHECKED BY _____ DATE _____ SEC. SHT. NO. 287
CALCULATIONS FOR Quantities