San Francisco-Oakland Bay Bridge
Seismic Retrofit Project

Independent Review of
Analysis and Strategy to Shim Bearings
at Pier E2 to Achieve Seismic Design Requirements
Modjeski and Masters, Inc.
August 9, 2013

Appendices 1-5

Appendices

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Appendix 1

Capacity of Bearings and Shear Keys

- FIND TRANSVERSE CAPACITIES OF BEARINGS GIVEN THE VERTICAL & LONGIT. LOADS OF THE MAX. TRANSVERSE & MAX UPLIFT CASES. BY INSP. THESE ARE 2 CONTROLLING CASES.

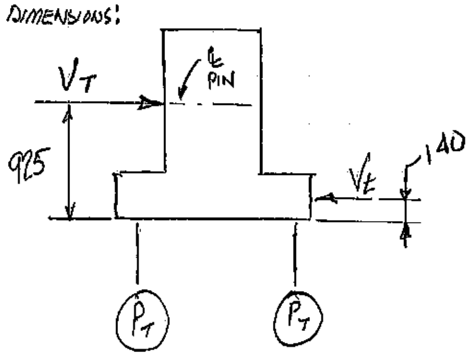
GIVEN:

BOLT FORCE AFTER LOSSES: 2510 kN (2789 kN LOAD WITH 10% LOSSES PER MARWAN)

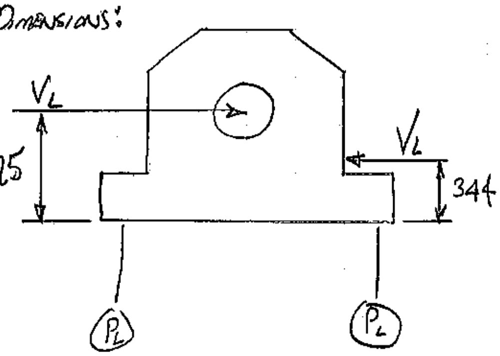
$\mu_{s,\,concrete} = 0.67$

$\mu_{s,\,steel} = 0.50$ (CLASS B COATING)

TRANSVERSE DIMENSIONS:

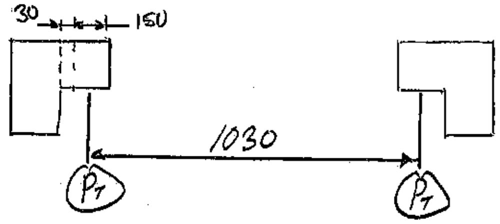


LONGIT. DIMENSIONS:



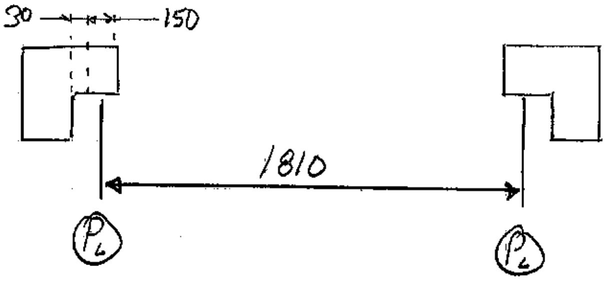
ASSUME P_T & P_L ARE AT MIDPOINT OF CONTACT AREA BETWEEN BRG. BTM. HOUSING & HOLD DOWN

TRANSV.



$$P_T = \frac{V_T(925-140)}{1030} = \frac{785}{1030} V_T$$

LONGIT.

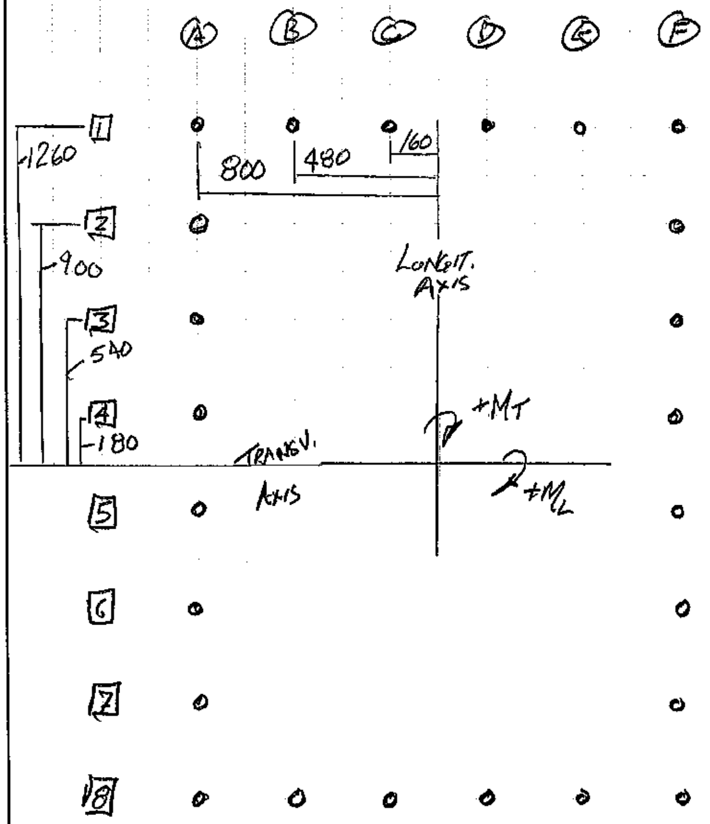


$$P_L = \frac{V_L(925-344)}{1810} = \frac{581}{1810} V_L$$

LOADCASE 4: (MAX. UPLIFT)
TRANSV: 25287 kN
LONGIT: 1628 kN
UPLIFT: 9539 kN

LOADCASE 5: (MAX. TRANSVERSE)
TRANSV: 30496 kN
LONGIT: 8186 kN
DOWNFORCE: 16441 kN

LOOK AT SYSTEM IN SIMILAR MANNER TO A PILE CAP WITH AXIAL & MOMENTS.



$I_T = 16(800)^2 + 4(480)^2 + 4(160)^2$

$= 11\,264\,000 \ mm^2$

$\times \frac{1}{800} = 14\,080 \ mm$ A/F

$\times \frac{1}{480} = 23\,467 \ mm$ B/E

$\times \frac{1}{160} = 70\,400 \ mm$ C/D

$I_L = 12(1260)^2 + 4(900)^2 + 4(540)^2 + 4(180)^2$

$= 23\,587\,200 \ mm^2$

$\times \frac{1}{1260} = 18\,720 \ mm$ $\boxed{1}/\boxed{8}$

$\times \frac{1}{900} = 26\,208 \ mm$ $\boxed{2}/\boxed{7}$

$\times \frac{1}{540} = 43\,680 \ mm$ $\boxed{3}/\boxed{6}$

$\times \frac{1}{180} = 131\,040 \ mm$ $\boxed{4}/\boxed{5}$

LOADCASE 4:

- NET CLAMPING = 24(2510 kN) − 9539 kN = 50 701 kN (2113 kN/BOLT)
  - LONGIT. COUPLE = 581/1810 × 1628 kN = 523 kN ↓↑
  - MOMENT FROM LONGIT COUPLE = 523 × 1810 = 946 630 kN-mm
  - LONGIT COUPLE EFFECTS:
    - $\boxed{1}/\boxed{8}$ = 946 630/18 720 = +/− 51 kN
    - $\boxed{2}/\boxed{7}$ = 946 630/26 208 = +/− 36 kN
    - $\boxed{3}/\boxed{6}$ = 946 630/43 680 = +/− 22 kN
    - $\boxed{4}/\boxed{5}$ = 946 630/131 040 = +/− 7 kN
  - TRANSV. COUPLE = $785/1030 \cdot V_T$
  - MOMENT FROM TRANSV. COUPLE = $785/1030 \cdot V_T \cdot 1030 \longrightarrow 785\, V_T$
  - TRANSV. COUPLE EFFECTS:
    - A/F = $\pm\ 785/14\,080 \cdot V_T$
    - B/E = $\pm\ 785/23\,467 \cdot V_T$
    - C/D = $\pm\ 785/70\,400 \cdot V_T$

BASIC EQUATION TO SOLVE:

$\mu_{S,STEEL} \cdot \Sigma \text{ BOLT LOADS} = V_T$, WHERE V_T IS TRANSV. CAPACITY WITH CONSTANT UPLIFT & V_L FORCES.

L.C. 4, CONT'D

SOLVE THIS IN EXCEL, AS BOLT LOADS ARE ALSO A FUNCTION OF V_T. ADDITIONALLY, THEY HAVE TO BE LIMITED TO PRELOAD OF 2510 kN ON "COMPRESSION" SIDE, WHERE V_T HELPS REDUCE UPLIFT EFFECTS,

(SEE PAGE 4 FOR RESULTS)

V_T CAPACITY = 21 712 kN/BRG (25 287 kN APPLIED, + 16 %)

LOADCASE 5:

- NET CLAMPING = 24(2510 kN) = 60 240 kN
- LONGIT. COUPLE = 581/1810 × 8186 = 2628 kN ↓↑
- MOMENT FROM LONGIT. COUPLE = 2628 × 1810 = 4 756 680 kN-mm
- LONGIT. COUPLE EFFECTS:
  - $\boxed{1}$ / $\boxed{8}$ = 4 756 680/18 720 = +/- 254 kN
  - $\boxed{2}$ / $\boxed{7}$ = 4 756 680/26 208 = +/- 181 kN
  - $\boxed{3}$ / $\boxed{6}$ = 4 756 680/43 680 = +/- 109 kN
  - $\boxed{4}$ / $\boxed{5}$ = 4 756 680/131 040 = +/- 36 kN
- TRANSV. COUPLE = 785/1030 · V_T
- MOMENT FROM TRANSV. COUPLE = 785 · V_T
- TRANSV. COUPLE EFFECTS:
  - A / F = +/- 785/14080 · V_T
  - B / E = +/- 785/23 467 · V_T
  - C / D = +/- 785/70 400 · V_T
- IN ADDITION, VERTICAL DOWNFORCE CAN NEGATE SOME UPLIFT:
  (16 441)/24 = 685 kN/BOLT CAN BE ADDED BACK IN, UP TO PRELOAD FORCE OF 2510 kN

SOLVE IN EXCEL:

(SEE PAGE 5 FOR RESULTS)

V_T CAPACITY = 25 652 kN/BRG (30496 kN APPLIED, +19 %)

Loadcase 4

V longit. 1628 kN

Vt **21,712 kN** (iterate this value)

V Resultant 21,773 kN

sum Anchor Forces 43545 kN
coeff friction 0.5
Friction Force 21773 kN (100.00% V Resultant)

	A1	B1	C1	D1	E1	F1	
	851	1336	1820	2304	2510	2510	
A2	866					2510	F2
A3	880					2510	F3
A4	895					2510	F4
A5	909					2510	F5
A6	924					2510	F6
A7	938					2510	F7
	953	1438	1922	2406	2510	2510	
	A8	B8	C8	D8	E8	F8	

Loadcase 5

V longit. 8186 kN

Vt 25,652 kN (iterate this value)

V Resultant 26,926 kN

sum Anchor Forces 53852 kN
coeff friction 0.5
Friction Force 26926 kN (100.00% V Resultant)

	A1	B1	C1	D1	E1	F1	
	1511	2083	2510	2510	2510	2510	
A2	1584					2510	F2
A3	1656					2510	F3
A4	1729					2510	F4
A5	1801					2510	F5
A6	1874					2510	F6
A7	1946					2510	F7
	2019	2510	2510	2510	2510	2510	
	A8	B8	C8	D8	E8	F8	

80 BOLTS – TOP HOUSING TO BOX @ 0.50 FRICTION (40)
48 BOLTS – STUB TO CONC. @ 0.67 FRICTION (32.16) ← CONTROLS

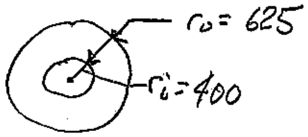
$32.16 \times 2510\ kN = 80\ 722\ kN$ TO SLIP

- CHECK BEARING ON RING/STUB, BRONZE PORTION HAS 414 MPa YIELD, SO CASTING STILL GOVERNS @ 345 MPa

WIDTH IN BRG = 320 mm
DIA OF BRG = 1250 mm
$\Rightarrow A = 400\ 000\ mm^2$

$$\sigma_{BRG} = \frac{80\ 722 \times 1000}{400\ 000} = 202\ MPa < 345$$, SO BEARING OK

- CHECK SHEAR STRESSES ON STUB



$$A_G = \pi(625^2 - 400^2) = 724\ 530\ mm^2$$

$$.58 \times 345 = 200\ MPa$$ (APPROX. SHEAR YIELD STRESS)

SHEAR STRESS ON A_G = $$\frac{80\ 722 \times 1000}{724\ 530} = 111\ MPa < 200\ MPa$$

AT YIELD, $\frac{V}{Y} = \frac{80\ 722 \times 200}{111 \times 2}$
$= 72\ 723\ kN$

- BENDING STRESSES

$$I = \tfrac{1}{4}\pi(625^4 - 400^4) = 9.974 \times 10^{10}\ mm^4$$
$$M = (80\ 722)(720) = 58\ 119\ 840\ kN\text{-}mm$$
$$\frac{Mc}{I} = \frac{58\ 119\ 840 \times 1000 \times 625}{9.974 \times 10^{10}} = 364\ MPa > 345$$

BACK SOLVE FOR YIELD M

$$\frac{345 \times 9.974 \times 10^{10}}{625 \times 1000} = 55\ 056\ 480,\ \div 720 = \text{YIELD } V$$

$= 76\ 467\ kN$

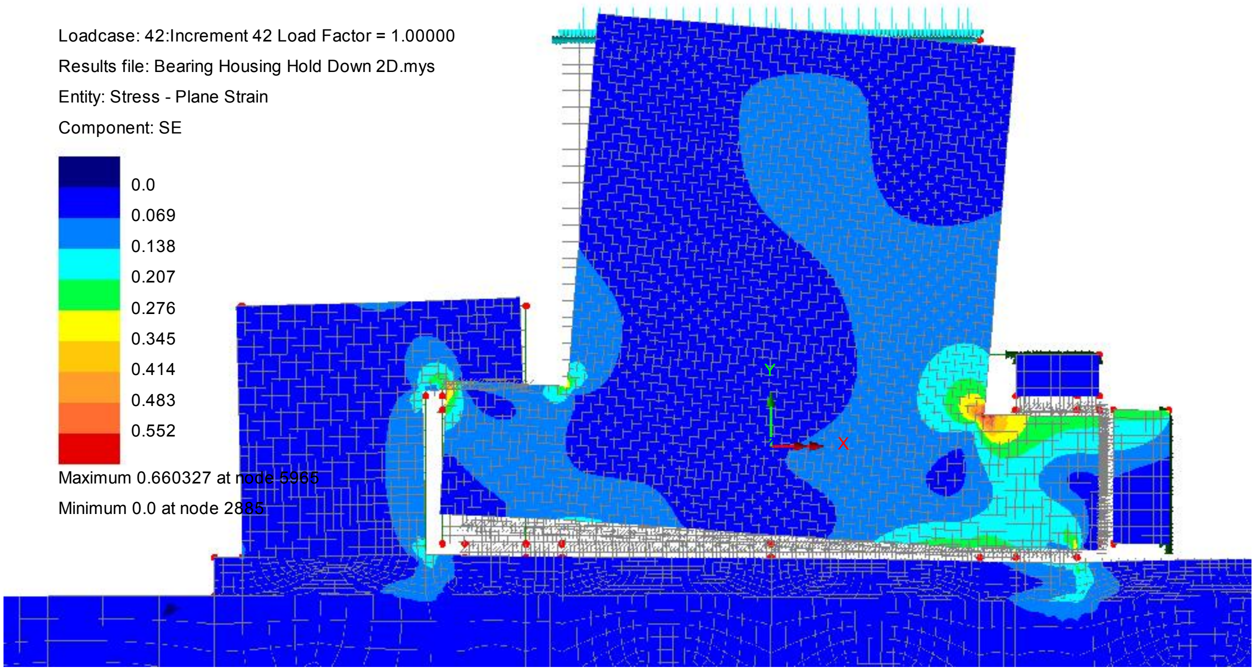
→ WHILE BENDING IS LIKELY NOT 100% APPLICABLE TO THIS SYSTEM, THERE IS LITTLE ELSE TO EXAMINE IN THE ABSENCE OF AN FEA (WHICH WAS DONE BY TPM), SO, TO SUMMARIZE THIS HAND-CALC. ANALYSIS IN A CONSERVATIVE MANNER, SAY CAPACITY OF SHEAR KEY IS $0.9 \times 72\ 723\ kN$
$= 65\ 451\ kN$

- SHEAR KEY CAPACITY APPROXIMATELY: 65451 kN
(CONSERV. ASSUMPTION BASED ON HAND-CALCS ONLY.)

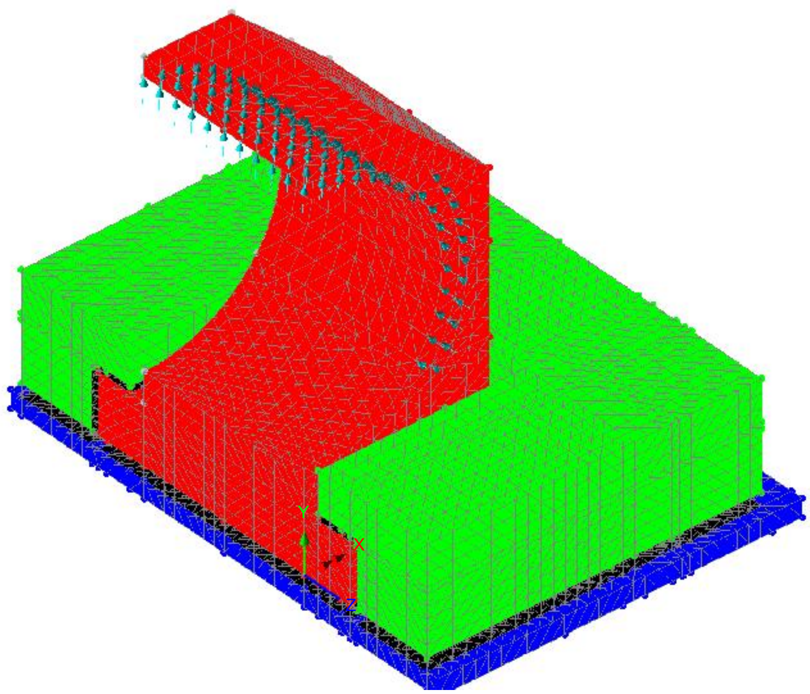
- IN LOADCASE 4, BEARINGS EACH HAVE 21712 kN CAPACITY TO RESIST V_T, ASSUMING V_L IS 1628 kN AND UPLIFT IS 9539 kN. LOADCASE 4 HAS 25287 kN PER BEARING. BY INSPECTION, THE COMBINATION OF 4 BEARINGS & 2 SHEAR KEYS SHOULD BE ADEQUATE.
- IN LOADCASE 5, BEARINGS EACH HAVE 25652 kN CAPACITY TO RESIST V_T, ASSUMING V_L IS 8186 kN AND DOWN FORCE IS 16441 kN. LOADCASE 5 HAS 30496 kN PER BEARING, BY INSPECTION, THE COMBINATION OF 4 BEARINGS AND 2 SHEAR KEYS SHOULD BE ADEQUATE.

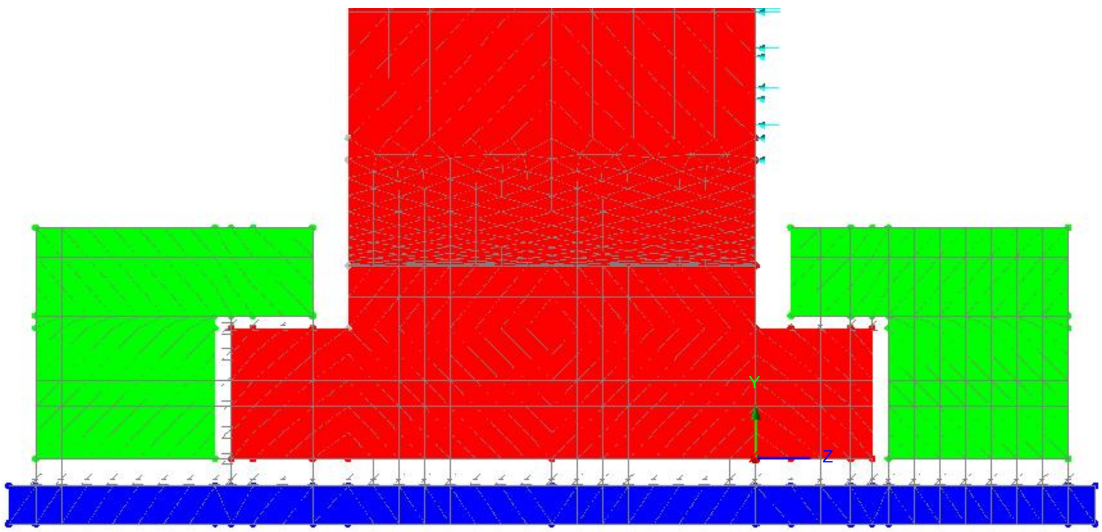
Appendix 2

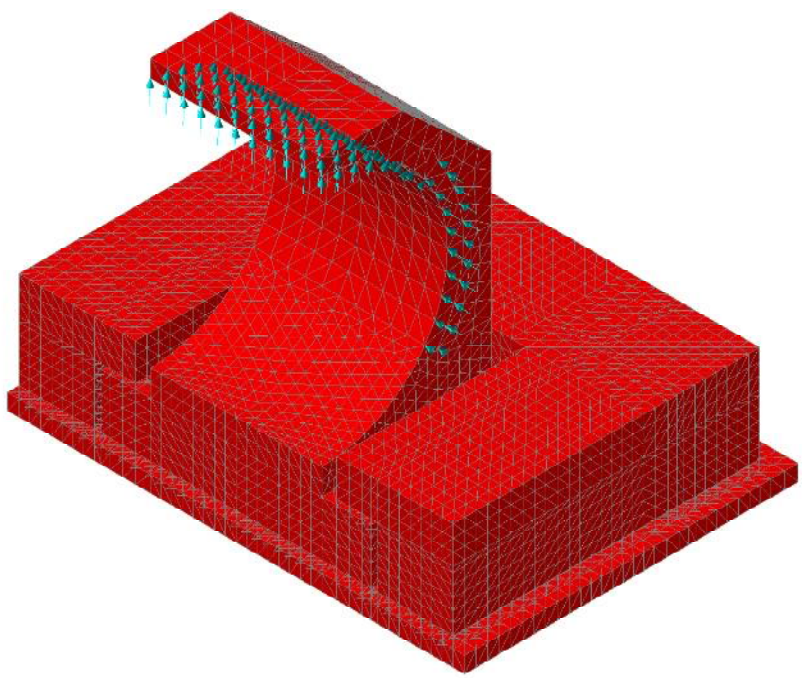
Finite Element Models of Bearings and Shear Keys

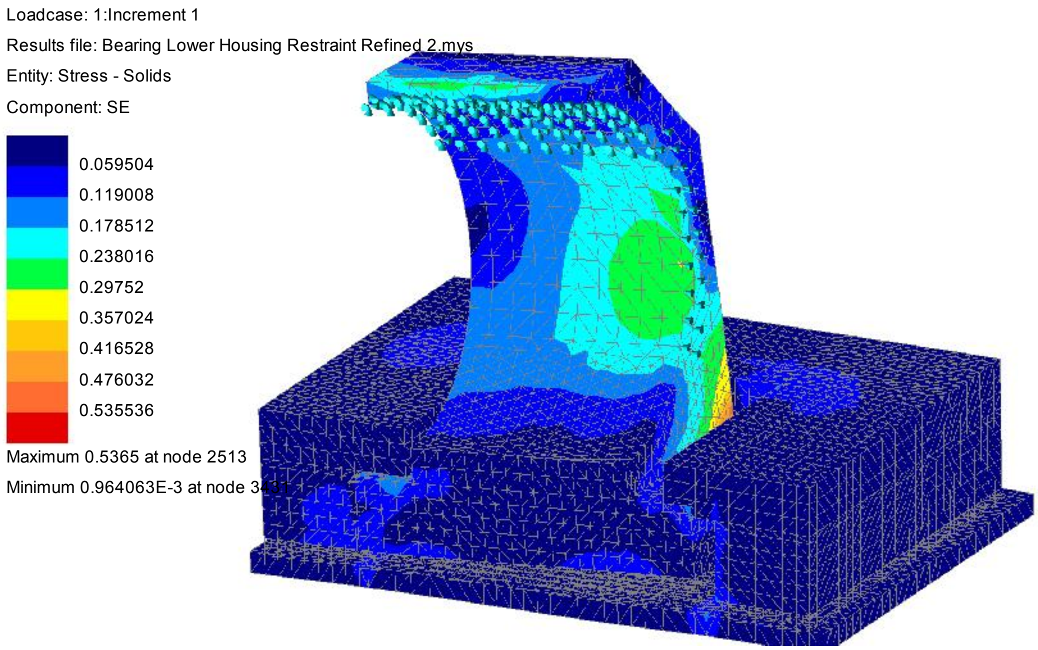


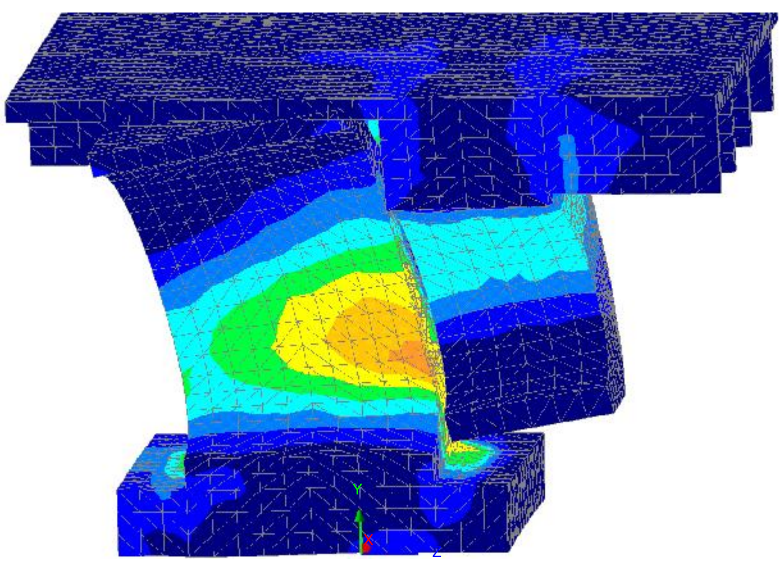
Loadcase: 42:Increment 42 Load Factor = 1.00000
Results file: Bearing Housing Hold Down 2D.mys
Entity: Stress - Plane Strain
Component: SE
0.0
0.069
0.138
0.207
0.276
0.345
0.414
0.483
0.552
Maximum 0.660327 at node 5965
Minimum 0.0 at node 2885
Y
X

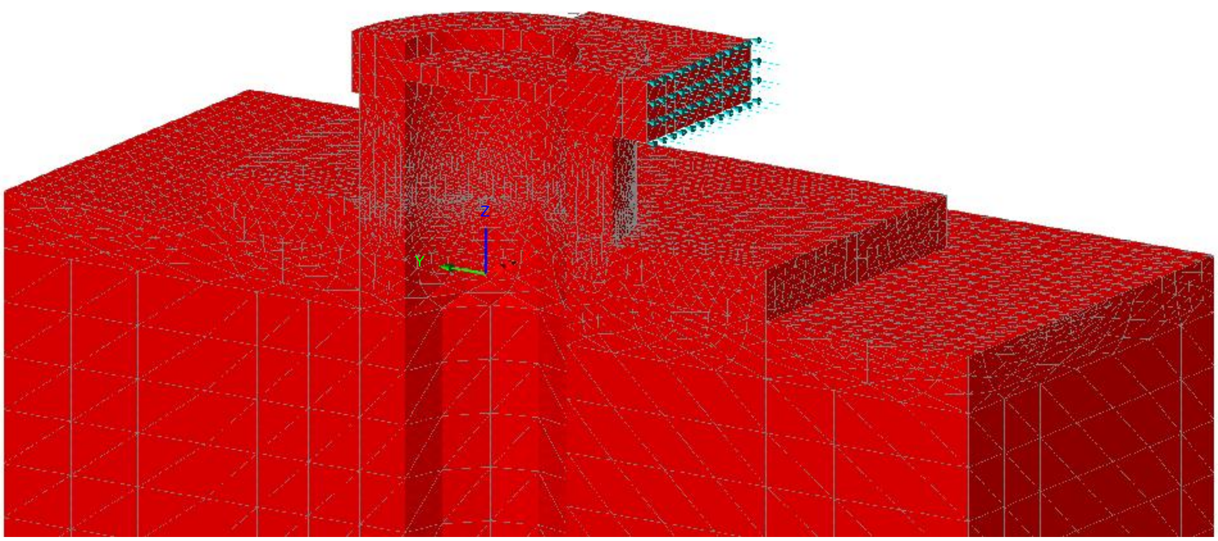

Y
X
Z

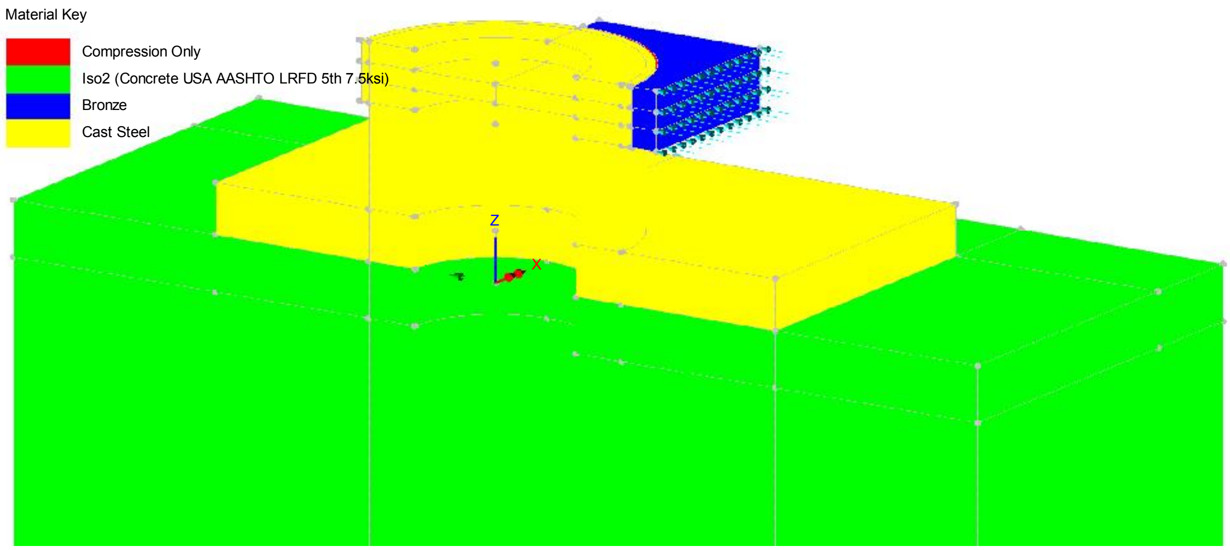




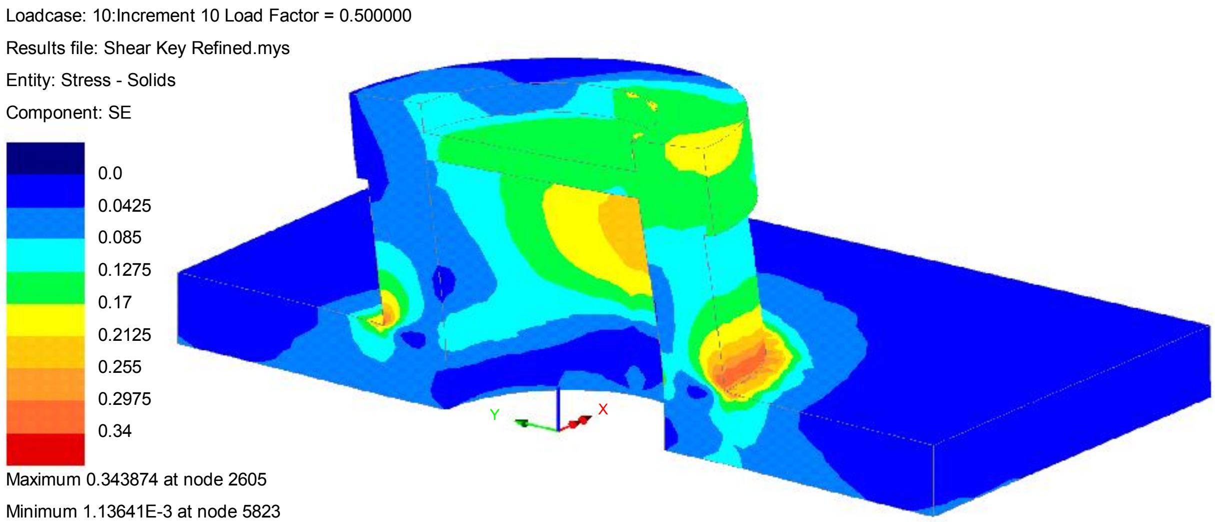

Loadcase: 1:Increment 1
Results file: Bearing Lower Housing Restraint Refined 2.mys
Entity: Stress - Solids
Component: SE
0.059504
0.119008
0.178512
0.238016
0.29752
0.357024
0.416528
0.476032
0.535536
Maximum 0.5365 at node 2513
Minimum 0.964063E-3 at node

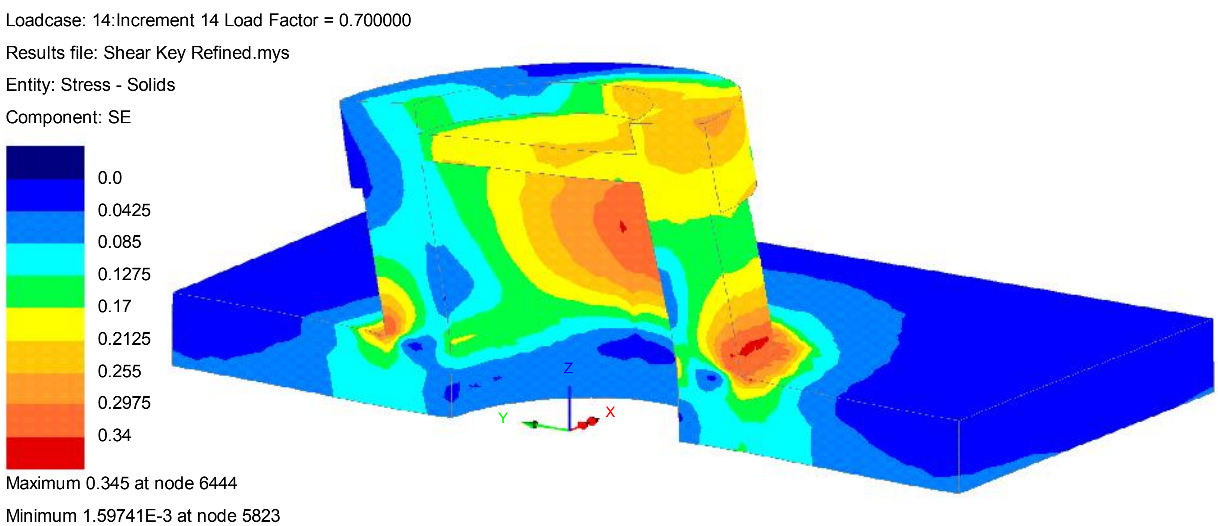

Y
X
Z

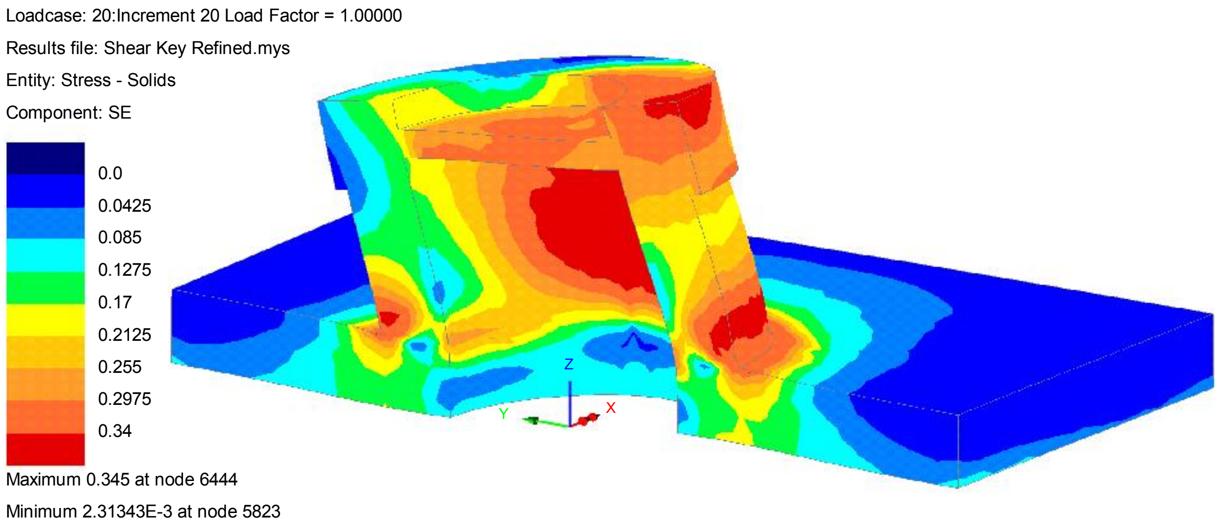

Z
Y



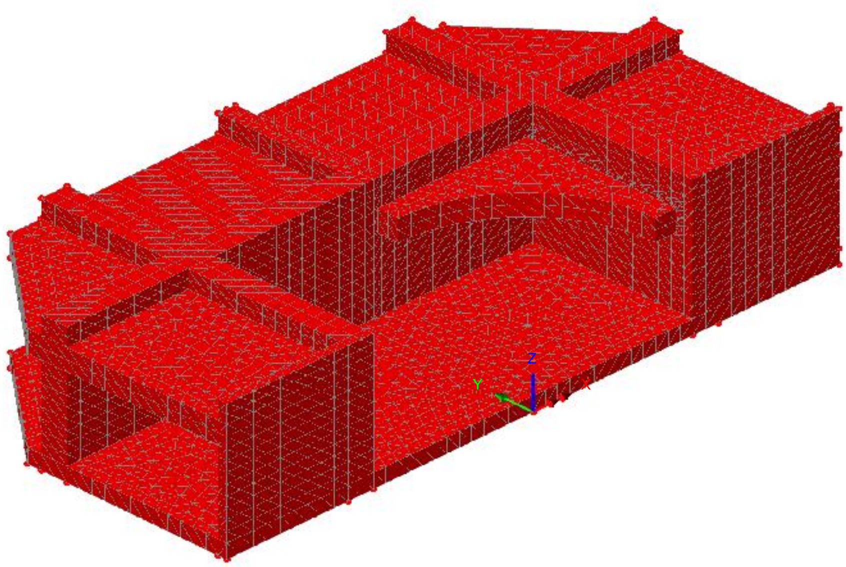
Material Key
Compression Only
Iso2 (Concrete USA AASHTO LRFD 5th 7.5ksi)
Bronze
Cast Steel
Z
X


Loadcase: 10:Increment 10 Load Factor = 0.500000
Results file: Shear Key Refined.mys
Entity: Stress - Solids
Component: SE
0.0
0.0425
0.085
0.1275
0.17
0.2125
0.255
0.2975
0.34
Maximum 0.343874 at node 2605
Minimum 1.13641E-3 at node 5823
Y
X

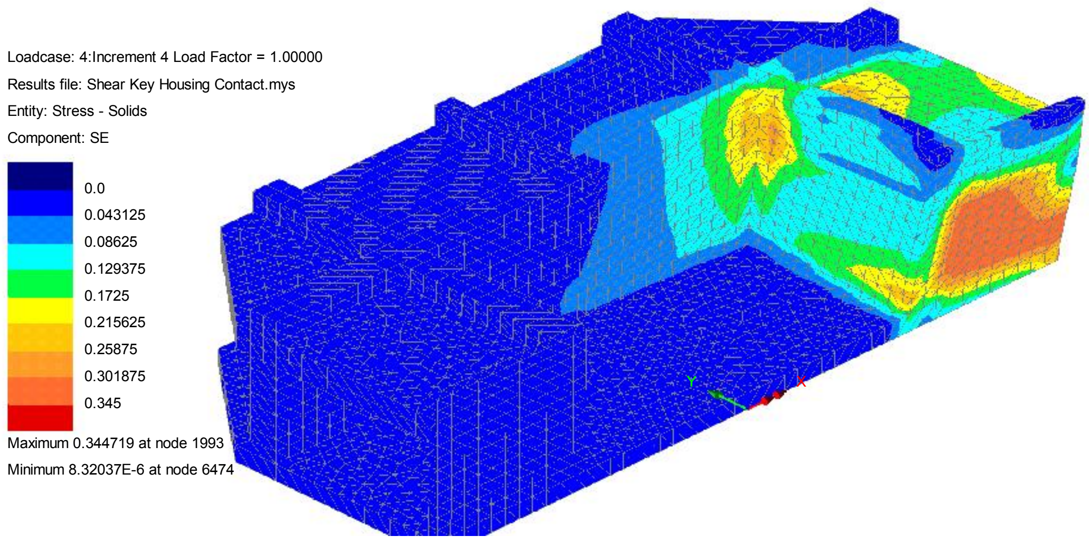

Loadcase: 14:Increment 14 Load Factor = 0.700000
Results file: Shear Key Refined.mys
Entity: Stress - Solids
Component: SE
0.0
0.0425
0.085
0.1275
0.17
0.2125
0.255
0.2975
0.34
Maximum 0.345 at node 6444
Minimum 1.59741E-3 at node 5823
Z
Y
X



Loadcase: 20:Increment 20 Load Factor = 1.00000
Results file: Shear Key Refined.mys
Entity: Stress - Solids
Component: SE
0.0
0.0425
0.085
0.1275
0.17
0.2125
0.255
0.2975
0.34
Z
Y
X
Maximum 0.345 at node 6444
Minimum 2.31343E-3 at node 5823



Z
Y


Loadcase: 4:Increment 4 Load Factor = 1.00000
Results file: Shear Key Housing Contact.mys
Entity: Stress - Solids
Component: SE
0.0
0.043125
0.08625
0.129375
0.1725
0.215625
0.25875
0.301875
0.345
Maximum 0.344719 at node 1993
Minimum 8.32037E-6 at node 6474
Y
X

Appendix 3

- **Appendix 3A** – Modjeski and Masters Calculations – Steel Orthotropic Girder at Bearings and Steel Crossbeam at Shear Keys
- **Appendix 3B** – TYLin Calculations – Steel Orthotropic Girder at Bearings
- **Appendix 3C** – Design Drawings – Orthotropic Box Girder at Pier E2, Crossbeam at Pier E2, Bearing and Shear Key Details

Appendix 3A

Modjeski and Masters Calculations

Steel Orthotropic Girder at Bearings and Steel Crossbeam at Shear Keys

Project:	*SFOBB - JN 3274*	Made by:	*PAR* *7/24/2013*
Subject:	*Girder capacity*	Checked by:	
Content:	*Lateral and longitudinal loads*	Sheet No.:	

Assumptions:
1. Look at it from a "block shear" perspective.
2. Local effects only - assumed that load spreads quickly away from bearing interface.
3. A709M Grade 345 (A709 Grade 50) steel.

Fy =	345	MPa	50	ksi
Fu =	450	MPa	65	ksi

Bearing Demands - Load Path C - compression and uplift from sheet 882AR2

	MN	k
Compression	68	15300
Uplift	17	3825
Longitudinal	15	3375
Transverse	30	6750

Bearing "footprint"

Length, L =	3500	mm	137.8	in
Width, w =	2900	mm	114.2	in

Block Shear Bottom Flange

If Atn ≤ 0.58 Avn then	Rr = ϕbs (0.58 Fy Avg + Fu Atn)	6.13.4-1
otherwise	Rr = ϕbs (0.58 Fu Avn + Fy Atg)	6.14.4-2
ϕbs = 0.80		6.5.4.2

Case 1 - Looking at bottom flange from the most conservative perspective

1. Ignore presence of transverse webs and longitudinal shear plates
2. Assume free edges on one longitudinal and one transverse side

3. Assume bottom flange thickness =			60	mm	2.36	in	transverse
4. Assume bottom flange thickness =			85	mm	3.35	in	longitudinal
5. Assume	8	diameter	63	mm	2.48	in	holes transverse
6. Assume	10	diameter	63	mm	2.48	in	holes longitudinal

Transverse

Axial and shear areas

Atg =	297500	mm2	461.1	in2			
Avg =	174000	mm2	269.7	in2			
Atn =	243950	mm2	378.1	in2			
Avn =	143760	mm2	222.8	in2			
Rr =	115675.9	kN	26004	k	D/C =	0.26	Ok

Shear areas only

Atg =	0	mm2	0.0	in2			
Avg =	174000	mm2	269.7	in2			
Atn =	0	mm2	0.0	in2			
Avn =	143760	mm2	222.8	in2			
Rr =	30017.09	kN	6748	k	D/C =	1.00	Ok

Axial areas only

Atg =	297500	mm2	461.1	in2			
Avg =	0	mm2	0.0	in2			
Atn =	243950	mm2	378.1	in2			
Avn =	0	mm2	0.0	in2			
Rr =	87822	kN	19742	k	D/C =	0.34	Ok

Longitudinal

Axial and shear areas

Atg =	174000	mm2	269.7	in2			
Avg =	297500	mm2	461.1	in2			
Atn =	143760	mm2	222.8	in2			
Avn =	243950	mm2	378.1	in2			
Rr =	99377.4	kN	22340	k	D/C =	0.15	Ok

Shear areas only

Atg =	0	mm2	0.0	in2			
Avg =	297500	mm2	461.1	in2			
Atn =	0	mm2	0.0	in2			
Avn =	243950	mm2	378.1	in2			
Rr =	50936.76	kN	11451	k	D/C =	0.29	Ok

Axial areas only

Atg =	174000	mm2	269.7	in2			
Avg =	0	mm2	0.0	in2			
Atn =	143760	mm2	222.8	in2			
Avn =	0	mm2	0.0	in2			
Rr =	51753.6	kN	11634	k	D/C =	0.29	Ok

Longitudinal shear plate

Assume 60% of longitudinal shear is transferred into longitudinal shear plate (rest stays in bottom flange)

	MN	k
Demand	9	2025

Capacity

t =	18	mm	0.71	in			
Shear capacity =	3602	N/mm	20.6	k/in			
Length required =	2499	mm	98.5	in			
Length of bearing =			3500	mm			
			137.8	in	D/C =	0.72	Ok

Transverse web plates

	MN	k
Demand	30	6750

Capacity (3 webs)

t =	35	mm	1.38	in			
Shear capacity =	21011	N/mm	119.9	k/in			
Length required =	1428	mm	56.3	in			
Width of bearing =			2900	mm			
			114.2	in	D/C =	0.49	Ok

Using very conservative assumptions, there is sufficient capacity to carry longitudinal and transverse shear loads up into the girder.

Project:	*SFOBB - JN 3274*
Subject:	*Girder capacity*
Content:	*Lateral and longitudinal loads*

Made by:	*PAR*	*7/24/2013*
Checked by:		
Sheet No.:		

Area of steel within bearing footprint

Webs (3)

Width =	2900	mm	114.2	in
Thickness =	35	mm	1.38	in
Area (total) =	304500	mm2	472.0	in2

Shear plate

Width =	3500	mm	137.8	in
Thickness =	18	mm	0.71	in
Area (total) =	63000	mm2	97.7	in2

Longitudinal stiffeners (4)

Width =	3500	mm	137.8	in
Thickness =	45	mm	1.77	in
Area (total) =	630000	mm2	976.5	in2

Transverse shear plate stiffeners (8)

Width =	250	mm	9.8	in
Thickness =	25	mm	0.98	in
Area (total) =	50000	mm2	77.5	in2

Transverse plate stiffeners A (2)

Width =	550	mm	21.7	in
Thickness =	50	mm	1.97	in
Area (total) =	55000	mm2	85.3	in2

Transverse plate stiffeners B (2)

Width =	475	mm	18.7	in
Thickness =	50	mm	1.97	in
Area (total) =	47500	mm2	73.6	in2

Typical bearing assembly (14 pairs)

outer plates (2)				
Width =	200	mm	7.9	in
Thickness =	35	mm	1.38	in
inner plate (1)				
Width =	200	mm	7.9	in
Thickness =	40	mm	1.57	in
Area (1) =	22000	mm2	34.1	in2
Area (total) =	616000	mm2	954.8	in2

Total plate bearing area above bottom flange within bearing footprint

Area (total) =	1766000	mm2	2737.3	in2

Prestressing force

56 50 diameter A354 Grade BD anchor bolts

Area =	1612.9	mm2	2.50	in2
Fu =	1034	MPa	150	ksi

Conservatively assume that bolts are tensioned to Fu

Tension/bolt =	1.67	MN/bolt	375	k/bolt
Total force =	93	MN	21000	k

Conservatively assume that force transferred thru stiffeners only

Stress =	151.7	MPa	21.99	ksi	D/C =	0.44	Ok

Force through stiffeners webs and diaphragms

Stress =	81.2	MPa	11.8	ksi	D/C =	0.24	Ok

Axial reaction

Compression

Conservatively assume that force transferred thru shear and web plates only

Demand =	68000	kN	15289	k			
	185.03	MPa	26.84	ksi			

Tension

Conservatively assume that bolt pretension is 0.6Fu

Tension/bolt =	1.00	MN	225	k			
Total pretension =	56062	kN	12600	k			
Demand =	17000	kN	3822	k	D/C =	0.30	Ok

Pretension is never overcome - all plates active in tension

	9.63	MPa	1.40	ksi			

Conservatively assume that force transferred thru shear and web plates only

	46.26	MPa	6.71	ksi	D/C =	0.13	Ok

Look at prestressing stiffener assembly

Material properties

Steel plate

Fy =	345	MPa	50	ksi
Fu =	450	MPa	65	ksi

Weld metal

Fexx =	485	MPa	70	ksi
ϕe =	0.80			
Rexx =	232.8	MPa	33.6	ksi

50 diameter A354 Grade BD anchor bolts

Area =	1612.9	mm2	2.50	in2
Fu =	1034	MPa	150	ksi

Conservatively assume load on each bolt = Fu

Pu =	1.67	MN/bolt	375	k/bolt

Geometry

Top bearing plate

b =	200	mm	7.9	in
t =	100	mm	3.9	in
L =	200	mm	7.9	in
hole diameter =	63	mm	2.5	in

Center stiffener

b =	200	mm	7.9	in
t =	40	mm	1.6	in
L =	600	mm	23.6	in
tw =	35	mm	1.4	in

Outer stiffener

b =	200	mm	7.9	in
t =	35	mm	1.4	in
L =	600	mm	23.6	in
tw =	35	mm	1.4	in

Bearing plate capacity

Sxx (at hole) =	228333	mm3	13.93	in3			
My =	78775000	N-mm	696.7	k-in			

Assume elastic simply supported bearing plate
Mmax = 1/4 Pmax L

Pmax =	1.58	MN	354.5	k	D/C =	1.06	NG

Assume elastic continuous bearing plate
Mmax = 5/32 Pmax L

Pmax =	2.52	MN	567.2	k	D/C =	0.66	Ok

Reactions at stiffeners (conservatively use max of continuous or simple support)

Rcenter =	2.29	MN	515.7	k
Router =	0.83	MN	187.5	k

Center stiffener capacity

Compression - column analogy
Include 200mm of web

Imin =	25466667	mm4	61.2	in4			
A =	23000	mm2	35.7	in2			
r =	33.28	mm	1.31	in			
use k =	1.00	for mill to bear ends					
kL/r =	18.03						
sqrt(2 x π^2 x E / Fy) =	107.0						
Fcr =	340.1	MPa	49.3	ksi			
Pmax =	2.72	MN	612	k	D/C =	0.84	Ok

Shear

D/t =	15.00						
6000 sqrt(k) / sqrt(Fy) =	60.00						
C =	1.00						
ϕs =	1.00						
Vr =	4.8	MN	1080	k	D/C =	0.48	Ok

Check weld

PJP =	34.0	mm	1.34	in			
Weld strength =	4.7	MN	1068	k	D/C =	0.48	Ok

Outer stiffener capacity

Compression - column analogy
Include 200mm of web

Imin =	24762500	mm4	59.5	in4			
A =	21000	mm2	32.6	in2			
r =	34.34	mm	1.35	in			
use k =	1.00	for mill to bear ends					
kL/r =	17.47						
sqrt(2 x π^2 x E / Fy) =	107.0						
Fcr =	340.4	MPa	49.4	ksi			
Pmax =	2.38	MN	536	k	D/C =	0.35	Ok

Shear							
D/t =	17.14						
6000 sqrt(k) / sqrt(Fy) =	60.00						
C =	1.00						
ϕs =	1.00						
Vr =	4.2	MN	945	k	D/C =	0.20	Ok
Check weld							
PJP =	29.0	mm	1.14	in			
Weld strength =	4.1	MN	911	k	D/C =	0.21	Ok

Check block shear tearout of webs

Shear length =	1200	mm	47.24	in			
Axial length =	400	mm	15.75	in			
t =	35	mm	1.38	in			
Atn = Atg =	14000	mm2	21.7	in2			
Avn = Avg =	42000	mm2	65.1	in2			
ϕbs =	0.80						
Rr =	12.6336	MN	3	k	D/C =	0.53	Ok

Check shear of 50x700mm plates

Shear capacity							
Vr =	7.0	MN	1574	k	D/C =	0.95	Ok
Weld capacity							
Vr =	7.2	MN	1612	k	D/C =	0.93	Ok

Area of steel within bearing footprint

Webs (3)

Width =	2900	mm	114.2	in
Thickness =	35	mm	1.38	in
Area (total) =	304500	mm2	472.0	in2

Shear plate

Width =	3500	mm	137.8	in
Thickness =	18	mm	0.71	in
Area (total) =	63000	mm2	97.7	in2

Longitudinal stiffeners (4)

Width =	3500	mm	137.8	in
Thickness =	45	mm	1.77	in
Area (total) =	630000	mm2	976.5	in2

Transverse shear plate stiffeners (8)

Width =	250	mm	9.8	in
Thickness =	25	mm	0.98	in
Area (total) =	50000	mm2	77.5	in2

Transverse plate stiffeners A (2)

Width =	550	mm	21.7	in
Thickness =	50	mm	1.97	in
Area (total) =	55000	mm2	85.3	in2

Transverse plate stiffeners B (2)

Width =	475	mm	18.7	in
Thickness =	50	mm	1.97	in
Area (total) =	47500	mm2	73.6	in2

Typical bearing assembly (14 pairs)

outer plates (2)				
Width =	200	mm	7.9	in
Thickness =	35	mm	1.38	in
inner plate (1)				
Width =	200	mm	7.9	in
Thickness =	40	mm	1.57	in
Area (1) =	22000	mm2	34.1	in2
Area (total) =	616000	mm2	954.8	in2
Ixx =	754141666.7	mm4	1811.8	in4
Iyy =	1124991667	mm4	2702.8	in4

Total plate bearing area above bottom flange within bearing footprint

Area (total) =	1766000	mm2	2737.3	in2

Moments of inertia assuming all plates active

Assume symmetry - not really correct

	Longit. mm4	Transv. mm4	Longit. in4	Transv. in4
Shear plate	64312500000	1701000	154511.3993	4.086668847
Web 1	2.28385E+11	71134583333	548698.0257	170901.5201
Web 2	10361458.33	71134583333	24.89350322	170901.5201
Web 3	2.28385E+11	71134583333	548698.0257	170901.5201
Longit stiff 1	1.60781E+11	1.27602E+11	386278.4982	306564.0177
Longit stiff 2	1.60781E+11	39401578125	386278.4982	94662.6701
Longit stiff 3	1.60781E+11	39401578125	386278.4982	94662.6701
Longit stiff 4	1.60781E+11	1.27602E+11	386278.4982	306564.0177
Transv shear plate stiff 1	5348307292	260416666.7	12849.35967	625.6535443
Transv shear plate stiff 2	1410807292	260416666.7	3389.478076	625.6535443
Transv shear plate stiff 3	1410807292	260416666.7	3389.478076	625.6535443
Transv shear plate stiff 4	5348307292	260416666.7	12849.35967	625.6535443
Transv plate stiff 1	18957588542	37460880534	45545.78865	90000.12548
Transv plate stiff 2	18957588542	37460880534	45545.78865	90000.12548
Transv plate stiff 3	18957588542	37460880534	45545.78865	90000.12548
Transv plate stiff 4	18957588542	37460880534	45545.78865	90000.12548
Bearing assemblies	4.71643E+11	15762143333	1133126.319	37868.70083
Total moment of inertia =	1.72521E+12	7.14059E+11	4144833.487	1715533.839
	Longit. mm3	Transv. mm3	Longit. in3	Transv. in3
Section modulus =	985834258	492454549	2368.5	1183.1

Loadings

			Arm =	750	mm
	VT (MN)	VL (MN)	P (MN)	MT (MN-mm)	ML (MN-mm)
U	25.3	1.6	-9.5	18975	1200
T	30.5	8.2	16.4	22875	6150
L	1.3	13.2	19.3	975	9900
C	30	15	68	22500	11250

Max/Min Stress P/A + Mx/Sx + My/Sy

	Max (MPa)	Min (MPa)	Max (ksi)	Min (ksi)
U	34.37	-45.13	4.98	-6.54
T	61.98	-43.40	8.99	-6.29
L	22.95	-1.09	3.33	-0.16
C	95.61	-18.60	13.86	-2.70

Add prestressing force

Conservatively assume that force transferred thru stiffeners only

Stress = 151.68 MPa 21.99 ksi

	Max (MPa)	Min (MPa)	Max (ksi)	Min (ksi)	D/C	
U	186.05	106.56	26.98	15.45	0.54	Ok
T	213.66	108.28	30.98	15.70	0.62	Ok
L	174.63	150.59	25.32	21.84	0.51	Ok
C	247.29	133.09	35.86	19.30	0.72	Ok

Area of steel within bearing footprint

Webs (3)				
Width =	2900	mm	114.2	in
Thickness =	35	mm	1.38	in
Area (total) =	304500	mm2	472.0	in2
Shear plate				
Width =	3500	mm	137.8	in
Thickness =	18	mm	0.71	in
Area (total) =	63000	mm2	97.7	in2
Longitudinal stiffeners (4)				
Width =	3500	mm	137.8	in
Thickness =	45	mm	1.77	in
Area (total) =	630000	mm2	976.5	in2
Transverse shear plate stiffeners (8)				
Width =	250	mm	9.8	in
Thickness =	25	mm	0.98	in
Area (total) =	50000	mm2	77.5	in2
Transverse plate stiffeners A (2)				
Width =	550	mm	21.7	in
Thickness =	50	mm	1.97	in
Area (total) =	55000	mm2	85.3	in2
Transverse plate stiffeners B (2)				
Width =	475	mm	18.7	in
Thickness =	50	mm	1.97	in
Area (total) =	47500	mm2	73.6	in2
Typical bearing assembly (14 pairs)				
outer plates (2)				
Width =	200	mm	7.9	in
Thickness =	35	mm	1.38	in
inner plate (1)				
Width =	200	mm	7.9	in
Thickness =	40	mm	1.57	in
Area (1) =	22000	mm2	34.1	in2
Area (total) =	616000	mm2	954.8	in2
Ixx =	754141666.7	mm4	1811.8	in4
Iyy =	1124991667	mm4	2702.8	in4

Total plate bearing area above bottom flange within bearing footprint

Area (total) =	1766000	mm2	2737.3	in2

Moments of inertia assuming all plates active

Assume symmetry - not really correct

	Longit. mm4	Transv. mm4	Longit. in4	Transv. in4
Shear plate	64312500000	1701000	154511.3993	4.086668847
Web 1	2.28385E+11	71134583333	548698.0257	170901.5201
Web 2	10361458.33	71134583333	24.89350322	170901.5201
Web 3	2.28385E+11	71134583333	548698.0257	170901.5201
Longit stiff 1				
Longit stiff 2				
Longit stiff 3				
Longit stiff 4				
Transv shear plate stiff 1				
Transv shear plate stiff 2				
Transv shear plate stiff 3				
Transv shear plate stiff 4				
Transv plate stiff 1				
Transv plate stiff 2				
Transv plate stiff 3				
Transv plate stiff 4				
Bearing assemblies				
Total moment of inertia =	5.21094E+11	2.13405E+11	1251932.344	512708.6469
	Longit. mm3	Transv. mm3	Longit. in3	Transv. in3
Section modulus =	297767763	147176173	715.4	353.6

Loadings

			Arm =	750	mm
	VT (MN)	VL (MN)	P (MN)	MT (MN-mm)	ML (MN-mm)
U	25.3	1.6	-9.5	18975	1200
T	30.5	8.2	16.4	22875	6150
L	1.3	13.2	19.3	975	9900
C	30	15	68	22500	11250

Max/Min Stress P/A + Mx/Sx + My/Sy

	Max (MPa)	Min (MPa)	Max (ksi)	Min (ksi)
U	127.58	-138.34	18.50	-20.06
T	185.37	-166.79	26.88	-24.19
L	50.80	-28.94	7.37	-4.20
C	229.16	-152.15	33.23	-22.06

Add prestressing force

Force through stiffeners webs and diaphragms

Stress = 81.25 MPa 11.78 ksi

	Max (MPa)	Min (MPa)	Max (ksi)	Min (ksi)	D/C	
U	208.83	-57.09	30.28	-8.28	0.61	Ok
T	266.62	-85.54	38.66	-12.40	0.77	Ok
L	132.05	52.31	19.15	7.58	0.38	Ok
C	310.41	-70.90	45.01	-10.28	0.90	Ok

Prestressing force

56 50 diameter A354 Grade BD anchor bolts

Area =	1612.9	mm2	2.50	in2
Fu =	1034	MPa	150	ksi

Assume that bolts are tensioned to 0.68 Fu

Tension/bolt =	1.13	MN/bolt	253	k/bolt
Total force =	63	MN	14175	k

Friction coefficient, μ, steel to steel
- assume class B surface

μ = 0.5 Table 6.13.2.8-3

Lateral capacity at interface

- With no other loads present

P =	63	MN	14175	k
μ x P =	31.5	MN	7088	k

- With uplift of 13.3 MN 2989.84 k

P =	50	MN	11185	k
μ x P =	24.9	MN	5593	k

Demand

	VT (MN)	VL (MN)	P (MN)	Vtotal (MN)	Capacity	D/C	
U	25.3	1.6	-9.5	25.4	26.8	0.95	Ok
T	30.5	8.2	16.4	31.6	39.7	0.79	Ok
L	1.3	13.2	19.3	13.3	41.2	0.32	Ok
All maximums concurrent							
	30.5	13.2	-9.5	31.6	26.8	1.18	NG
Design demands							
C	30	15	-17	33.5	23.0	1.46	NG

Assumptions:
1. Look at it from a "block shear" perspective.
2. Local effects only - assumed that load spreads quickly away from bearing.
3. A709M Grade 345 (A709 Grade 50) steel.

Fy =	345	MPa	50	ksi
Fu =	450	MPa	65	ksi

Shear Key Demand

	MN	k
Compression	0	0
Uplift	0	0
Longitudinal	0	0
Transverse	60	13500

Shear Key "footprint"

Length, L =	3600	mm	141.7	in
Width, w =	3400	mm	133.9	in

Key plate "footprint"

Length, L =	4400	mm	173.2	in
Width, w =	4200	mm	165.4	in

Block Shear

If Atn ≤ 0.58 Avn then	Rr = ϕbs (0.58 Fy Avg + Fu Atn)	6.13.4-1
otherwise	Rr = ϕbs (0.58 Fu Avn + Fy Atg)	6.14.4-2
ϕbs = 0.80		6.5.4.2

Case 1 - Looking at bottom flange from the most conservative perspective

1. Ignore presence of transverse webs and longitudinal shear plates
2. Use minimum of key plate - holes or bottom flange no holes
3. Assume bottom flange thickness = 35 mm 1.38 in
4. Assume key plate thickness = 75 mm 2.95 in longitudinal
5. Assume 8 diameter 100 mm 3.94 in holes transverse
6. Assume 8 diameter 100 mm 3.94 in holes longitudinal

Transverse

Axial and shear areas

Atg =	308000	mm2	477.4	in2			
Avg =	294000	mm2	455.7	in2			
Atn =	308000	mm2	477.4	in2			
Avn =	294000	mm2	455.7	in2			
Rr =	157943.5	kN	35506	k	D/C =	0.38	Ok

Shear areas only

Atg =	0	mm2	0.0	in2			
Avg =	294000	mm2	455.7	in2			
Atn =	0	mm2	0.0	in2			
Avn =	294000	mm2	455.7	in2			
Rr =	61387.2	kN	13800	k	D/C =	0.98	Ok

Axial areas only

Atg =	308000	mm2	477.4	in2			
Avg =	0	mm2	0.0	in2			
Atn =	308000	mm2	477.4	in2			
Avn =	0	mm2	0.0	in2			
Rr =	110880	kN	24926	k	D/C =	0.54	Ok

Longitudinal

Axial and shear areas

Atg =	294000	mm2	455.7	in2			
Avg =	308000	mm2	477.4	in2			
Atn =	294000	mm2	455.7	in2			
Avn =	308000	mm2	477.4	in2			
Rr =	155144.6	kN	34877	k	D/C =	0.00	Ok

Shear areas only

Atg =	0	mm2	0.0	in2			
Avg =	308000	mm2	477.4	in2			
Atn =	0	mm2	0.0	in2			
Avn =	308000	mm2	477.4	in2			
Rr =	64310.4	kN	14457	k	D/C =	0.00	Ok

Axial areas only

Atg =	294000	mm2	455.7	in2			
Avg =	0	mm2	0.0	in2			
Atn =	294000	mm2	455.7	in2			
Avn =	0	mm2	0.0	in2			
Rr =	105840	kN	23793	k	D/C =	0.00	Ok

Transverse web plates

	MN	k
Demand	60	13500

Capacity (only consider 3 interior webs)

t =	40	mm	1.57	in			
Shear capacity =	24012	N/mm	137.0	k/in			
Length required =	2499	mm	98.5	in			
Width of bearing =			3400	mm			
			133.9	in	D/C =	0.74	Ok

Using conservative assumptions, there is sufficient capacity to carry transverse shear loads up into the girder.

Area of steel within shear key footprint

Webs (3)

Width =	3400	mm	133.9	in
Thickness =	40	mm	1.57	in
Area (total) =	408000	mm2	632.4	in2

Longitudinal stiffener

Width =	3600	mm	141.7	in
Thickness =	50	mm	1.97	in
Area (total) =	180000	mm2	279.0	in2

Longitudinal diaphragms (2)

Width =	3600	mm	141.7	in
Thickness =	50	mm	1.97	in
Area (total) =	720000	mm2	1116.0	in2

Bearing assembly 1 (4)

outer plates (2)				
Width =	300	mm	11.8	in
Thickness =	35	mm	1.38	in
inner plate (1)				
Width =	300	mm	11.8	in
Thickness =	50	mm	1.97	in
Area (1) =	36000	mm2	55.8	in2
Area (total) =	144000	mm2	223.2	in2

Bearing assembly 2 (4 pairs)

outer plates (2)				
Width =	300	mm	11.8	in
Thickness =	35	mm	1.38	in
inner plates (2)				
Width =	300	mm	11.8	in
Thickness =	40	mm	1.57	in
Area (1) =	45000	mm2	69.8	in2
Area (total) =	360000	mm2	558.0	in2

Bearing assembly 3 (2 pairs)

outer plates (2)				
Width =	300	mm	11.8	in
Thickness =	35	mm	1.38	in
inner plates (2)				
Width =	300	mm	11.8	in
Thickness =	40	mm	1.57	in
center plate (1)				
Width =	300	mm	11.8	in
Thickness =	50	mm	1.97	in
Area (1) =	60000	mm2	93.0	in2
Area (total) =	240000	mm2	372.0	in2

Total plate bearing area above bottom flange within shear key footprint

Area (total) =	2052000	mm2	3181	in2

Prestressing force (stiffened bolts)

48	76 diameter A354 Grade BD anchor bolts						
Area =	3851.6	mm2	5.97	in2			
Fu =	966	MPa	140	ksi			
Conservatively assume that bolts are tensioned to Fu							
Tension/bolt =	3.72	MN/bolt	836	k/bolt			
Total force =	179	MN	40118	k			
Conservatively assume that force transferred thru stiffeners only							
Stress =	239.9	MPa	34.8	ksi	D/C =	0.70	Ok
Force through stiffeners webs and diaphragms							
Stress =	95.4	MPa	13.8	ksi	D/C =	0.20	Ok

Look at prestressing stiffener assembly

Material properties

Steel plate				
Fy =	345	MPa	50	ksi
Fu =	450	MPa	65	ksi
Weld metal				
Fexx =	485	MPa	70	ksi
ϕe =	0.80			
Rexx =	232.8	MPa	33.6	ksi
50 diameter A354 Grade BD anchor bolts				
Area =	3851.6	mm2	5.97	in2
Fu =	966	MPa	140	ksi
Conservatively assume load on each bolt = 0.8Fu				
Pu =	2.98	MN/bolt	668.64	k/bolt

Geometry

Top bearing plate				
b =	300	mm	11.8	in
t =	200	mm	7.9	in
L =	400	mm	15.7	in
hole diameter =	100	mm	3.9	in
Center stiffener				
b =	300	mm	11.8	in
t =	50	mm	2.0	in
L =	600	mm	23.6	in
tw =	40	mm	1.6	in
Interior stiffener				
b =	300	mm	11.8	in
t =	50	mm	2.0	in
L =	600	mm	23.6	in
tw =	40	mm	1.6	in
Outer stiffener				
b =	300	mm	11.8	in
t =	35	mm	1.4	in
L =	600	mm	23.6	in
tw =	40	mm	1.6	in

Bearing plate capacity

Look at shear between 2 bearing plates

I =	200000000	mm4	480.5	in4		
Q =	1500000	mm3	91.5	in4		
t =	90	mm	3.54	in		
Vmax =	2.23	MN	501.5	k		
VQ/It =	185.94	MPa	26.96	ksi	D/C =	0.80

D/C <= 1.00 - therefore can assume bearing plates act as single unit

Sxx (at hole) =	1333333	mm3	81.36	in3			
My =	460000000	N-mm	4068.2	k-in			

Assume elastic simply supported bearing plate
Mmax = 1/4 Pmax L

Pmax =	4.60	MN	1035.0	k	D/C =	0.65	Ok

Reactions at stiffeners (conservatively use max of continuous or simple support)

Rcenter =	4.09	MN	919.6	k
Router =	1.49	MN	334.4	k

Center stiffener capacity

Compression - column analogy
Include 200mm of web

Imin =	32916667	mm4	79.1	in4			
A =	38000	mm2	58.9	in2			
r =	29.43	mm	1.16	in			
use k =	1.00	for mill to bear ends					
kL/r =	20.39						
sqrt(2 x π^2 x E / Fy) =	107.0						
Fcr =	338.7	MPa	49.1	ksi			
Pmax =	5.08	MN	1142	k	D/C =	0.81	Ok

Shear

D/t =	12.00						
6000 sqrt(k) / sqrt(Fy) =	60.00						
C =	1.00						
ϕs =	1.00						
Vr =	6.0	MN	1349	k	D/C =	0.68	Ok

Check weld

PJP =	32.0	mm	1.26	in			
Weld strength =	4.5	MN	1005	k	D/C =	0.92	Ok

Outer stiffener capacity

Compression - column analogy
Include 200mm of web

Imin =	28810417	mm4	69.2	in4
A =	29000	mm2	45.0	in2
r =	31.52	mm	1.24	in
use k =	1.00	for mill to bear ends		
kL/r =	19.04			

sqrt(2 x π^2 x E / Fy) =	107.0						
Fcr =	339.5	MPa	49.2	ksi			
Pmax =	3.57	MN	801	k	D/C =	0.42	Ok
Shear							
D/t =	17.14						
6000 sqrt(k) / sqrt(Fy) =	60.00						
C =	1.00						
ϕs =	1.00						
Vr =	4.2	MN	945	k	D/C =	0.35	Ok
Check weld							
PJP =	29.0	mm	1.14	in			
Weld strength =	4.1	MN	911	k	D/C =	0.37	Ok
Check block shear tearout of diaphragms							
Shear length =	1200	mm	47.24	in			
Axial length =	1100	mm	43.31	in			
t =	50	mm	1.97	in			
Atn = Atg =	55000	mm2	85.3	in2			
Avn = Avg =	60000	mm2	93.0	in2			
ϕbs =	0.80						
Rr =	29.4	MN	6.6	k	D/C =	0.76	Ok
Check block shear tearout of webs							
Shear length =	1800	mm	70.87	in			
Axial length =	1220	mm	48.03	in			
teff =	43.3	mm	1.71	in			
Atn = Atg =	52867	mm2	81.9	in2			
Avn = Avg =	78000	mm2	120.9	in2			
ϕbs =	0.80						
Rr =	31.5	MN	7.1	k	D/C =	0.94	Ok

Area of steel within bearing footprint

Webs (3)				
Width =	3400	mm	133.9	in
Thickness =	40	mm	1.57	in
Area (total) =	408000	mm2	632.4	in2
Diaphragms (2)				
Width =	3600	mm	141.7	in
Thickness =	50	mm	1.97	in
Area (total) =	540000	mm2	837.0	in2

Total plate bearing area above bottom flange within bearing footprint

Area (total) =	948000	mm2	632.4	in2

Moments of inertia assuming all plates active

Assume symmetry

	Transv. mm4	Transv. in4
Web 1	1.31013E+11	314760.7924
Web 2	0	0
Web 3	1.31013E+11	314760.7924
Diaphragm 1	3.52838E+11	847695.4845
Diaphragm 2	3.52838E+11	847695.4845
Total moment of inertia =	9.67702E+11	2324912.554
	Transv. mm3	Transv. in3
Section modulus =	691215476	42180.6

Loadings

Transverse Demand =	42	MN
Arm =	750	mm
Moment =	31500	MN-mm

Max/Min Stress = My/Sy

	Max (MPa)	Min (MPa)	Max (ksi)	Min (ksi)
Normal Stress	45.57	-45.57	6.61	-6.61

Add prestressing force

Force through stiffeners webs and diaphragms

Stress =	95.35	MPa	13.83	ksi

	Max (MPa)	Min (MPa)	Max (ksi)	Min (ksi)	D/C	
T	140.93	49.78	20.43	7.22	0.41	Ok

Prestressing force

80 76 diameter A354 Grade BD anchor bolts

Area =	3852	mm2	5.97	in2
Fu =	966	MPa	140	ksi

Assume that bolts are tensioned to 0.68 Fu

Tension/bolt =	2.510	MN/bolt	564	k/bolt
Total force =	201	MN	45133	k

Friction coefficient, μ, steel to steel
- assume class B surface

μ = 0.5 Table 6.13.2.8-3

Lateral capacity at interface

- No other axial loads present

P =	201	MN	45133	k			
Shear capacity							
μ x P =	100	MN	22567	k	D/C =	0.60	Ok

Cross Beam Estimated Section Properties

Top flange					
	Width =	10000	mm	393.7	in
	Thickness =	20	mm	0.8	in
	Stiffeners				
	Number =	20			
	Width =	205	mm	8.1	in
	Thickness =	22	mm	0.9	in
"Effective" top flange					
	Width =	10000	mm	393.7	in
	Thickness =	29.02	mm	1.1	in
Outer webs					
	Width =	5500	mm	216.5	in
	Thickness =	20	mm	0.8	in
	Stiffeners				
	Number =	13			
	Width =	205	mm	8.1	in
	Thickness =	22	mm	0.9	in
"Effective" outer web					
	Width =	5500	mm	216.5	in
	Thickness =	30.66	mm		
Inner webs1					
	Width1 =	3275	mm	128.9	in
	Thickness1 =	20	mm	0.8	in
	Stiffeners1				
	Number =	7			
	Width =	205	mm	8.1	in
	Thickness =	22	mm	0.9	in
	Width2 =	2225	mm	87.6	in
	Thickness2 =	40	mm	1.6	in
	Stiffeners2				
	Number =	4			
	Width =	200	mm	7.9	in
	Thickness =	40	mm	1.6	in
"Effective" inner web1					
	Width1 =	3275	mm	128.9	in
	Thickness1 =	29.64	mm	1.2	in
	Width2 =	2225	mm	87.6	in
	Thickness2 =	54.38	mm	2.1	in

Inner webs2

Width1 =	3275	mm	128.9	in
Thickness1 =	20	mm	0.8	in
Stiffeners1				
Number =	7			
Width =	205	mm	8.1	in
Thickness =	22	mm	0.9	in
Width2 =	2225	mm	87.6	in
Thickness2 =	40	mm	1.6	in
Stiffeners2				
Number =	5			
Width =	200	mm	7.9	in
Thickness =	40	mm	1.6	in

"Effective" inner web2

Width1 =	3275	mm	128.9	in
Thickness1 =	29.64	mm	1.2	in
Width2 =	2225	mm	87.6	in
Thickness2 =	57.98	mm	2.3	in

Bottom flange

Width1 =	3000	mm	118.1	in
Thickness1 =	35	mm	1.4	in
Stiffeners1				
Number =	7			
Width =	310	mm	12.2	in
Thickness =	35	mm	1.4	in
Width2 =	4000	mm	157.5	in
Thickness2 =	35	mm	1.4	in
Width3 =	3000	mm	118.1	in
Thickness3 =	35	mm	1.4	in
Stiffeners 3				
Number =	7			
Width =	310	mm	12.2	in
Thickness =	35	mm	1.4	in

"Effective" bottom flange

Width1 =	3000	mm	118.1	in
Thickness1 =	60.3	mm	2.4	in
Width2 =	4000	mm	157.5	in
Thickness2 =	35.0	mm	1.4	in
Width3 =	3000	mm	118.1	in
Thickness3 =	60.3	mm	2.4	in

Area =	1791570	mm2	2776.9	in2
NAxx (from top) =	3236	mm	127.4	in
NAyy (from center) =	0	mm	0.0	in
Ixx =	8.19082E+12	mm4	19678521	in4
Sxtop =	2531494140	mm3	154481.3	in3
Sxbot =	3617160932	mm3	220732.7	in3
Iyy =	1.55463E+13	mm4	37350231	in4
Sy =	777317000	mm3	47434.79	in3

Cable geometry (from Suspension Cable Layout No. 3)

	WP1	WPA	EP1	EPA
X	393	35852	393	34114
Y	36045	29461	-36025	-29939
Z	266	-3106	-1176	-4339

Angle in XY plane	West	0.18359	East	0.17856	radians
		10.519		10.231	degrees
Angle in XZ plane	West	0.09481	East	0.09353	radians
		5.432		5.359	degrees

Estimated max cable force

Fa =	689	MPa	100	ksi	(From General Notes No. 1)
net area =	0.39848	m2	617.6	in2	(From Suspension Cable Layout No. 1)
Pmax =	275	MN	61719	k	

Estimated forces in crossbeam at Pier E2

X =	50.1	MN	11267	k
ex =	500	mm	20	in
Y =	4.6	MN	1033	k
ey =	10785	mm	425	in
Z =	0.4	MN	98	k
ez =	2986	mm	118	in
Mx = X ez + Z ey =	154353.5	MN-mm	113841	k-ft
My = Y ex =	2296.6	MN-mm	1694	k-ft
P/A + Mx/Sx + My/Sy =	73.6	MPa	10.67	ksi

Seismic shear force

Peq =	120.0	MN	26976	k			
ez =	3014.4	mm	118.7	in			
P/A + Mx/Sx =	67.8	MPa	9.83	ksi			
Total stress =	141.4	MPa	20.51	ksi	D/C =	0.41	Ok

Cross Beam Estimated Section Properties

Top flange					
	Width =	4000	mm	157.5	in
	Thickness =	20	mm	0.8	in
	Stiffeners				
	Number =	6			
	Width =	205	mm	8.1	in
	Thickness =	22	mm	0.9	in
"Effective" top flange					
	Width =	4000	mm	157.5	in
	Thickness =	26.765	mm	1.1	in
Outer webs					
	Width =	5500	mm	216.5	in
	Thickness =	0	mm	0.0	in
	Stiffeners				
	Number =	0			
	Width =	205	mm	8.1	in
	Thickness =	22	mm	0.9	in
"Effective" outer web					
	Width =	5500	mm	216.5	in
	Thickness =	0	mm		
Inner webs1					
	Width1 =	3275	mm	128.9	in
	Thickness1 =	20	mm	0.8	in
	Stiffeners1				
	Number =	7			
	Width =	205	mm	8.1	in
	Thickness =	22	mm	0.9	in
	Width2 =	2225	mm	87.6	in
	Thickness2 =	40	mm	1.6	in
	Stiffeners2				
	Number =	4			
	Width =	200	mm	7.9	in
	Thickness =	40	mm	1.6	in
"Effective" inner web1					
	Width1 =	3275	mm	128.9	in
	Thickness1 =	29.64	mm	1.2	in
	Width2 =	2225	mm	87.6	in
	Thickness2 =	54.38	mm	2.1	in

Inner webs2				
Width1 =	3275	mm	128.9	in
Thickness1 =	20	mm	0.8	in
Stiffeners1				
Number =	7			
Width =	205	mm	8.1	in
Thickness =	22	mm	0.9	in
Width2 =	2225	mm	87.6	in
Thickness2 =	40	mm	1.6	in
Stiffeners2				
Number =	5			
Width =	200	mm	7.9	in
Thickness =	40	mm	1.6	in
"Effective" inner web2				
Width1 =	3275	mm	128.9	in
Thickness1 =	29.64	mm	1.2	in
Width2 =	2225	mm	87.6	in
Thickness2 =	57.98	mm	2.3	in
Bottom flange				
Width1 =	3000	mm	118.1	in
Thickness1 =	0	mm	0.0	in
Stiffeners1				
Number =	0			
Width =	310	mm	12.2	in
Thickness =	35	mm	1.4	in
Width2 =	4000	mm	157.5	in
Thickness2 =	35	mm	1.4	in
Width3 =	3000	mm	118.1	in
Thickness3 =	0	mm	0.0	in
Stiffeners 3				
Number =	0			
Width =	310	mm	12.2	in
Thickness =	35	mm	1.4	in
"Effective" bottom flange				
Width1 =	3000	mm	118.1	in
Thickness1 =	0.0	mm	0.0	in
Width2 =	4000	mm	157.5	in
Thickness2 =	35.0	mm	1.4	in
Width3 =	3000	mm	118.1	in
Thickness3 =	0.0	mm	0.0	in
Area =	909270	mm2	1409.4	in2
NAxx (from top) =	3163	mm	124.5	in
NAyy (from center) =	0	mm	0.0	in
Ixx =	3.48304E+12	mm4	8368031	in4
Sxtop =	1101169898	mm3	67197.51	in3
Sxbot =	1490409752	mm3	90950.38	in3
Iyy =	1.31073E+12	mm4	3149037	in4
Sy =	163841041.7	mm3	9998.194	in3

Project: *SFOBB - JN 3274*		Made by: *PAR*	*7/24/2013*
Subject: *Crossbeam capacity*		Checked by:	
Content: *Cable load*		Sheet No.:	

Cable geometry (from Suspension Cable Layout No. 3)

	WP1	WPA	EP1	EPA
X	393	35852	393	34114
Y	36045	29461	-36025	-29939
Z	266	-3106	-1176	-4339

Angle in XY plane	West	0.18359	East	0.17856	radians
		10.519		10.231	degrees
Angle in XZ plane	West	0.09481	East	0.09353	radians
		5.432		5.359	degrees

Estimated max cable force

Fa =	689	MPa	100	ksi	(From General Notes No. 1)
net area =	0.39848	m2	617.6	in2	(From Suspension Cable Layout No. 1)
Pmax =	275	MN	61719	k	

Estimated forces in crossbeam at Pier E2

X =	50.1	MN	11267	k
ex =	500	mm	20	in
Y =	4.6	MN	1033	k
ey =	10785	mm	425	in
Z =	0.4	MN	98	k
ez =	2913	mm	115	in
Mx = X ez + Z ey =	150718.0	MN-mm	111159	k-ft
My = Y ex =	2296.6	MN-mm	1694	k-ft
P/A + Mx/Sx + My/Sy =	170.3	MPa	24.69	ksi

Seismic shear force

Peq =	0.0	MN	0	k			
ez =	3087.0	mm	121.5	in			
P/A + Mx/Sx =	2.1	MPa	0.30	ksi			
Total stress =	172.3	MPa	24.99	ksi	D/C =	0.50	Ok

Appendix 3B

TYLin Calculations

Steel Orthotropic Girder at Bearings

TRANSFER OF LOADS TO OBG

Self-Anchored Suspension Bridge

San Francisco Oakland Bay Bridge East Span Seismic Safety Project

Caltrans Project No. 04-0120F4

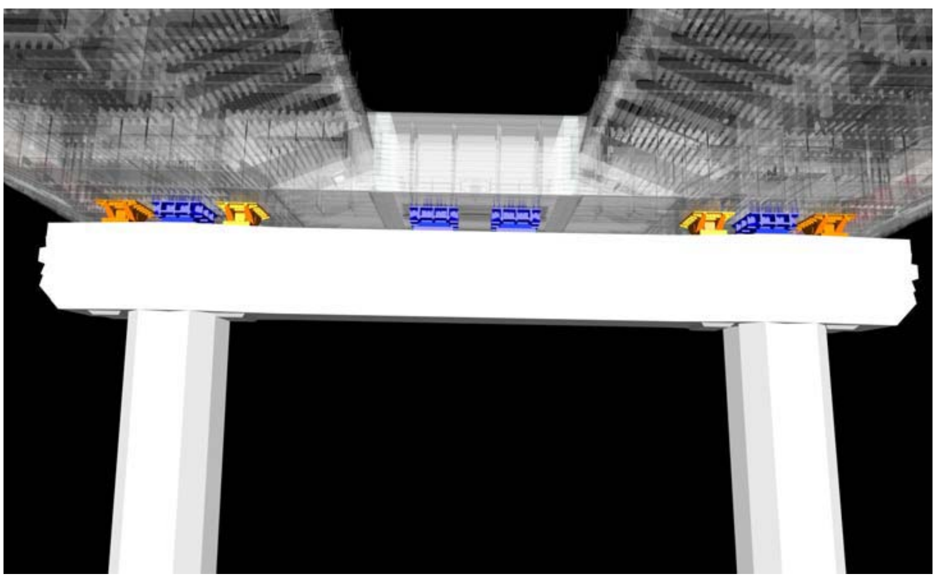
T.Y. Lin International / Moffatt & Nichol Joint Venture

July 27, 2013

BOX GIRDER SEISMIC CAPACITY DURING NEW DESIGN OF SHEAR KEYS S1, S2

These calculations evaluate the box girder at Pier E2 for seismic forces applied through the bearings prior to completing the new design of the shear keys S1 and S2. The modification of the bearings B1, B2, B3 and B4 by adding shims will not change the demands envisioned as part of the original design, shown in Page 3.

The connections of the bearings and shear keys to the OBG are achieved by the use of A354 BD rods anchored into stiffened anchor seats within the OBG. The tension of the rods generates friction to carry the longitudinal and transverse forces into a thickened key plate that forms the soffit of the box girder over Pier E2. The Key Plate appears in Section A-A of Page 4 and Section B-B of Page 5. This plate, 100 mm thick and 5 meters wide, forms a rigid platform to distribute the horizontal seismic loads coming from the shear keys and bearings, as well as from the global moments in the girders and crossbeam at Pier E2.



This figure shows the girders and crossbeams over the Pier E2 supports. The steel box crossbeam over the cap beam is subject to bending as it enters the main box girders. The box girders themselves bend as cantilevers over the support of the bearings and shear keys. These bendings produce compression in the key plates both transversely and longitudinally. These axial stresses have been added to the shear and axial stresses due to the bearings during an earthquake.

Vertical loads are carried by the tension of the rods, and by bearing against the stiffeners of the rod anchor seats. The layout of anchor seats is shown in Detail A on Page 4.

Pages 6 through 8 show the passage of seismic demands between the box girders and Pier E2. Page 6 shows the conceptual separation of the two structures. Page 7 shows the decomposition of force components at the bearing pins, as well as the vertical reactions into the girder due to the vertical and transverse demands. Page 8 shows the break-down of the longitudinal demands and resulting reactions. Page six also gives a table of the governing combined demands on the bearings for load path A (with shear keys engaged) and load path B (with shear keys disengaged).

Pages 9 and 10 show the upper housing of the bearing. A total of 52 rods clamp this housing against the key plate, and the large width to height ratio results in relatively small values of vertical reactions RT and RL into the box girder.

Pages 4, 5, 11-13 show the locations of shear, bearing and tension that have been checked to verify the capacity of the bearing to box girder connections. Pages 14-16 show representative calculations for the demand/capacity ratios for these locations.

The floorbeams and longitudinal shear plates are stiffened compact elements that engage the seismic demands into the beam action of the overall box girder.

A summary of these ratios is provided in the following table, numbered per the figures in the appendix:

Component	Force	Demand/Capacity
1. Key plate	Longitudinal Transverse Combined Global and Local	10% 35% 62%
2. Longitudinal shear plate	Longitudinal to the Key Plate	76% 80%
3. Floorbeams at Pier E2	Transverse to the Key Plate Combined Global and Local	50% 56%
4. Anchor seat middle stiffeners	Tension of anchor rods	79%
5. Anchor seat side stiffeners	Tension of anchor rods	45%
6. Floorbeam web	Shear tearout at anchor seats	87%
7. Bearing upper housing	Shear friction against sliding	45%
8. A354 anchor rods	Axial tension	93%

For the vertical reactions the pretension of the anchor rods carries effectively most of the load. Only a small increase in tension results from the seismic response. Note that anchor rods were installed at 85% of the yield capacity.

In conclusion, the use of load path B during the retrofit of shear keys S1 and S2 engages the capacities anticipated in the original design, and effectively protects the bridge in the event of the Safety Evaluation Earthquake.

Caltrans Metric

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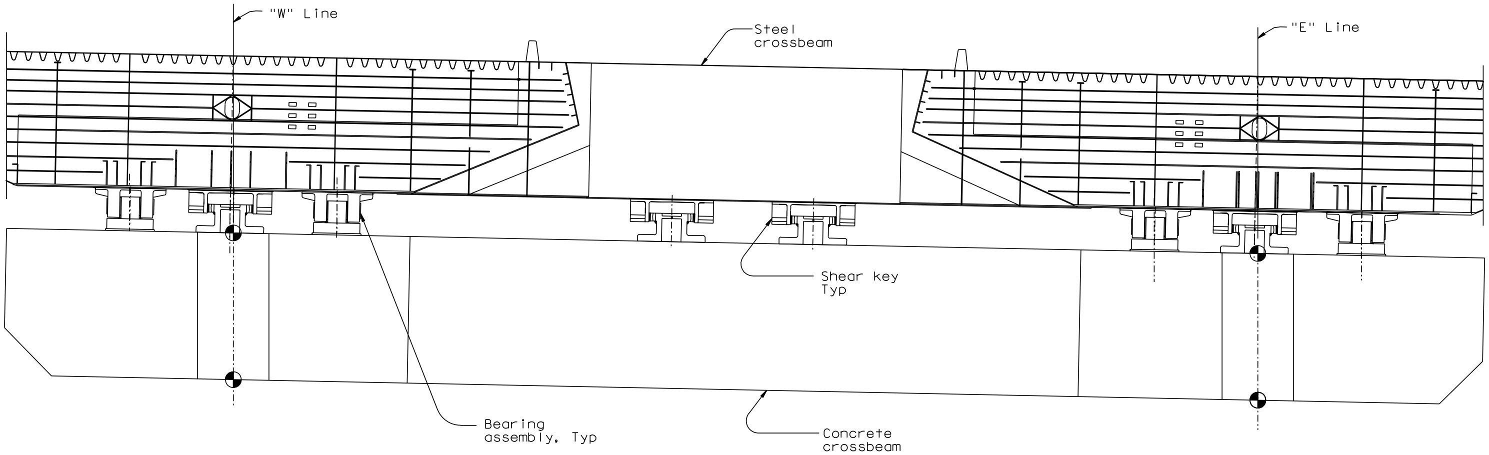
12-6-04

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Marwan N. Nader No. C 054426 Exp. 12/31/13 CIVIL STATE OF CALIFORNIA

T.Y. LIN / MOFFATT & NICHOL
825 BATTERY STREET
SAN FRANCISCO, CA 94111



ELEVATION (Looking East)

BEARING ASSEMBLY TABLE

Location	Bearing Typ	Number of bearing units per location	Design Load (kN per bearing) Service Compression	Service Uplift	Service Long or Trans	Ultimate Load Condition	Ultimate Compression	Ultimate Uplift	Ultimate Long / Trans	Design rotation (radians) Service Trans Axis	Service Long Axis	Service Vert Axis	Ultimate Trans Axis	Ultimate Long Axis	Ultimate Vert Axis	Design Displacement (mm) Service Trans Axis	Service Long Axis	Service Vert Axis	Ultimate Trans Axis	Ultimate Long Axis	Ultimate Vert Axis
"W" Line	Spherical Bushing	2	35000	0	0	Shear Key Engaged	86000	32000	0	009	0	0	0.130	0.032	0.032	5	5	0	20	20	2**
						Shear Key Failed*	68000	17000	13300 / 30500												
"E" Line	Spherical Bushing	2	35000	0	0	Shear Key Engaged	86000	32000	0	009	0	0	0.130	0.032	0.032	5	5	0	20	20	2**
						Shear Key Failed*	68000	17000	13300 / 30500												

* Seismic load factor $\gamma = 1.0$ (For shear key engaged load condition, $\gamma = 1.4$).
** For uplifting only.

Installation Misalignment Tolerances (Per bearing/ Shear Key)	Trans./Long./Vertical	Rotational
	± 2 mm	± 0.5°

SHEAR KEY ASSEMBLY TABLE

Location	Number of Shear Key Units Per Location	Design Load (kN per shear key) Service Trans	Service Long	Service Vert	Ultimate Trans	Ultimate Long	Ultimate Vert	Design rotation (radians) Service Trans Axis	Service Long Axis	Service Vert Axis	Ultimate Trans Axis	Ultimate Long Axis	Ultimate Vert Axis	Design Displacement (mm) Service Trans Axis	Service Long Axis	Service Vert Axis	Ultimate Trans Axis	Ultimate Long Axis	Ultimate Vert Axis
"W" Line	1	9000	4500	0	42500	35000	0	0.009	0	0	0.130	0.130	0.130	0	0	10	0	0	20
"E" Line	1	9000	4500	0	42500	35000	0	0.009	0	0	0.130	0.130	0.130	0	0	10	0	0	20
Crossbeam	2	9000	1	0	42500	0	0	0.009	0	0	0.130	0.130	0.130	0	5	10	0	20	20

CONTRACT CHANGE ORDER NO. ____
SHEET ____ OF ____

REQUESTS FOR INFORMATION NOT ADDRESSED IN THIS CCO REMAIN IN FORCE

MARK	DATE	DESCRIPTIONS	BY	CH'D	CCO#
2	XX/XX/13	E2 CROSS BEAM	MN	NV	XX
1	09/26/08	E2 CROSS BEAM	MN	NV	71

REVISIONS

R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE XX/XX/13

DESIGN BY M. Nader — CHECKED L. Rus
DETAILS BY N. Vo — CHECKED J. Leventini
QUANTITIES BY N. Vo — CHECKED J. Leventini

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

ALL DIMENSIONS ARE IN MILLIMETERS

SAN FRANCISCO OAKLAND BAY BRIDGE

Bearing Demands

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 04
EA 0120F1

DISREGARD PRINTS BEARING EARLIER REVISION DATES → 12/07/05

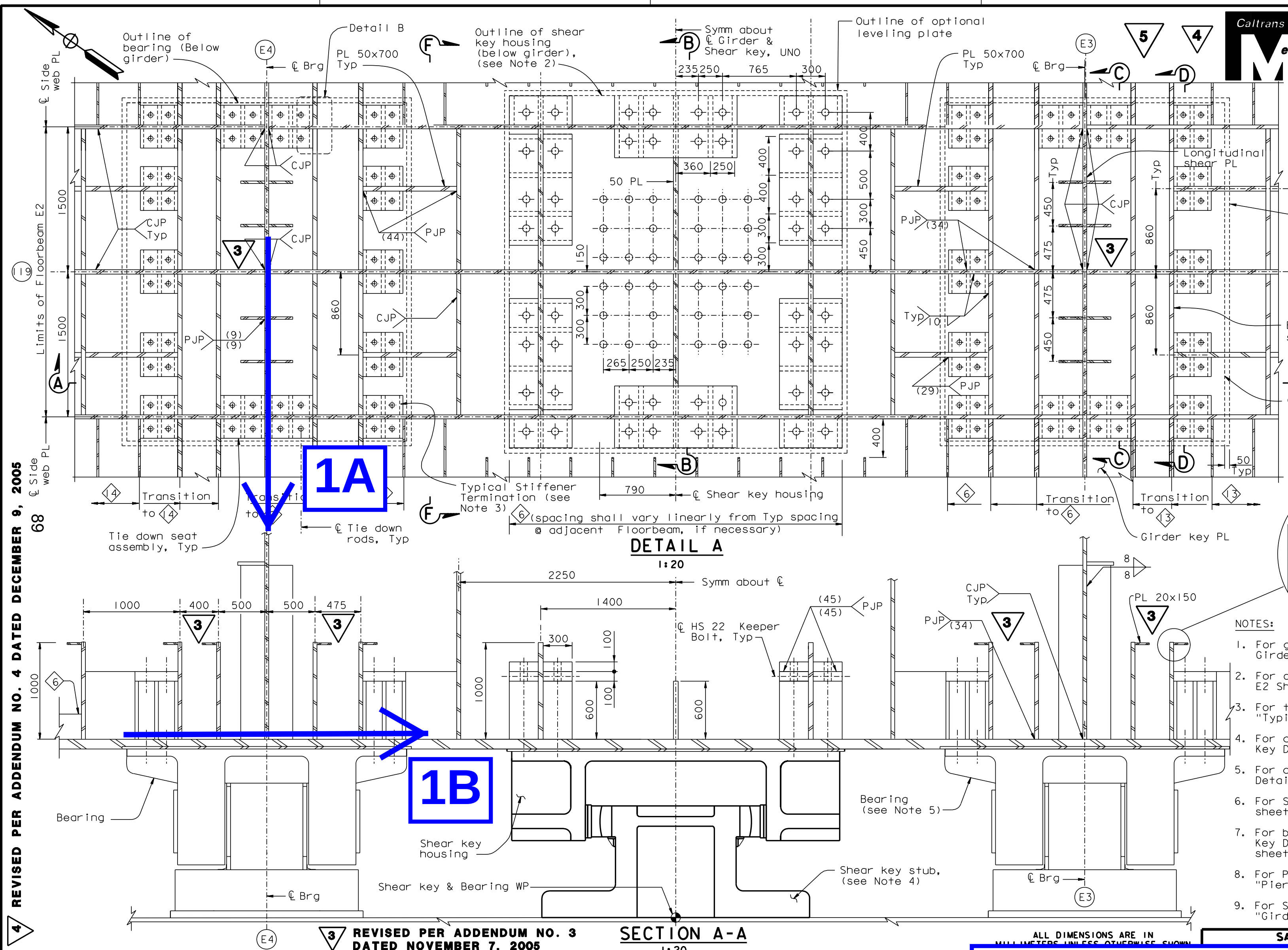
REVISION DATES (PRELIMINARY STAGE ONLY)

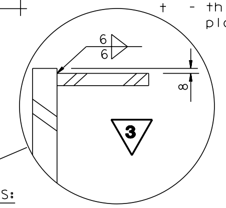
SHEET 465AR2 OF

FILE => L:\BB\04-0120I\SAS\Contract Plans and CCO\CCO\CCO#XXX\I-Original XX-XX-13\DGN\aksrd02.dgn

USERNAME => arivera DATE PLOTTED => 19 JUL 2013 TIME PLOTTED => 15:31:41

100 % PS&E





Outline of optional leveling plate, Typ

Symm about ℄ Floorbeam E2 & ℄ Shear key

Bearing stiffener

Outline of Bearing (see Note 5)

LEGEND

- 75 Dia anchor bolt (A354 Gr. BD)
- 64 Dia anchor bolt (A354 Gr. BD)
- 29 Dia anchor bolt (A354 Gr. BD)
- 50 Dia anchor bolt (A354 Gr. BD)
- † - thickness of terminating plate in T-joint

NOTES:

1. For girder plate thickness, see "Typical Girder Details No. 1" sheet.
2. For details of shear key housing, see "Pier E2 Shear Key" sheets.
3. For typical girder stiffener termination, see "Typical Girder Details No. 16" sheet.
4. For details of shear key, see "Pier E2 Shear Key Details" sheets.
5. For details of bearing, see "Pier E2 Bearing Details" sheets.
6. For Section F-F, see "Girder At Pier E2 No. 6" sheet.
7. For bolt and rod locations, see "Pier E2 Shear Key Details" and "Pier E2 Bearing Details" sheets.
8. For Pier E2 bearing and shear key location, see "Pier E2 Bearing And Shear Key Layout" sheet.
9. For Section B-B, C-C, D-D and detail B, see "Girder At Pier E2 No. 3" sheet.

ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN

3 REVISED PER ADDENDUM NO. 3 DATED NOVEMBER 7, 2005

4 REVISED PER ADDENDUM NO. 4 DATED DECEMBER 9, 2005

5 REVISED PER ADDENDUM NO. 5 DATED DECEMBER 21, 2005

68

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	675	1204

George Baker
REGISTERED ENGINEER - CIVIL
12-6-04
PLANS APPROVAL DATE

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

REGISTERED PROFESSIONAL ENGINEER George S. Baker No. C57112 Exp. 12/31/05 CIVIL STATE OF CALIFORNIA

T.Y. LIN / MOFFATT & NICHOL
825 BATTERY STREET
SAN FRANCISCO, CA 94111

Caltrans now has a web site! To get to the web site, go to: http://www.dot.ca.gov

Caltrans Metric

R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 12/07/05

DESIGN	BY G. Baker	CHECKED P. Ritchie
DETAILS	BY C. Mauch	CHECKED T. McMeans
QUANTITIES	BY D. Turner	CHECKED M. Roberts

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

SAN FRANCISCO OAKLAND BAY BRIDGE

Key Plate - Axial Check

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 04
EA 0120F1

DISREGARD PRINTS BEARING EARLIER REVISION DATES

SHEET 258

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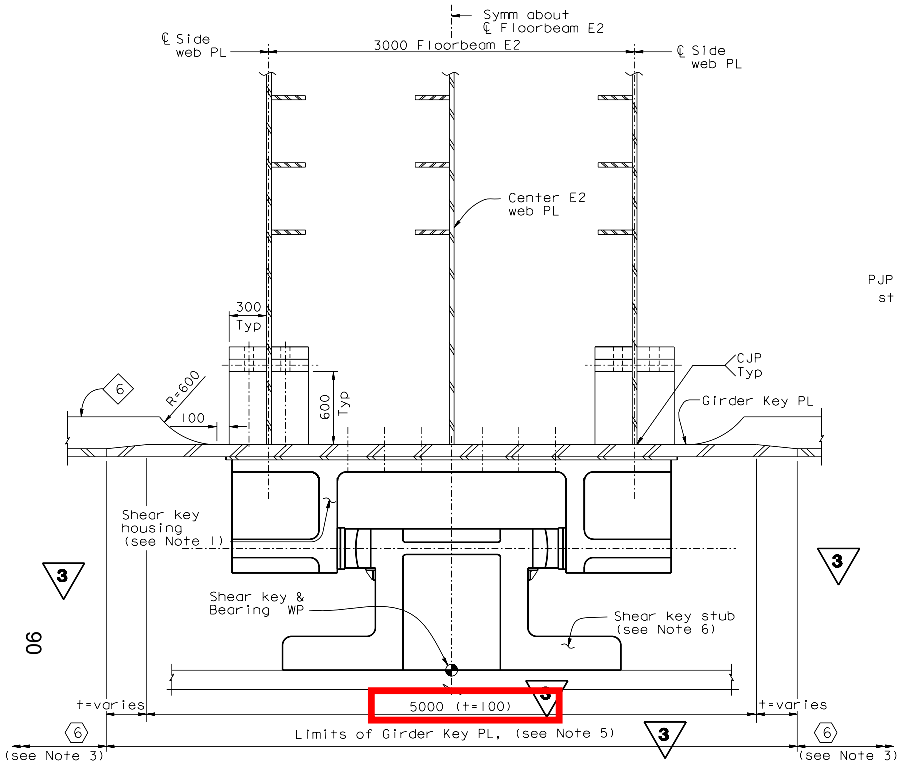
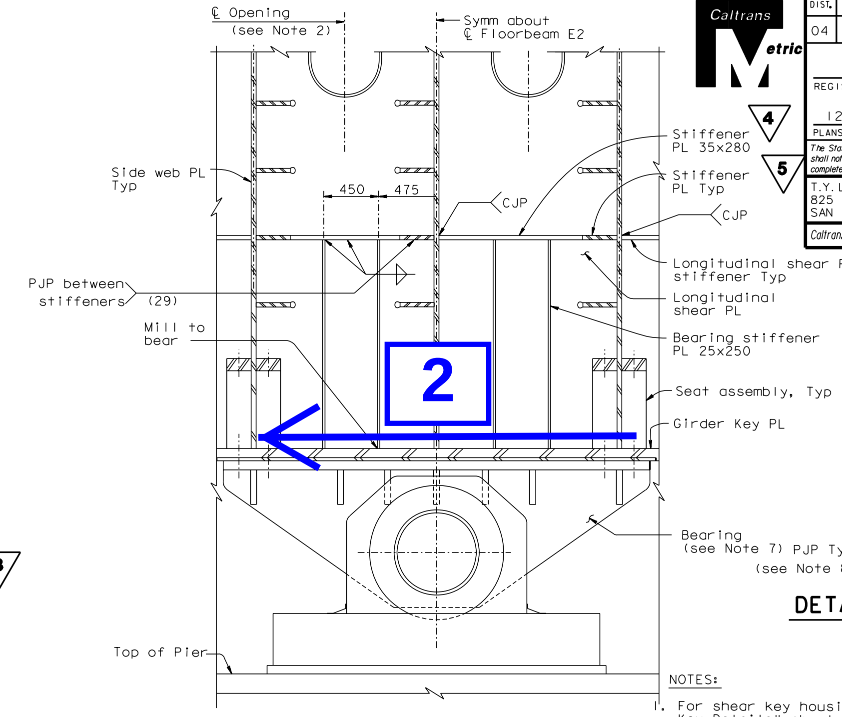
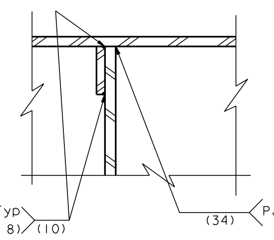
Longitudinal Shear Plate - Shear Check



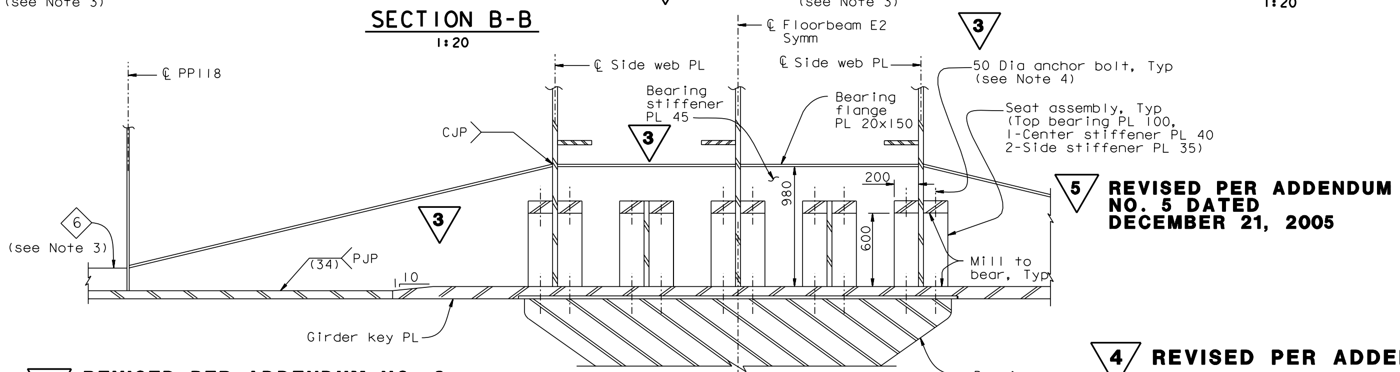
SECTION B-B
1:20



SECTION C-C
1:20



DETAIL B - ALTERNATIVE STIFFENER WELD
1:10



SECTION D-D
1:20

NOTES:

1. For shear key housing details, see "Pier E2 Shear Key Details" sheet.
2. For opening details, see "Typical Girder Details No. 7" sheet.
3. For typical girder details not shown, see "Typical Girder Details" sheets.
4. For anchor bolt, see "Pier E2 Bearing Details" sheets.
5. For transverse limits of girder key PL, see "Girder At Pier E2 No. 1" sheet.
6. For shear key details, see "Pier E2 Shear Key Details" sheets.
7. For Pier E2 bearing details, see "Pier E2 Bearing Details" sheets.
8. For bearing stiffeners in faying contact with diaphragms or webs, stiffener corners and welds may be ground to allow fit-up.
9. Leveling plate may be used to compensate for the permissible out-of-flatness of the girder key plate. Leveling plates shall be tapered as required to ensure proper alignment of the bearings and shear keys in both longitudinal and transverse directions.

3 REVISED PER ADDENDUM NO. 3 DATED NOVEMBER 7, 2005

4 REVISED PER ADDENDUM NO. 4 DATED DECEMBER 9, 2005

5 REVISED PER ADDENDUM NO. 5 DATED DECEMBER 21, 2005

ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE SHOWN

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	676	1204

George Baker
REGISTERED ENGINEER - CIVIL
12-6-04
PLANS APPROVAL DATE
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R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 12/07/05

DESIGN BY G. Bo
DETAILS BY C. Mo
QUANTITIES BY D. Tu

Rev. Date: 5-18-98

EA 0120F1

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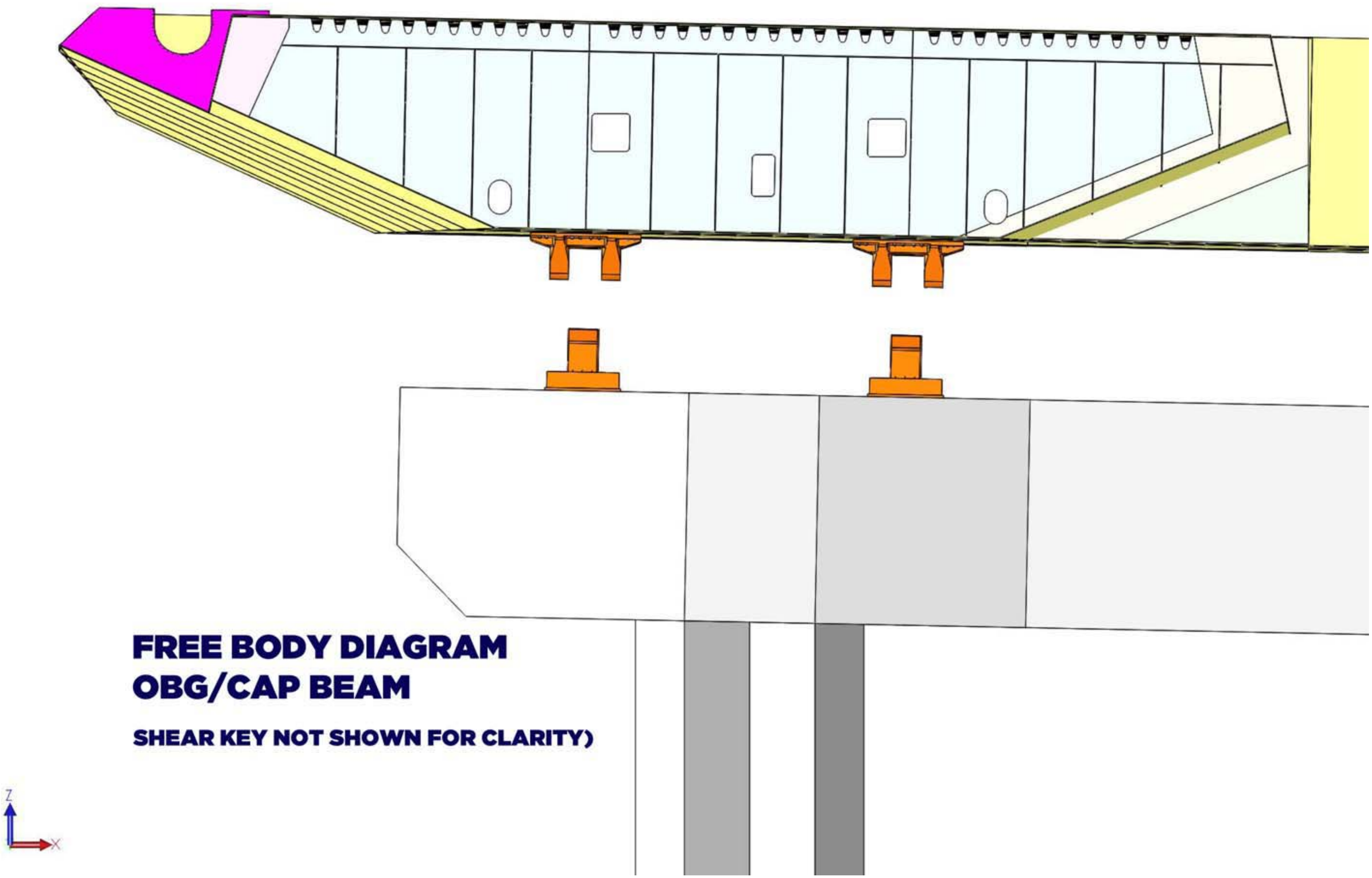
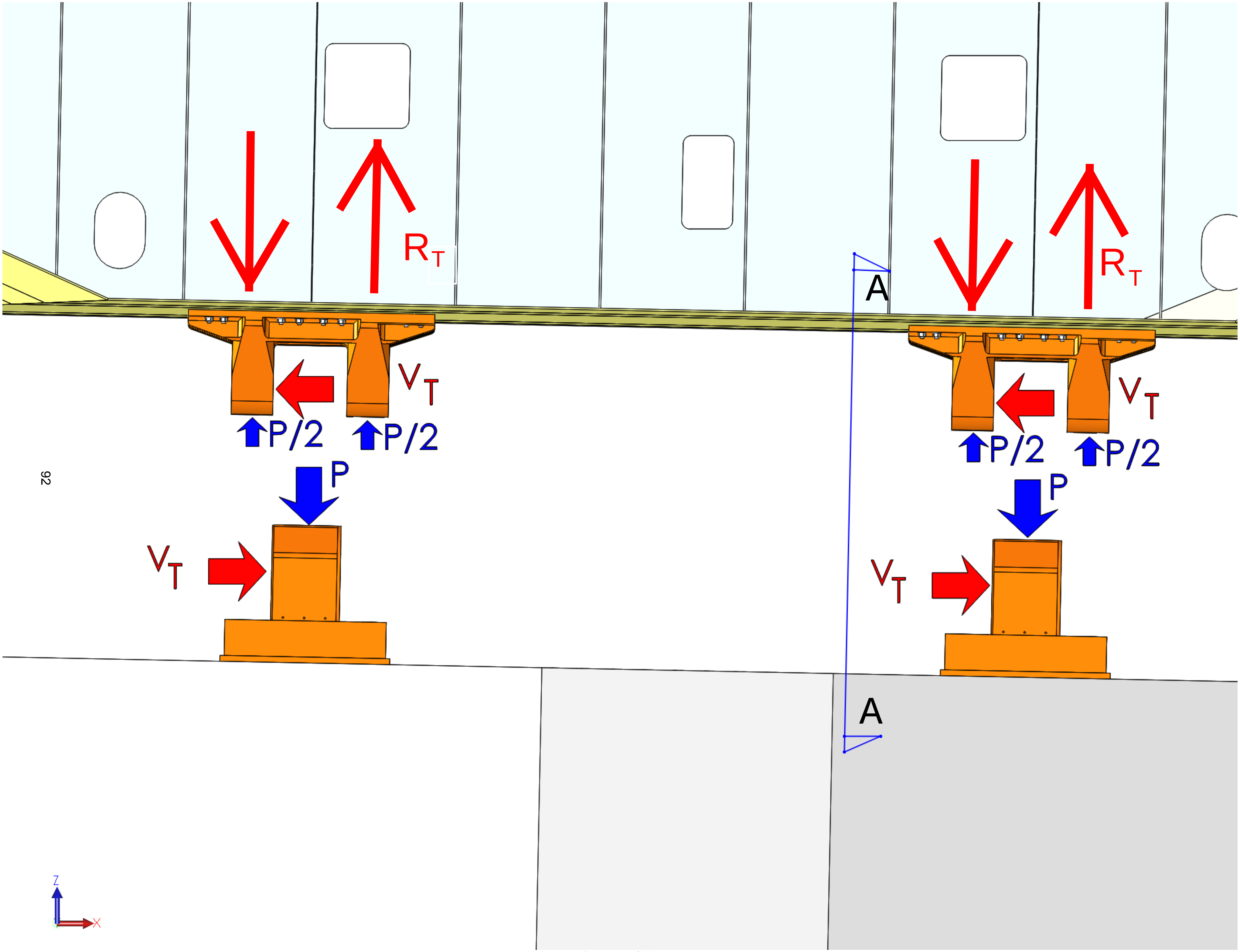

FREE BODY DIAGRAM
OBG/CAP BEAM
SHEAR KEY NOT SHOWN FOR CLARITY)
Z
X



R_T
V_T
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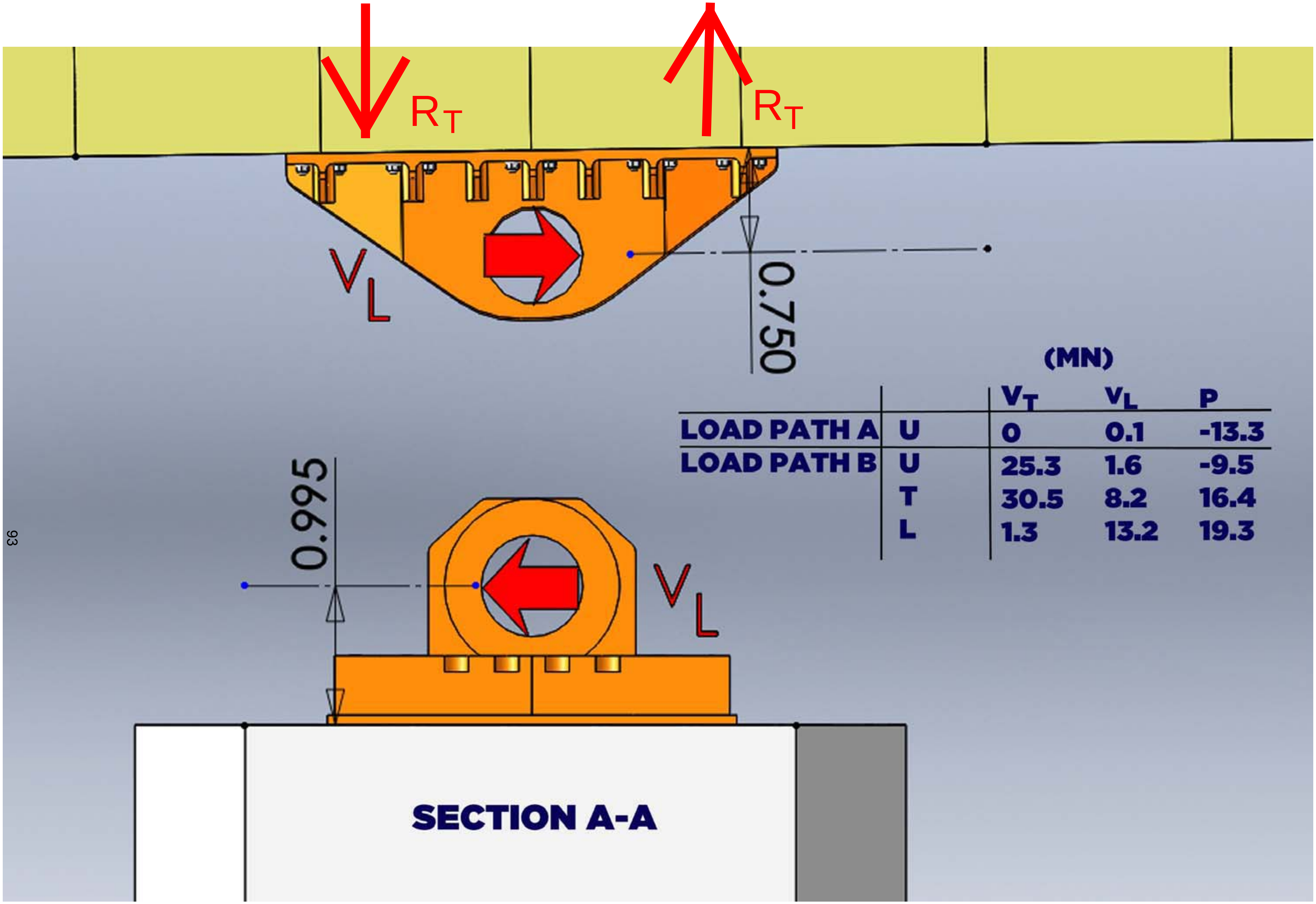

(MN)

		V_T	V_L	P
LOAD PATH A	U	0	0.1	-13.3
LOAD PATH B	U	25.3	1.6	-9.5
	T	30.5	8.2	16.4
	L	1.3	13.2	19.3

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	883	1204

REGISTERED ENGINEER - CIVIL

12-6-04

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Marwan N. Nader No. C 054426 Exp. 12/31/05 CIVIL STATE OF CALIFORNIA

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825 BATTERY STREET
SAN FRANCISCO, CA 94111

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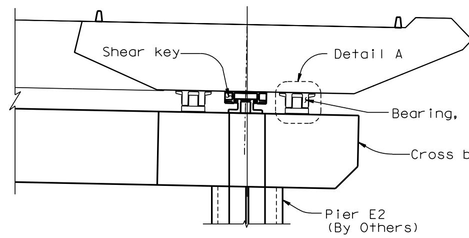
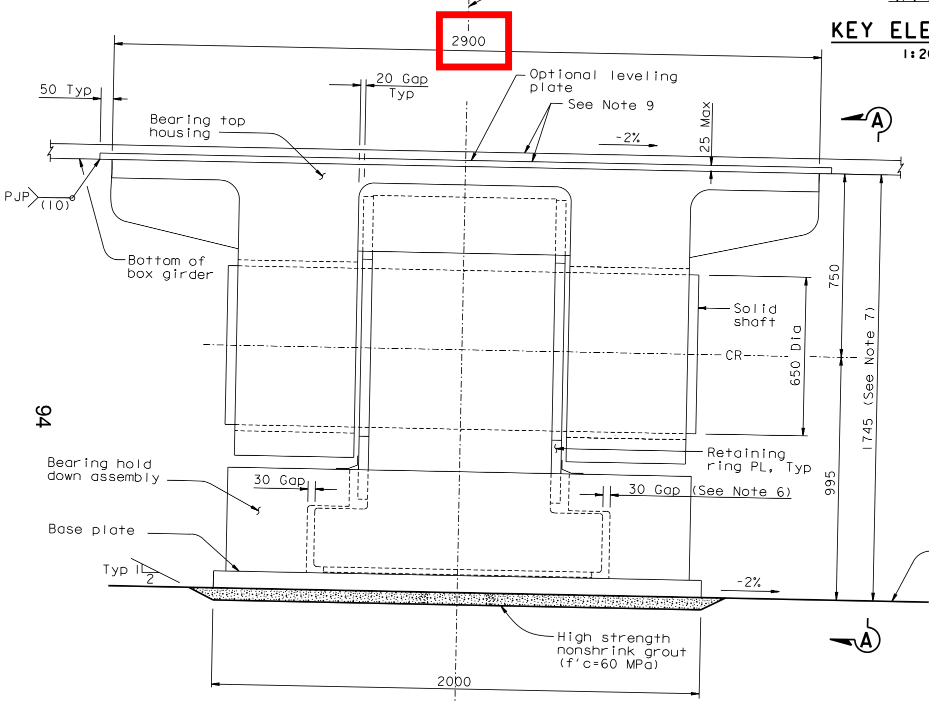
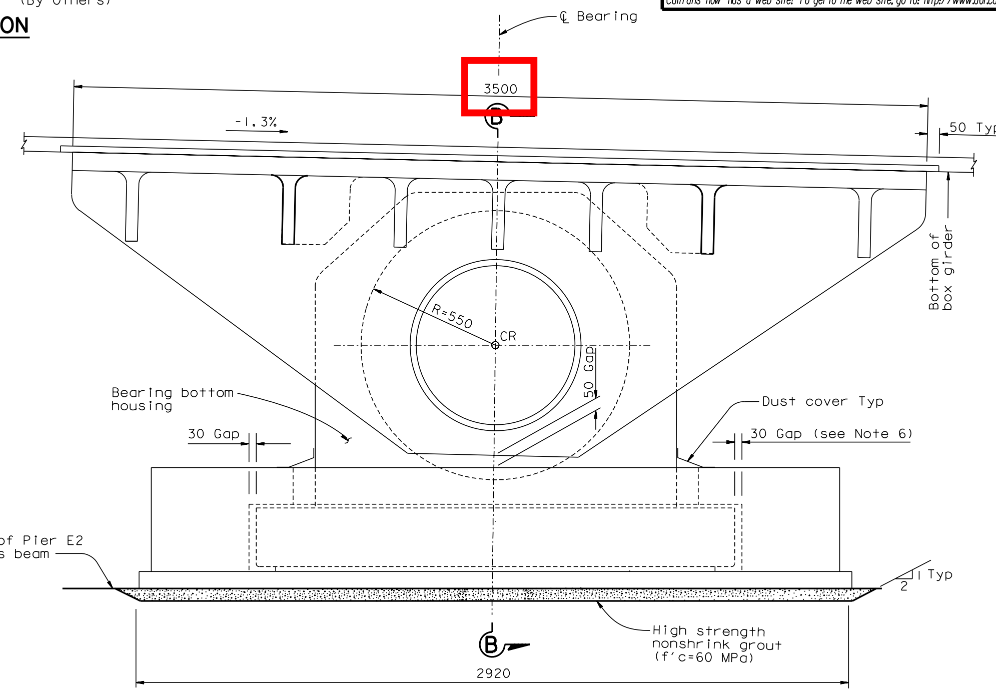


KEY ELEVATION
1:200



DETAIL A (Bearing Assembly)
1:10



VIEW A-A
1:10

5 REVISED PER ADDENDUM NO. 5 DATED DECEMBER 21, 2005

LEGEND:

CR Center of Rotation

NOTES:

1. For Section B-B, see "Pier E2 Bearing Details No.2" sheet.
2. Connections to box girder and Pier E2 are not shown for clarity.
3. The bearing top housing and bearing hold down assembly shall be Structural Casting Grade 345.
4. The bearing bottom housing and the solid shaft shall be Structural Casting Grade 550.
5. The grout pad thickness is shown for information only. Before grout pour, Contractor shall verify in the field the grout pad thickness required to align the center of rotation of the shear key and bearings at 0.750 m from the bottom surface of the box girder and ensure center of rotation of all bearings and shear keys are aligned in the same axis.
6. Gaps shall be maintained during installation by using shims placed below the dust cover locations. Remove shims after grouting.
7. The Contractor may provide optional leveling plates to achieve fit-up and level contact surfaces for the bearings and shear keys. The Contractor shall erect the E2 cap beam and E2 girders to the elevations and tolerances specified in the plans and special provisions.
8. Leveling plate shall be attached with 20 mm plug welds at 0.5 m Max spacing, and a perimeter PJP weld.
9. All faying surfaces of the girder key plate, the bearing top housing, and the optional leveling plate shall be machined after all welding for flatness and smoothness as specified for the top of the bearing top housing. See "Pier E2 Bearing Detail No. 3" sheet.

ALL DIMENSIONS ARE IN

SAN FRANCISCO OAKLAND BAY BRIDGE

Bearing Dimensions

R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 12/07/05

DESIGN	BY M. Nader	CHECKED L. Rus
DETAILS	BY N. Vo	CHECKED J. Leventini
QUANTITIES	BY N. Vo	CHECKED J. Leventini

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 04
EA 0120F1

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 05/14/01 04/08/02 07/01/02 12/19/02 12/07/05

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12-6-04
PLANS APPROVAL DATE

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825 BATTERY STREET
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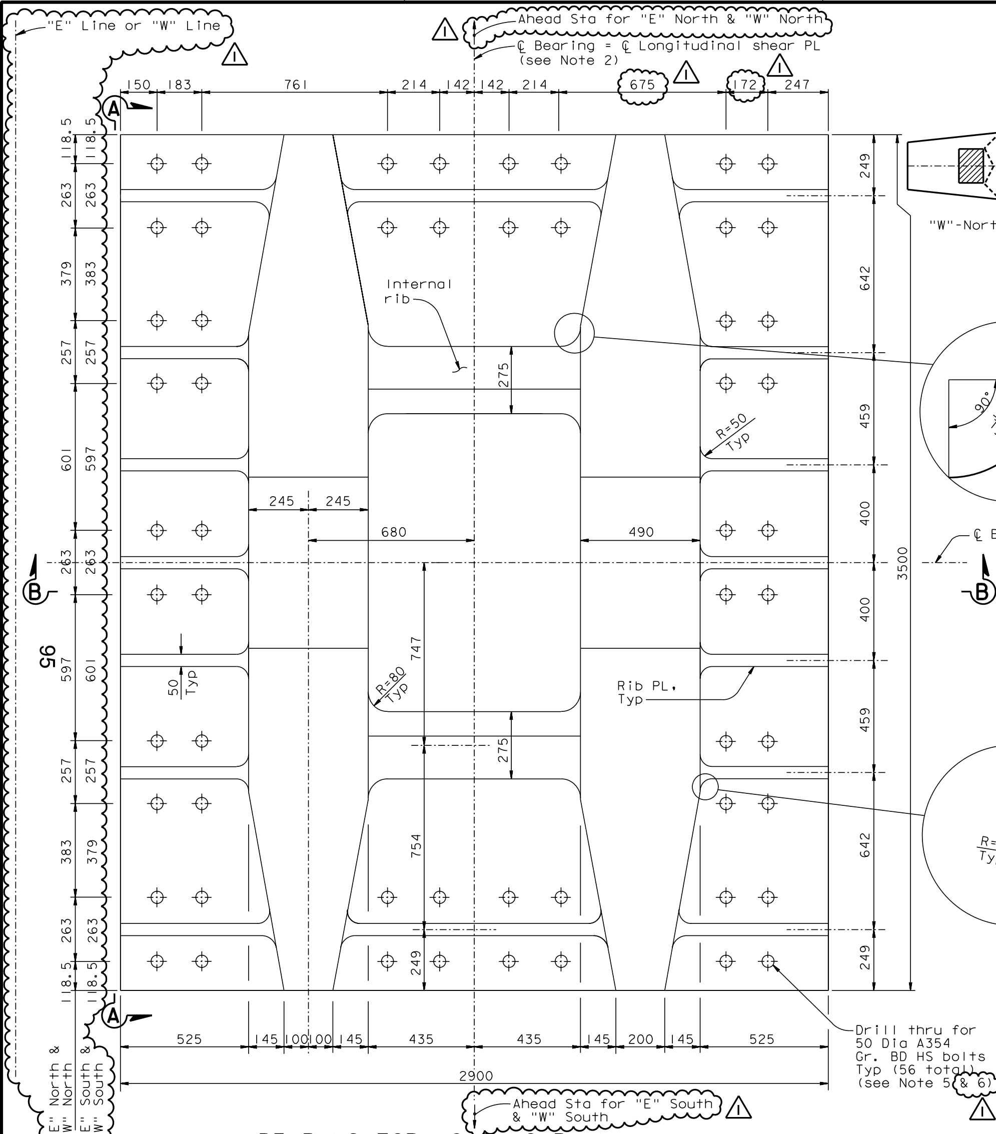
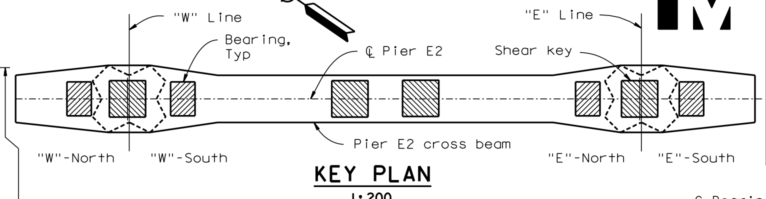
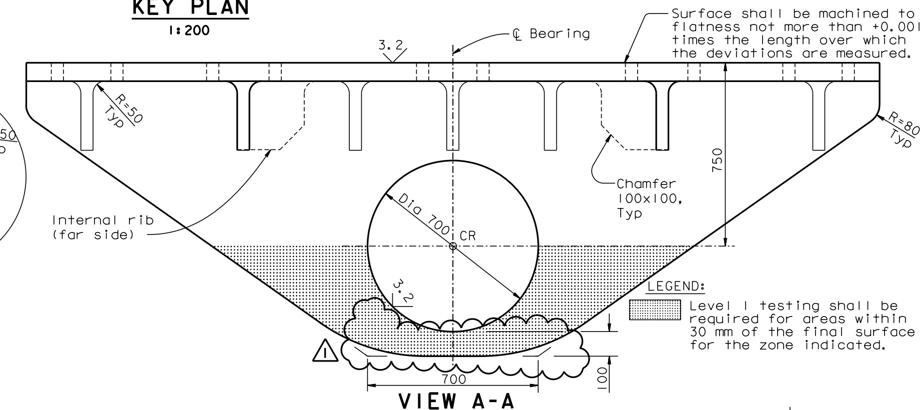


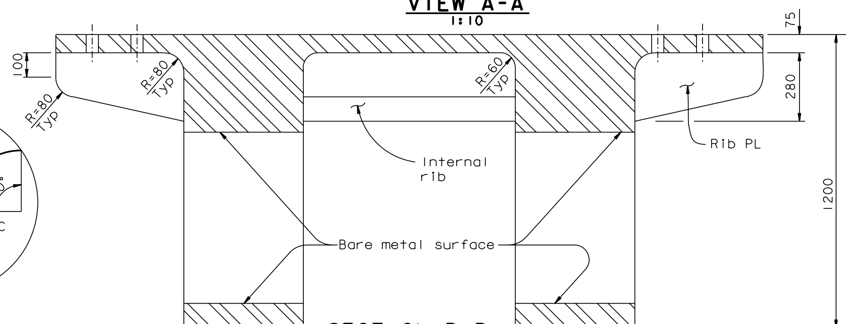
BEARING TOP HOUSING PLAN (Looking Up) ("E" - South Bearing shown. "W" - South similar. See Note 3 and 4)
1:10



KEY PLAN
1:200



VIEW A-A
1:10



SECTION B-B
1:10

NOTES:

1. Solid shaft in bushings is press fit.
2. For longitudinal shear plate location on box girder, see "Girder At East Transition No.1" sheet.
3. "E"-South bearing and "W"-South bearing are similar. "E"-North bearing and "W"-North bearing are similar.
4. Bolt hole location shall be cross checked with the box girder stiffener layout before drilling holes. Bolt edge distance and clearance for tightening shall be verified and submitted for review and approval by the Engineer. A template shall be used to drill bolt holes in bearing top housing and girder key plate to ensure hole matching.
5. The anchor bolt shall be pretensioned to 70% of ultimate tensile strength.
6. At the Contractor's option, oversized holes and appropriate washer plates may be used in the bearing top housing.

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REQUESTS FOR INFORMATION NOT ADDRESSED IN THIS CCO REMAIN IN FORCE

MARK	DATE	DESCRIPTIONS	BY	CH'D	CCO#
1	09/26/08	E2 CROSS BEAM	MN	NV	71

REVISIONS

R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 09/26/08

DESIGN BY M. Nader — CHECKED L. Rus
DETAILS BY N. Vo — CHECKED J. Leventini
QUANTITIES BY N. Vo — CHECKED J. Leventini

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE (SUPERSTRUCTURE & TOWER)

PIER E2 BEARING DETAILS NO. 3

CONTRACT CHANGE ORDER NO. ____
SHEET ____ OF ____

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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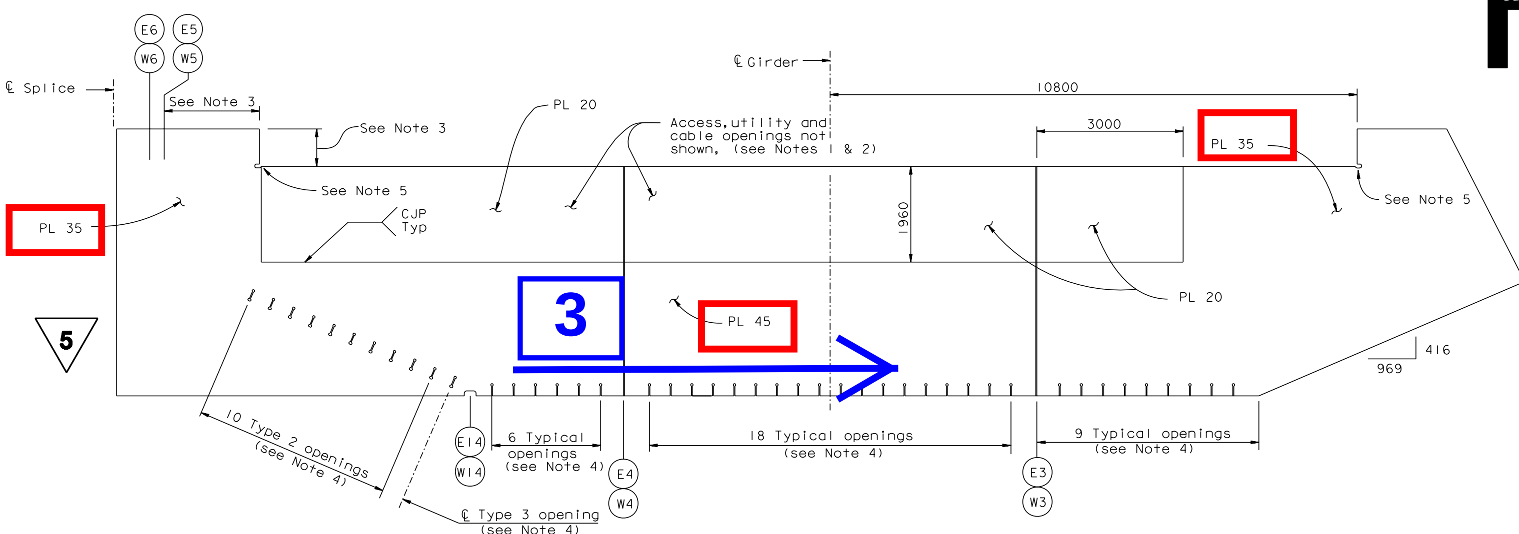
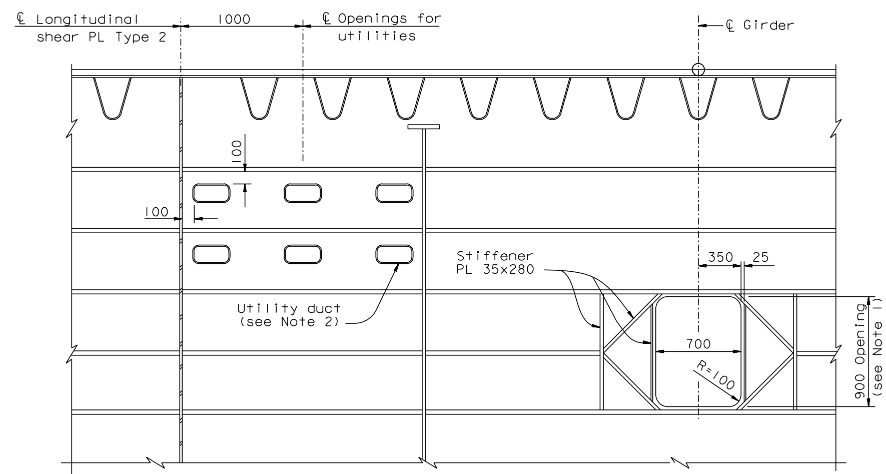
Floorbeam Web - Shear Check



AT CROSSBEAM
FLOORBEAM WEB LAYOUT
1:50



DETAIL C
1:20

NOTES:

1. For opening details not shown, see "Typical Girder Details No.7" sheets.
2. For utility details not shown, see "Utility Details" sheets.
3. For dimensions not shown, see "Partial Typical Sections", "Typical Girder Details" and "Girder At Pier E2 No.1" sheets.
4. For stiffener cutouts, see "Typical Girder Details No.3", "Typical Girder Details No.5" and "Typical Girder Details No.6" sheets.
5. For cutout detail, see "Typical Floorbeam Details No.3" sheet.

5 REVISED PER ADDENDUM NO. 5 DATED DECEMBER 21, 2005

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	677	1204

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12-6-04
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SIGN OFF DATE 12/07/05

DESIGN
DETAILS
QUANTITIES

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 260

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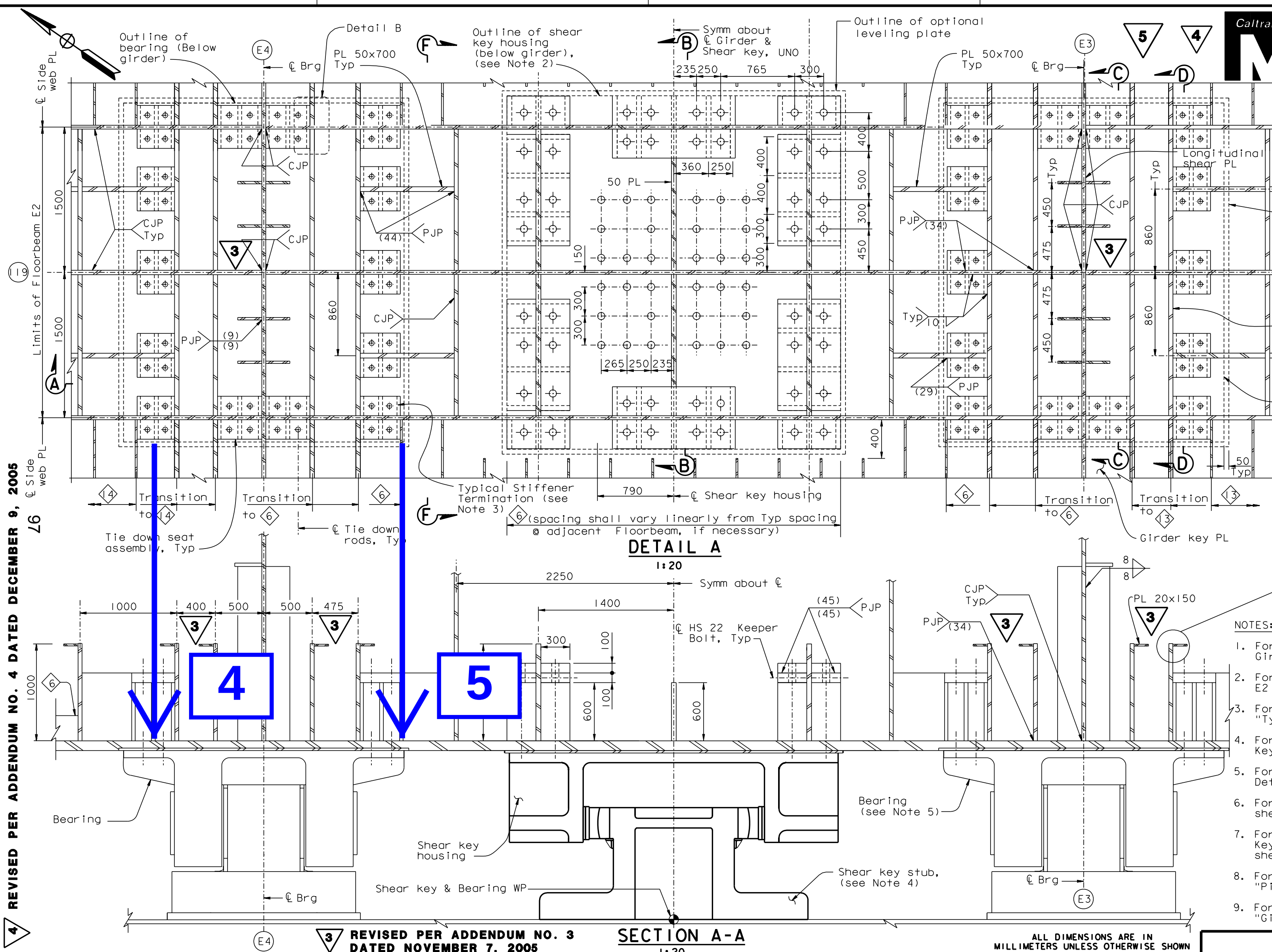

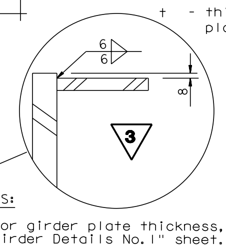

LEGEND

- ⌖ - 75 Dia anchor bolt (A354 Gr. BD)
- ⌖ - 64 Dia anchor bolt (A354 Gr. BD)
- ◆ - 29 Dia anchor bolt (A354 Gr. BD)
- ⌖ - 50 Dia anchor bolt (A354 Gr. BD)
- † - thickness of terminating plate in T-joint

NOTES:

1. For girder plate thickness, see "Typical Girder Details No.1" sheet.
2. For details of shear key housing, see "Pier E2 Shear Key" sheets.
3. For typical girder stiffener termination, see "Typical Girder Details No.16" sheet.
4. For details of shear key, see "Pier E2 Shear Key Details" sheets.
5. For details of bearing, see "Pier E2 Bearing Details" sheets.
6. For Section F-F, see "Girder At Pier E2 No.6" sheet.
7. For bolt and rod locations, see "Pier E2 Shear Key Details" and "Pier E2 Bearing Details" sheets.
8. For Pier E2 bearing and shear key location, see "Pier E2 Bearing And Shear Key Layout" sheet.
9. For Section B-B, C-C, D-D and detail B, see "Girder At Pier E2 No.3" sheet.

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REVISED PER ADDENDUM NO. 4 DATED DECEMBER 9, 2005

97

REVISED PER ADDENDUM NO. 3 DATED NOVEMBER 7, 2005

REVISED PER ADDENDUM NO. 5 DATED DECEMBER 21, 2005

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	675	1204

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12-6-04
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DESIGN OVERSIGHT
SIGN OFF DATE 12/07/05

DESIGN	BY G. Baker	CHECKED P. Ritchie
DETAILS	BY C. Mauch	CHECKED T. McMeans
QUANTITIES	BY D. Turner	CHECKED M. Roberts

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPPERSTRUCTURE & TOWER)

GIRDER AT PIER E2 NO. 2

SHEET 258

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 05/15/99 08/02/99 05/31/01 04/06/02 07/31/02 12/19/02 10/15/05 11/15/05 12/07/05

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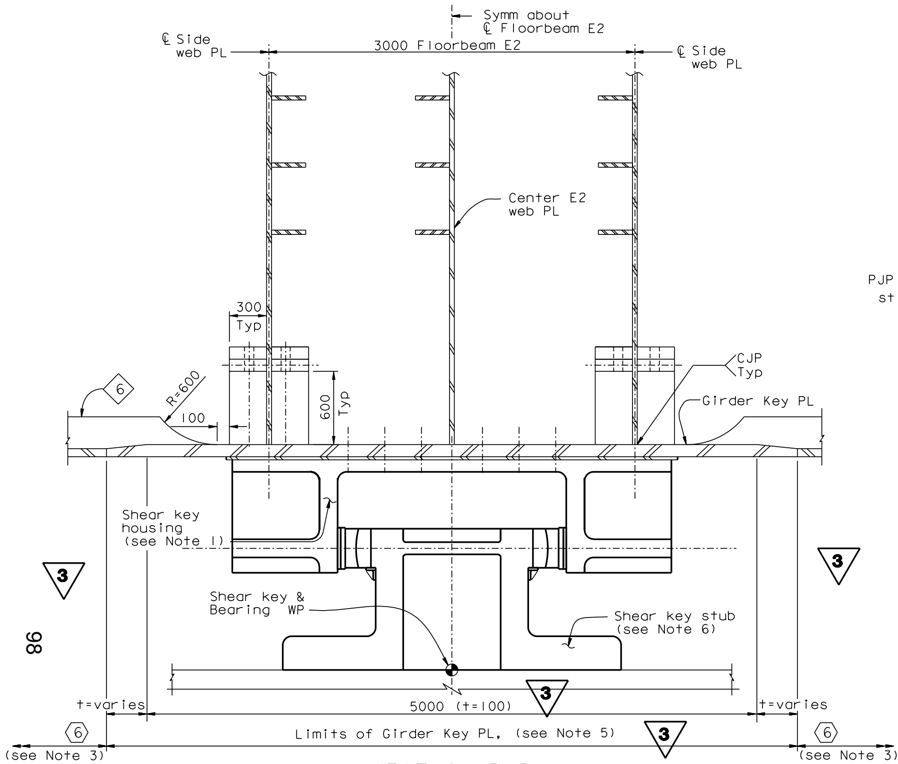
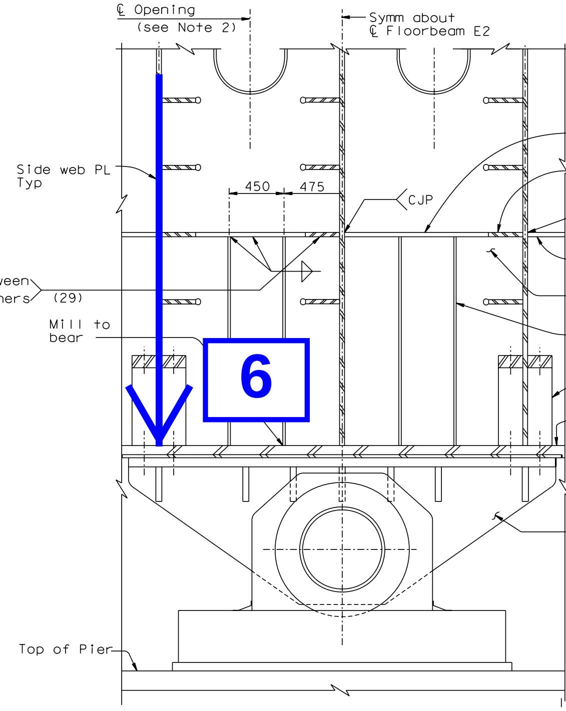
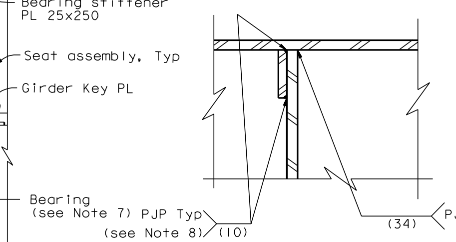


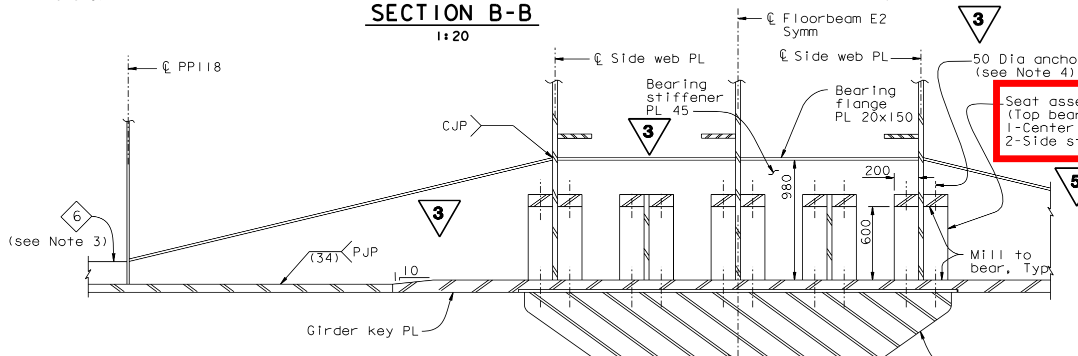
SECTION B-B
1:20



SECTION C-C
1:20



DETAIL B - ALTERNATIVE STIFFENER WELD
1:10



SECTION D-D
1:20

NOTES:

1. For shear key housing details, see "Pier E2 Shear Key Details" sheet.
2. For opening details, see "Typical Girder Details No. 7" sheet.
3. For typical girder details not shown, see "Typical Girder Details" sheets.
4. For anchor bolt, see "Pier E2 Bearing Details" sheets.
5. For transverse limits of girder key PL, see "Girder At Pier E2 No. 1" sheet.
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3 REVISED PER ADDENDUM NO. 3 DATED NOVEMBER 7, 2005

4 REVISED PER ADDENDUM NO. 4 DATED DECEMBER 9, 2005

5 REVISED PER ADDENDUM NO. 5 DATED DECEMBER 21, 2005

98

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DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	676	1204

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REGISTERED ENGINEER - CIVIL
12-6-04
PLANS APPROVAL DATE

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R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 12/07/05

DESIGN	BY G. Baker	CHECKED P. Ritchie
DETAILS	BY C. Mauch	CHECKED T. McMeans
QUANTITIES	BY D. Turner	CHECKED M. Roberts

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

GIRDER AT PIER E2 NO. 3

CU 04
EA 0120F1

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

DISREGARD PRINTS BEARING EARLIER REVISION DATES

SHEET 259 OF

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OBG Plates at Bearings

stresses neglect global OBG stresses

Formulas

		Reference	Comment
Steel Plate Properties			
Fy	345 MPa	Sheet 423	
E	200,000 MPa		
Force Applied to OBG per Bearing			
Transverse	30.5 MN	Sheet 882A	
Longitudinal	13.3 MN	Sheet 882A	
1) Key Plate			
Transverse dimension	2900 mm		
Longitudinal dimension	5000 mm	Sheet 676	
t	85 mm	Sheet 676	TC-RFI-34R1 allows key plate to be milled to 85mm
Axial Check			
1A) Longitudinal Axial Capacity	146.6 MN		
Longitudinal Axial Demand	13.3 MN		
D/C	**0.09**		
1B) Transverse Axial Capacity	85.0 MN		
Transverse Axial Demand	30.5 MN		
D/C	**0.36**		
2) Longitudinal Shear Plates			
t	25 mm	Sheet 691	
Longitudinal dimension	3500 mm	Sheet 883	
Shear Check			
Design Shear Capacity	17.5 MN		
Shear Demand	13.3 MN		
D/C	**0.76**		
3) Floorbeam Webs			
t	35 mm	Sheet 677	
Transverse dimension	2900 mm	Sheet 883	
no. of webs	3		
Shear Check			
Design Shear Capacity	60.9 MN		
Shear Demand	30.5 MN		
D/C	**0.50**		

Bearing to OBG Bolted Connection

Formulas

	Value	Unit	Reference		Comment
Steel Plate Properties					
Fy	345	MPa	Sheet 423		
E	200,000	MPa			
F_{exx}	483	MPa			
Anchor Bolt Properties					
Diameter	50	mm	Sheet 675		
Tensile Strength	375	kips	ASTM A354		
	1674	kN			
Prestress	1.0	Fu			
Pretension	1.67	MN			
4) Vertical Stiffeners - Middle					
D	550	mm	Sheet 676		50mm cope hole at bottom
b	200	mm	Sheet 676		
t	40	mm	Sheet 676		
Load on Stiffener	1.67	MN			Middle stiffener reaction = 2 rods x 50%
- Axial Check					
r	11.5	mm			
kL/r	47.6				
sqrt (2 x π^2 x E / Fy)	107.0				
F_{cr}	311	MPa	BDS 10-152		
Design Axial Strength	2.11	MN	BDS 10-150	**OK**	
D/C	0.79				
- Shear Check					
D/t_w	13.8				
6000 x sqrt(k) / sqrt F_y	323.0				
C	1.0				
Design Shear Strength	4.40	MN	BDS 10-115	**OK**	
D/C	0.38				
Weld Check					
PJP (E) Web to Stiff	34	mm			
Weld Factored Resistance	232	MPa			
Design Weld Strength	4.34	MN		**OK**	
D/C	0.39				
- Bearing Check					
Allowable	466	MPa	BDS 10.48.7	$1.35F_y$	
Bearing Strength	3.73	MN		**OK**	
D/C	0.45				

5) Vertical Stiffeners - Side

D	550	mm	Sheet 676	50mm cope hole at bottom
b	200	mm	Sheet 676	
t	35	mm	Sheet 676	
Load on Stiffener	**0.84**	MN		Side stiffener reaction = 1 rod x 50%
- Axial Check				
r	10.1	mm		
kL/r	54.4			
sqrt (2 x π^2 x E / Fy)	107.0			
F_{cr}	311	MPa	BDS 10-152	
Design Axial Strength	**1.85**	MN	BDS 10-150	**OK**
D/C	**0.45**			
- Shear Check				
D/t_w	15.7			
6000 x sqrt(k) / sqrt F_y	323.0			
C	1.0			
Design Shear Strength	**3.85**	MN	BDS 10-115	**OK**
D/C	**0.22**			
Weld Check				
PJP (E) Web to Stiff	29	mm		
Weld Factored Resistance	232	MPa		
Design Weld Strength	**3.70**	MN		**OK**
D/C	**0.23**			
- Bearing Check				
Allowable	466	MPa	BDS 10.48.7	$1.35F_y$
Bearing Strength	**3.26**	MN		**OK**
	0.26			

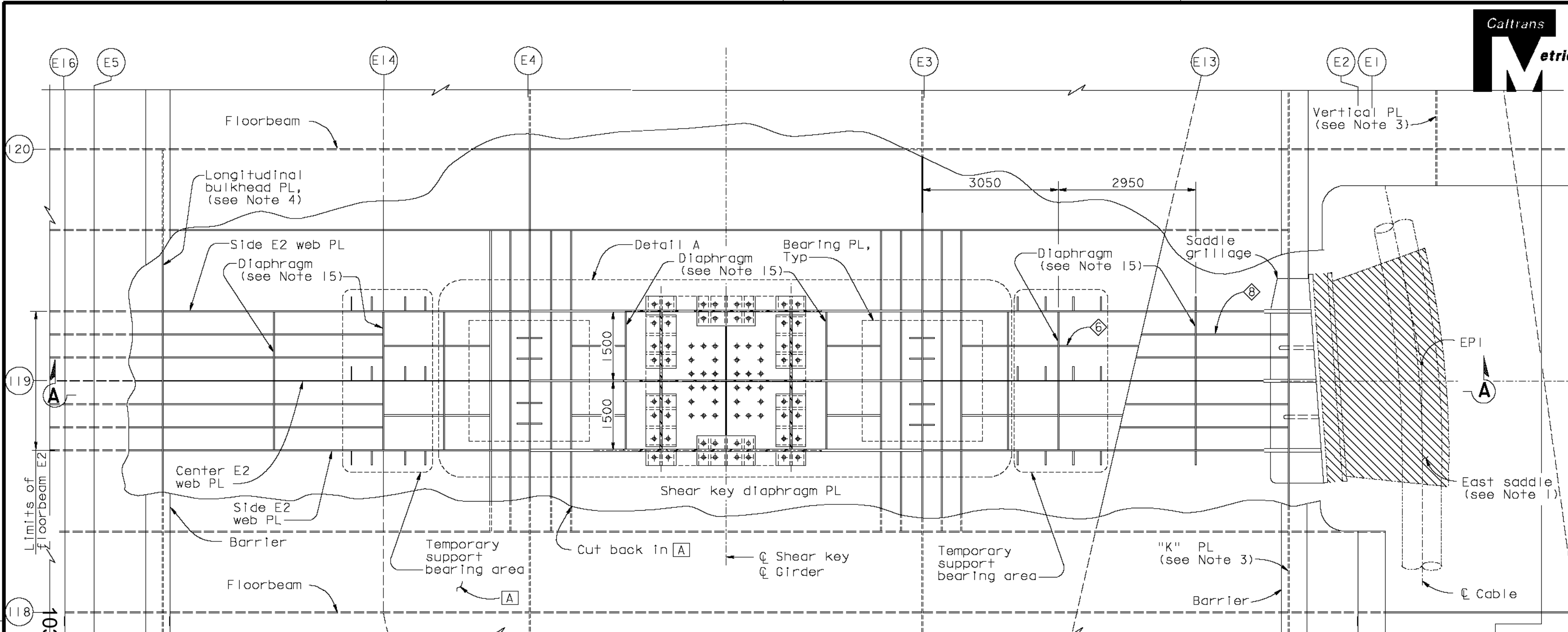
6) Floorbeam Web

D	550	mm	Sheet 676	
t	35	mm	Sheet 677	
Load on Web	**3.35**	MN		50% reaction from 4 rods
Shear Tearout of Web				
Shear Capacity of Web	**3.85**	MN		**OK**
D/C	**0.87**			

Appendix 3C

Design Drawings

Orthotropic Box Girder at Pier E2, Crossbeam at Pier E2, Bearing and Shear Key Details

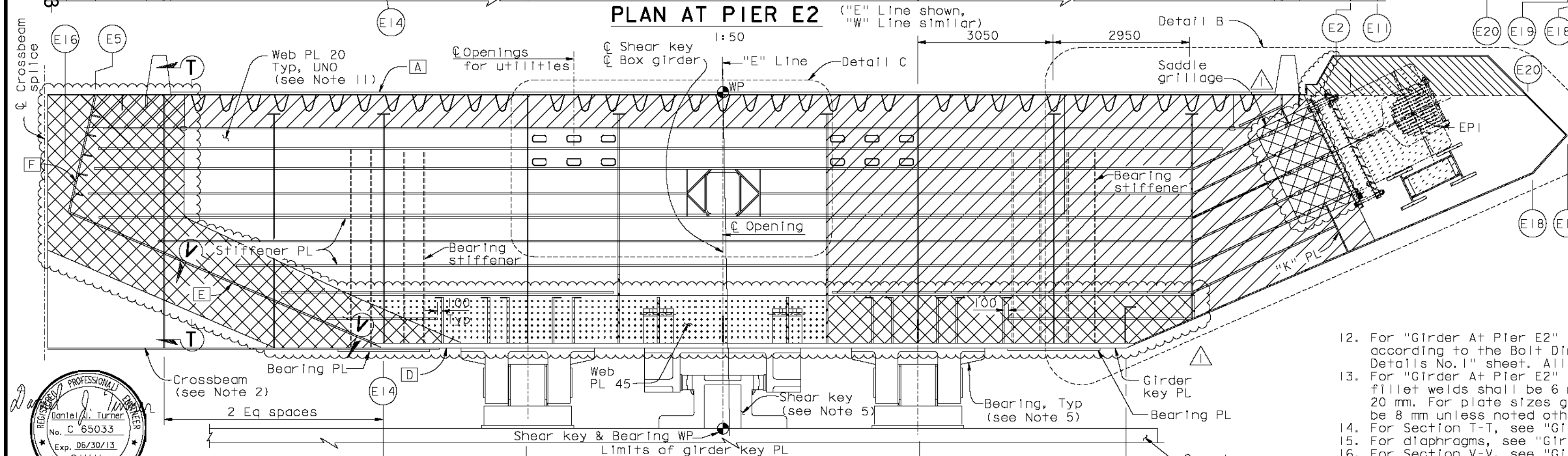


PLAN AT PIER E2

1:50

("E" Line shown, "W" Line similar)

103



SECTION A-A

1:50

(Center web PL shown, side web PL's similar)

LEGEND:

- East saddle (see Note 1)
- Saddle housing (see Note 7)
- "Indicates SPCM web PL"
- "Indicates TTP web PL"
- "Indicates SPCM & TTP web PL"

NOTES:

1. For saddle details, see "East Saddle Details" sheets.
2. For crossbeam, see "Crossbeam At Pier E2" sheets.
3. For "K" PL and vertical PL details, see "Girder At East Transition Details" sheets.
4. For longitudinal bulkhead PL details, see "Girder At Pier E2 No.9" sheet.
5. For shear key & bearing details, see "Pier E2 Shear Key Details" and "Pier E2 Bearing Details" sheets.
6. For clarity, typical girder stiffeners are not shown in plan.
7. For saddle housing details, see "Saddle Housing Details" sheets.
8. For Detail A, see "Girder At Pier E2 No.2" sheet.
9. For Detail B, see "Girder At Pier E2 No.5" sheet.
10. For Detail C, see "Girder At Pier E2 No.4" sheet.
11. For E2 floorbeam web layout, see "Girder At Pier E2 No.4" sheet.
12. For "Girder At Pier E2" sheets, all bolts shall be detailed according to the Bolt Dimension Table. See "Typical Girder Details No.1" sheet. All bolts are HS unless noted otherwise.
13. For "Girder At Pier E2" sheets, unless noted otherwise, all fillet welds shall be 6 mm for plate sizes not greater than 20 mm. For plate sizes greater than 20 mm, fillet welds shall be 8 mm unless noted otherwise.
14. For Section T-T, see "Girder At Pier E2 No.9" sheet.
15. For diaphragms, see "Girder At Pier E2 No.6" sheet.
16. For Section V-V, see "Girder at Pier E2 No.9" sheet.

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(SUPERSTRUCTURE & TOWER)

GIRDER AT PIER E2 NO. 1

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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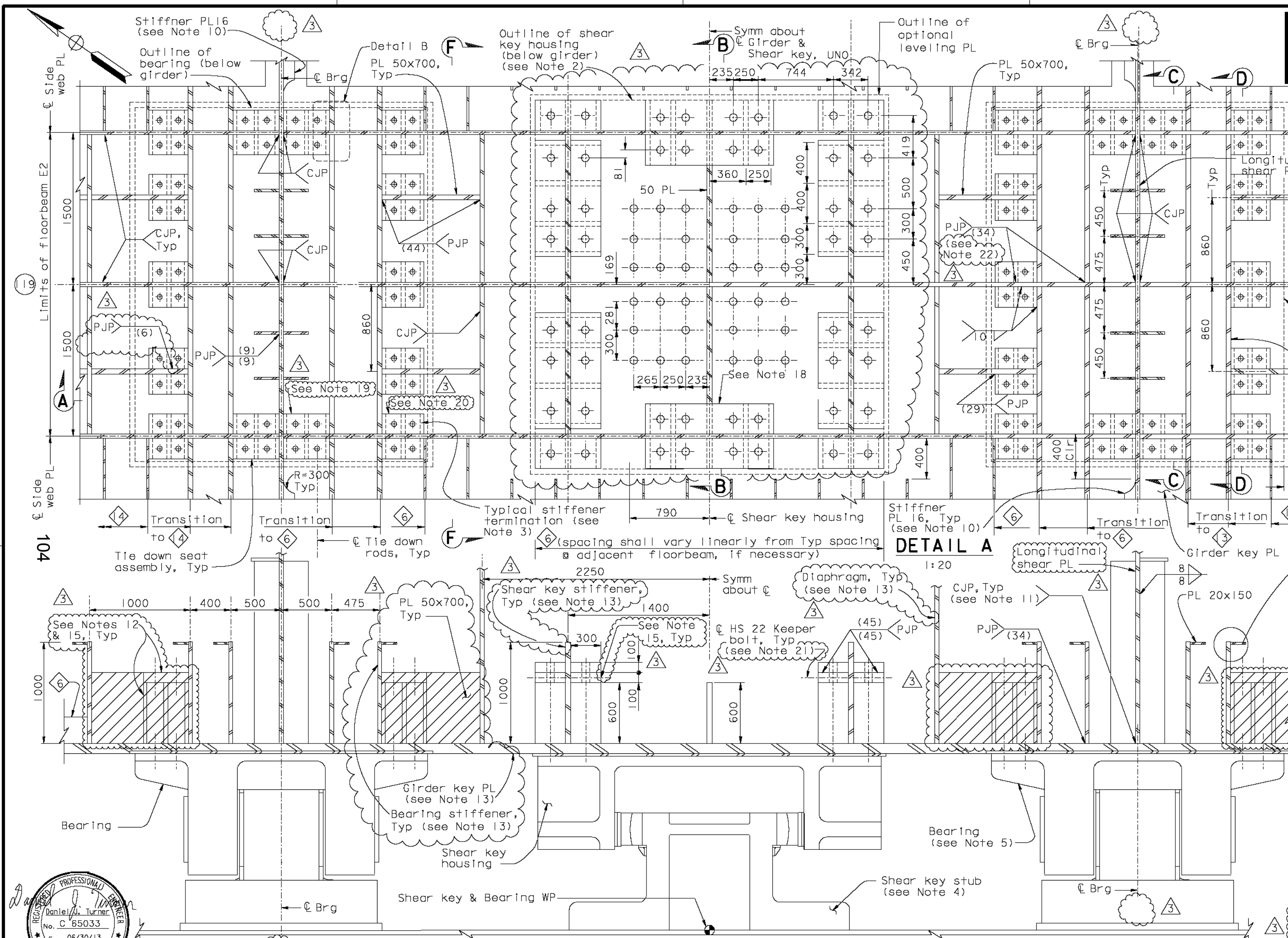

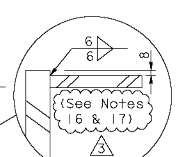

LEGEND

- ⌖ - 75 Dia anchor bolt (A354 Gr. BD) (see Note 15)
- ⌖ - 50 Dia anchor bolt (A354 Gr. BD) (see Note 15)
- + - Thickness of terminating PL in T-joint
- "Indicates TTP Web PL"

NOTES:

1. For girder PL thickness, see "Typical Girder Details No.1" sheet.
2. For details of shear key housing, see "Pier E2 Shear Key" sheets.
3. For typical girder stiffener termination, see "Typical Girder Details No.16" sheet.
4. For details of shear key, see "Pier E2 Shear Key Details" sheets.
5. For details of bearing, see "Pier E2 Bearing Details" sheets.
6. For Section F-F, see "Girder At Pier E2 No.6" sheet.
7. For bolt and rod locations, see "Pier E2 Shear Key Details" and "Pier E2 Bearing Details" sheets.
8. For Pier E2 bearing and shear key location, see "Pier E2 Bearing And Shear Key Layout" sheet.
9. For Sections B-B, C-C, D-D and Detail B, see "Girder At Pier E2 No.3" sheet.
10. For termination of stiffener PL 16 at E2 floorbeam, see Detail G on "Typical Floorbeam Details No.4" sheet. For other stiffener details, see "Typical Girder Details No.7" sheet.
11. For extent of CJP weld, see Note 12 on "Typical Girder Details No.7" sheet.
12. TTP not required for seat assembly, Typ.
13. For other TTP plates, see " General Notes No.4" sheet.
14. For Notes 15 and above, see "Girder at Pier E2 No. 2A" sheet.

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1	07/30/07	HINGE A REACTION-SHEAR PLATES	DT	GB	50

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GIRDER AT PIER E2 NO. 2

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EA 0120F1

SHEET 258R3

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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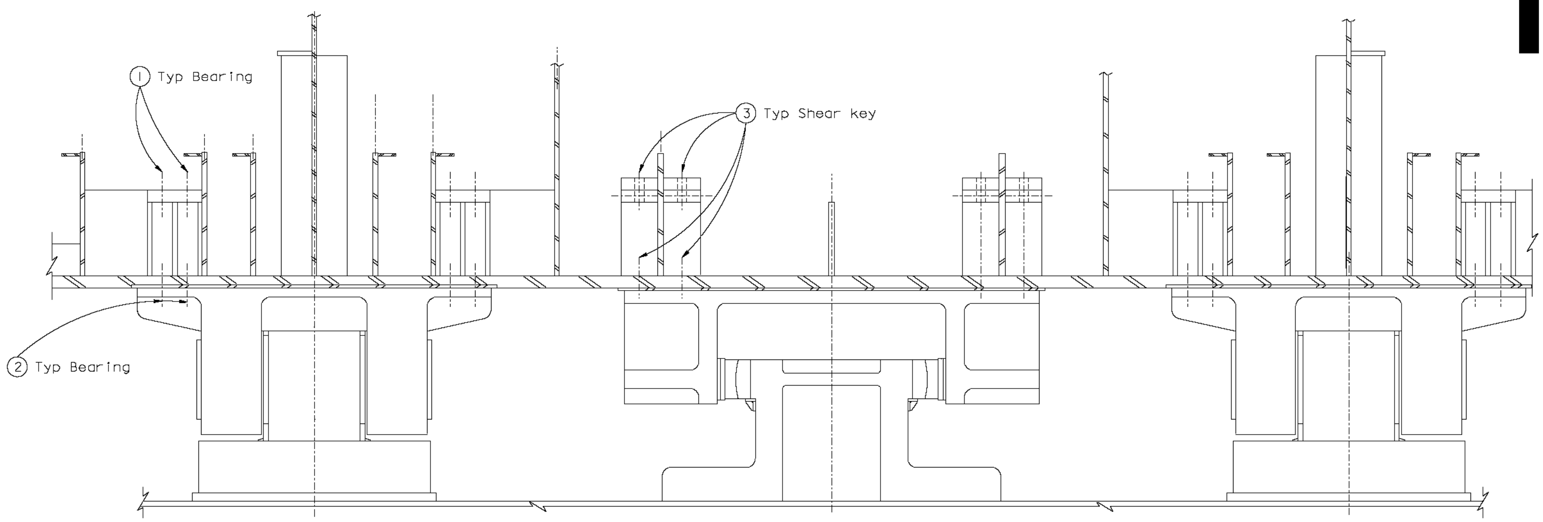

SECTION A-A (See Note 15)

1:20

NOTES:

15. At the Contractor's option, oversize holes may be used for the anchor bolt and keeper bolt in the top anchor plates of the seat assemblies, and in the key PL.

16. At the Contractor's option, an equivalent PJP may be used in lieu of a double sided fillet weld.

17. At the Contractor's option, the flange may be placed on the opposite side of the bearing stiffener.

18. At the Contractor's option, the top bearing PL may be detailed to overhang the side stiffener plates by 5 mm, subject to review and approval of the Engineer.

19. At the Contractor's option, the top bearing PL may be detailed to provide 5 mm gap at the floorbeams and abutting longitudinal members.

20. At the Contractor's option, the top bearing PL may be detailed to provide 5 mm gap at the abutting longitudinal members.

21. At the Contractor's option, keeper bolts may be placed in the upper top bearing plate. Oversized holes may be used in the bearing plates and vertical plates at keeper bolt locations, subject to review and approval of the Engineer.

22. At the Contractor's option, double-sided 24 mm fillet may be used on selected stiffener to floorbeam welds, subject to review and approval of the Engineer.

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GIRDER AT PIER E2 NO. 2A

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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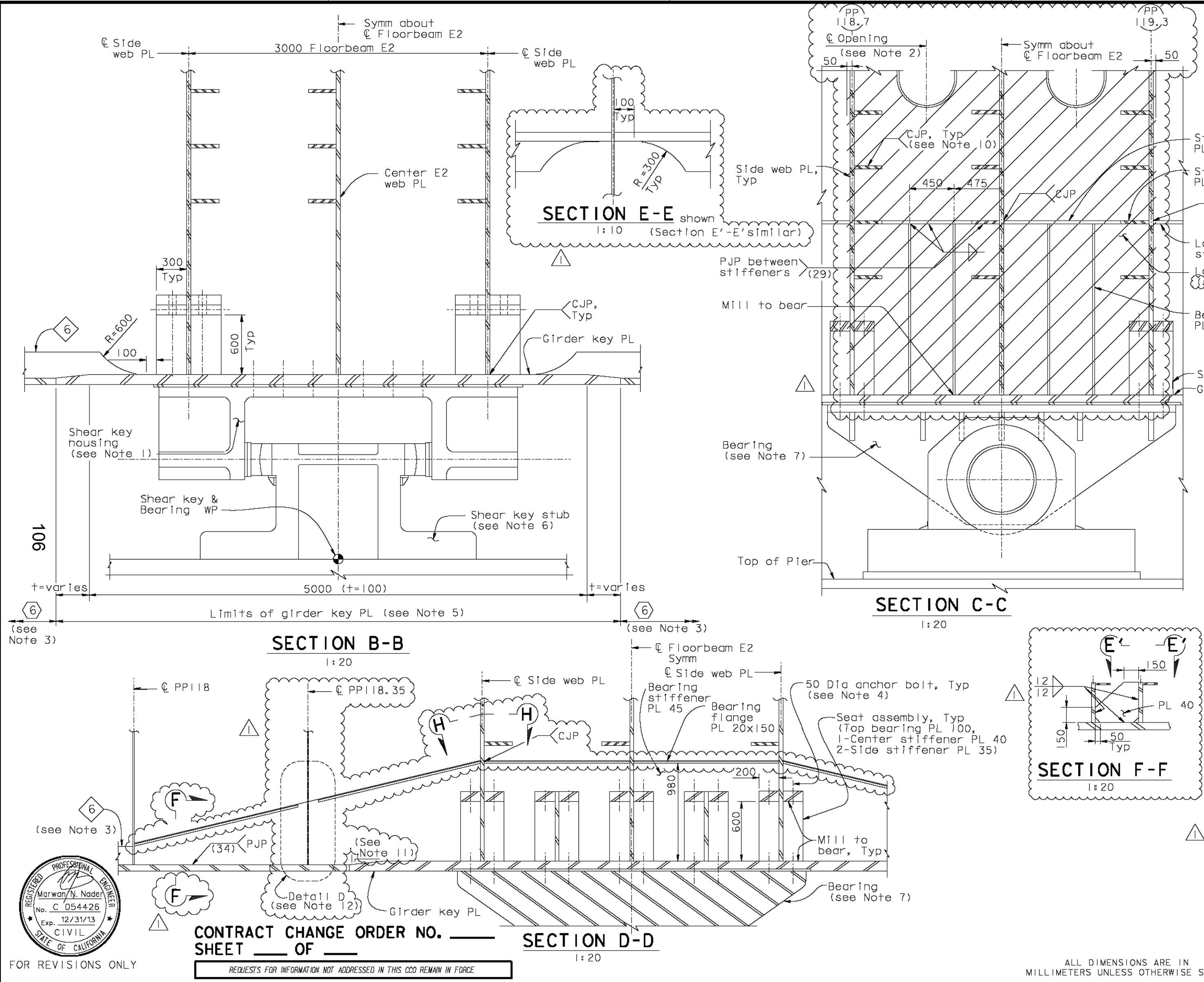

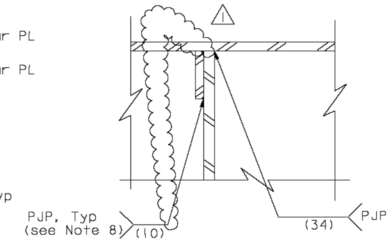

DETAIL B - ALTERNATIVE STIFFENER WELD

1:10

LEGEND:

"Indicates TTP Web PL"

NOTES:

1. For shear key housing details, see "Pier E2 Shear Key Details" sheet.
2. For opening details, see "Typical Girder Details No. 7" sheet.
3. For typical girder details not shown, see "Typical Girder Details" sheets.
4. For anchor bolt, see "Pier E2 Bearing Details" sheets.
5. For transverse limits of girder key PL, see "Girder At Pier E2 No. 1" sheet.
6. For shear key details, see "Pier E2 Shear Key Details" sheets.
7. For Pier E2 bearing details, see "Pier E2 Bearing Details" sheets.
8. For bearing stiffeners in faying contact with diaphragms or webs, stiffener corners and welds may be ground to allow fit-up.
9. Leveling PL may be used to compensate for the permissible out-of-flatness of the girder key plate. Leveling plates shall be tapered as required to ensure proper alignment of the bearings and shear keys in both longitudinal and transverse directions.
10. Stiffeners welded to the longitudinal shear PL shall be coped. See stiffener type 9 in similar Section D-D on "Typical Crossbeam Details No. 6" sheet.
11. At the Contractor's option and subject to review and approval of the Engineer, a 1:2.5 taper may be used in lieu of the 1:10 taper. The overall width of the girder key PL shall remain 5800 mm.
12. For Detail D and Section H-H, see "Girder At Pier E2 No. 3A" sheet.
13. The longitudinal shear PL shall have Through Thickness Properties (TTP).

106

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EAST SPAN SEISMIC SAFETY PROJECT
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GIRDER AT PIER E2 NO. 3

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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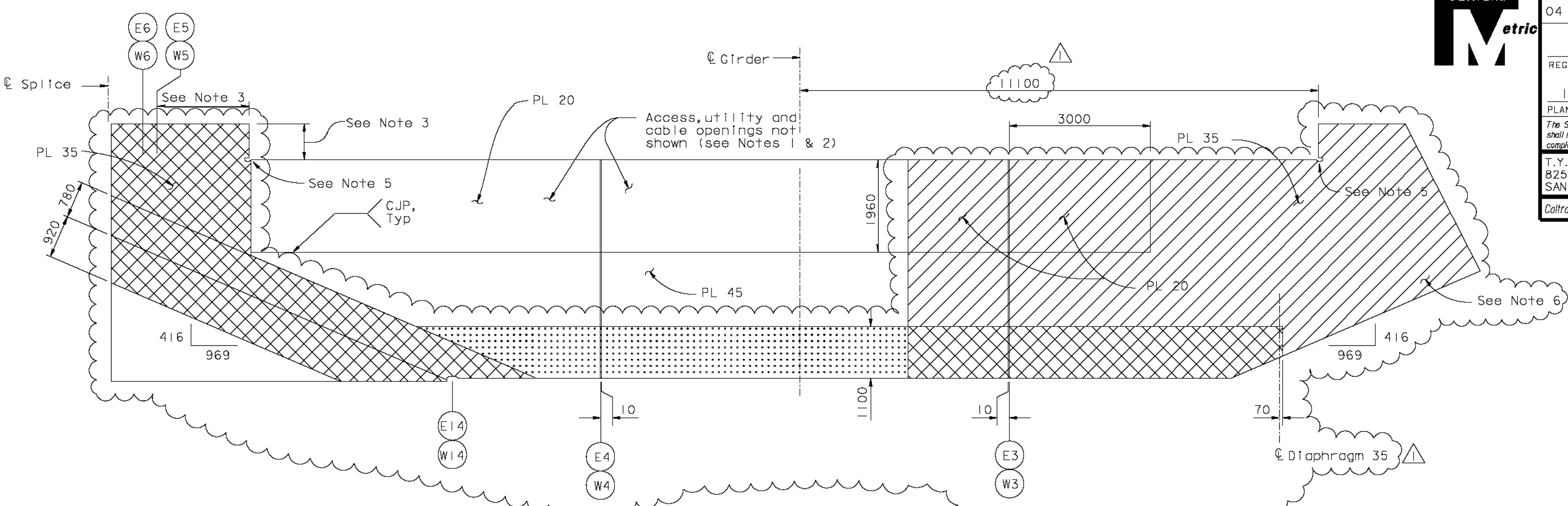

AT CROSSBEAM
FLOORBEAM WEB LAYOUT
(PP 118.7, 119.0, 119.3)
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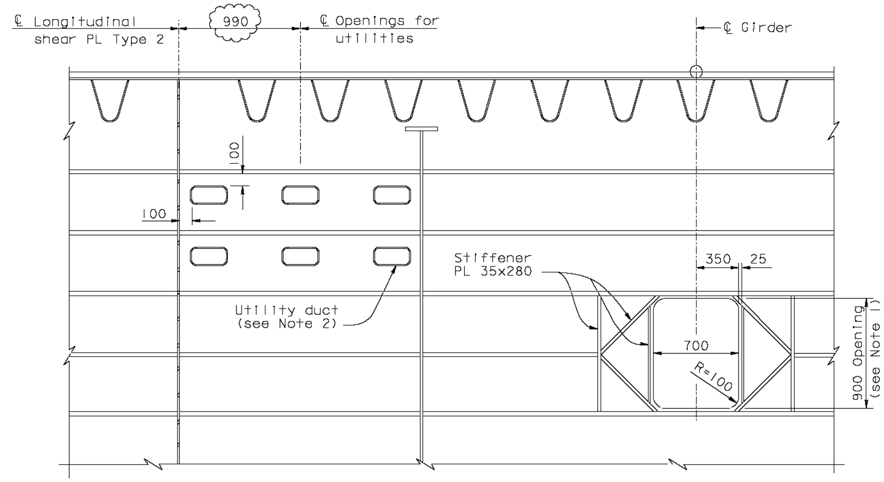

DETAIL C
1:20

LEGEND:

- "Indicates SPCM web PL"
- "Indicates TTP web PL"
- "Indicates SPCM & TTP web PL"

NOTES:

1. For opening details not shown, see "Typical Girder Details No.7" sheets.
2. For utility details not shown, see "Utility Details" sheets.
3. For dimensions not shown, see "Partial Typical Sections", "Typical Girder Details" and "Girder At Pier E2 No.1" sheets.
4. For stiffener cutouts, see "Typical Girder Details No.3", "Typical Girder Details No.5" and "Typical Girder Details No.6" sheets.
5. For cutout detail, see "Typical Floorbeam Details No.3" sheet.
6. For floorbeam web cutout at East Saddle Grillage, see "Girder At Pier E2 No.5" sheet.

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GIRDER AT PIER E2 NO.4

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REVISION DATES (PRELIMINARY STAGE ONLY) 5/15/98 08/02/99 05/01/01 04/08/02 07/01/02 12/17/02 12/01/05
SHEET 260R1 OF

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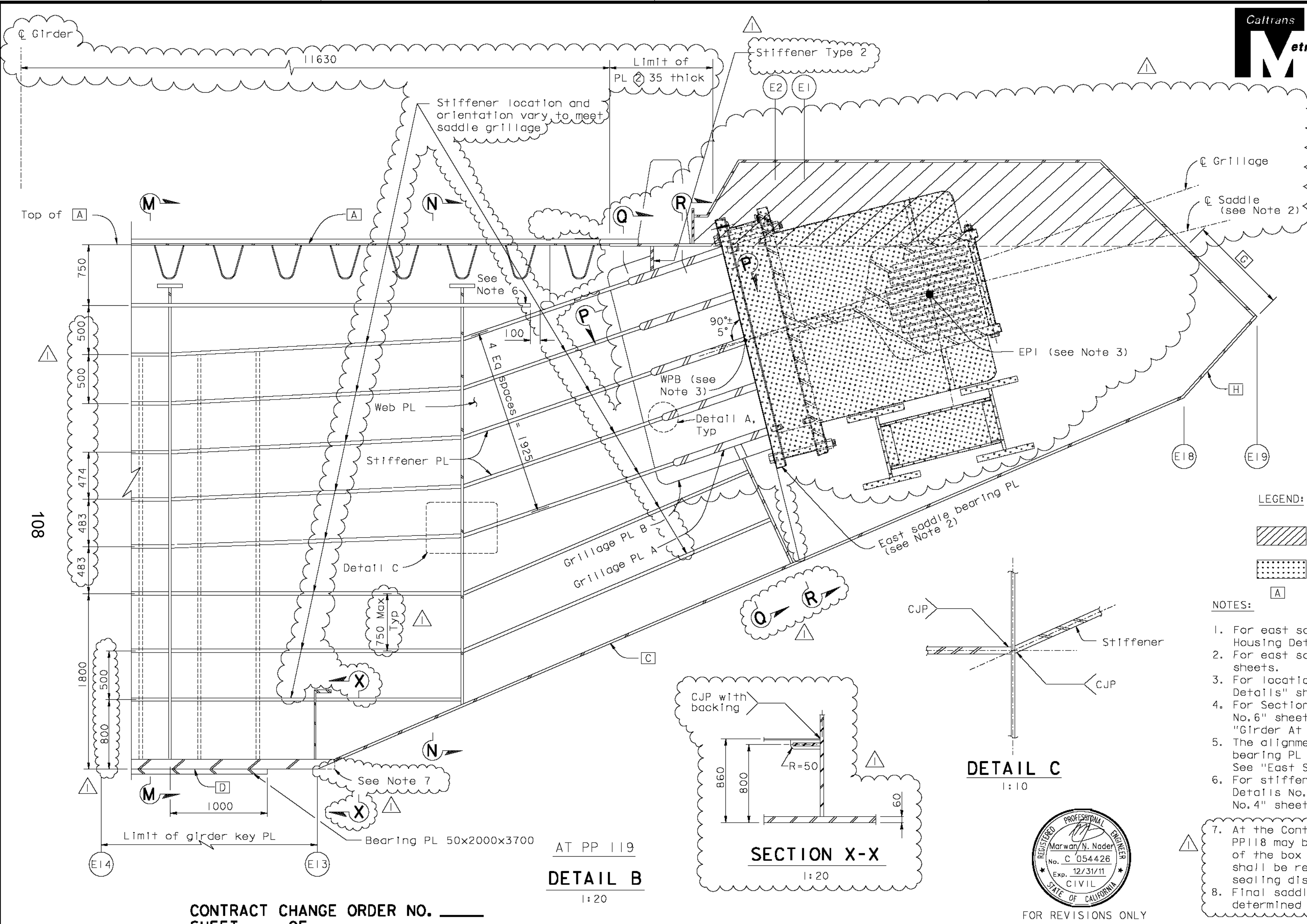

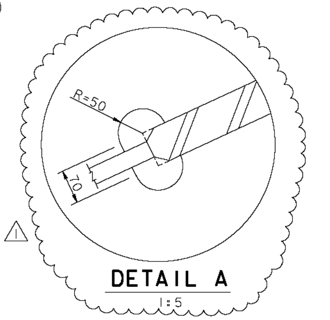

LEGEND:

Limits of east saddle housing (see Note 1)

East saddle (see Note 2)

A Girder PL A

NOTES:

1. For east saddle housing details, see "East Saddle Housing Details" sheets.
2. For east saddle details, see "East Saddle Details" sheets.
3. For locations of EP1 and WPB, see "East Saddle Details" sheets.
4. For Sections M-M and N-N, see "Girder At Pier E2 No.6" sheet. For Sections P-P, Q-Q and R-R, see "Girder At Pier E2 No.7" sheet.
5. The alignment of the saddle grillage and saddle bearing PL is related to the saddle layout. See "East Saddle Details" sheets.
6. For stiffener terminations, see "Typical Floorbeam Details No.3" and "Typical Floorbeam Details No.4" sheets.
7. At the Contractor's option, the bearing PL 50 at PP118 may be trimmed flush with the bottom edge of the box along line W13. The bearing PL bolts shall be relocated so as to maintain minimum sealing distance along the bearing PL edge.
8. Final saddle grillage orientation shall be determined from final cable geometry.

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12-6-04
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PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
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SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

GIRDER AT PIER E2 NO.5

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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EA 0120F1

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

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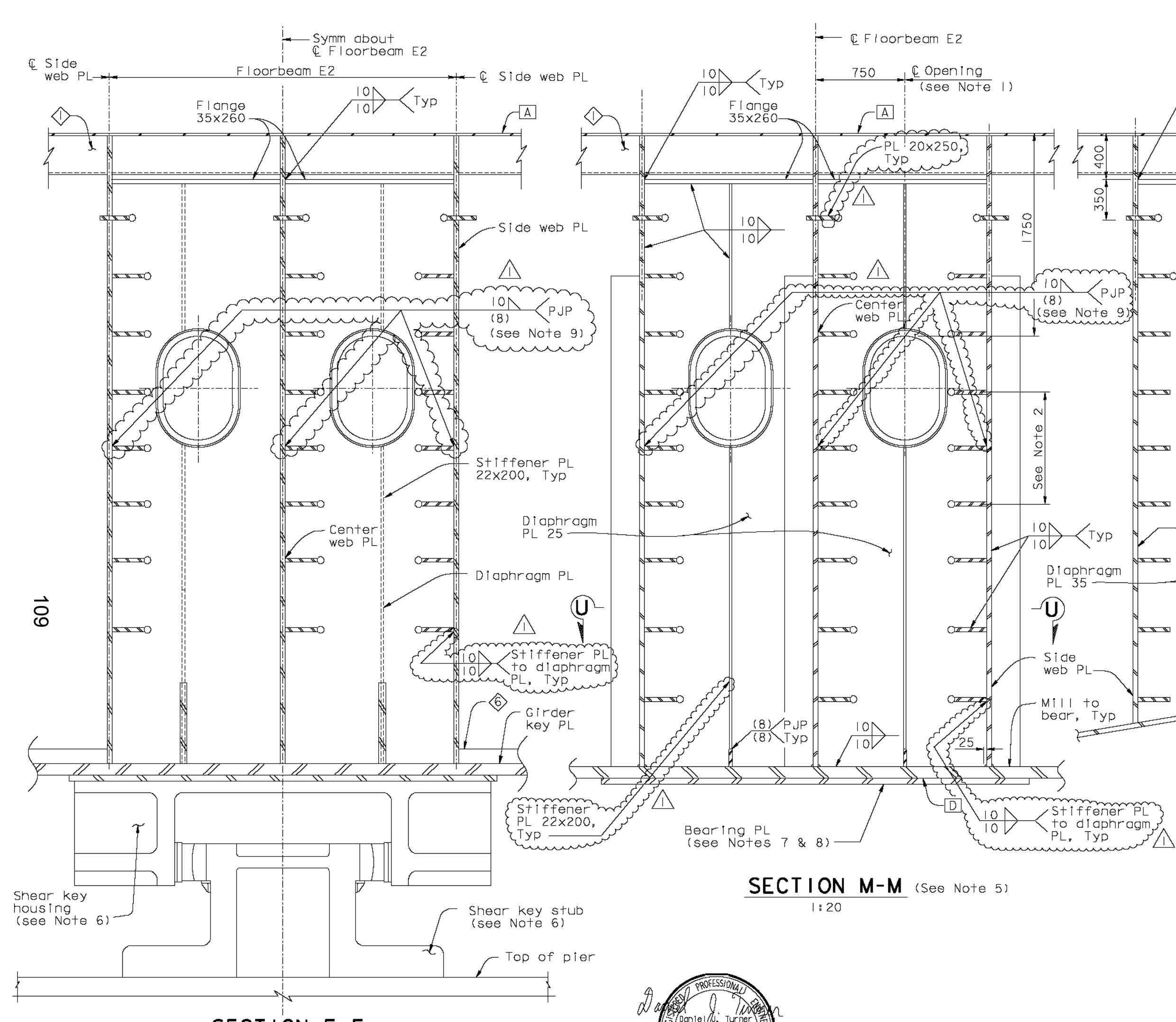
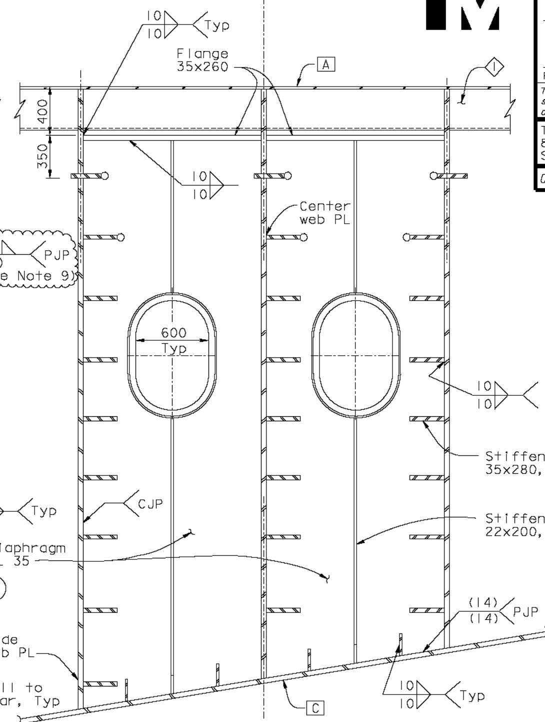


SECTION F-F

1:20

SECTION M-M (See Note 5)

1:20



SECTION N-N

1:20

NOTES:

1. For typical opening detail, see "Typical Girder Details No. 7" sheet.
2. For stiffener penetration details, see "Typical Girder Details No. 5" sheet and "Girder At Pier E2, No. 6A" sheet.
3. Access walkways not shown. For access details, see "Girder Access Details" sheets.
4. For Section U-U, see "Girder At Pier E2 No. 8" sheet.
5. For typical diaphragm details, Section M-M shall apply.
6. For shear key stub and shear key housing details, see "Pier E2 Shear Key Details" sheets.
7. For plan of bearing PL, see "Crossbeam at Pier E2 No. 7" sheet.
8. Bearing PL is counterbored to accomodate bolt heads. See Detail J on "Crossbeam At Pier E2 No. 3" sheet.
9. At web PL taper, provide (8) PJP weld in lieu of fillet weld.

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PROJECT ENGINEER

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SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

GIRDER AT PIER E2 NO. 6

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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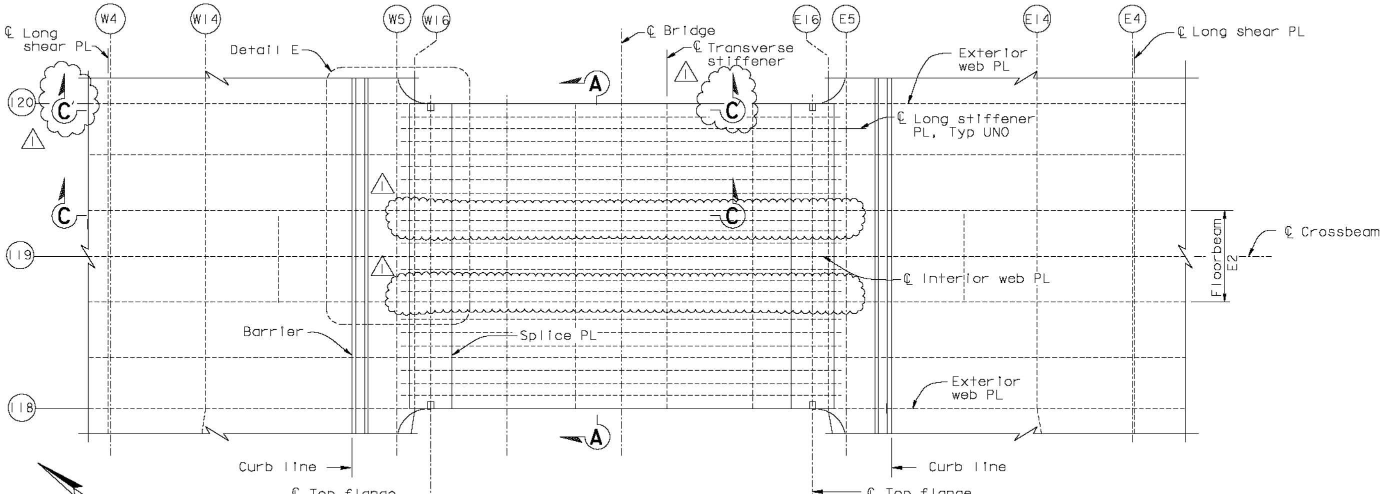
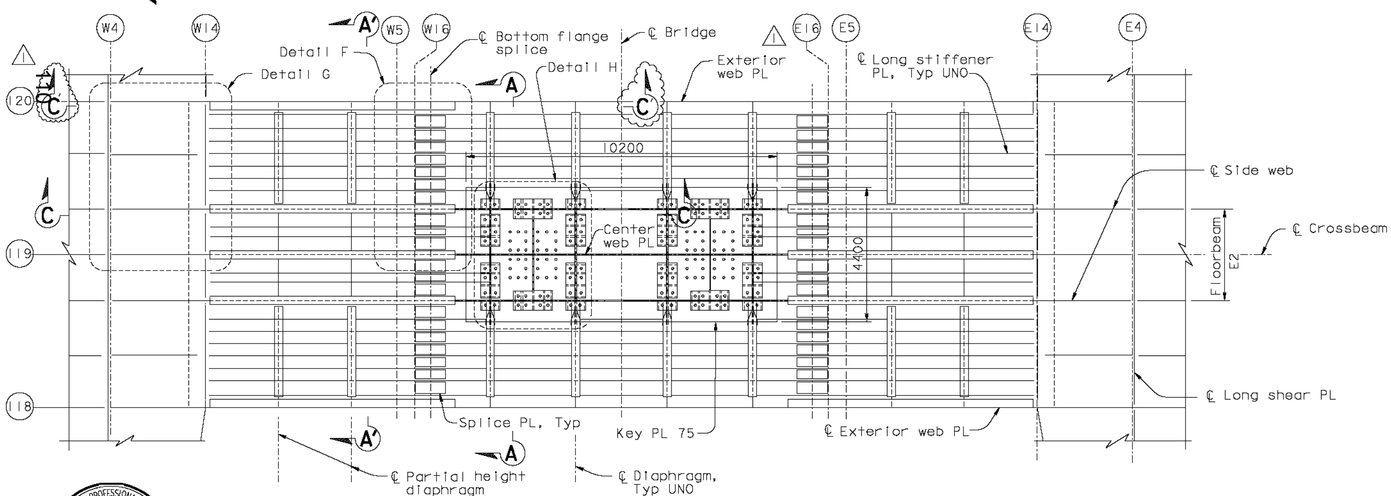


TOP PLAN OF CROSSBEAM

1:80



BOTTOM PLAN OF CROSSBEAM

1:80

NOTES:

1. For details not shown, see "Typical Crossbeam Details" sheets.
2. For Section C-C, see "Crossbeam At Pier E2 No.6" sheet. For Section C'-C', see "Crossbeam At Pier E2 No.6A" sheet.
3. For locations of W14, W16, E14 & E16, see "Girder At East Transition No. 1" and "Girder At East Transition No.3" sheets.
4. For all "Crossbeam At Pier E2" sheets, all bolts shall be detailed according to the Bolt Dimension Table. See "Typical Girder Details No.1" sheet. All bolts are HS unless noted otherwise.
5. For "Crossbeam At Pier E2" sheets unless noted otherwise, all fillet welds shall be 6 mm for plate sizes not greater than 20 mm. For plates greater than 20 mm, fillet welds shall be 8 mm unless noted otherwise.
6. For Detail E, see "Crossbeam At Pier E2 No.4" sheet.
7. For Detail F, see "Crossbeam At Pier E2 No.5" sheet.
8. For Detail G, see "Crossbeam At Pier E2 No.7" sheet.
9. For Sections A-A and A'-A', see "Crossbeam At Pier E2 No.2" sheet.

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PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CROSSBEAM AT PIER E2 NO. 1

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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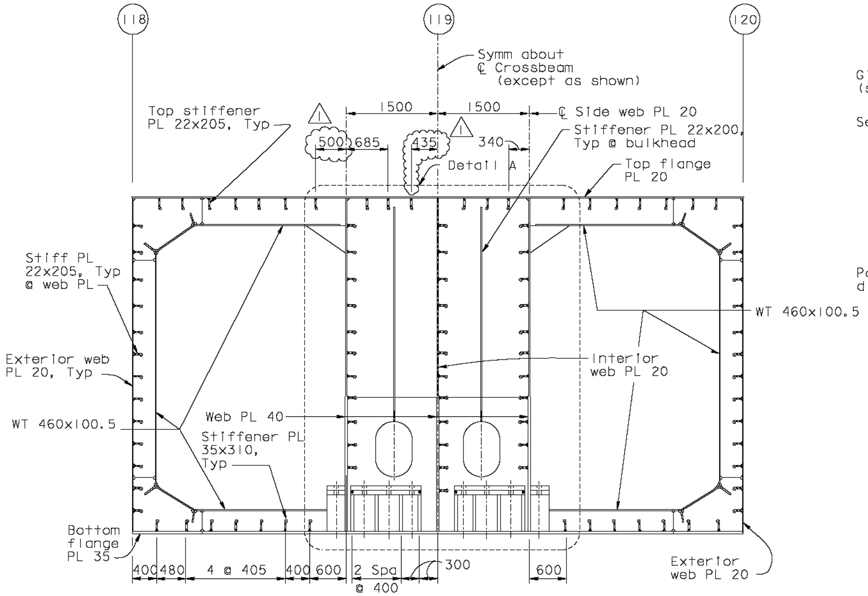
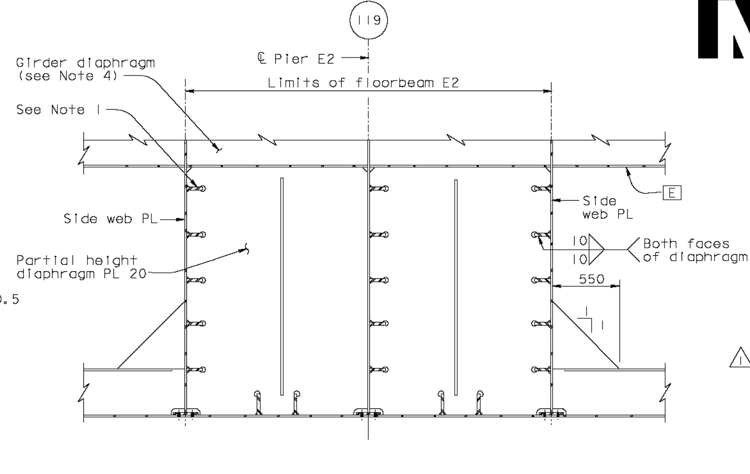


TYPICAL DIAPHRAGM
SECTION A-A
1:40



DETAIL B
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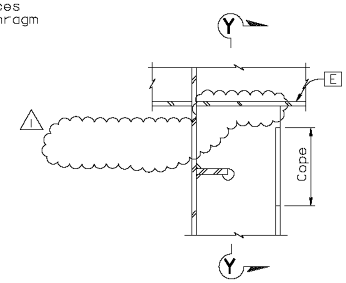
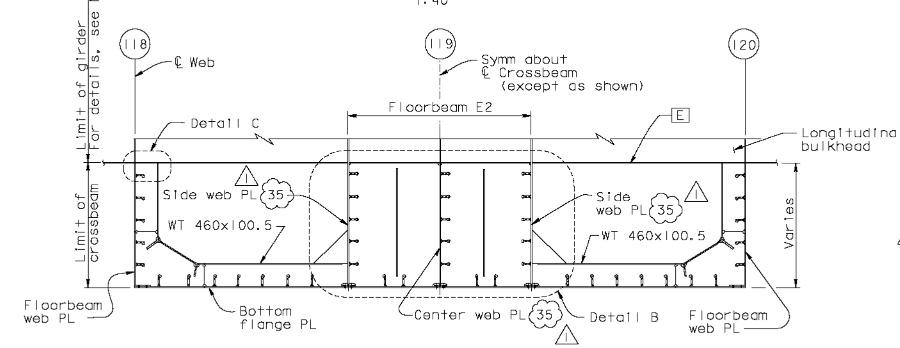
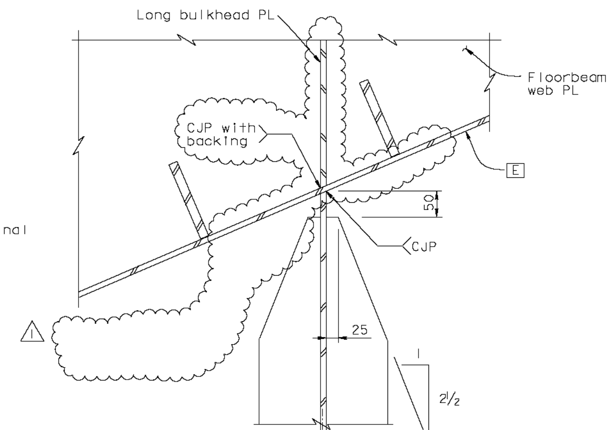


DETAIL C
1:10

111



PARTIAL HEIGHT DIAPHRAGM
SECTION A'-A'
1:40



SECTION Y-Y
1:5

NOTES:

1. For details not shown, see "Typical Crossbeam Details" sheets.
2. For girder details, see "Girder At Pier E2" sheets
3. For Detail A, see "Crossbeam At Pier E2 No.3" sheet.
4. For diaphragm in girder, see "Girder At Pier E2 No.6" sheet.

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PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
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SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
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(SUPERSTRUCTURE & TOWER)

CROSSBEAM AT PIER E2 NO. 2

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ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100
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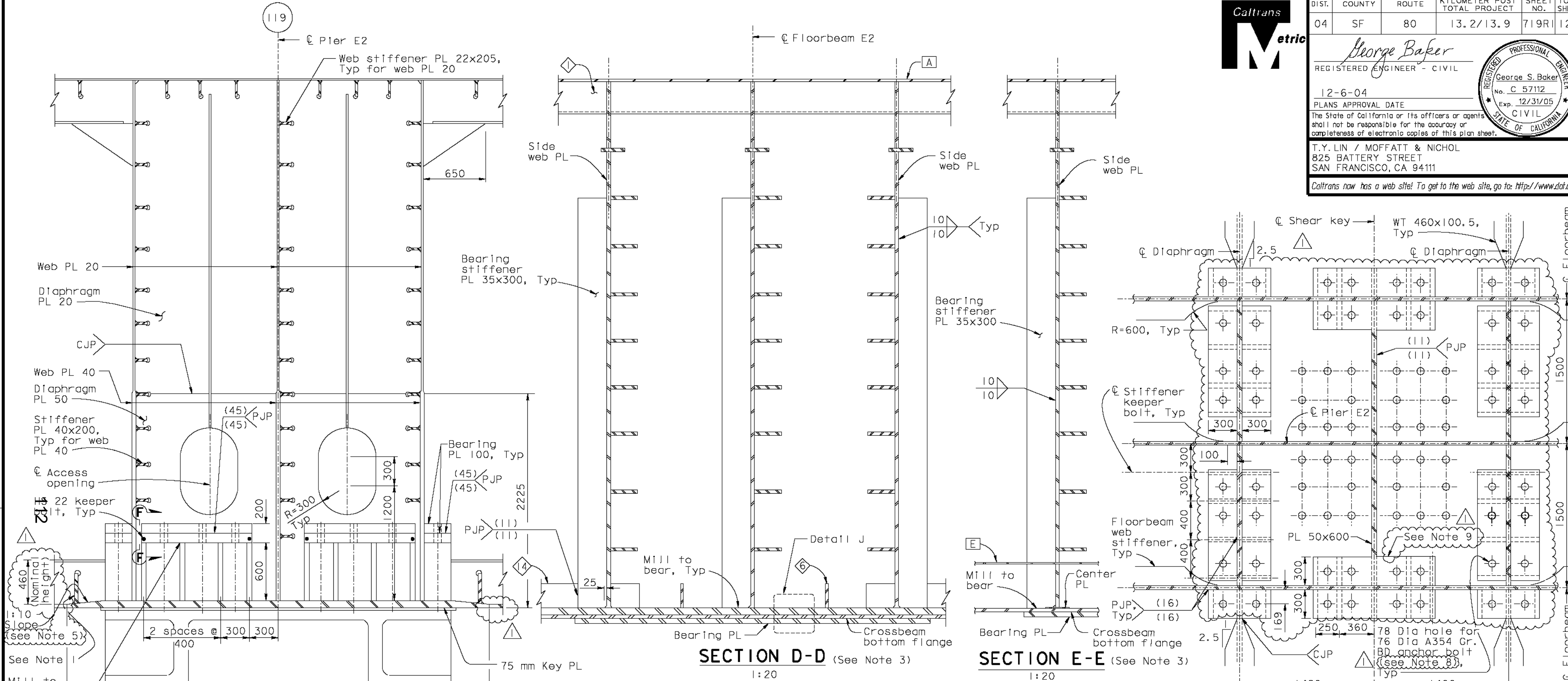

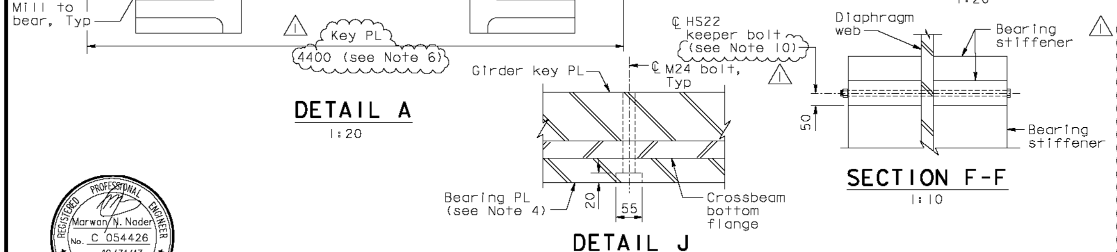

LEGEND:

76 Dia anchor bolt (A354 Gr. BD)

7. At the Contractor's option, standard oversized holes may be used in the key PL and the temporary bearing PL. For this option, a counterbore diameter of 61 mm and a counterbore depth of 25 mm shall be provided.
8. At the Contractor's option, 100 Dia holes may be used in the key PL and bearing PL, subject to review and approval of the Engineer.
9. At the Contractor's option, the top bearing plates may be detailed to overhang the side stiffener plates by 5 mm, subject to review and approval of the Engineer.
10. At the Contractor's option, keeper bolts may be placed in the upper bearing plate. Oversized holes may be used in the bearing plates and vertical plates at keeper bolt locations, subject to review and approval of the Engineer.

NOTES:

1. No weld to diaphragm this face of stiffener.
2. For details not shown, see "Typical Crossbeam Details" sheets.
3. For Section D-D and E-E callouts, see "Crossbeam At Pier E2 No. 7" sheet.
4. Bearing PL is counterbored to accomodate bolt heads.
5. Cope WT460 as required over the key PL.
6. At the Contractor's option and subject to review and approval of the Engineer, it is acceptable to use a taper at 1:2.5 slope at the Key PL. The taper shall end 100 clear of the nearest stiffener. Key PL width shall be 4000 Min.

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SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
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CROSSBEAM AT PIER E2 NO. 3

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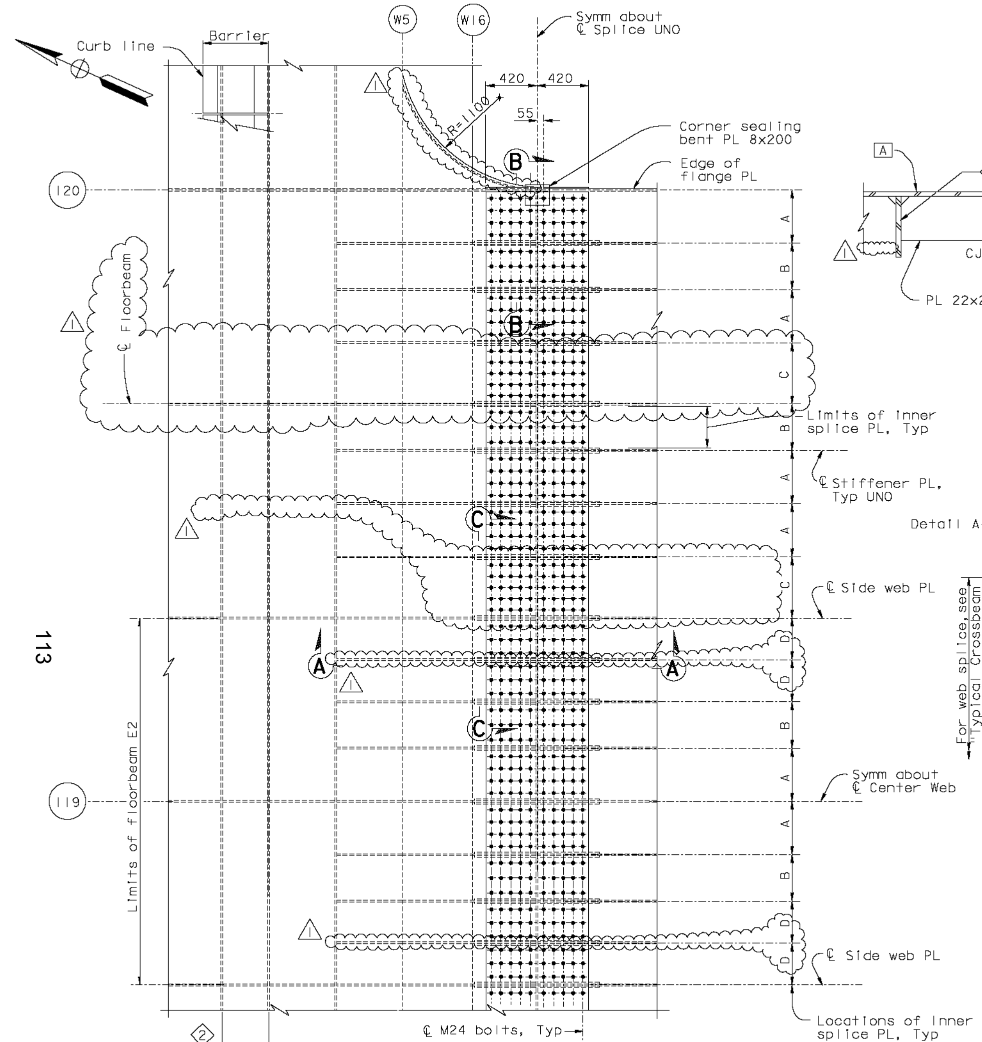
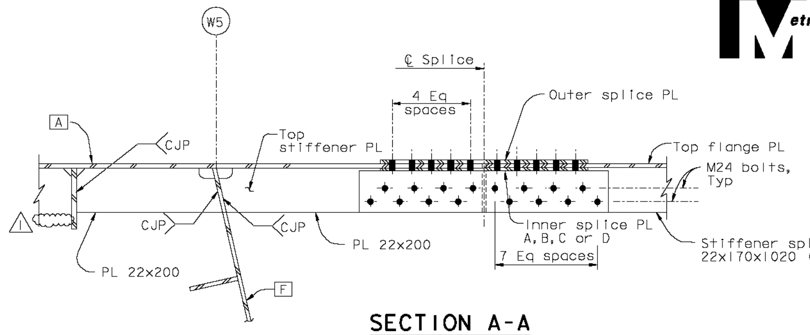


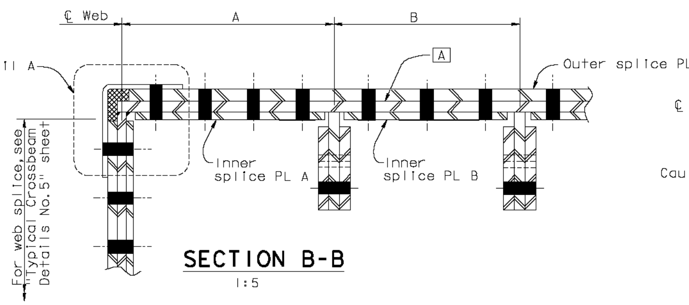
TOP FLANGE SPLICE DETAIL E

1:20



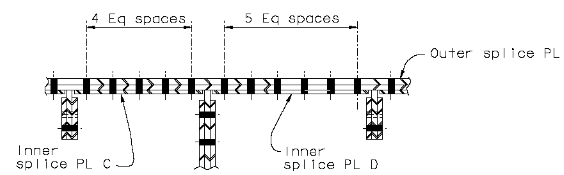
SECTION A-A

1:10



SECTION B-B

1:5



SECTION C-C

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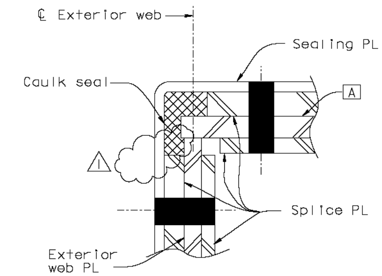

DETAIL A

1:2.5

SPLICE PLATE TABLE	
Plate Type	Dimensions
Inner PL "A"	PL 16x900x375
Inner PL "B"	PL 16x900x320
Inner PL "C"	PL 16x900x440
Inner PL "D"	PL 18x900x270
Outer PL	PL 25x900x9960

NOTES:

1. For details not shown, see "Typical Crossbeam Details" sheets.

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DETAILS	S. Camo	CHECKED	T. McMeans
QUANTITIES	D. Turner	CHECKED	M. Roberts

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE (SUPERSTRUCTURE & TOWER)

CROSSBEAM AT PIER E2 NO. 4

Rev. Date: 5-18-98
ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100
CU 04
EA 0120F1
DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES (PRELIMINARY STAGE ONLY)
SHEET OF 303R1

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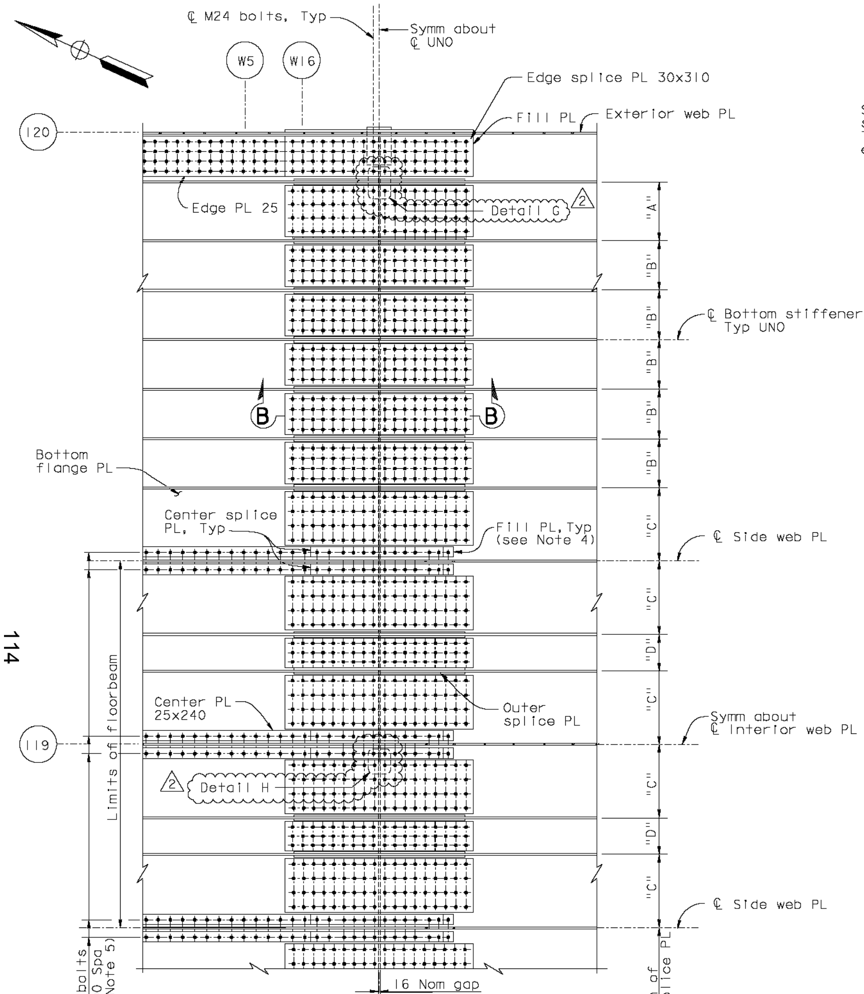
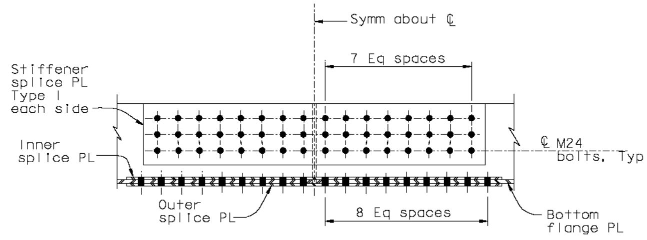


BOTTOM FLANGE SPLICE

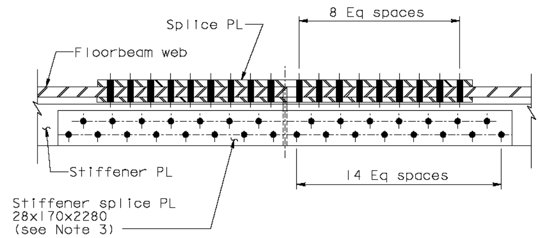
DETAIL F

1:20



SECTION B-B

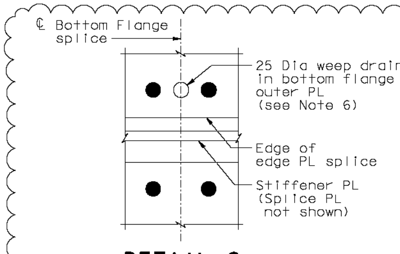
1:10



SECTION D-D

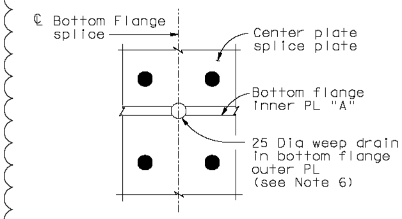
1:10

SPLICE PLATE TABLE	
Plate Type	Dimensions
Inner PL "A"	PL 25x1620x420
Inner PL "B"	PL 25x1620x340
Inner PL "C"	PL 25x1620x440
Inner PL "D"	PL 25x1620x240
Outer PL	PL 30x1620x9980
Stiff PL Type I	PL 30x250x1440



DETAIL G

1:4



DETAIL H

1:4

NOTES:

1. For details not shown, see "Typical Crossbeam Details No.8" sheet.
2. For details not shown, see "Typical Crossbeam Details" sheets.
3. At the Contractor's option, the stiffener splice PL length may be reduced to 1740, subject to review and approval of the Engineer.
4. For crossbeam segments which vary in depth within the specified tolerances, the Contractor shall use fill plates in the bottom splice to correct for the variation. Fill plates shown on the plans shall be adjusted as required to bring splice plates into contact.
5. At the Contractor's option, it is acceptable to space the M30 bolts at 90 mm, subject to review and approval of the Engineer.
6. Inside surfaces of weep drain shall be finish painted.

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DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	721R2	1204

George Baker
REGISTERED ENGINEER - CIVIL
12-6-04
PLANS APPROVAL DATE
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DESIGN OVERSIGHT
SIGN OFF DATE 04/30/12

MARK	DATE	DESCRIPTIONS	BY	CH'D	CCO#
2	04/30/12	OBG & CROSSBEAM SEALING	DT	HT	244
1	09/01/11	EAST END OBG	DT	GB	87
		REVISIONS			

	BY		CHECKED
DESIGN	G. Baker		P. Ritchie
DETAILS	D. Veytsman		T. McMeans
QUANTITIES	D. Turner		M. Roberts

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CROSSBEAM AT PIER E2 NO. 5

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 04
EA 0120F1

DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 304R2 OF

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DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	722R2	1204

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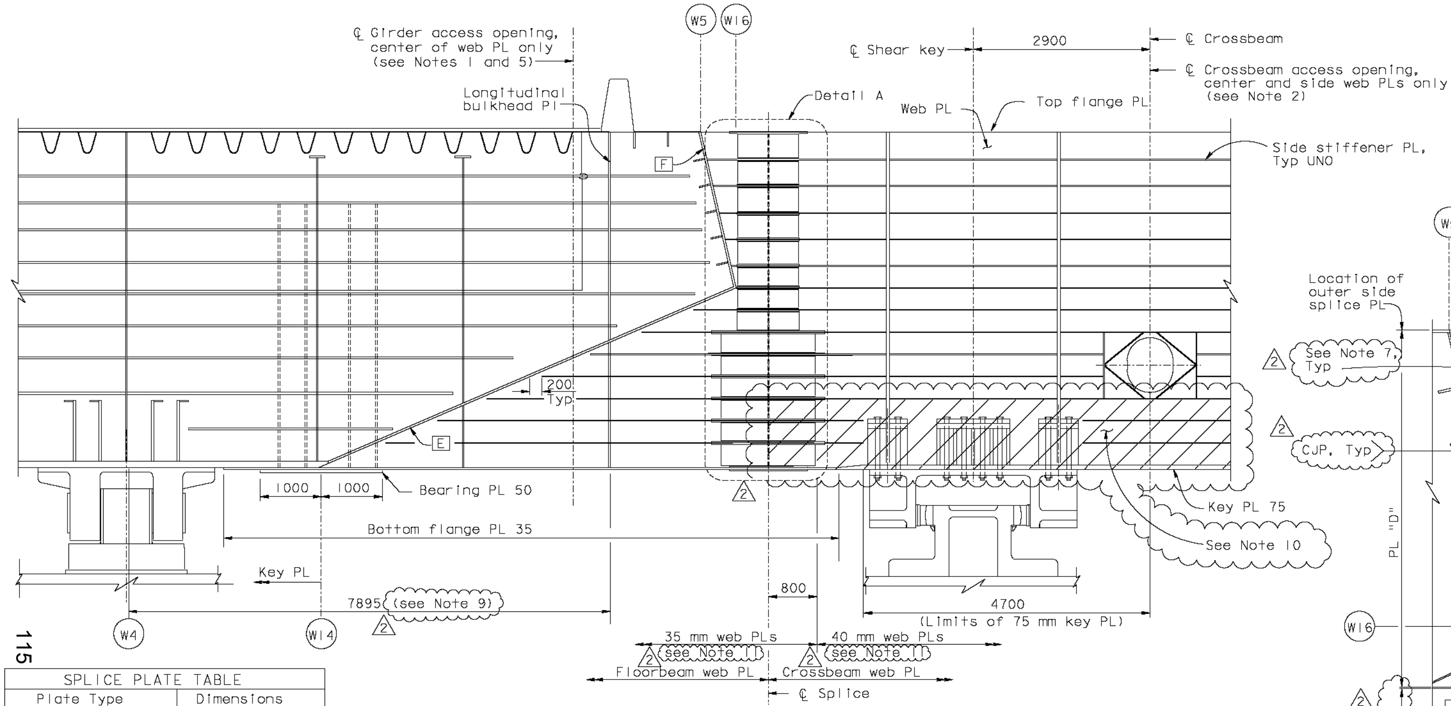
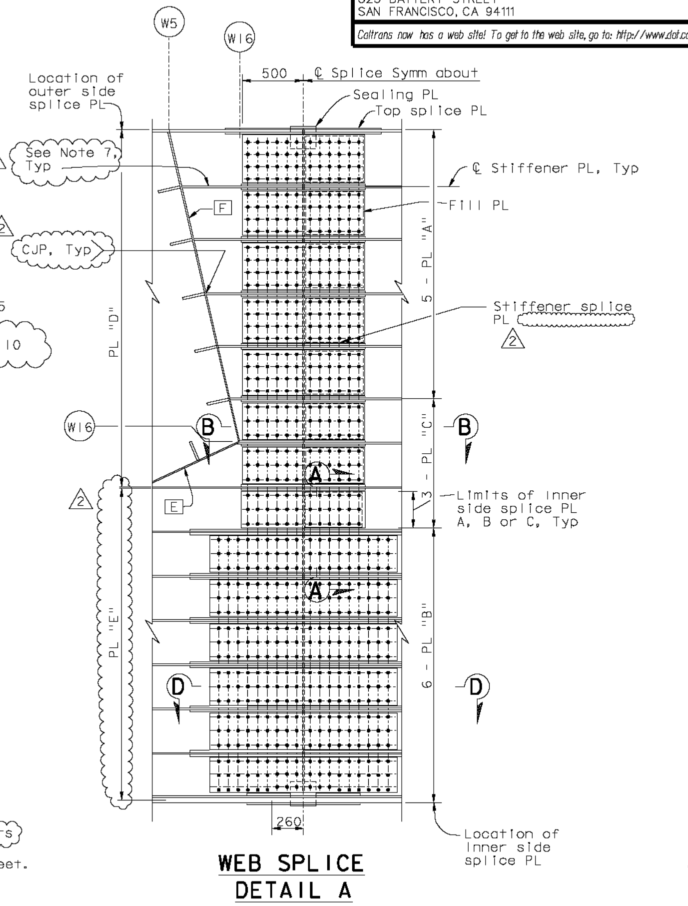


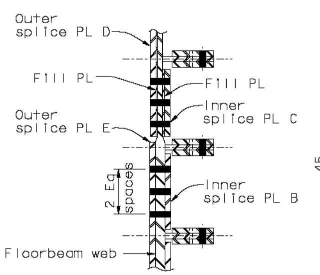
SECTION C-C
1:40



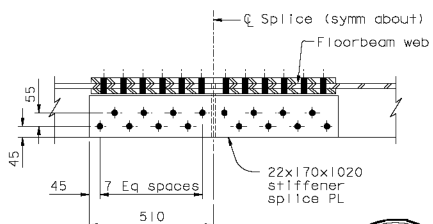
WEB SPLICE DETAIL A
1:20

115

SPLICE PLATE TABLE	
Plate Type	Dimensions
Inner Splice PL "A"	PL 22x1000x390
Inner Splice PL "B"	PL 22x1620x310
Outer Splice PL "D"	PL 28x1000x3170
Inner Splice PL "C"	PL 22x1000x310
Outer Splice PL "E"	PL 28x1620x2140



SECTION A-A
1:10



SECTION B-B
1:10

LEGEND:

"Indicates TTP Web PL"

NOTES:

1. For details at girder access opening, see "Typical Girder Details No.15" sheet.
2. For details of crossbeam access opening not shown, see "Typical Crossbeam Details No.6" sheet.
3. For details not shown, see "Typical Crossbeam Details" sheets.
4. For layout of E2 floorbeam web, see "Girder At Pier E2 No.4" sheet.
5. For girder access locations, see "Girder Access Details No.3" sheet.
6. For details not shown, see "Typical Crossbeam Details No.5" sheet.
7. Transitions not required for crossbeam web stiffeners terminating at Face F.
8. For Section D-D, see "Crossbeam At Pier E2 No.5" sheet.
9. For the EB girder, this dimension is 7885.
10. Through Thickness Properties (TTP) area shown applies to PP118.7, 119.0 and 119.3.
11. It is acceptable to extend the limit of the 40 mm web PL to the centerline of the splice from 800 mm from the splice, provided the fill plates on the 35 mm web are placed on the interior faces of the crossbeam.

PROFESSIONAL ENGINEER Daniel A. Turner No. C 65033 Exp. 06/30/13 CIVIL STATE OF CALIFORNIA

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2	09/01/11	EAST END OBG	DT	GB	87
1	11/05/10	PP119 ACCESS & PP125 PLATES	GB	DT	86
		REVISIONS			

R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 09/01/11

	BY	CHECKED
DESIGN	G. Baker	P. Ritchie
DETAILS	D. Veytsman	T. McMeans
QUANTITIES	D. Turner	M. Roberts

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R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CROSSBEAM AT PIER E2 NO. 6

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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SHEET 305R2

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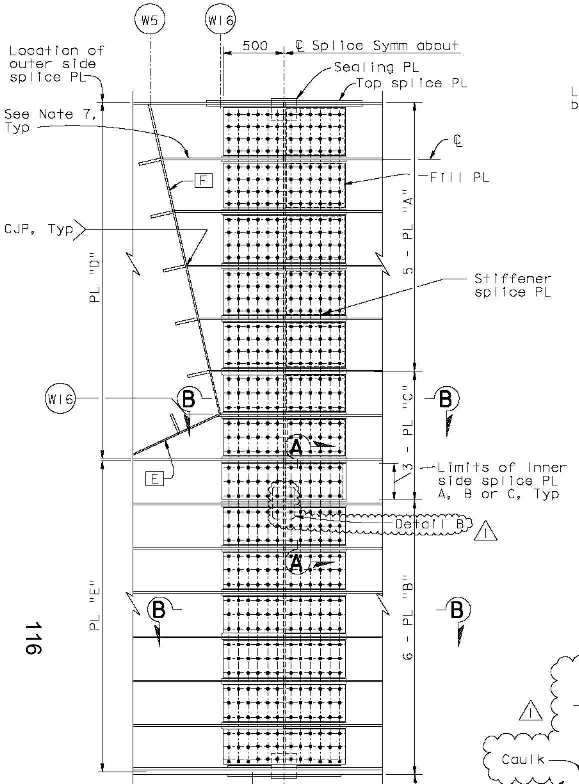
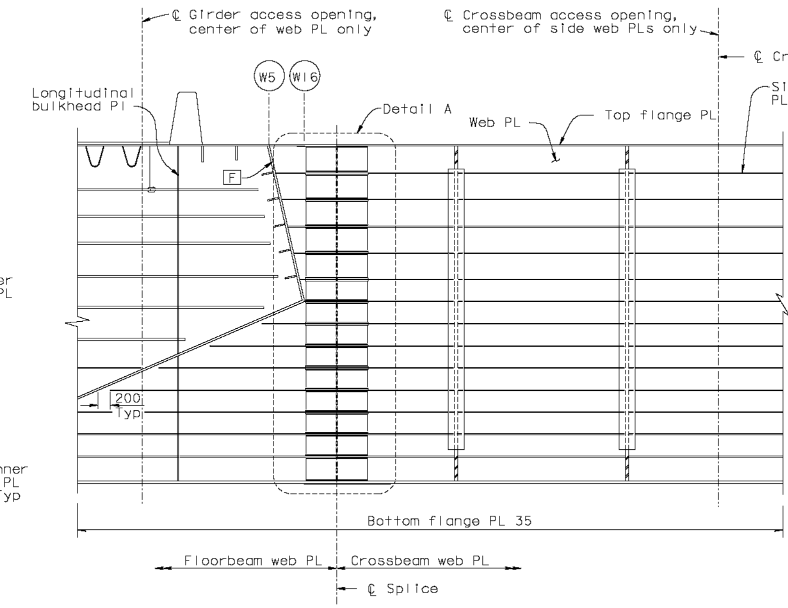


WEB SPLICE DETAIL A

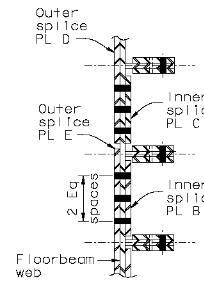
1:20

SPLICE PLATE TABLE	
Plate Type	Dimensions
Inner Splice PL "A"	PL 22x1000x390
Inner Splice PL "B"	PL 22x1000x310
Outer Splice PL "D"	PL 28x1000x3170
Inner Splice PL "C"	PL 22x1000x310
Outer Splice PL "E"	PL 28x1000x2140



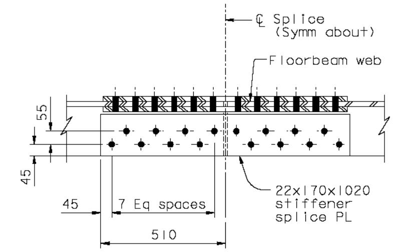
SECTION C'-C'

1:40



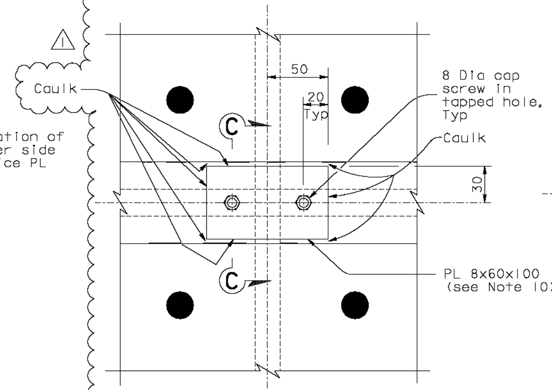
SECTION A-A

1:10



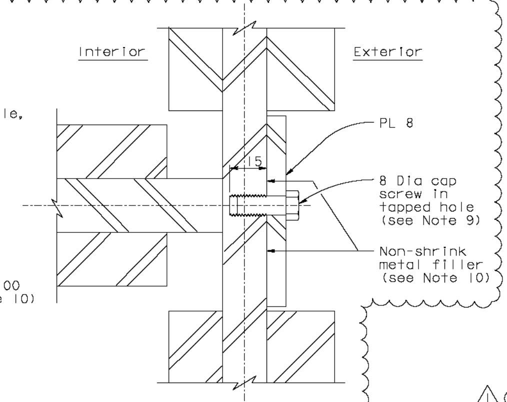
SECTION B-B

1:10



DETAIL B

1:2



SECTION C-C

1:1

NOTES:

1. For details at girder access opening, see "Typical Girder Details No.15" sheet.
2. For details of crossbeam access opening not shown, see "Typical Crossbeam Details No.6" sheet.
3. For details not shown, see "Typical Crossbeam Details" sheets.
4. For layout of E2 floorbeam web, see "Girder At Pier E2 No.4" sheet.
5. For girder access locations, see "Girder Access Details No.3" sheet.
6. For details not shown, see "Typical Crossbeam Details No.5" sheet.
7. Transitions not required for crossbeam web stiffeners terminating at Face F.
8. For Section D-D, see "Crossbeam At Pier E2 No.5" sheet.
9. Cap screws shall have a minimum embedment length of 12 mm in the crossbeam web plate.
10. Non-shrink metal filler shall be applied to fill gaps and provide a level faying surface for the PL 8.

116

CONTRACT CHANGE ORDER NO. ____
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1	04/30/12	OBG & CROSSBEAM SEALING	DT	HT	244
0	04/18/12	EAST END OBG	GB	MN	87
REVISIONS					

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DESIGN OVERSIGHT
SIGN OFF DATE 04/30/12

	BY	CHECKED
DESIGN	G. Baker	M. Nader
DETAILS	G. Baker	D. Turner
QUANTITIES	G. Baker	D. Turner

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CROSSBEAM AT PIER E2 NO. 6A

Rev. Date: 5-18-98

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 04
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DISREGARD PRINTS BEARING EARLIER REVISION DATES 04/18/12

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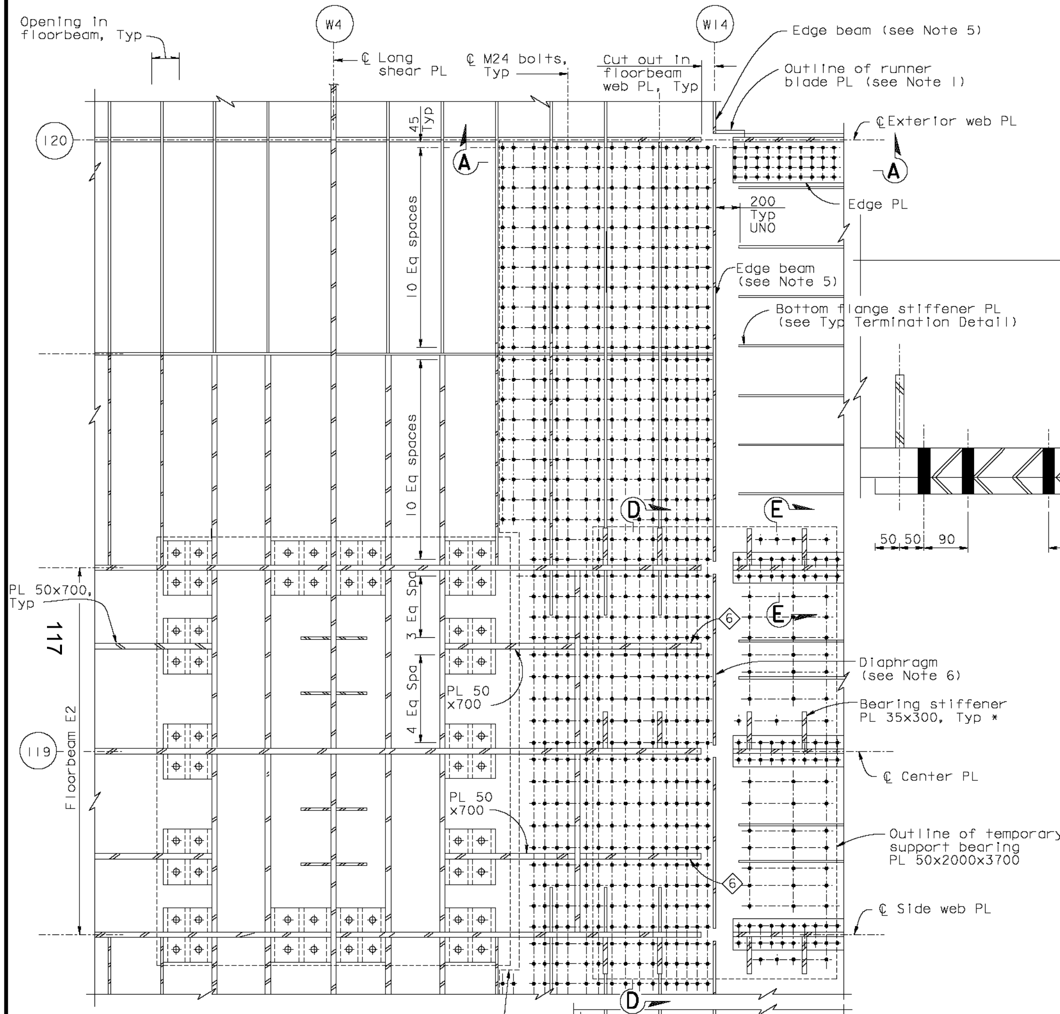
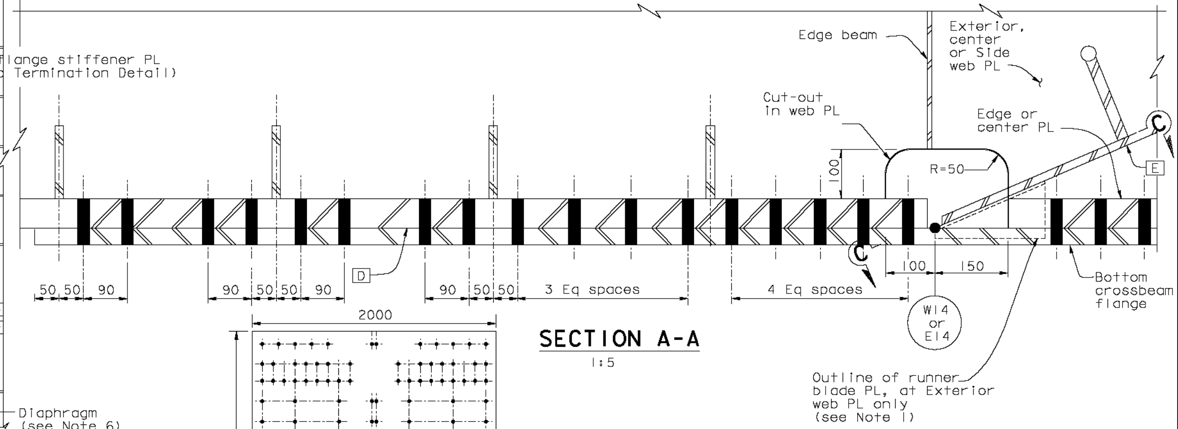


117

BOTTOM SPLICE TO GIRDER

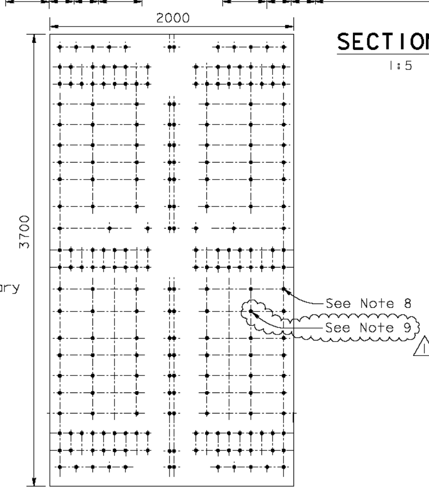
DETAIL G *(For bearing replacement)

1:20



SECTION A-A

1:5



BEARING PLATE AT TEMPORARY SUPPORT AT LINE E13/W13 - PLAN

NOTES:

1. For details of runner blade, see "Typical Crossbeam Details No.10" sheet.
2. For Section C-C, see "Typical Crossbeam Details No.9" sheet.
3. For details not shown, see "Typical Crossbeam Details" sheets.
4. For Sections D-D and E-E, see "Crossbeam At Pier E2 No.3" sheet.
5. For edge beam, see longitudinal shear PL Type I in "Typical Girder Details No.7" sheet.
6. For diaphragm details, see "Girder At Pier E2 No.6" sheet.
7. Cope crossbeam bottom flange to clear bearing top PL.
8. Stitch and sealing bolts shall be detailed to avoid stiffeners.
9. At the Contractor's option, standard oversize holes may be used for anchor bolts of the temporary bearings at E13/W13 and E14/W14.

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REQUESTS FOR INFORMATION NOT ADDRESSED IN THIS CCO REMAIN IN FORCE

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1	09/01/11	EAST END OBG	DT	GB	87

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DESIGN	G. Baker	P. Ritchie
DETAILS	D. Veytsman	T. McMeans
QUANTITIES	D. Turner	M. Roberts

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R. Manzanarez
PROJECT ENGINEER

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BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

CROSSBEAM AT PIER E2 NO. 7

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	723R1	1204

George Baker
REGISTERED ENGINEER - CIVIL
12-6-04
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DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 08/02/99 05/31/01 04/02/02 07/31/02 12/19/02 10/13/05 12/07/05

SHEET 306R1 OF

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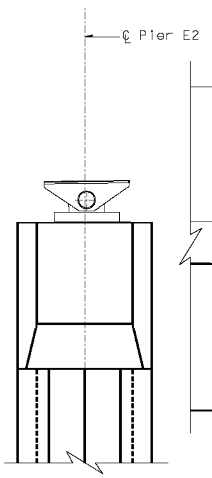
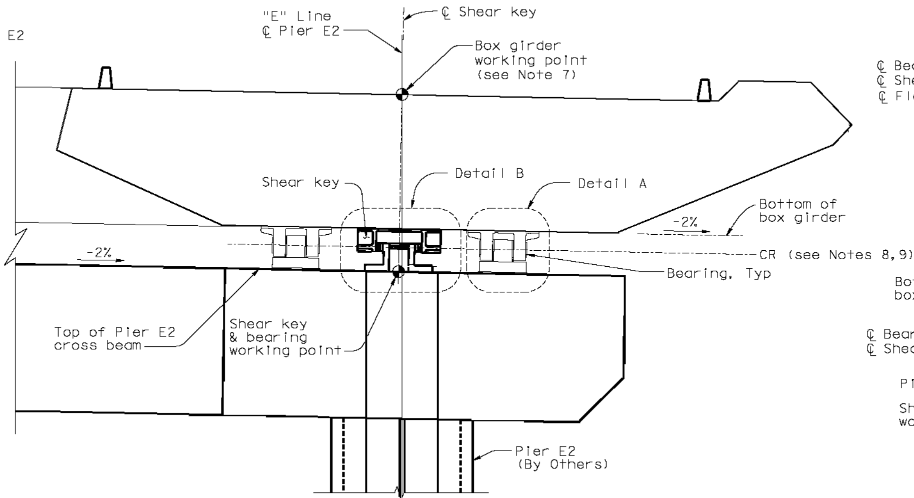
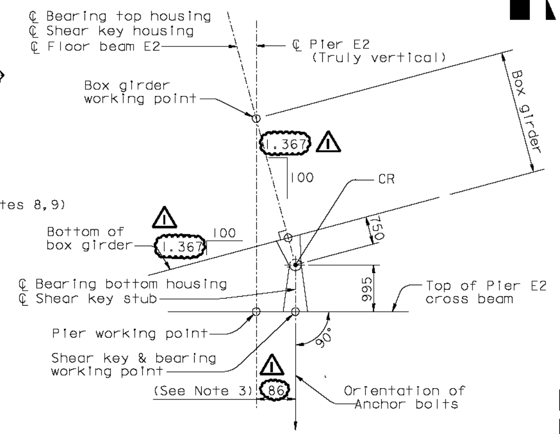


SIDE ELEVATION
1:200
(Looking South)

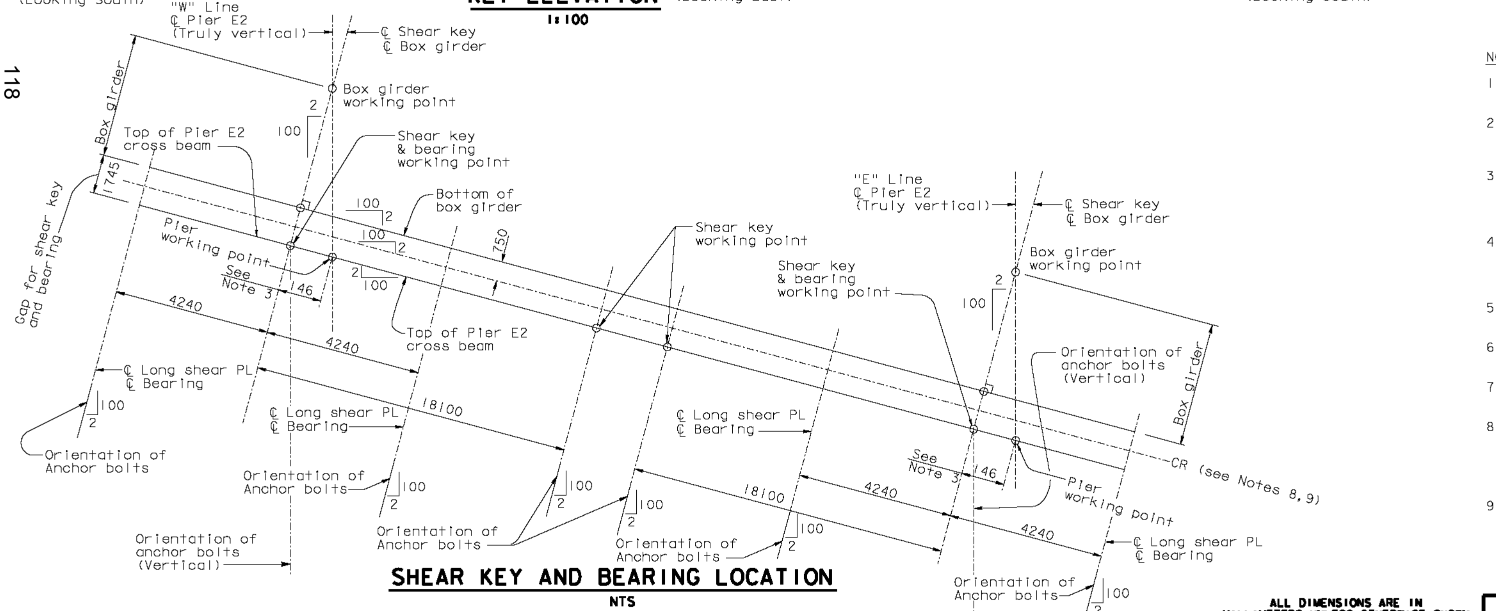


KEY ELEVATION (Looking East)
1:100

LEGEND:
CR Center of Rotation



SHEAR KEY AND BEARING LOCATION
NTS
(Looking South)



SHEAR KEY AND BEARING LOCATION
NTS
(Looking East)

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NOTES:

1. For Detail A, see "Pier E2 Bearing Details No. 1" sheet.
2. Detail B is a section view at center line of shear key. For Detail B, see "Pier E2 Shear Key Details No. 1" sheet.
3. Offset value is shown for information only. The Contractor shall calculate the offset and verify it in the field before placing the bearings and shear key.
4. The Contractor shall submit the construction sequence of the shear key and bearings based on his ways and means for the Engineer's approval.
5. For floor beam E2, see "Girder at Pier E2" sheets.
6. For longitudinal shear plate, see "Girder at East Transition No. 1" sheet.
7. Box girder work point is on the finished profile grade which is the top of 50 mm overlay.
8. At the Contractor's option, small holes may be drilled through the bearings and shear keys for use with a laser alignment method. All holes shall be plugged with tapped screws after bearing and shear key installation.
9. The center of rotation CR shall be a straight line through the pin center of all four bearings, and perpendicular to the bridge axis direction.

DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	80	13.2/13.9	882R1	1204

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1	09/26/08	E2 CROSS BEAM	MN	NV	71

REVISIONS

CONTRACT CHANGE ORDER NO. ____
SHEET ___ OF ___

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R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 09/26/08
Rev. Date: 5-18-98

DESIGN	BY M. Nader	CHECKED L. Rus
DETAILS	BY N. Vo	CHECKED J. Leventini
QUANTITIES	BY N. Vo	CHECKED J. Leventini

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

PREPARED FOR THE
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R. Manzanarez
PROJECT ENGINEER

CU 04
EA 0120F1

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

PIER E2 BEARING AND SHEAR KEY LAYOUT

DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES (PRELIMINARY STAGE ONLY) 04/08/01 07/01/02 12/15/03 12/07/05
SHEET 465R1

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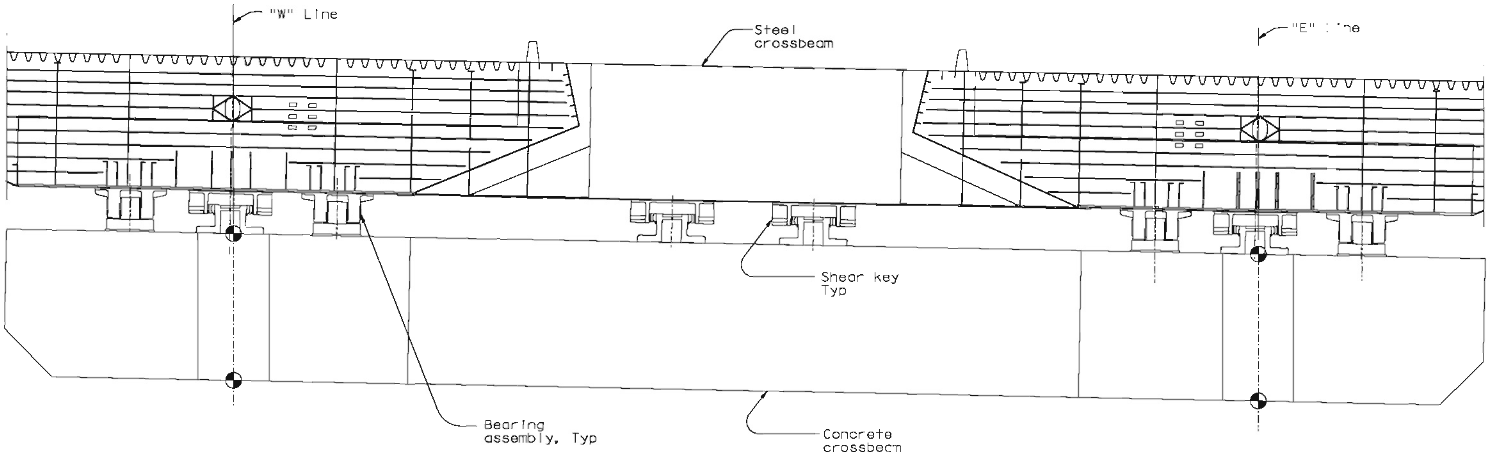

ELEVATION (Looking East)

CLEAN UP/ CLARIFICATION

BEARING ASSEMBLY TABLE

Location	Bearing Typ	Number of bearing units per location	Design Load (kN per bearing)							Design rotation (radians)						Design Displacement (mm)					
			Service			Ultimate				Service			Ultimate			Service			Ultimate		
			Compression	Uplift	Long or Trans	Load Condition	Compression	Uplift	Long / Trans	Trans Axis	Long Axis	Vert Axis	Trans Axis	Long Axis	Vert Axis	Trans Axis	Long Axis	Vert Axis	Trans Axis	Long Axis	Vert Axis
"W" Line	Spherical Bushing	2	35000	0	0	Shear Key Engaged	86000	32000	0	0.009	0	0	0.130	0.032	0.032	5	5	0	20	20	2**
						Shear Key Failed*	68000	17000	13300 / 30500												
"E" Line	Spherical Bushing	2	35000	0	0	Shear Key Engaged	86000	32000	0	0.009	0	0	0.130	0.032	0.032	5	5	0	20	20	2**
						Shear Key Failed*	68000	17000	13300 / 30500												

RFI #760 R0

* Seismic load factor ɣ= 1.0 (For shear key engaged load condition, ɣ= 1.4).
** For uplifting only.

Installation Misalignment Tolerances (Per bearing/ Shear Key)	Trans./Long./Vertical	Rotational
	± 2 mm	± 0.5°

SHEAR KEY ASSEMBLY TABLE

Location	Number of Shear Key Units Per Location	Design Load (kN per shear key)						Design rotation (radians)						Design Displacement (mm)					
		Service			Ultimate			Service			Ultimate			Service			Ultimate		
		Trans	Long	Vert	Trans	Long	Vert	Trans Axis	Long Axis	Vert Axis	Trans Axis	Long Axis	Vert Axis	Trans Axis	Long Axis	Vert Axis	Trans Axis	Long Axis	Vert Axis
"W" Line	1	9000	4500	0	42500	35000	0	0.009	0	0	0.130	0.130	0.130	0	0	10	0	0	20
"E" Line	1	9000	4500	0	42500	35000	0	0.009	0	0	0.130	0.130	0.130	0	0	10	0	0	20
Crossbeam	2	9000	1	0	42500	0	0	0.009	0	0	0.130	0.130	0.130	0	5	10	0	20	20

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DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
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R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 09/26/08

MARK	DATE	DESCRIPTIONS	BY	CH'D	CCO#
1	09/26/08	E2 CROSS BEAM	VN	NV	71

REVISIONS

DESIGN BY M. Nader — CHECKED L. Rus
DETAILS BY N. Vo — CHECKED J. Leventini
QUANTITIES BY N. Vo — CHECKED J. Leventini

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

PIER E2 BEARING AND SHEAR KEY DESIGN FORCES

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100
CU 04
EA 0120F1
DISREGARD PRINTS BEARING EARLIER REVISION DATES
SHEET 465ARIC

Rev. Date 3-19-98
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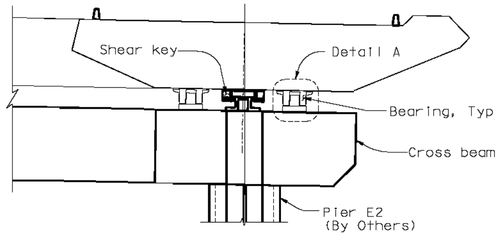
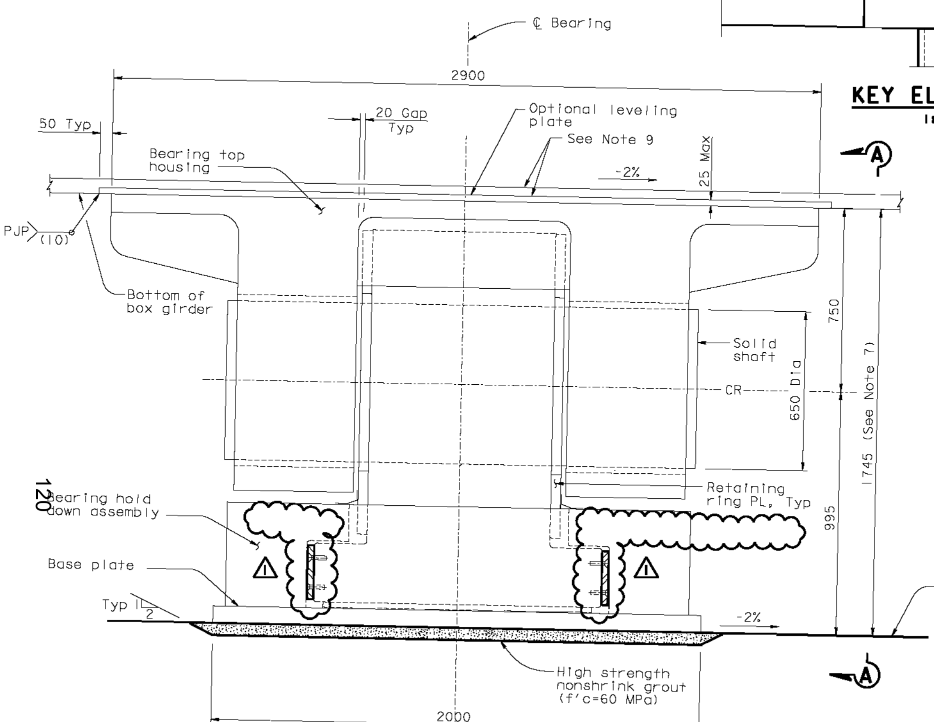
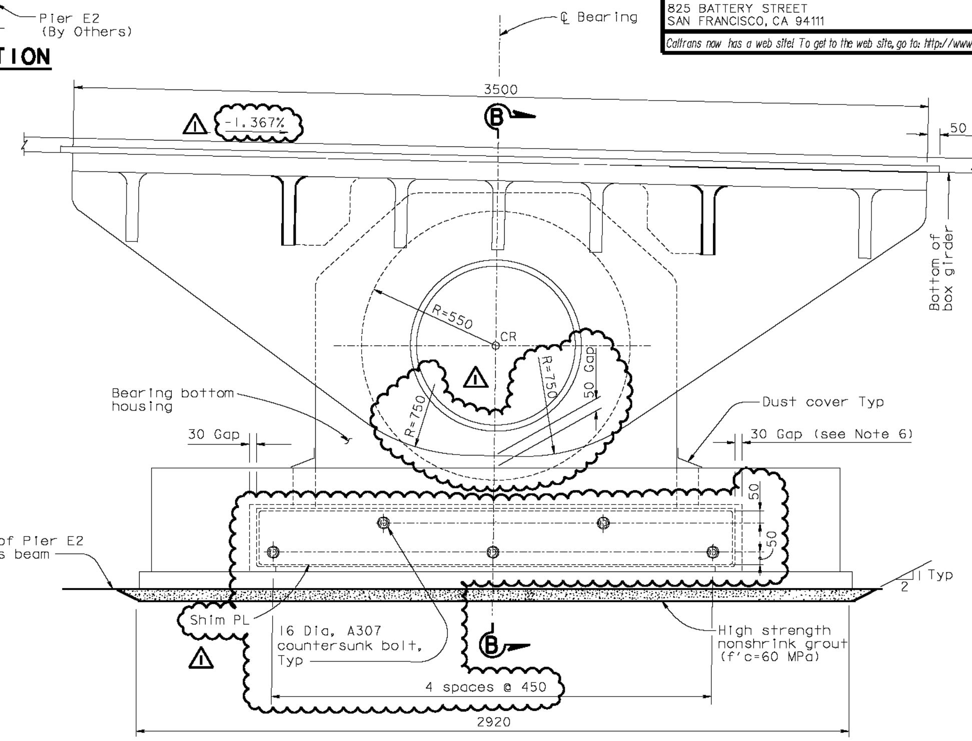


KEY ELEVATION
1:200



DETAIL A (Bearing Assembly)
1:10



VIEW A-A
1:10

LEGEND:

CR Center of Rotation

NOTES:

1. For Section B-B, see "Pier E2 Bearing Details No.2" sheet.
2. Connections to box girder and Pier E2 are not shown for clarity.
3. The bearing top housing and bearing hold down assembly shall be Structural Casting Grade 345.
4. The bearing bottom housing and the solid shaft shall be Structural Casting Grade 550.
5. The grout pad thickness is shown for information only. Before grout pour, Contractor shall verify in the field the grout pad thickness required to align the center of rotation of the shear key and bearings at 0.750 m from the bottom surface of the box girder and ensure center of rotation of all bearings and shear keys are aligned in the same axis.
6. Gaps shall be maintained during installation by using shims placed below the dust cover locations. Remove shims after grouting. Alternatively, 3/4" diameter spacing bolts may be installed. These shall remain undamaged during installation and shall be replaced with plug bolts.
7. The Contractor may provide optional leveling plates to achieve fit-up and level contact surfaces for the bearings and shear keys. The Contractor shall erect the E2 cap beam and E2 girders to the elevations and tolerances specified in the plans and special provisions.
8. Leveling plate shall be attached with plug welds at 0.5 m Max spacing, and a perimeter PJP weld.
9. All faying surfaces of the girder key plate, the bearing top housing, and the optional leveling plate shall be machined after all welding for flatness and smoothness as specified for the top of the bearing top housing. See "Pier E2 Bearing Details No. 3" sheet.

CONTRACT CHANGE ORDER NO. ____
SHEET ____ OF ____

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MARK	DATE	DESCRIPTIONS	BY	CH'D	CCO#
2	07/17/09	E2 CROSS BEAM	MN	NV	71
1	09/26/08	E2 CROSS BEAM	MN	NV	71
		REVISIONS			

R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 07/17/09

DESIGN BY M. Nader — CHECKED L. Rus
DETAILS BY N. Vo — CHECKED J. Leventini
QUANTITIES BY N. Vo — CHECKED J. Leventini

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE (SUPERSTRUCTURE & TOWER)

PIER E2 BEARING DETAILS NO. 1

SHEET 466R2

CU 04
EA 0120F1

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS

DISREGARD PRINTS BEARING EARLIER REVISION DATES — 05/31/01 04/26/02 07/01/02 12/18/02 12/01/05

Rev. Date: 5-18-98

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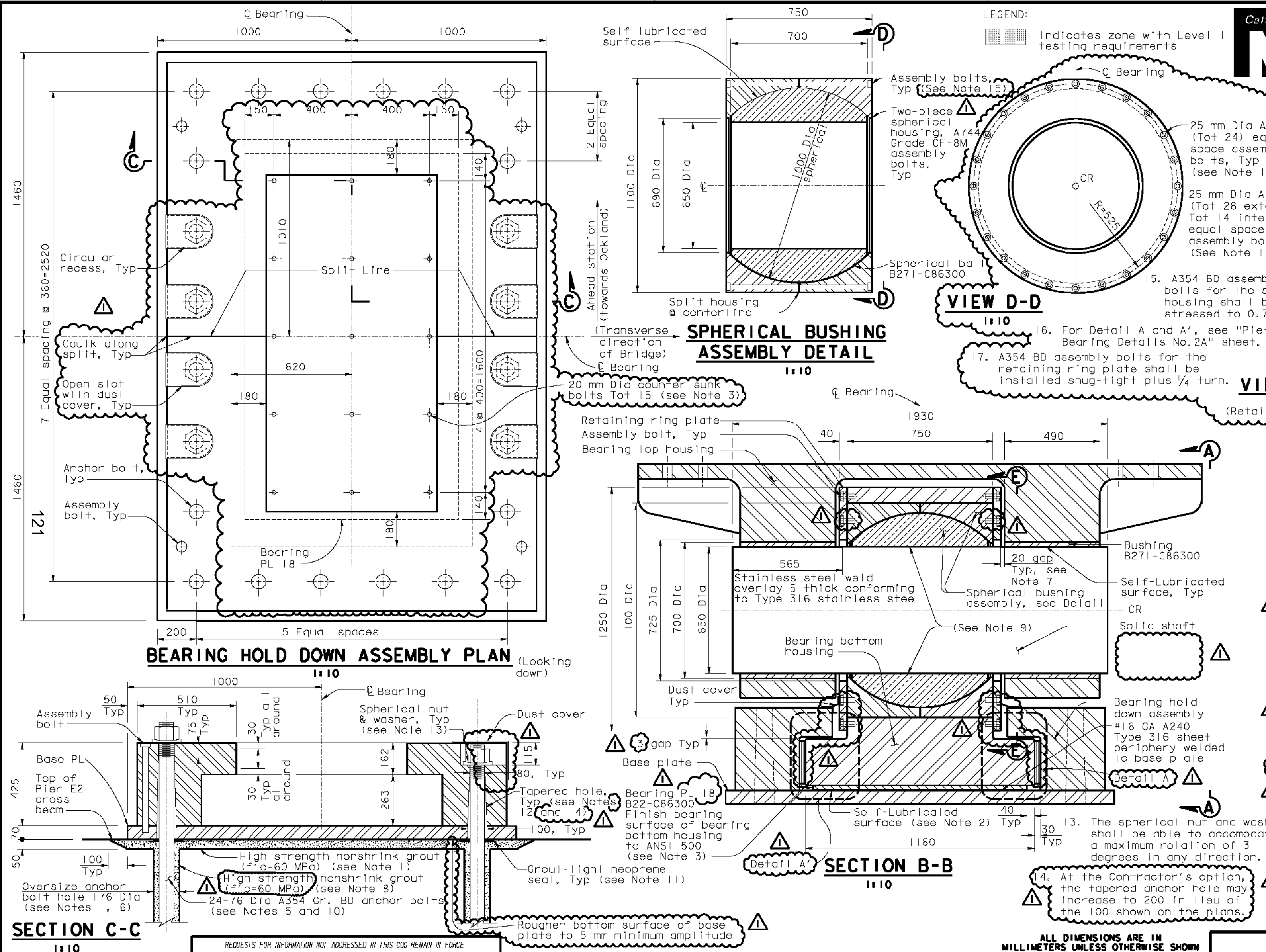

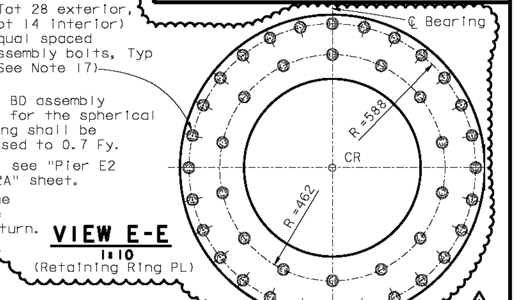

NOTES:

1. The grout pad thickness is shown for information only. The Contractor shall verify in the field the grout pad thickness required to align the center of rotation of the shear key and bearings at 0.750 m from the bottom surface of the box girder.
2. The lubricant shall be Self-lubricated.
3. Bearing plate shall be fastened to bottom of bottom housing using A240 Type 316 counter sunk bolts.
4. All assembly bolts shall be A240 Type 316 unless noted otherwise.
5. The anchor bolt shall be pretensioned to 70% of ultimate tensile strength. Wherever there is no access from the top, anchor bolt shall be prestressed from the bottom.
6. The oversize anchor bolt holes are provided to accommodate construction tolerances.
7. 20 mm gap ±2 mm shall be maintained on both sides during installation.
8. For additional prestressing details, see "Prestressing Notes" sheet.
9. Solid shaft in bushing is press fit.
10. The prestressed anchor bolts shall be wrapped in a PT supplier recommended debonding tape, subject to the review and approval of the Engineer.
11. Grout-tight neoprene seal shown is schematic and is for information only. The seal shall prevent any high strength nonshrink grout from seeping inside the anchor bolt tapered holes during grouting of the base plate. This is necessary for proper stressing of the anchor bolts. The Contractor may propose an alternate detail to suit his erection means and methods, subject to review and approval of the Engineer.
12. For tapered hole details, see "Pier E2 Shear Key Details No. 1" sheet.
13. The spherical nut and washer shall be able to accomodate a maximum rotation of 3 degrees in any direction.
14. At the Contractor's option, the tapered anchor hole may increase to 200 in lieu of the 100 shown on the plans.

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DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
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△	09/26/08	E2 CROSS BEAM	MN	NV	71
		REVISIONS			

R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 09/26/08

CONTRACT CHANGE ORDER NO. ____
SHEET ____ OF ____

DESIGN	BY M. Nader	CHECKED L. Rus
DETAILS	BY N. Vo	CHECKED J. Leventini
QUANTITIES	BY N. Vo	CHECKED J. Leventini

PREPARED FOR THE STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

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ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

PIER E2 BEARING DETAILS NO. 2

DISREGARD PRINTS BEARING EARLIER REVISION DATES
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SHEET 467R1 OF

Rev. Date: 5-18-98

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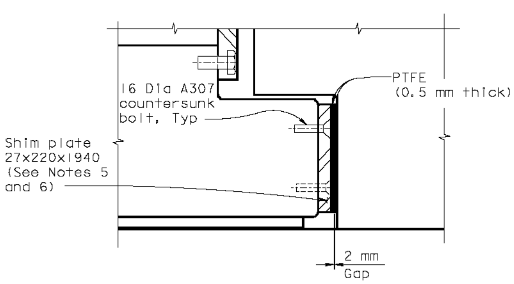
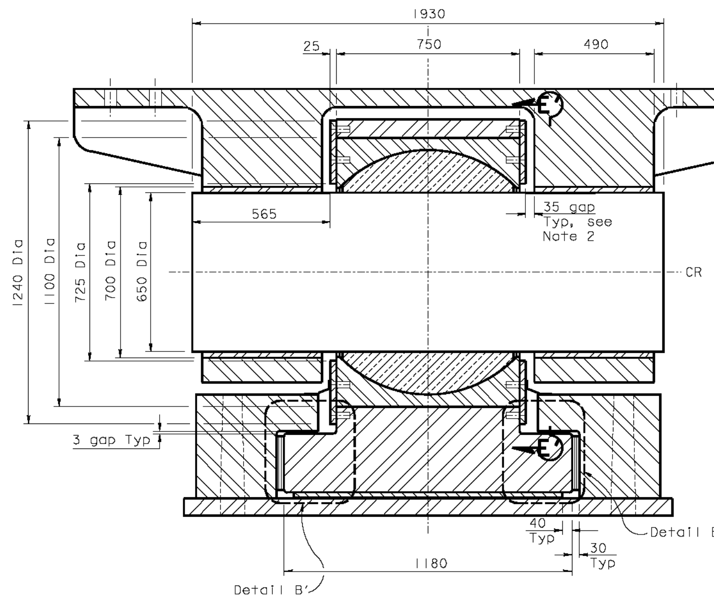
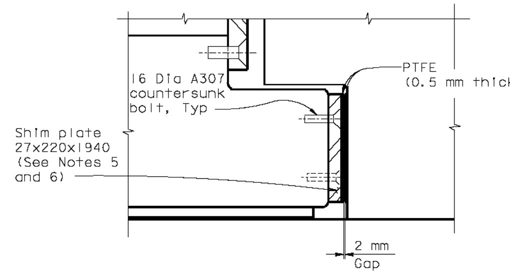


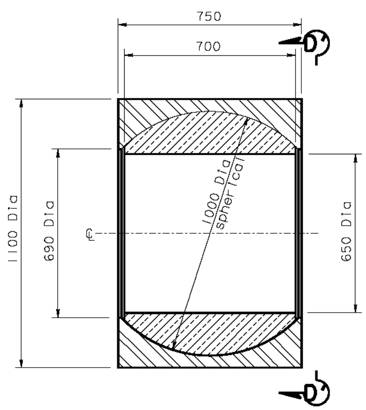
DETAIL A
(Detail A' similar)
1:5



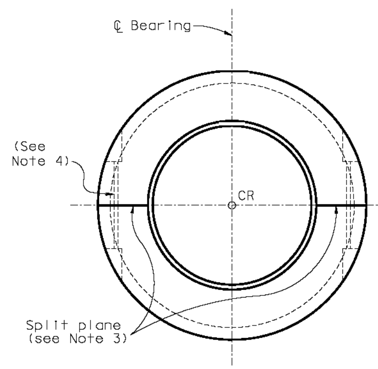
SECTION B-B
1:10



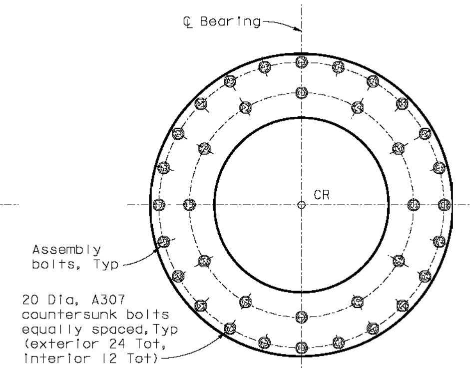
DETAIL B
(Detail B' similar)
1:5



SPHERICAL BUSHING ASSEMBLY DETAIL
1:10



VIEW D'-D'
1:10



VIEW E'-E'
1:10
(Retaining Ring PL)

ALTERNATE BEARING DETAILS (See Note 1)
1:10

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△	09/25/08	E2 CROSS BEAM	MN	NV	71

REVISIONS

CONTRACT CHANGE ORDER NO. ____
SHEET ____ OF ____

NOTES:

1. For details not shown, see "Pier E2 Bearing Details No. 2" sheet.
2. 35 mm gap ±2 mm shall be maintained on both sides during installation.
3. The split plane shall be parallel to the base plate in the final erected position.
4. Assembly bolts and recesses are shown schematically. Design and details of assembly bolts for spherical housing shall be per bearing manufacturer.
5. Shim plate and PTFE film shall be provided along the 2 sides of the bearing housing in the longitudinal direction of the bridge.
6. Shim plate details may vary with field conditions subject to review and approval of the Engineer.

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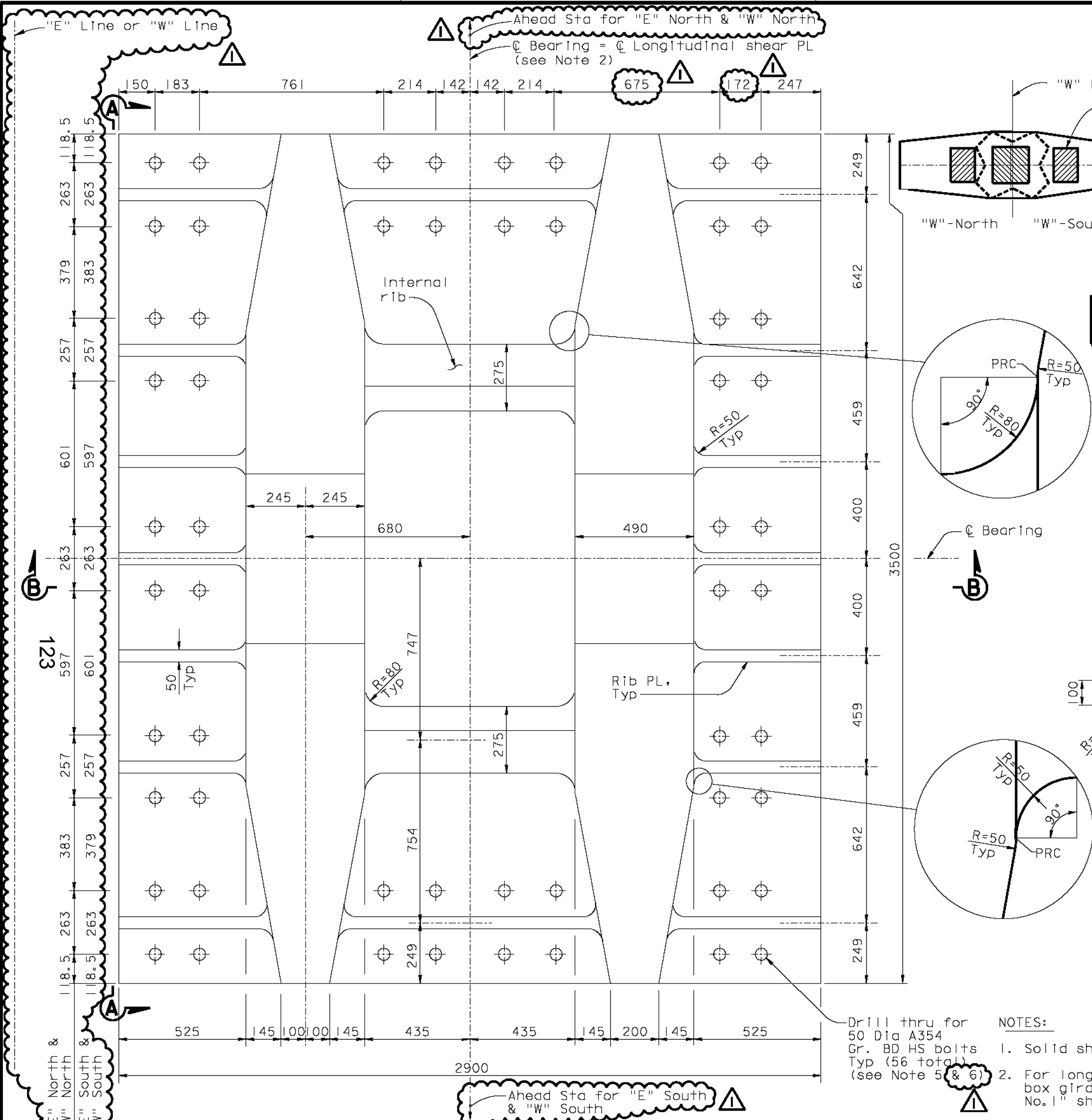
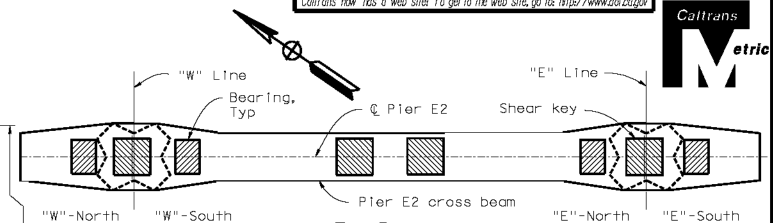
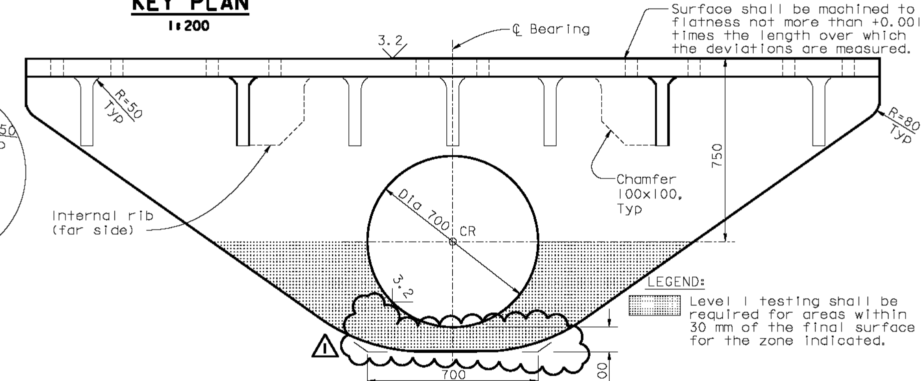


BEARING TOP HOUSING PLAN
1:10

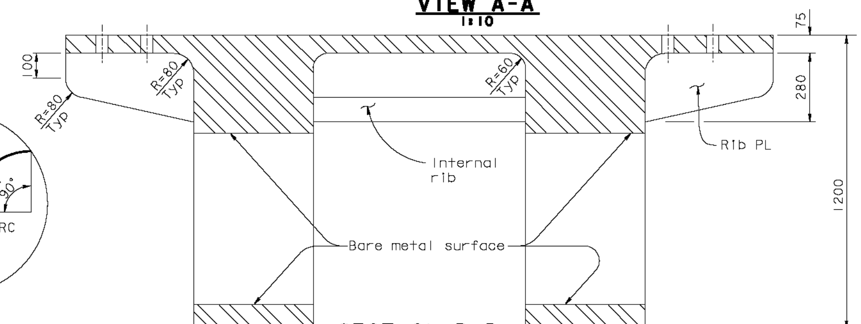
(Looking Up) ("E" - South Bearing shown. "W" - South similar. See Note 3 and 4)



KEY PLAN
1:200



VIEW A-A
1:10



SECTION B-B
1:10

NOTES:

1. Solid shaft in bushings is press fit.
2. For longitudinal shear plate location on box girder, see "Girder At East Transition No. 1" sheet.
3. "E"-South bearing and "W"-South bearing are similar. "E"-North bearing and "W"-North bearing are similar.
4. Bolt hole location shall be cross checked with the box girder stiffener layout before drilling holes. Bolt edge distance and clearance for tightening shall be verified and submitted for review and approval by the Engineer. A template shall be used to drill bolt holes in bearing top housing and girder key plate to ensure hole matching.
5. The anchor bolt shall be pretensioned to 70% of ultimate tensile strength.
6. At the Contractor's option, oversized holes and appropriate washer plates may be used in the bearing top housing.

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	BY	CHECKED
DESIGN	M. Nader	L. Rus
DETAILS	N. Vo	J. Leventini
QUANTITIES	N. Vo	J. Leventini

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

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SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE (SUPERSTRUCTURE & TOWER)

PIER E2 BEARING DETAILS NO. 3

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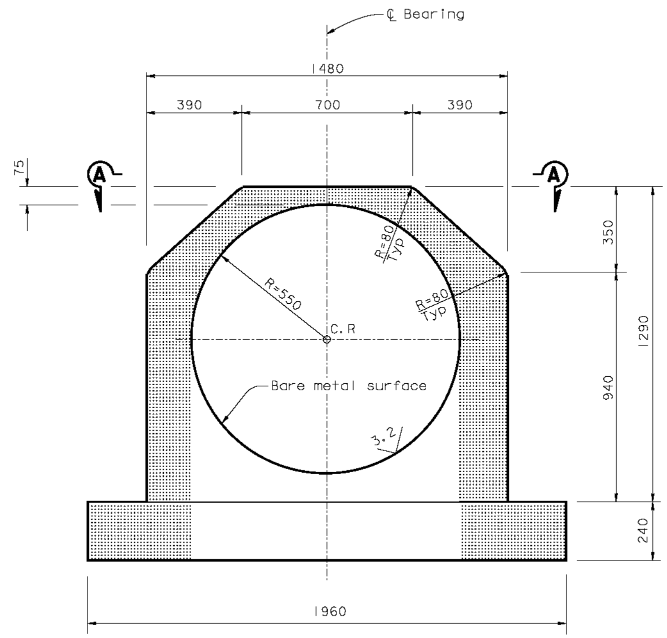
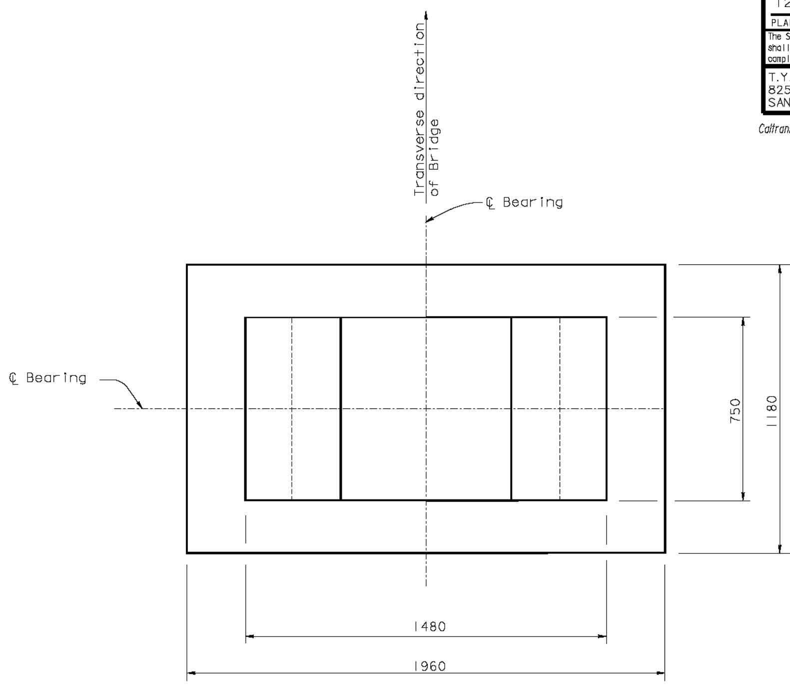


BEARING BOTTOM HOUSING (Looking North)
1:10



VIEW A-A
1:10

LEGEND:
Indicates zone with Level I testing requirements

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DIST.	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
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SIGN OFF DATE 12/19/02

DESIGN	BY M. Nader	CHECKED L. Rus
DETAILS	BY N. Vo	CHECKED J. Leventini
QUANTITIES	BY N. Vo	CHECKED J. Leventini

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DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
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SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

PIER E2 BEARING DETAILS NO. 4

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REVISION DATES (PRELIMINARY STAGE ONLY) 04/08/02 07/01/02 12/19/02
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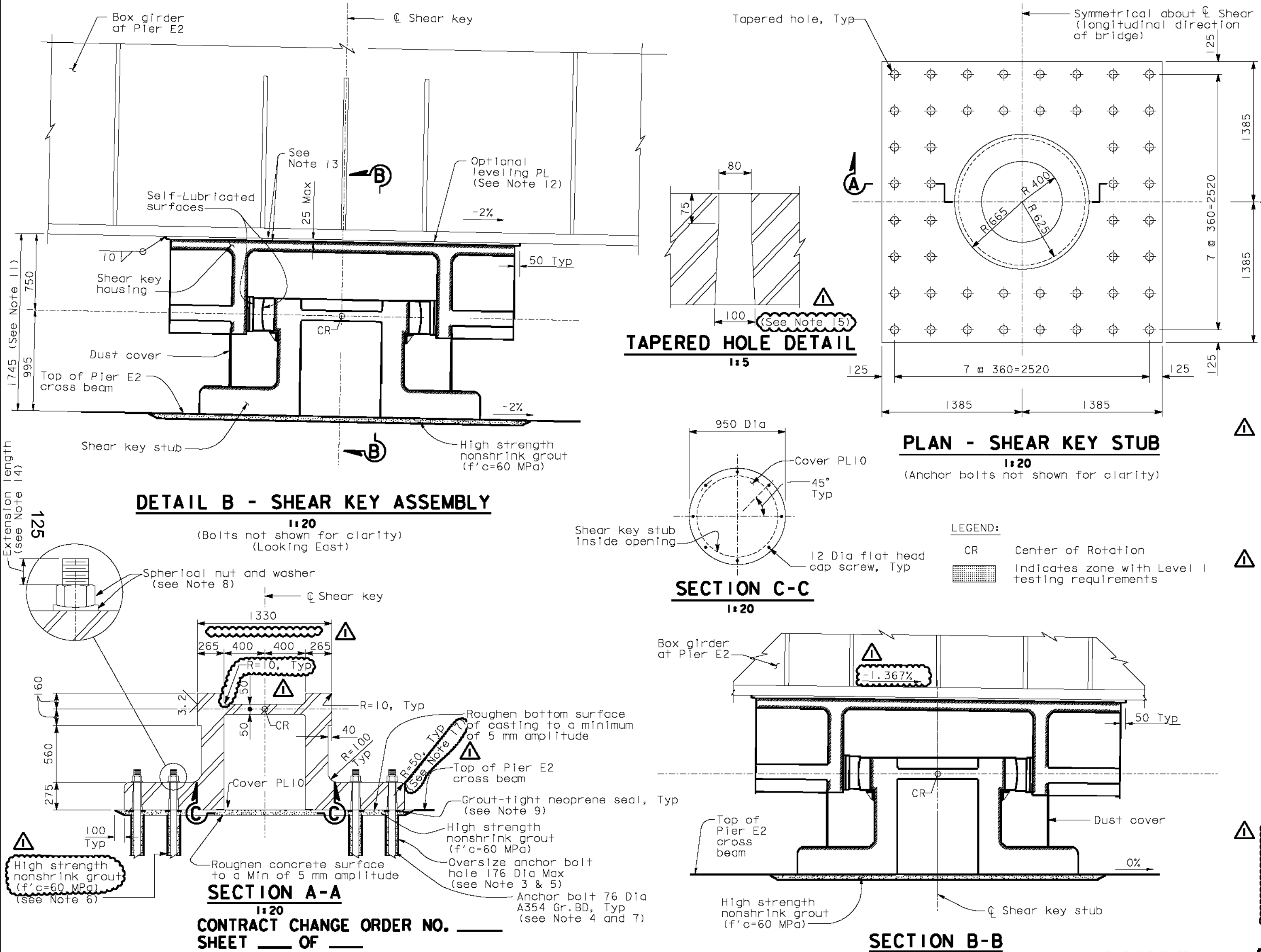

NOTES:

1. The shear key stub shall be Structural Casting Grade 345.
2. The lubricant shall be self-lubricated.
3. The grout pad thickness is shown for information only. The Contractor shall verify in the field the grout pad thickness required to align the center of rotation of the shear key and bearings at 0.750 m from the bottom surface of the box girder.
4. The anchor bolts shall be pretensioned by jacking to 70% ultimate tensile strength.
5. The oversize anchor bolt holes are provided to accommodate construction tolerances.
6. For additional prestressing details, see "Prestressing Notes" sheet.
7. The prestressed anchor bolts shall be wrapped in a PT supplier recommended debonding tape, subject to the review and approval of the Engineer.
8. The spherical nut and washer shall be able to accomodate a maximum rotation of 3 degrees in any direction.
9. Grout-tight neoprene seal shown is schematic and is for information only. The seal shall prevent any high strength nonshrink grout from seeping inside the anchor bolt tapered holes during grouting of the shear key stub. This is necessary for proper stressing of the anchor bolts. The Contractor may propose an alternate detail to suit their erection means and methods, subject to review and approval of the Engineer.
10. Bolted connection between shear key housing and box girder is slip-critical connection with class B contact surface.
11. The Contractor may provide optional leveling plates to achieve fit-up and level contact surfaces for the bearings and shear keys. The Contractor shall erect the E2 cap beam and E2 girders to the elevations and tolerances specified in the plans and special provisions.
12. Leveling plate shall be attached with plug welds at 0.5 m Max spacing, and a perimeter fillet weld.
13. All faying surfaces of the girder key plate, the shear key housing, and the optional leveling plate shall be machined after all welding for flatness and smoothness as specified for the top of the shear key housing. See "Pier E2 Shear Key Detail No. 2" sheet.
14. Extension length for anchor bolts at "E" Line and "W" Line shear keys shall be long enough to engage hydraulic jack.
15. At the Contractor's option, the tapered anchor hole may increase to 200 in lieu of the 100 shown on the plans.
16. The top shear key stub shaft section shall be "snug-fit" inside the spherical ring with a diameter gap between ring and stub of 1 mm Max.
17. At the Contractor's option, 10 mm radius may be used to facilitate stressing of anchor bolts.

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
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12-6-04
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MARK	DATE	DESCRIPTIONS	BY	CH'D	CCO#
2	07/17/09	E2 CROSS BEAM	MN	NV	71
1	09/26/08	E2 CROSS BEAM	MN	NV	71

REVISIONS

R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 07/17/09

	BY	CHECKED
DESIGN	M. Nader	J. Denis
DETAILS	N. Vo	J. Denis
QUANTITIES	N. Vo	J. Denis

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

PIER E2 SHEAR KEY DETAILS NO. 1

CU 04
EA 0120F1

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

SHEET 470R2

Rev. Date: 5-18-98

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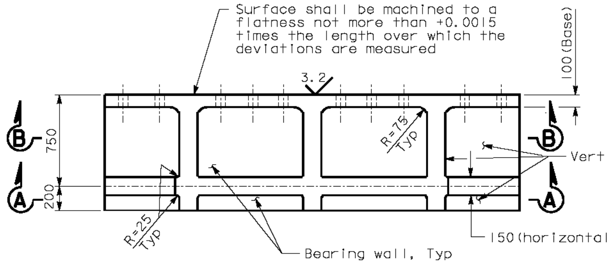
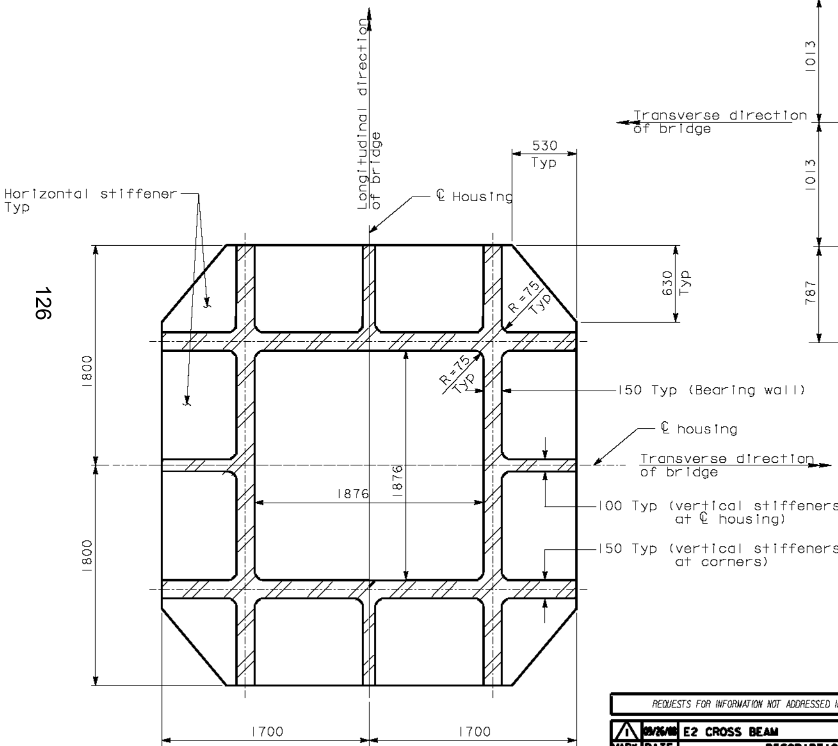


ELEVATION - SHEAR KEY HOUSING
1:20



SECTION A-A
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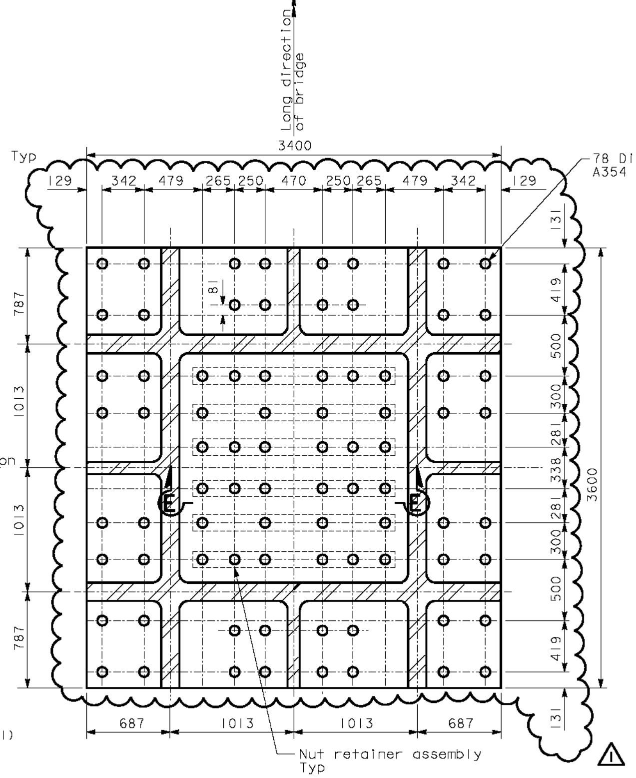

SECTION B-B
(For details not shown see Section A-A)
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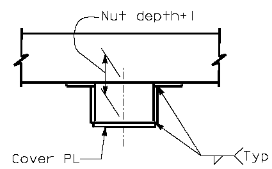
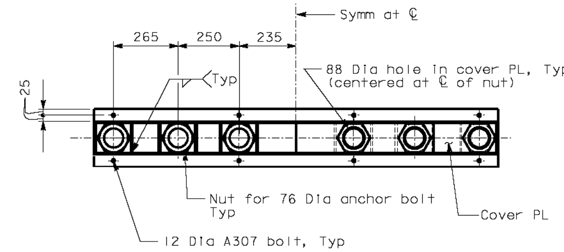
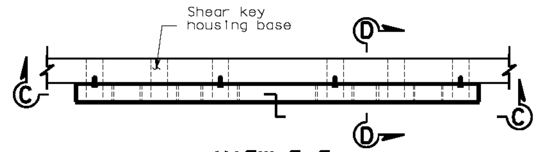


SECTION D-D
1:5



SECTION C-C
1:10



VIEW E-E
NUT RETAINER ASSEMBLY-ELEVATION
1:10

(All plates 8 mm thick)

NOTES:

1. The shear key housing shall be Structural Casting Grade 345.
2. The shear key housing bolt layout shall accommodate the girder and crossbeam plate stiffener location. For girder details at Pier E2, see "Girder At Pier E2 details" sheet. For crossbeam details at pier E2, see "Crossbeam at Pier E2" sheets.
3. Bolt hole location shall be cross checked with the box girder and crossbeam stiffener layout before drilling holes. Bolt edge distance and clearance for tightening shall be verified and submitted for review and approval by the Engineer. A template shall be used to drill bolt holes in shear key housing and girder key plate to ensure hole matching.
4. The anchor bolts shall be pretensioned to 70% of ultimate tensile strength.
5. Nut retainer assemblies including nuts shall be installed to shear key housing prior to assembling housing to bushing and stub.
6. Machined washers of bolt heads shall be used if necessary at stiffener fillets.

REQUESTS FOR INFORMATION NOT ADDRESSED IN THIS CCO REMAIN IN FORCE

MARK	DATE	DESCRIPTIONS	BY	CH'D	CCO#
1	09/26/08	E2 CROSS BEAM	MN	NV	71

REVISIONS

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R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 09/26/08

DESIGN	BY M. Nader	CHECKED J. Denis
DETAILS	BY N. Vo	CHECKED J. Denis
QUANTITIES	BY N. Vo	CHECKED J. Denis

PREPARED FOR THE STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE (SUPERSTRUCTURE & TOWER)

PIER E2 SHEAR KEY DETAILS NO. 2

Rev. Date: 5-18-98
ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100
CU 04
EA 0120F1
DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES (PRELIMINARY STAGE ONLY) 04/08/01 07/01/02 12/15/02 05/14/03 12/07/05
SHEET 471R1 OF

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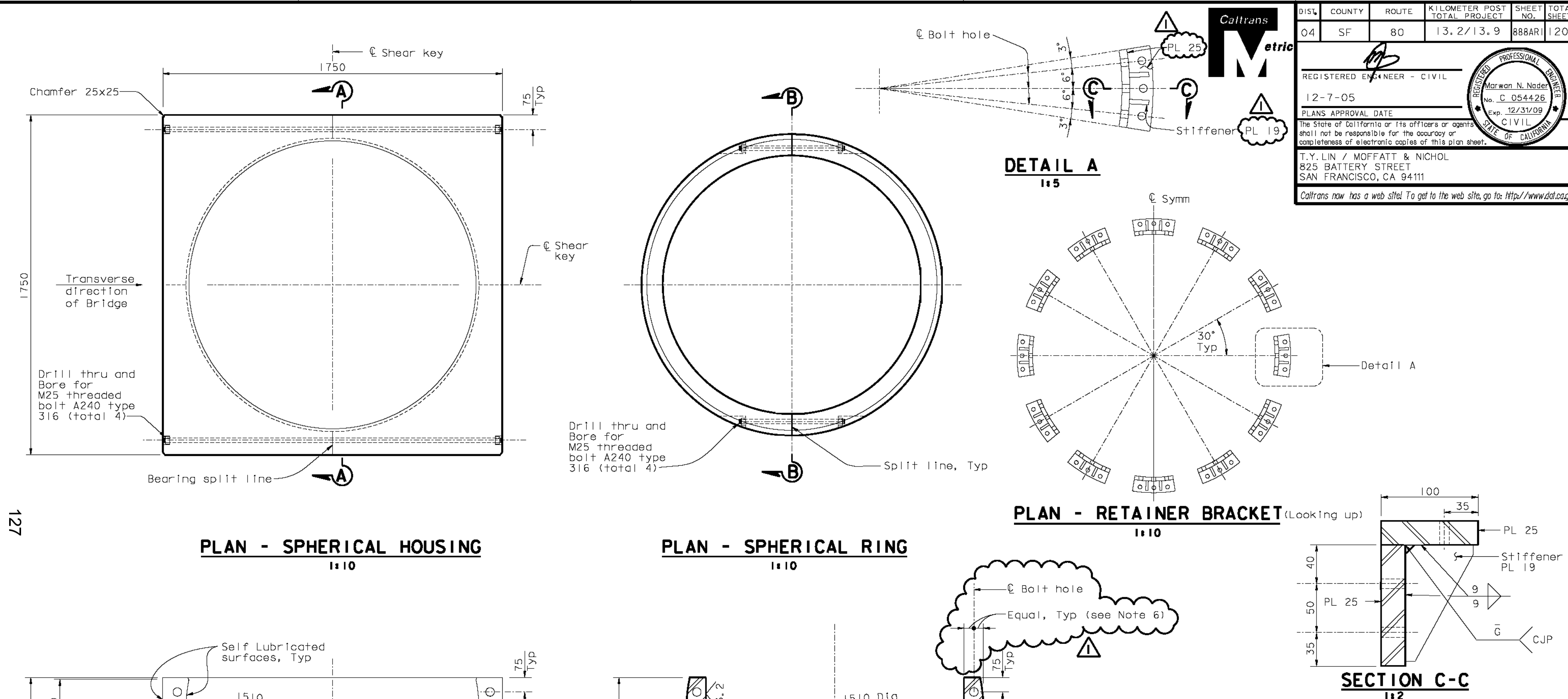

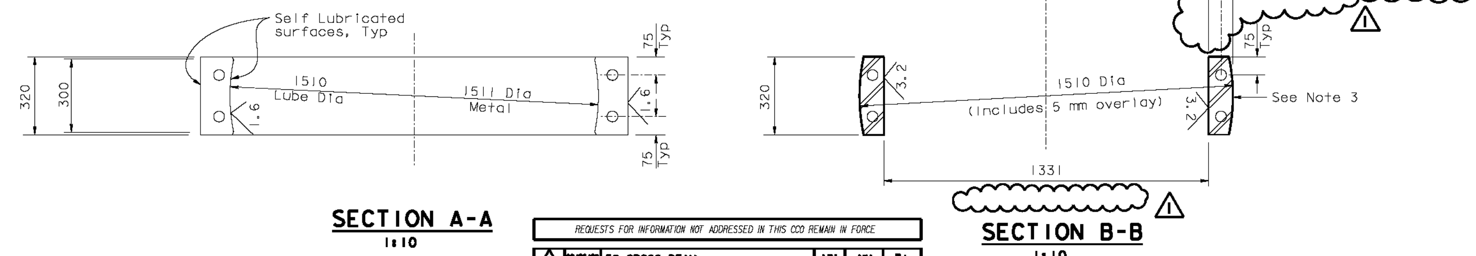

NOTES:

1. The spherical ring shall be Structural Casting Grade 345.
2. The spherical housing shall be high strength manganese bronze centrifugally cast to the requirements of B271-C86300. All assembly bolts shall be stainless steel A240 Type 316.
3. The mating surfaces of the spherical ring shall be 5 mm thick stainless steel weld overlay conforming to the requirements of A240, Type 316.
4. The lubricant shall be self-lubricated.
5. The spherical ring shall be "snug fit" over the shaft section of the shear key stub with a diameter gap of 1 mm Max.
6. At the Contractor's option, bolt hole location may be adjusted to accomodate seating of bolt nut and washer, subject to review and approval of the Engineer.

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MARK	DATE	DESCRIPTIONS	BY	CH'D	CCO#
1	09/26/08	E2 CROSS BEAM	MN	NV	71

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CONTRACT CHANGE ORDER NO. ____
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SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

PIER E2 SHEAR KEY DETAILS NO. 3

R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 09/26/08

DESIGN	BY M. Nader	CHECKED J. Denis
DETAILS	BY N. Vo	CHECKED J. Denis
QUANTITIES	BY N. Vo	CHECKED J. Denis

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

CU 04
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DISREGARD PRINTS BEARING EARLIER REVISION DATES
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Rev. Date: 5-18-98
ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

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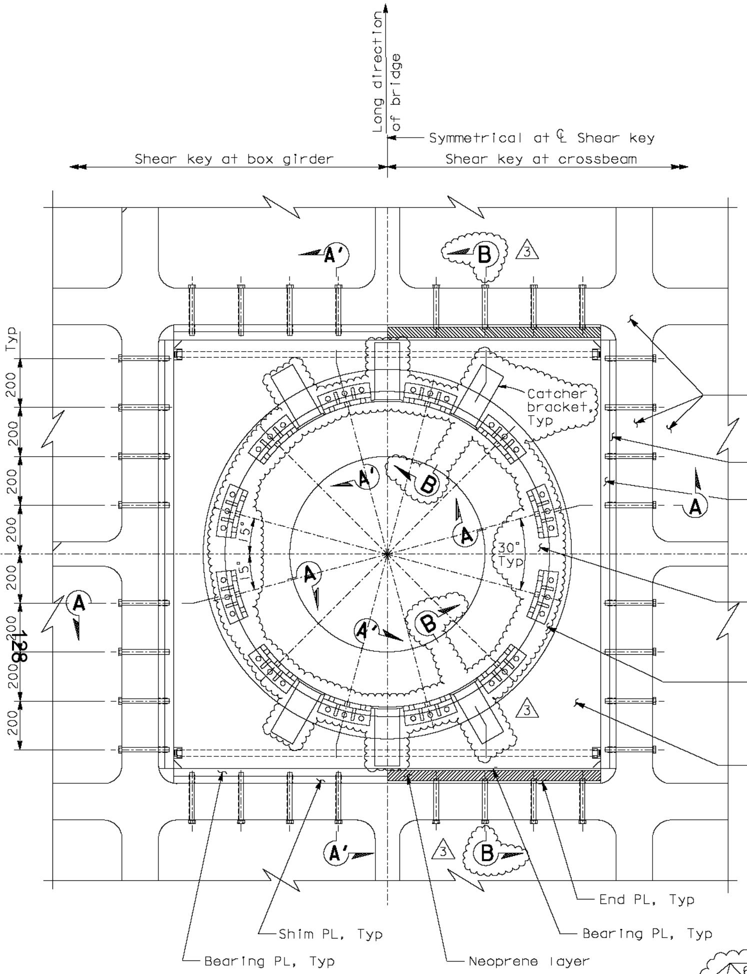
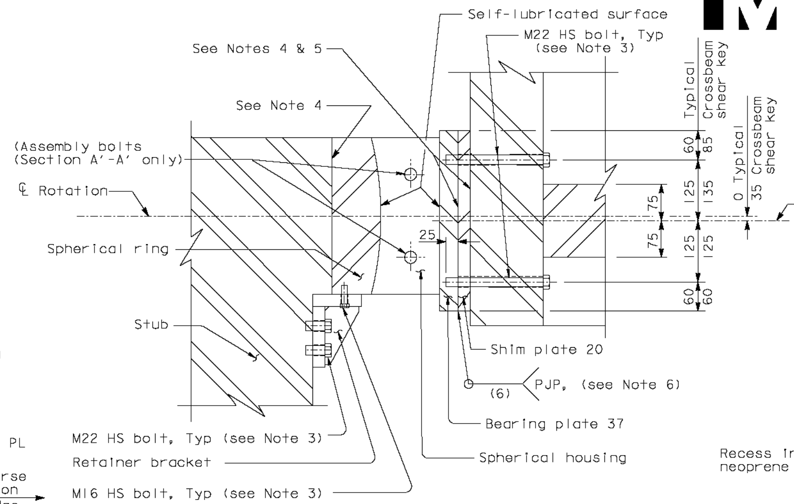


BUSHING CONNECTION DETAIL (Looking up)
1:10



SECTION A-A (Section A'-A' similar)
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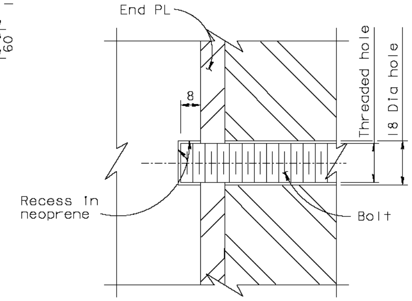
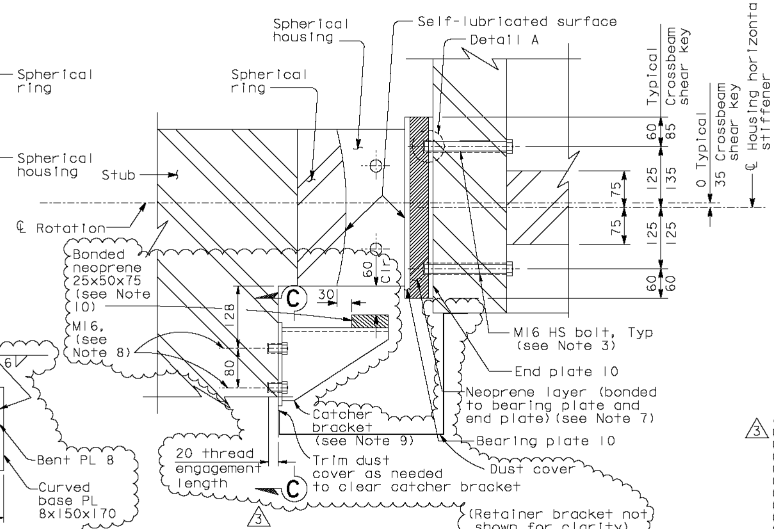


DETAIL A
1:1



SECTION B-B
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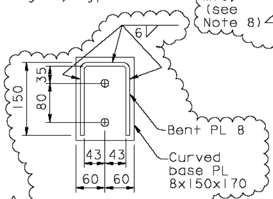

SECTION C-C
1:5

NOTES:

1. The mating surfaces of the spherical ring and bearing plate shall be 5 mm thick stainless steel weld overlay conforming to the requirements of A240, Type 316.
2. The lubricant shall be self-lubricated.
3. High strength bolts shall be installed snug-tight only.
4. Contact steel surfaces shall comply with Class A surface (slip coefficient 0.33).
5. The Contractor shall ensure the flatness requirements are met at the faying surfaces.
6. Welding shall be made after field measurement and actual shim plate thickness has been determined.
7. Field measurement shall be made to determine actual thickness of neoprene layer.
8. Bolts shall be installed snug-tight plus 1/4 turn.
9. Catcher brackets shall be installed for shear keys at crossbeam only.
10. Neoprene pad shall satisfy the requirements of Caltrans Standard Specifications Section 51-1.14.

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MARK	DATE	DESCRIPTIONS	BY	CH'D	CCO#
3	01/04/11	E2 SHEAR KEY CATCHER BRACKETS	MN	NV	71S1
2	02/13/09	E2 CROSS BEAM	MN	NV	71
1	09/26/08	E2 CROSS BEAM	MN	NV	71

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R. Valizadeh/V. Toan/Y.L./W.L./F.C.
DESIGN OVERSIGHT
SIGN OFF DATE 01/04/11

DESIGN	BY M. Nader	CHECKED L. Rus
DETAILS	BY A. Sanjines	CHECKED J. Leventini
QUANTITIES	BY A. Sanjines	CHECKED J. Leventini

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

R. Manzanarez
PROJECT ENGINEER

BRIDGE NO. 34-0006L/R
KILOMETER POST 13.2/13.9

SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE & TOWER)

PIER E2 SHEAR KEY DETAILS NO. 4

Rev. Date: 5-18-98
ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100
CU 04
EA 0120F1
DISREGARD PRINTS BEARING EARLIER REVISION DATES 12/07/05
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Appendix 4

Strut Sectional Checks

0 Specifications: **AASHTO LRFD Design Specs. 2012**

1 Material properties:

				Metric	
- Concrete density:	Wc =	0.15	kcf	24	KN/m3
- Concrete 28-day compressive strength:	f'c =	8	ksi	55	MPa
- Reinforcement strength:	fy =	60	ksi	414	MPa
- Prestressing steel strand:	Diameter =	0.60	in	15	mm
Strand cross-sectional area:	A =	0.217	in2	140	mm2
Modulus of Elasticity:	Ep =	28478	ksi	196500	MPa
Ultimate tensile strength:	Fpu =	58.43	kips	260	KN
	fpu =	269	ksi	1858	MPa
Minimum yield strength:	Fpy =	52.58	kips	234	KN
	fpy =	90%	fpu		
Maximum stressing force per strand:	Fp max =	43.82	kips	195	KN
	fp jacking =	75%	fpu		
% prestressing stress after transfer with Max losses:	fpt =	67%	fpu		
% Effective prestressing stress considering Max losses:	fpe =	63%	fpu		
Effective prestressing strain considering Max losses:	ε_{so} =	0.00592			
Wedge seating:	Δ =	0.39	in	10	mm
Friction coefficient:	μ =	0.15	/rad	0.15	/rad
Wobble coefficient:	k =	0.000201	/ft	0.00066	/m
- Tendons:					
Duct diameter:	OD =	5.51	in	140	mm
Location of tendon CG with respect to duct CG	Z =	0.47	in	12	mm

2 Bearing location:

- Eccentricity between shaft and top of pier cap:	e1 =	39.17	in	995	mm

3 Loads:

- Case — Maximum Bearing: Mux max

- Loading effects at the section of maximum effect:		PT + EQ + DL		PT + EQ + DL	
Axial (if <0 → Tension)	Pu =	37045	kips	165	MN
In-plane shear	Vuy =	15649	kips	70	MN
Out-of-plane shear	Vux =	0	kips	0	MN
In-plane-bending moment: (if >0 → Top fiber in tension)	Mux =	692959	kip-ft	941	MN-m
Out-of-plane bending moment	Muy =	2014	kip-ft	3	MN-m
Torsion	Tu =	0	kip-ft	0	MN-m

4 Pier cap section properties:

- Section No. 1 — Location = Bearing

- Dimensions:							
Location from cap end	x =	16.59	ft			5058	mm
Width	B =	175.3		167	in	4453	mm
Depth	H =	236.2		228	in	6000	mm
Concrete cover:	Top =	4.33	in			110	mm
	Bottom =	4.33	in			110	mm
	Lateral =	4.33	in			110	mm

- Section properties:		Gross	Eff at f'_{cu}	
Area	A =	41414	37924	in2
Centroid (with respect to left bottom corner)	yc =	118.110	114	in
	xc =	88	83	in
Moment of inertia	Ix =	192573235	163652090	in4
	Iy =	106074258	87775676	in4
Section modulus	Sx =	1630453	1438326	in3
	Sy =	1210084	1053377	in3

- *Longitudinal tendon locations:*

*from section centerline
**from bottom of gross section

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	α (rad)	x (mm)	y (mm)
1	B6.1	-54.09	22.60	36	7.81	0.25	-1374	492
2	B6.2	-25.75	22.60	37	8.03	0.25	-654	492

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	α (rad)	x (mm)	y (mm)
3	B6.3	18.03	22.60	37	8.03	0.25	458	492
4	B6.4	61.81	22.60	36	7.81	0.25	1570	492
5	B5.1	-52.83	28.19	36	7.81	0.43	-1342	634
6	B5.2	-24.49	28.19	37	8.03	0.43	-622	634
7	B5.3	16.77	28.19	37	8.03	0.43	426	634
8	B5.4	60.55	28.19	36	7.81	0.43	1538	634
9	B4.1	-53.46	35.59	37	8.03	0.48	-1358	822
10	B4.2	-25.12	35.59	37	8.03	0.48	-638	822
11	B4.3	17.40	35.59	37	8.03	0.48	442	822
12	B4.4	61.18	35.59	37	8.03	0.48	1554	822
13	T3.1	-54.09	206.02	36	7.81	0.21	-1374	5175
14	T3.2	-25.75	206.02	37	8.03	0.21	-654	5175
15	T3.3	3.86	206.02	37	8.03	0.21	98	5175
16	T3.4	63.11	206.02	36	7.81	0.21	1603	5175
17	T2.1	-39.29	211.61	37	8.03	0.12	-998	5317
18	T2.2	-10.94	211.61	37	8.03	0.12	-278	5317
19	T2.3	17.40	211.61	37	8.03	0.12	442	5317
20	T2.4	45.75	211.61	37	8.03	0.12	1162	5317
21	T1.1	-52.83	211.61	36	7.81	0.00	-1342	5317
22	T1.2	-24.49	211.61	37	8.03	0.00	-622	5317
23	T1.3	2.60	211.61	37	8.03	0.00	66	5317
24	T1.4	61.85	211.61	36	7.81	0.00	1571	5317
					190.96	0.25		
	Centroids =	-0.58	119.29 in					
	Eccentricity =	-0.58	1.18 in					

- *Longitudinal reinforcement locations:*

**from bottom of gross section

No.	Name	db (in)	y**(in)	Qty	Area (in2)	db (mm)	y (mm)
1	B1	2.24	6.44	16	63.28	57	164
2	B2	2.24	12.31	14	55.37	57	313
3	B3	2.24	18.19	14	55.37	57	462
4	B4	2.24	24.07	6	23.73	57	611
5	B5	2.24	29.95	6	23.73	57	761
6	B6	2.24	35.82	6	23.73	57	910
7	B7	2.24	41.70	6	23.73	57	1059
8	B8	2.24	47.58	14	55.37	57	1209
9	B9	2.24	53.46	14	55.37	57	1358
10	B10	2.24	59.33	14	55.37	57	1507
11	B11	2.24	65.21	14	55.37	57	1656
12	M1	1.69	71.09	6	13.51	43	1806
13	M2	1.69	76.97	6	13.51	43	1955
14	M3	1.69	82.85	6	13.51	43	2104
15	M4	1.69	88.72	6	13.51	43	2254
16	M5	1.69	94.60	6	13.51	43	2403
17	M6	1.69	100.48	6	13.51	43	2552
18	M7	1.69	106.36	6	13.51	43	2701
19	M8	1.69	112.23	6	13.51	43	2851
20	M9	1.69	118.11	6	13.51	43	3000
21	M10	1.69	123.99	6	13.51	43	3149
22	M11	1.69	129.87	6	13.51	43	3299
23	M12	1.69	135.74	6	13.51	43	3448
24	M13	1.69	141.62	6	13.51	43	3597
25	M14	1.69	147.50	6	13.51	43	3746
26	M15	1.69	153.38	6	13.51	43	3896
27	M16	1.69	159.25	6	13.51	43	4045
28	T1	2.24	165.13	13	51.42	57	4194
29	T2	2.24	171.01	5	19.78	57	4344
30	M17	1.69	176.89	6	13.51	43	4493
31	M18	1.69	182.76	6	13.51	43	4642
32	M19	1.69	188.64	6	13.51	43	4791
33	M20	1.69	194.52	6	13.51	43	4941
34	M21	1.69	200.40	6	13.51	43	5090
35	M22	1.69	206.27	6	13.51	43	5239
36	T3	2.24	212.15	20	79.10	57	5389

37	T4	2.24	218.03	20	79.10	57	5538
38	T5	2.24	223.91	19	75.15	57	5687
39	T6	2.24	229.78	19	75.15	57	5837
					1167.27		

5 Check for axial and flexural force effects:

- *Resistance factor:* A.5.10.11.4.1b ϕ = 0.90

- *Loading effects:*

1 Prestressing loading effects:	Pu =	32218	kips	143	MN
	Mux =	-3171	kip-ft	-4	MN-m
	Muy =	-1551	kip-ft	-2	MN-m
2 Net loading effects:	Pu =	4827	kips	21	MN
	Mux =	689788	kip-ft	936	MN-m
	Muy =	463	kip-ft	1	MN-m
3 Total loading stresses:					
	Pu/A =	0.895	ksi	6.2	MPa
	Mux/Sx =	5.100	ksi	35.2	MPa
	Muy/Sy =	0.020	ksi	0.1	MPa
	Max =	6.015	ksi	41.5	MPa
		75%	f'c	75%	
	Min =	-4.226	ksi	-29.2	MPa
		53%	f'c	53%	

- *Analysis approach:* *Strain compatibility*

- *Concrete behavior assumptions:* *Unconfined*

Stress distribution: A.5.7.2.2 *Rectangular*

α = 0.85

β = 0.65

Compressive strength: f'c = 8 ksi

Ultimate compression strain: ε_{cu} = 0.004 *

* Seismic Design of Reinforced Concrete and Masonry Buildings, Paulay and Priestly, pag. 98

Ignore concrete cover for calculations at this strain level

Modulus of Elasticity: A.5.4.2.4-1 E = 5422 ksi

Effective section properties: b = 167 in 4233 mm

h = 228 in 5780 mm

- *Major-axis bending resistance: Pnx, Mnx*

Eccentricity: ey = 1714.90 in

Neutral axis depth: c = 72.81 in

a = 47.32 in

Concrete forces: Pc = 53630 kips

Mc = 402752 kip-ft

Tendon forces:

Effective initial prestressing strain: ε_{so} = 0.00592

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B6.1	18.27	0.00293	83.39	651	-96	-5185
2	B6.2	18.27	0.00293	83.39	670	-96	-5329
3	B6.3	18.27	0.00293	83.39	670	-96	-5329
4	B6.4	18.27	0.00293	83.39	651	-96	-5185
5	B5.1	23.86	0.00324	92.13	720	-90	-5393
6	B5.2	23.86	0.00324	92.13	740	-90	-5543
7	B5.3	23.86	0.00324	92.13	740	-90	-5543

8	B5.4	23.86	0.00324	92.13	720	-90	-5393
9	B4.1	31.26	0.00364	103.71	833	-83	-5726
10	B4.2	31.26	0.00364	103.71	833	-83	-5726
11	B4.3	31.26	0.00364	103.71	833	-83	-5726
12	B4.4	31.26	0.00364	103.71	833	-83	-5726
13	T3.1	201.69	0.01301	262.59	2051	88	15028
14	T3.2	201.69	0.01301	262.59	2108	88	15446
15	T3.3	201.69	0.01301	262.59	2108	88	15446
16	T3.4	201.69	0.01301	262.59	2051	88	15028
17	T2.1	207.28	0.01331	262.91	2111	94	16448
18	T2.2	207.28	0.01331	262.91	2111	94	16448
19	T2.3	207.28	0.01331	262.91	2111	94	16448
20	T2.4	207.28	0.01331	262.91	2111	94	16448
21	T1.1	207.28	0.01331	262.91	2054	94	16004
22	T1.2	207.28	0.01331	262.91	2111	94	16448
23	T1.3	207.28	0.01331	262.91	2111	94	16448
24	T1.4	207.28	0.01331	262.91	2054	94	16004
					33984		125840

Reinforcement forces:

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B1	2.11	-0.00388	-60.00	-3797	-112	35335
2	B2	7.98	-0.00356	-60.00	-3322	-106	29291
3	B3	13.86	-0.00324	-60.00	-3322	-100	27664
4	B4	19.74	-0.00292	-60.00	-1424	-94	11159
5	B5	25.62	-0.00259	-60.00	-1424	-88	10461
6	B6	31.49	-0.00227	-60.00	-1424	-82	9764
7	B7	37.37	-0.00195	-56.46	-1340	-76	8531
8	B8	43.25	-0.00162	-47.09	-2608	-71	15327
9	B9	49.13	-0.00130	-37.73	-2089	-65	11256
10	B10	55.00	-0.00098	-28.36	-1571	-59	7693
11	B11	60.88	-0.00066	-19.00	-1052	-53	4637
12	M1	66.76	-0.00033	-9.63	-130	-47	510
13	M2	72.64	-0.00001	-0.27	-4	-41	12
14	M3	78.51	0.00031	9.09	123	-35	-361
15	M4	84.39	0.00064	18.46	249	-29	-611
16	M5	90.27	0.00096	27.82	376	-24	-736
17	M6	96.15	0.00128	37.19	502	-18	-738
18	M7	102.02	0.00161	46.55	629	-12	-616
19	M8	107.90	0.00193	55.92	755	-6	-370
20	M9	113.78	0.00225	60.00	810	0	0
21	M10	119.66	0.00257	60.00	810	6	397
22	M11	125.53	0.00290	60.00	810	12	794
23	M12	131.41	0.00322	60.00	810	18	1191
24	M13	137.29	0.00354	60.00	810	24	1588
25	M14	143.17	0.00387	60.00	810	29	1984
26	M15	149.04	0.00419	60.00	810	35	2381
27	M16	154.92	0.00451	60.00	810	41	2778
28	T1	160.80	0.00483	60.00	3085	47	12088
29	T2	166.68	0.00516	60.00	1187	53	5231
30	M17	172.55	0.00548	60.00	810	59	3969
31	M18	178.43	0.00580	60.00	810	65	4366
32	M19	184.31	0.00613	60.00	810	71	4763
33	M20	190.19	0.00645	60.00	810	76	5160
34	M21	196.07	0.00677	60.00	810	82	5557
35	M22	201.94	0.00709	60.00	810	88	5953
36	T3	207.82	0.00742	60.00	4746	94	37195
37	T4	213.70	0.00774	60.00	4746	100	39520
38	T5	219.58	0.00806	60.00	4509	106	39752
39	T6	225.45	0.00839	60.00	4509	112	41961
					13254		384836

Load Contour Method:

Mux/Mrx =	0.84				
Muy/Mry =	0.02				
IE =	**0.86**	**OK!**	**0.86**		

6 Check for shear and torsional force effects:

- *Resistance factor:* A.5.5.4.2 — ϕ = 0.90

- *Torsional effects:*

Total area enclosed by outside perimeter of concrete:	Acp =	41414	in2		
Outside perimeter of the concrete section:	pc =	823	in		
Diameter of exterior stirrups:	d =	0.98	in	25	mm
Spacing of exterior stirrups:	s =	5.91	in	150	mm
Area enclosed by the shear path:	Ao =	37537	in2		
Perimeter of centerline of exterior stirrups:	ph =	784	in		
Torsional cracking moment:	Tcr =	109830	kip-ft	149	MN-m
	0.25ϕTcr=	24712	kip-ft	34	MN-m
	Tu =	0	kip-ft	0	MN-m

Torsional effects DO NOT need to be considered for shear checking

Torsional resistance:	ϕTn=	60191	kip-ft	82	MN-m
	Tu/ϕTn =	**0.00**	**OK!**	**0.00**	

- *In-plane shear:*

Factored load:	Vuy =	15649	Kips	70	MN
Prestressing force in direction of Vu:	Vp =	2405	Kips	11	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2		
Effective width	bv =	175	in		
Minimum Shear depth:	dv =	189	in		
Net tensile Strains:	$\varepsilon_{average}$ =	0.00196			
Crack angle:	θ =	35.9			
Parameter β assuming min. transverse reinforcement:	β=	1.941			
In-plane concrete shear resistance:	Vc =	5743	Kips	26	MN
In-plane steel shear resistance:	Vs =	16145	Kips	72	MN
Inclination of reinforcement:	α=	90	deg		
Diameter of stirrups:	d =	0.98	in		
Spacing of stirrups:	s =	5.91	in		
No. of reinforcement legs:	n =	8			
	ϕVn =	19699	Kips	88	MN
	Vu/ϕVn =	**0.79**	**OK!**	**0.79**	

- *Out-of-plane shear:*

Factored load:	Vux =	0	Kips	0	MN
Prestressing force in direction of Vu:	Vp =	0	Kips	0	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2		
Effective width	bv =	236	in		
Minimum Shear depth:	dv =	126	in		
Net tensile Strains:	$\varepsilon_{average}$ =	0.00000			
Parameter β assuming min. transverse reinforcement:	β=	4.800			
In-plane concrete shear resistance:	Vc =	12792	Kips	57	MN
	ϕVc =	11513	Kips	51	MN
	Vu/ϕVc =	**0.00**	**OK!**	**0.00**	

Resultant forces:	Pnx =	6392	kips	28	MN
	Mnx =	913429	kip-ft	1239	MN-m
	ey =	1714.95	in		
	ey req/ ey res =	1.00		1.00	

- *Minor-axis bending resistance: Pny, Mny*

Eccentricity:	ex =	1.15	in
Neutral axis depth:	c =	254.97	in
	a =	165.73	in
Concrete forces:	Pc =	256452	kips
	Mc =	9887	kip-ft

Tendon forces:

Effective initial prestressing strain: ε_{po}= 0.00592

No.	Name	xeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B6.1	137.42	0.00408	116.20	908	54	4092
2	B6.2	109.08	0.00364	103.54	831	26	1784
3	B6.3	65.30	0.00295	83.98	674	-18	-1013
4	B6.4	21.52	0.00226	64.42	503	-62	-2592
5	B5.1	136.16	0.00406	115.64	903	53	3977
6	B5.2	107.82	0.00362	102.97	827	24	1687
7	B5.3	66.56	0.00297	84.54	679	-17	-949
8	B5.4	22.78	0.00228	64.98	508	-61	-2561
9	B4.1	136.79	0.00407	115.92	931	53	4147
10	B4.2	108.45	0.00363	103.25	829	25	1735
11	B4.3	65.93	0.00296	84.26	677	-17	-981
12	B4.4	22.15	0.00227	64.70	519	-61	-2648
13	T3.1	137.42	0.00408	116.20	908	54	4092
14	T3.2	109.08	0.00364	103.54	831	26	1784
15	T3.3	79.47	0.00317	90.31	725	-4	-233
16	T3.4	20.22	0.00224	63.84	499	-63	-2623
17	T2.1	122.62	0.00385	109.59	880	39	2881
18	T2.2	94.27	0.00340	96.92	778	11	710
19	T2.3	65.93	0.00296	84.26	677	-17	-981
20	T2.4	37.58	0.00251	71.59	575	-46	-2191
21	T1.1	136.16	0.00406	115.64	903	53	3977
22	T1.2	107.82	0.00362	102.97	827	24	1687
23	T1.3	80.73	0.00319	90.87	730	-3	-158
24	T1.4	21.48	0.00226	64.40	503	-62	-2593
					17624		13029

Resultant forces:	Pny =	238828	kips	1063	MN
	Mny =	22917	kip-ft	31	MN-m
	ex =	1.15	in		
	ex req/ ex res =	1.00		1.00	

- *Biaxial flexure check:* A.5.7.4.5

Minimum axial load to be considered as a column:	0.10ϕf'cAg =	29818	kips	133	MN
Factored axial load:	Pu =	4827	kips	21	MN
Nominal axial resistance:	Po =	246120	kips	1095	MN
Method to check biaxial flexure:		Load Contour Method			
Reciprocal Load Method:	Pr =	NA	Kips		
	Pu/Pr =	**NA**	**NA**	**NA**	

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	ε (rad)	x (mm)	y (mm)
5	B5.1	-52.83	28.19	36	7.81	0.43	-1342	634
6	B5.2	-24.49	28.19	37	8.03	0.43	-622	634
7	B5.3	16.77	28.19	37	8.03	0.43	426	634
8	B5.4	60.55	28.19	36	7.81	0.43	1538	634
9	B4.1	-53.46	35.59	37	8.03	0.48	-1358	822
10	B4.2	-25.12	35.59	37	8.03	0.48	-638	822
11	B4.3	17.40	35.59	37	8.03	0.48	442	822
12	B4.4	61.18	35.59	37	8.03	0.48	1554	822
13	T3.1	-54.09	206.02	36	7.81	0.21	-1374	5175
14	T3.2	-25.75	206.02	37	8.03	0.21	-654	5175
15	T3.3	3.86	206.02	37	8.03	0.21	98	5175
16	T3.4	63.11	206.02	36	7.81	0.21	1603	5175
17	T2.1	-39.29	211.61	37	8.03	0.12	-998	5317
18	T2.2	-10.94	211.61	37	8.03	0.12	-278	5317
19	T2.3	17.40	211.61	37	8.03	0.12	442	5317
20	T2.4	45.75	211.61	37	8.03	0.12	1162	5317
21	T1.1	-52.83	211.61	36	7.81	0.00	-1342	5317
22	T1.2	-24.49	211.61	37	8.03	0.00	-622	5317
23	T1.3	2.60	211.61	37	8.03	0.00	66	5317
24	T1.4	61.85	211.61	36	7.81	0.00	1571	5317
					190.96	0.25		
	Centroids =	-0.58	119.29 in					
	Eccentricity =	-0.58	1.18 in					

- *Longitudinal reinforcement locations:*

**from bottom of gross section

No.	Name	db (in)	y**(in)	Qty	Area (in2)	db (mm)	y (mm)
1	B1	2.24	6.44	16	63.28	57	164
2	B2	2.24	12.31	14	55.37	57	313
3	B3	2.24	18.19	14	55.37	57	462
4	B4	2.24	24.07	6	23.73	57	611
5	B5	2.24	29.95	6	23.73	57	761
6	B6	2.24	35.82	6	23.73	57	910
7	B7	2.24	41.70	6	23.73	57	1059
8	B8	2.24	47.58	14	55.37	57	1209
9	B9	2.24	53.46	14	55.37	57	1358
10	B10	2.24	59.33	14	55.37	57	1507
11	B11	2.24	65.21	14	55.37	57	1656
12	M1	1.69	71.09	6	13.51	43	1806
13	M2	1.69	76.97	6	13.51	43	1955
14	M3	1.69	82.85	6	13.51	43	2104
15	M4	1.69	88.72	6	13.51	43	2254
16	M5	1.69	94.60	6	13.51	43	2403
17	M6	1.69	100.48	6	13.51	43	2552
18	M7	1.69	106.36	6	13.51	43	2701
19	M8	1.69	112.23	6	13.51	43	2851
20	M9	1.69	118.11	6	13.51	43	3000
21	M10	1.69	123.99	6	13.51	43	3149
22	M11	1.69	129.87	6	13.51	43	3299
23	M12	1.69	135.74	6	13.51	43	3448
24	M13	1.69	141.62	6	13.51	43	3597
25	M14	1.69	147.50	6	13.51	43	3746
26	M15	1.69	153.38	6	13.51	43	3896
27	M16	1.69	159.25	6	13.51	43	4045
28	T1	2.24	165.13	13	51.42	57	4194
29	T2	2.24	171.01	5	19.78	57	4344
30	M17	1.69	176.89	6	13.51	43	4493
31	M18	1.69	182.76	6	13.51	43	4642
32	M19	1.69	188.64	6	13.51	43	4791
33	M20	1.69	194.52	6	13.51	43	4941
34	M21	1.69	200.40	6	13.51	43	5090
35	M22	1.69	206.27	6	13.51	43	5239
36	T3	2.24	212.15	20	79.10	57	5389
37	T4	2.24	218.03	20	79.10	57	5538
38	T5	2.24	223.91	19	75.15	57	5687
39	T6	2.24	229.78	19	75.15	57	5837
					1167.27		

0 Specifications: — **AASHTO LRFD Design Specs. 2012**

1 Material properties:

		Value	Unit	Metric	Unit
- Concrete density:	Wc =	0.15	kcf	24	KN/m3
- Concrete 28-day compressive strength:	f'c =	8	ksi	55	MPa
- Reinforcement strength:	fy =	60	ksi	414	MPa
- Prestressing steel strand:	Diameter =	0.60	in	15	mm
Strand cross-sectional area:	A =	0.217	in2	140	mm2
Modulus of Elasticity:	Ep =	28478	ksi	196500	MPa
Ultimate tensile strength:	Fpu =	58.43	kips	260	KN
	fpu =	269	ksi	1858	MPa
Minimum yield strength:	Fpy =	52.58	kips	234	KN
	fpy =	90%	fpu		
Maximum stressing force per strand:	Fp max =	43.82	kips	195	KN
	fp jacking =	75%	fpu		
% prestressing stress after transfer with Max losses:	fpt =	67%	fpu		
% Effective prestressing stress considering Max losses:	fpe =	63%	fpu		
Effective prestressing strain considering Max losses:	ε_{po} =	0.00592			
Wedge seating:	Δ =	0.39	in	10	mm
Friction coefficient:	μ =	0.15	/rad	0.15	/rad
Wobble coefficient:	k =	0.000201	/ft	0.00066	/m
- Tendons:					
Duct diameter:	OD =	5.51	in	140	mm
Location of tendon CG with respect to duct CG	Z =	0.47	in	12	mm

2 Bearing location:

		Value	Unit	Metric	Unit
- Eccentricity between shaft and top of pier cap:	e1 =	39.17	in	995	mm

3 Loads:

- Case — Maximum Bearing: Mux min

- Loading effects at the section of maximum effect:

		PT + EQ + DL	Unit	PT + EQ + DL	Unit
Axial (if <0 → Tension)	Pu =	27908	kips	124	MN
In-plane shear	Vuy =	11341	kips	50	MN
Out-of-plane shear	Vux =	0	kips	0	MN
In-plane-bending moment: (if >0 → Top fiber in tension)	Mux =	-671357	kip-ft	-912	MN-m
Out-of-plane bending moment	Muy =	2014	kip-ft	3	MN-m
Torsion	Tu =	0	kip-ft	0	MN-m

4 Pier cap section properties:

- Section No. 1 — Location = Bearing

- Dimensions:

		Value	Unit		Unit	Metric	Unit
Location from cap end	x =	16.59	ft			5058	mm
Width	B =	175.3		167	in	4453	mm
Depth	H =	236.2		228	in	6000	mm
Concrete cover:	Top =	4.33	in			110	mm
	Bottom =	4.33	in			110	mm
	Lateral =	4.33	in			110	mm

- Section properties:

		Gross	Eff at ε_{cu}	Unit
Area	A =	41414	37924	in2
Centroid (with respect to left bottom corner)	yc =	118.110	114	in
	xc =	88	83	in
Moment of inertia	Ix =	192573235	163652090	in4
	Iy =	106074258	87775676	in4
Section modulus	Sx =	1630453	1438326	in3
	Sy =	1210084	1053377	in3

- Longitudinal tendon locations:

*from section centerline
**from bottom of gross section

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	ε (rad)	x (mm)	y (mm)
1	B6.1	-54.09	22.60	36	7.81	0.25	-1374	492
2	B6.2	-25.75	22.60	37	8.03	0.25	-654	492
3	B6.3	18.03	22.60	37	8.03	0.25	458	492
4	B6.4	61.81	22.60	36	7.81	0.25	1570	492

5 Check for axial and flexural force effects:

- *Resistance factor:* A.5.10.11.4.1b ϕ = 0.90

- *Loading effects:*

1 Prestressing loading effects:	Pu =	32218	kips	143	MN
	Mux =	-3171	kip-ft	-4	MN-m
	Muy =	-1551	kip-ft	-2	MN-m
2 Net loading effects:	Pu =	27908	kips	124	MN
	Mux =	-671357	kip-ft	-911	MN-m
	Muy =	2014	kip-ft	3	MN-m
3 Total loading stresses:	Pu/A =	0.674	ksi	4.6	MPa
	Mux/Sx =	-4.941	ksi	-34.1	MPa
	Muy/Sy =	0.020	ksi	0.1	MPa
	Max =	-4.247	ksi	-29.3	MPa
		-53%	f'c	-53%	
	Min =	5.595	ksi	38.6	MPa
		70%	f'c	70%	

- *Analysis approach:* *Strain compatibility*

- *Concrete behavior assumptions:* *Unconfined*

Stress distribution: A.5.7.2.2 *Rectangular*
α = 0.85
β = 0.65
Compressive strength: f'c = 8 ksi
Ultimate compression strain: ε_{cu} = 0.004 *

* Seismic Design of Reinforced Concrete and Masonry Buildings, Paulay and Priestly, pag. 98
Ignore concrete cover for calculations at this strain level

Modulus of Elasticity: A.5.4.2.4-1 E = 5422 ksi
Effective section properties: b = 167 in 4233 mm
h = 228 in 5780 mm

- *Major-axis bending resistance: Pnx, Mnx*

Eccentricity: ey = -288.68 in
Neutral axis depth: c = 75.36 in
a = 48.98 in

Concrete forces: Pc = 55509 kips
Mc = 413025 kip-ft

Tendon forces:
Effective initial prestressing strain: ε_{po} = 0.00592

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B6.1	18.27	0.01303	262.62	2052	96	16329
2	B6.2	18.27	0.01303	262.62	2109	96	16783
3	B6.3	18.27	0.01303	262.62	2109	96	16783
4	B6.4	18.27	0.01303	262.62	2052	96	16329
5	B5.1	23.86	0.01274	262.28	2049	90	15353
6	B5.2	23.86	0.01274	262.28	2106	90	15780
7	B5.3	23.86	0.01274	262.28	2106	90	15780
8	B5.4	23.86	0.01274	262.28	2049	90	15353
9	B4.1	31.26	0.01234	261.76	2102	83	14453
10	B4.2	31.26	0.01234	261.76	2102	83	14453
11	B4.3	31.26	0.01234	261.76	2102	83	14453
12	B4.4	31.26	0.01234	261.76	2102	83	14453
13	T3.1	201.69	0.00330	93.90	734	-88	-5374

14	T3.2	201.69	0.00330	93.90	754	-88	-5524
15	T3.3	201.69	0.00330	93.90	754	-88	-5524
16	T3.4	201.69	0.00330	93.90	734	-88	-5374
17	T2.1	207.28	0.00300	85.45	686	-94	-5346
18	T2.2	207.28	0.00300	85.45	686	-94	-5346
19	T2.3	207.28	0.00300	85.45	686	-94	-5346
20	T2.4	207.28	0.00300	85.45	686	-94	-5346
21	T1.1	207.28	0.00300	85.45	668	-94	-5202
22	T1.2	207.28	0.00300	85.45	686	-94	-5346
23	T1.3	207.28	0.00300	85.45	686	-94	-5346
24	T1.4	207.28	0.00300	85.45	668	-94	-5202
					33463		122025

Reinforcement forces:

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B1	2.11	0.00797	60.00	3797	112	35335
2	B2	7.98	0.00766	60.00	3322	106	29291
3	B3	13.86	0.00734	60.00	3322	100	27664
4	B4	19.74	0.00703	60.00	1424	94	11159
5	B5	25.62	0.00672	60.00	1424	88	10461
6	B6	31.49	0.00641	60.00	1424	82	9764
7	B7	37.37	0.00610	60.00	1424	76	9066
8	B8	43.25	0.00578	60.00	3322	71	19528
9	B9	49.13	0.00547	60.00	3322	65	17900
10	B10	55.00	0.00516	60.00	3322	59	16273
11	B11	60.88	0.00485	60.00	3322	53	14646
12	M1	66.76	0.00454	60.00	810	47	3175
13	M2	72.64	0.00422	60.00	810	41	2778
14	M3	78.51	0.00391	60.00	810	35	2381
15	M4	84.39	0.00360	60.00	810	29	1984
16	M5	90.27	0.00329	60.00	810	24	1588
17	M6	96.15	0.00298	60.00	810	18	1191
18	M7	102.02	0.00266	60.00	810	12	794
19	M8	107.90	0.00235	60.00	810	6	397
20	M9	113.78	0.00204	59.15	799	0	0
21	M10	119.66	0.00173	50.10	677	-6	-331
22	M11	125.53	0.00142	41.05	554	-12	-543
23	M12	131.41	0.00110	32.00	432	-18	-635
24	M13	137.29	0.00079	22.96	310	-24	-607
25	M14	143.17	0.00048	13.91	188	-29	-460
26	M15	149.04	0.00017	4.86	66	-35	-193
27	M16	154.92	-0.00014	-4.19	-57	-41	194
28	T1	160.80	-0.00046	-13.23	-680	-47	2666
29	T2	166.68	-0.00077	-22.28	-441	-53	1942
30	M17	172.55	-0.00108	-31.33	-423	-59	2072
31	M18	178.43	-0.00139	-40.38	-545	-65	2938
32	M19	184.31	-0.00170	-49.42	-667	-71	3923
33	M20	190.19	-0.00202	-58.47	-790	-76	5028
34	M21	196.07	-0.00233	-60.00	-810	-82	5557
35	M22	201.94	-0.00264	-60.00	-810	-88	5953
36	T3	207.82	-0.00295	-60.00	-4746	-94	37195
37	T4	213.70	-0.00326	-60.00	-4746	-100	39520
38	T5	219.58	-0.00358	-60.00	-4509	-106	39752
39	T6	225.45	-0.00389	-60.00	-4509	-112	41961
					15201		401308

Resultant forces:

Pnx =	39063	kips	174	MN
Mnx =	939529	kip-ft	1274	MN-m
ey =	-288.62	in		
ey req/ ey res =	1.00		1.00	

6 Check for shear and torsional force effects:

- *Resistance factor:* A.5.5.4.2	ϕ =	0.90				
- *Torsional effects:*						
Total area enclosed by outside perimeter of concrete:	Acp =	41414	in2			
Outside perimeter of the concrete section:	pc =	823	in			
Diameter of exterior stirrups:	d =	0.98	in		25	mm
Spacing of exterior stirrups:	s =	5.91	in		150	mm
Area enclosed by the shear path:	Ao =	37537	in2			
Perimeter of centerline of exterior stirrups:	ph =	784	in			
Torsional cracking moment:	Tcr =	109830	kip-ft		149	MN-m
	0.25ϕTcr=	24712	kip-ft		34	MN-m
	Tu =	0	kip-ft		0	MN-m
Torsional effects DO NOT need to be considered for shear checking						
Torsional resistance:	ϕTn=	60279	kip-ft		82	MN-m
	Tu/ϕTn =	0.00	OK!		0.00	
- *In-plane shear:*						
Factored load:	Vuy =	11341	Kips		50	MN
Prestressing force in direction of Vu:	Vp =	2405	Kips		11	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2			
Effective width	bv =	175	in			
Minimum Shear depth:	dv =	187	in			
Net tensile Strains:	$\varepsilon_{average}$ =	0.00195				
Crack angle:	θ =	35.8				
Parameter β assuming min. transverse reinforcement:	β=	1.948				
In-plane concrete shear resistance:	Vc =	5698	Kips		25	MN
In-plane steel shear resistance:	Vs =	15986	Kips		71	MN
Inclination of reinforcement:	α=	90	deg			
Diameter of stirrups:	d =	0.98	in			
Spacing of stirrups:	s =	5.91	in			
No. of reinforcement legs:	n =	8				
	ϕVn =	19516	Kips		87	MN
	Vu/ϕVn =	0.58	OK!		0.58	
- *Out-of-plane shear:*						
Factored load:	Vux =	0	Kips		0	MN
Prestressing force in direction of Vu:	Vp =	0	Kips		0	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2			
Effective width	bv =	236	in			
Minimum Shear depth:	dv =	126	in			
Net tensile Strains:	$\varepsilon_{average}$ =	0.00000				
Parameter β assuming min. transverse reinforcement:	β=	4.800				
In-plane concrete shear resistance:	Vc =	12792	Kips		57	MN
	ϕVc =	11513	Kips		51	MN
	Vu/ϕVc =	0.00	OK!		0.00	

- *Minor-axis bending resistance: Pny, Mny*

Eccentricity:	ex =	0.87	in		
Neutral axis depth:	c =	255.67	in		
	a =	166.18	in		
Concrete forces:	Pc =	257151	kips		
	Mc =	5069	kip-ft		
Tendon forces:					
Effective initial prestressing strain:	ε_{po}=	0.00592			

No.	Name	xeff (in)	ε_{si}	fsi (ksi)	Tsi (kips)	di (in)	Msi (kip-ft)
1	B6.1	137.42	0.00407	116.03	906	54	4086
2	B6.2	109.08	0.00363	103.40	830	26	1781
3	B6.3	65.30	0.00295	83.90	674	-18	-1012
4	B6.4	21.52	0.00226	64.39	503	-62	-2591
5	B5.1	136.16	0.00405	115.47	902	53	3972
6	B5.2	107.82	0.00361	102.84	826	24	1685
7	B5.3	66.56	0.00297	84.46	678	-17	-948
8	B5.4	22.78	0.00228	64.95	507	-61	-2560
9	B4.1	136.79	0.00406	115.75	929	53	4141
10	B4.2	108.45	0.00362	103.12	828	25	1733
11	B4.3	65.93	0.00296	84.18	676	-17	-980
12	B4.4	22.15	0.00227	64.67	519	-61	-2647
13	T3.1	137.42	0.00407	116.03	906	54	4086
14	T3.2	109.08	0.00363	103.40	830	26	1781
15	T3.3	79.47	0.00317	90.21	724	-4	-233
16	T3.4	20.22	0.00224	63.81	498	-63	-2622
17	T2.1	122.62	0.00384	109.44	879	39	2877
18	T2.2	94.27	0.00340	96.81	777	11	709
19	T2.3	65.93	0.00296	84.18	676	-17	-980
20	T2.4	37.58	0.00251	71.55	574	-46	-2190
21	T1.1	136.16	0.00405	115.47	902	53	3972
22	T1.2	107.82	0.00361	102.84	826	24	1685
23	T1.3	80.73	0.00319	90.77	729	-3	-158
24	T1.4	21.48	0.00226	64.37	503	-62	-2592
					17604		12995

Resultant forces:	Pny =	271765	kips		1209	MN
	Mny =	19615	kip-ft		27	MN-m
	ex =	0.87	in			
	ex req/ ex res =	1.00			1.00	
- *Biaxial flexure check:* A.5.7.4.5						
Minimum axial load to be considered as a column:	0.1ϕf'cAg =	29818	kips		133	MN
Factored axial load:	Pu =	27908	kips		124	MN
Nominal axial resistance:	Po =	267050	kips		1188	MN
Method to check biaxial flexure:		Load Contour Method				
Reciprocal Load Method:	Pr =	NA	Kips			
	Pu/Pr =	NA	NA		NA	
Load Contour Method:	Mux/Mrx =	0.79				
	Muy/Mry =	0.11				
	IE =	0.91	OK!		0.91	

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	(rad)	x (mm)	y (mm)
5	B5.1	-52.83	28.19	36	7.81	0.43	-1342	634
6	B5.2	-24.49	28.19	37	8.03	0.43	-622	634
7	B5.3	16.77	28.19	37	8.03	0.43	426	634
8	B5.4	60.55	28.19	36	7.81	0.43	1538	634
9	B4.1	-53.46	35.59	37	8.03	0.48	-1358	822
10	B4.2	-25.12	35.59	37	8.03	0.48	-638	822
11	B4.3	17.40	35.59	37	8.03	0.48	442	822
12	B4.4	61.18	35.59	37	8.03	0.48	1554	822
13	T3.1	-54.09	206.02	36	7.81	0.21	-1374	5175
14	T3.2	-25.75	206.02	37	8.03	0.21	-654	5175
15	T3.3	3.86	206.02	37	8.03	0.21	98	5175
16	T3.4	63.11	206.02	36	7.81	0.21	1603	5175
17	T2.1	-39.29	211.61	37	8.03	0.12	-998	5317
18	T2.2	-10.94	211.61	37	8.03	0.12	-278	5317
19	T2.3	17.40	211.61	37	8.03	0.12	442	5317
20	T2.4	45.75	211.61	37	8.03	0.12	1162	5317
21	T1.1	-52.83	211.61	36	7.81	0.00	-1342	5317
22	T1.2	-24.49	211.61	37	8.03	0.00	-622	5317
23	T1.3	2.60	211.61	37	8.03	0.00	66	5317
24	T1.4	61.85	211.61	36	7.81	0.00	1571	5317
					190.96	0.25		
	Centroids =	-0.58	119.29 in					
	Eccentricity =	-0.58	1.18 in					

- *Longitudinal reinforcement locations:*

**from bottom of gross section

No.	Name	db (in)	y**(in)	Qty	Area (in2)	db (mm)	y (mm)
1	B1	2.24	6.44	16	63.28	57	164
2	B2	2.24	12.31	14	55.37	57	313
3	B3	2.24	18.19	14	55.37	57	462
4	B4	2.24	24.07	6	23.73	57	611
5	B5	2.24	29.95	6	23.73	57	761
6	B6	2.24	35.82	6	23.73	57	910
7	B7	2.24	41.70	6	23.73	57	1059
8	B8	2.24	47.58	14	55.37	57	1209
9	B9	2.24	53.46	14	55.37	57	1358
10	B10	2.24	59.33	14	55.37	57	1507
11	B11	2.24	65.21	14	55.37	57	1656
12	M1	1.69	71.09	6	13.51	43	1806
13	M2	1.69	76.97	6	13.51	43	1955
14	M3	1.69	82.85	6	13.51	43	2104
15	M4	1.69	88.72	6	13.51	43	2254
16	M5	1.69	94.60	6	13.51	43	2403
17	M6	1.69	100.48	6	13.51	43	2552
18	M7	1.69	106.36	6	13.51	43	2701
19	M8	1.69	112.23	6	13.51	43	2851
20	M9	1.69	118.11	6	13.51	43	3000
21	M10	1.69	123.99	6	13.51	43	3149
22	M11	1.69	129.87	6	13.51	43	3299
23	M12	1.69	135.74	6	13.51	43	3448
24	M13	1.69	141.62	6	13.51	43	3597
25	M14	1.69	147.50	6	13.51	43	3746
26	M15	1.69	153.38	6	13.51	43	3896
27	M16	1.69	159.25	6	13.51	43	4045
28	T1	2.24	165.13	13	51.42	57	4194
29	T2	2.24	171.01	5	19.78	57	4344
30	M17	1.69	176.89	6	13.51	43	4493
31	M18	1.69	182.76	6	13.51	43	4642
32	M19	1.69	188.64	6	13.51	43	4791
33	M20	1.69	194.52	6	13.51	43	4941
34	M21	1.69	200.40	6	13.51	43	5090
35	M22	1.69	206.27	6	13.51	43	5239
36	T3	2.24	212.15	20	79.10	57	5389
37	T4	2.24	218.03	20	79.10	57	5538
38	T5	2.24	223.91	19	75.15	57	5687
39	T6	2.24	229.78	19	75.15	57	5837
					1167.27		

0 Specifications: **AASHTO LRFD Design Specs. 2012**

1 Material properties:

				Metric	
- Concrete density:	Wc =	0.15	kcf	24	KN/m3
- Concrete 28-day compressive strength:	f'c =	8	ksi	55	MPa
- Reinforcement strength:	fy =	60	ksi	414	MPa
- Prestressing steel strand:	Diameter =	0.60	in	15	mm
Strand cross-sectional area:	A =	0.217	in2	140	mm2
Modulus of Elasticity:	Ep =	28478	ksi	196500	MPa
Ultimate tensile strength:	Fpu =	58.43	kips	260	KN
	fpu =	269	ksi	1858	MPa
Minimum yield strength:	Fpy =	52.58	kips	234	KN
	fpy =	90%	fpu		
Maximum stressing force per strand:	Fp max =	43.82	kips	195	KN
	fp jacking =	75%	fpu		
% prestressing stress after transfer with Max losses:	fpt =	67%	fpu		
% Effective prestressing stress considering Max losses:	fpe =	63%	fpu		
Effective prestressing strain considering Max losses:	ε_{po} =	0.00592			
Wedge seating:	Λ =	0.39	in	10	mm
Friction coefficient:	μ =	0.15	/rad	0.15	/rad
Wobble coefficient:	k =	0.000201	/ft	0.00066	/m
- Tendons:					
Duct diameter:	OD =	5.51	in	140	mm
Location of tendon CG with respect to duct CG	Z =	0.47	in	12	mm

2 Bearing location:

- Eccentricity between shaft and top of pier cap:	e1 =	39.17	in	995	mm

3 Loads:

- Case: Maximum Bearing: Pu min

		PT + EQ + DL		PT + EQ + DL	
- *Loading effects at the section of maximum effect:*					
Axial (if <0 → Tension)	Pu =	46182	kips	206	MN
In-plane shear	Vuy =	15932	kips	71	MN
Out-of-plane shear	Vux =	0	kips	0	MN
In-plane-bending moment: (if >0 → Top fiber in tension)	Mux =	588677	kip-ft	799	MN-m
Out-of-plane bending moment	Muy =	2014	kip-ft	3	MN-m
Torsion	Tu =	0	kip-ft	0	MN-m

4 Pier cap section properties:

- Section No.		1					
	Location =	Bearing					
- *Dimensions:*							
Location from cap end	x =	16.59	ft			5058	mm
Width	B =	175.3		167	in	4453	mm
Depth	H =	236.2		228	in	6000	mm
Concrete cover:	Top =	4.33	in			110	mm
	Bottom =	4.33	in			110	mm
	Lateral =	4.33	in			110	mm

- *Section properties:*		Gross	Eff at ε_{cu}	
Area	A =	41414	37924	in2
Centroid (with respect to left bottom corner)	yc =	118.110	114	in
	xc =	88	83	in
Moment of inertia	Ix =	192573235	163652090	in4
	Iy =	106074258	87775676	in4
Section modulus	Sx =	1630453	1438326	in3
	Sy =	1210084	1053377	in3

- *Longitudinal tendon locations:*

*from section centerline
**from bottom of gross section

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	ε_{γ} (rad)	x (mm)	y (mm)
1	B6.1	-54.09	22.60	36	7.81	0.25	-1374	492
2	B6.2	-25.75	22.60	37	8.03	0.25	-654	492
3	B6.3	18.03	22.60	37	8.03	0.25	458	492
4	B6.4	61.81	22.60	36	7.81	0.25	1570	492

5 Check for axial and flexural force effects:

- *Resistance factor:* A.5.10.11.4.1b ϕ = 0.90

- *Loading effects:*

1 Prestressing loading effects:	Pu =	32218	kips	143	MN
	Mux =	-3171	kip-ft	-4	MN-m
	Muy =	-1551	kip-ft	-2	MN-m
2 Net loading effects:	Pu =	13964	kips	62	MN
	Mux =	585506	kip-ft	794	MN-m
	Muy =	463	kip-ft	1	MN-m
3 Total loading stresses:	Pu/A =	1.115	ksi	7.7	MPa
	Mux/Sx =	4.333	ksi	29.9	MPa
	Muy/Sy =	0.020	ksi	0.1	MPa
	Max =	5.468	ksi	37.7	MPa
		68%	f'c	68%	
	Min =	-3.237	ksi	-22.3	MPa
		40%	f'c	40%	

- *Analysis approach:* *Strain compatibility*

- *Concrete behavior assumptions:* *Unconfined*

Stress distribution: A.5.7.2.2 *Rectangular*

α = 0.85

β = 0.65

Compressive strength: f'c = 8 ksi

Ultimate compression strain: ε_{cu} = 0.004 *

* Seismic Design of Reinforced Concrete and Masonry Buildings, Paulay and Priestly, pag. 98

Ignore concrete cover for calculations at this strain level

Modulus of Elasticity: A.5.4.2.4-1 E = 5422 ksi

Effective section properties: b = 167 in 4233 mm

h = 228 in 5780 mm

- *Major-axis bending resistance: Pnx, Mnx*

Eccentricity: ey = 503.15 in

Neutral axis depth: c = 86.71 in

a = 56.36 in

Concrete forces: Pc = 63872 kips

Mc = 455612 kip-ft

Tendon forces:

Effective initial prestressing strain: ε_{po} = 0.00592

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B6.1	18.27	0.00277	78.80	616	-96	-4900
2	B6.2	18.27	0.00277	78.80	633	-96	-5036
3	B6.3	18.27	0.00277	78.80	633	-96	-5036
4	B6.4	18.27	0.00277	78.80	616	-96	-4900
5	B5.1	23.86	0.00303	86.15	673	-90	-5043
6	B5.2	23.86	0.00303	86.15	692	-90	-5183
7	B5.3	23.86	0.00303	86.15	692	-90	-5183
8	B5.4	23.86	0.00303	86.15	673	-90	-5043
9	B4.1	31.26	0.00337	95.87	770	-83	-5293
10	B4.2	31.26	0.00337	95.87	770	-83	-5293
11	B4.3	31.26	0.00337	95.87	770	-83	-5293
12	B4.4	31.26	0.00337	95.87	770	-83	-5293
13	T3.1	201.69	0.01123	259.79	2029	88	14868

14	T3.2	201.69	0.01123	259.79	2086	88	15281
15	T3.3	201.69	0.01123	259.79	2086	88	15281
16	T3.4	201.69	0.01123	259.79	2029	88	14868
17	T2.1	207.28	0.01149	260.33	2090	94	16287
18	T2.2	207.28	0.01149	260.33	2090	94	16287
19	T2.3	207.28	0.01149	260.33	2090	94	16287
20	T2.4	207.28	0.01149	260.33	2090	94	16287
21	T1.1	207.28	0.01149	260.33	2034	94	15847
22	T1.2	207.28	0.01149	260.33	2090	94	16287
23	T1.3	207.28	0.01149	260.33	2090	94	16287
24	T1.4	207.28	0.01149	260.33	2034	94	15847
					33144		128218

Reinforcement forces:

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B1	2.11	-0.00390	-60.00	-3797	-112	35335
2	B2	7.98	-0.00363	-60.00	-3322	-106	29291
3	B3	13.86	-0.00336	-60.00	-3322	-100	27664
4	B4	19.74	-0.00309	-60.00	-1424	-94	11159
5	B5	25.62	-0.00282	-60.00	-1424	-88	10461
6	B6	31.49	-0.00255	-60.00	-1424	-82	9764
7	B7	37.37	-0.00228	-60.00	-1424	-76	9066
8	B8	43.25	-0.00200	-58.14	-3219	-71	18923
9	B9	49.13	-0.00173	-50.28	-2784	-65	15000
10	B10	55.00	-0.00146	-42.42	-2349	-59	11504
11	B11	60.88	-0.00119	-34.55	-1913	-53	8434
12	M1	66.76	-0.00092	-26.69	-360	-47	1412
13	M2	72.64	-0.00065	-18.83	-254	-41	872
14	M3	78.51	-0.00038	-10.96	-148	-35	435
15	M4	84.39	-0.00011	-3.10	-42	-29	103
16	M5	90.27	0.00016	4.76	64	-24	-126
17	M6	96.15	0.00044	12.63	171	-18	-251
18	M7	102.02	0.00071	20.49	277	-12	-271
19	M8	107.90	0.00098	28.35	383	-6	-188
20	M9	113.78	0.00125	36.22	489	0	0
21	M10	119.66	0.00152	44.08	595	6	292
22	M11	125.53	0.00179	51.94	701	12	687
23	M12	131.41	0.00206	59.80	808	18	1187
24	M13	137.29	0.00233	60.00	810	24	1588
25	M14	143.17	0.00260	60.00	810	29	1984
26	M15	149.04	0.00288	60.00	810	35	2381
27	M16	154.92	0.00315	60.00	810	41	2778
28	T1	160.80	0.00342	60.00	3085	47	12088
29	T2	166.68	0.00369	60.00	1187	53	5231
30	M17	172.55	0.00396	60.00	810	59	3969
31	M18	178.43	0.00423	60.00	810	65	4366
32	M19	184.31	0.00450	60.00	810	71	4763
33	M20	190.19	0.00477	60.00	810	76	5160
34	M21	196.07	0.00504	60.00	810	82	5557
35	M22	201.94	0.00532	60.00	810	88	5953
36	T3	207.82	0.00559	60.00	4746	94	37195
37	T4	213.70	0.00586	60.00	4746	100	39520
38	T5	219.58	0.00613	60.00	4509	106	39752
39	T6	225.45	0.00640	60.00	4509	112	41961
					7166		404999

Resultant forces:

Pnx =	23561	kips	105	MN
Mnx =	988829	kip-ft	1341	MN-m
ey =	503.62	in		
ey req/ ey res =	1.00		1.00	

6 Check for shear and torsional force effects:

- *Resistance factor:* A.5.5.4.2 — ϕ = 0.90

- *Torsional effects:*

Total area enclosed by outside perimeter of concrete:	Acp =	41414	in2		
Outside perimeter of the concrete section:	pc =	823	in		
Diameter of exterior stirrups:	d =	0.98	in	25	mm
Spacing of exterior stirrups:	s =	5.91	in	150	mm
Area enclosed by the shear path:	Ao =	37537	in2		
Perimeter of centerline of exterior stirrups:	ph =	784	in		
Torsional cracking moment:	Tcr =	109830	kip-ft	149	MN-m
	0.25ϕTcr =	24712	kip-ft	34	MN-m
	Tu =	0	kip-ft	0	MN-m

Torsional effects DO NOT need to be considered for shear checking

Torsional resistance:	ϕTn =	64708	kip-ft	88	MN-m
	Tu/ϕTn =	0.00	OK!	0.00	

- *In-plane shear:*

Factored load:	Vuy =	15932	Kips	71	MN
Prestressing force in direction of Vu:	Vp =	2405	Kips	11	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2		
Effective width	bv =	175	in		
Minimum Shear depth:	dv =	189	in		
Net tensile Strains:	$\varepsilon_{average}$ =	0.00141			
Crack angle:	θ =	33.9			
Parameter β assuming min. transverse reinforcement:	β =	2.335			
In-plane concrete shear resistance:	Vc =	6907	Kips	31	MN
In-plane steel shear resistance:	Vs =	17356	Kips	77	MN
Inclination of reinforcement:	α =	90	deg		
Diameter of stirrups:	d =	0.98	in		
Spacing of stirrups:	s =	5.91	in		
No. of reinforcement legs:	n =	8			
	ϕVn =	21837	Kips	97	MN
	Vu/ϕVn =	0.73	OK!	0.73	

- *Out-of-plane shear:*

Factored load:	Vux =	0	Kips	0	MN
Prestressing force in direction of Vu:	Vp =	0	Kips	0	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2		
Effective width	bv =	236	in		
Minimum Shear depth:	dv =	126	in		
Net tensile Strains:	$\varepsilon_{average}$ =	0.00000			
Parameter β assuming min. transverse reinforcement:	β =	4.800			
In-plane concrete shear resistance:	Vc =	12792	Kips	57	MN
	ϕVc =	11513	Kips	51	MN
	Vu/ϕVc =	0.00	OK!	0.00	

- *Minor-axis bending resistance: Pny, Mny*

Eccentricity:	ex =	0.40	in
Neutral axis depth:	c =	422.49	in
	a =	166.66	in
Concrete forces:	Pc =	257883	kips
	Mc =	0	kip-ft

Tendon forces:

Effective initial prestressing strain: ε_{po} = 0.00592

No.	Name	xeff (in)	ε_{si}	fsi (ksi)	Tsi (kips)	di (in)	Msi (kip-ft)
1	B6.1	137.42	0.00323	91.86	718	54	3235
2	B6.2	109.08	0.00296	84.21	676	26	1451
3	B6.3	65.30	0.00254	72.41	581	-18	-874
4	B6.4	21.52	0.00213	60.60	473	-62	-2439
5	B5.1	136.16	0.00321	91.52	715	53	3148
6	B5.2	107.82	0.00295	83.87	673	24	1374
7	B5.3	66.56	0.00255	72.75	584	-17	-816
8	B5.4	22.78	0.00214	60.94	476	-61	-2402
9	B4.1	136.79	0.00322	91.69	736	53	3280
10	B4.2	108.45	0.00295	84.04	675	25	1412
11	B4.3	65.93	0.00255	72.58	583	-17	-845
12	B4.4	22.15	0.00213	60.77	488	-61	-2488
13	T3.1	137.42	0.00323	91.86	718	54	3235
14	T3.2	109.08	0.00296	84.21	676	26	1451
15	T3.3	79.47	0.00268	76.23	612	-4	-197
16	T3.4	20.22	0.00212	60.25	471	-63	-2476
17	T2.1	122.62	0.00309	87.86	705	39	2310
18	T2.2	94.27	0.00282	80.22	644	11	587
19	T2.3	65.93	0.00255	72.58	583	-17	-845
20	T2.4	37.58	0.00228	64.94	521	-46	-1988
21	T1.1	136.16	0.00321	91.52	715	53	3148
22	T1.2	107.82	0.00295	83.87	673	24	1374
23	T1.3	80.73	0.00269	76.57	615	-3	-133
24	T1.4	21.48	0.00213	60.59	473	-62	-2440
					14785		8063

Resultant forces:	Pny =	243098	kips	1082	MN
	Mny =	8063	kip-ft	11	MN-m
	ex =	0.40	in		
	ex req/ ex res =	1.00		1.00	

- *Biaxial flexure check:* A.5.7.4.5

Minimum axial load to be considered as a column:	0.10ϕf'cAg =	29818	kips	133	MN
Factored axial load:	Pu =	13964	kips	62	MN
Nominal axial resistance:	Po =	246120	kips	1095	MN
Method to check biaxial flexure:		Load Contour Method			
Reciprocal Load Method:	Pr =	NA	Kips		
	Pu/Pr =	NA	NA	NA	
Load Contour Method:	Mux/Mrx =	0.66			
	Muy/Mry =	0.06			
	IE =	0.72	OK!	0.72	

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	ε (rad)	x (mm)	y (mm)
5	B5.1	-52.83	28.19	36	7.81	0.43	-1342	634
6	B5.2	-24.49	28.19	37	8.03	0.43	-622	634
7	B5.3	16.77	28.19	37	8.03	0.43	426	634
8	B5.4	60.55	28.19	36	7.81	0.43	1538	634
9	B4.1	-53.46	35.59	37	8.03	0.48	-1358	822
10	B4.2	-25.12	35.59	37	8.03	0.48	-638	822
11	B4.3	17.40	35.59	37	8.03	0.48	442	822
12	B4.4	61.18	35.59	37	8.03	0.48	1554	822
13	T3.1	-54.09	206.02	36	7.81	0.21	-1374	5175
14	T3.2	-25.75	206.02	37	8.03	0.21	-654	5175
15	T3.3	3.86	206.02	37	8.03	0.21	98	5175
16	T3.4	63.11	206.02	36	7.81	0.21	1603	5175
17	T2.1	-39.29	211.61	37	8.03	0.12	-998	5317
18	T2.2	-10.94	211.61	37	8.03	0.12	-278	5317
19	T2.3	17.40	211.61	37	8.03	0.12	442	5317
20	T2.4	45.75	211.61	37	8.03	0.12	1162	5317
21	T1.1	-52.83	211.61	36	7.81	0.00	-1342	5317
22	T1.2	-24.49	211.61	37	8.03	0.00	-622	5317
23	T1.3	2.60	211.61	37	8.03	0.00	66	5317
24	T1.4	61.85	211.61	36	7.81	0.00	1571	5317
					190.96	0.25		
	Centroids =	-0.58	119.29 in					
	Eccentricity =	-0.58	1.18 in					

- *Longitudinal reinforcement locations:*

**from bottom of gross section

No.	Name	db (in)	y**(in)	Qty	Area (in2)	db (mm)	y (mm)
1	B1	2.24	6.44	16	63.28	57	164
2	B2	2.24	12.31	14	55.37	57	313
3	B3	2.24	18.19	14	55.37	57	462
4	B4	2.24	24.07	6	23.73	57	611
5	B5	2.24	29.95	6	23.73	57	761
6	B6	2.24	35.82	6	23.73	57	910
7	B7	2.24	41.70	6	23.73	57	1059
8	B8	2.24	47.58	14	55.37	57	1209
9	B9	2.24	53.46	14	55.37	57	1358
10	B10	2.24	59.33	14	55.37	57	1507
11	B11	2.24	65.21	14	55.37	57	1656
12	M1	1.69	71.09	6	13.51	43	1806
13	M2	1.69	76.97	6	13.51	43	1955
14	M3	1.69	82.85	6	13.51	43	2104
15	M4	1.69	88.72	6	13.51	43	2254
16	M5	1.69	94.60	6	13.51	43	2403
17	M6	1.69	100.48	6	13.51	43	2552
18	M7	1.69	106.36	6	13.51	43	2701
19	M8	1.69	112.23	6	13.51	43	2851
20	M9	1.69	118.11	6	13.51	43	3000
21	M10	1.69	123.99	6	13.51	43	3149
22	M11	1.69	129.87	6	13.51	43	3299
23	M12	1.69	135.74	6	13.51	43	3448
24	M13	1.69	141.62	6	13.51	43	3597
25	M14	1.69	147.50	6	13.51	43	3746
26	M15	1.69	153.38	6	13.51	43	3896
27	M16	1.69	159.25	6	13.51	43	4045
28	T1	2.24	165.13	13	51.42	57	4194
29	T2	2.24	171.01	5	19.78	57	4344
30	M17	1.69	176.89	6	13.51	43	4493
31	M18	1.69	182.76	6	13.51	43	4642
32	M19	1.69	188.64	6	13.51	43	4791
33	M20	1.69	194.52	6	13.51	43	4941
34	M21	1.69	200.40	6	13.51	43	5090
35	M22	1.69	206.27	6	13.51	43	5239
36	T3	2.24	212.15	20	79.10	57	5389
37	T4	2.24	218.03	20	79.10	57	5538
38	T5	2.24	223.91	19	75.15	57	5687
39	T6	2.24	229.78	19	75.15	57	5837
					1167.27		

0 Specifications: **AASHTO LRFD Design Specs. 2012**

1 Material properties:

				Metric	
- Concrete density:	Wc =	0.15	kcf	24	KN/m3
- Concrete 28-day compressive strength:	f'c =	8	ksi	55	MPa
- Reinforcement strength:	fy =	60	ksi	414	MPa
- Prestressing steel strand:	Diameter =	0.60	in	15	mm
Strand cross-sectional area:	A =	0.217	in2	140	mm2
Modulus of Elasticity:	Ep =	28478	ksi	196500	MPa
Ultimate tensile strength:	Fpu =	58.43	kips	260	KN
	fpu =	269	ksi	1858	MPa
Minimum yield strength:	Fpy =	52.58	kips	234	KN
	fpy =	90%	fpu		
Maximum stressing force per strand:	Fp max =	43.82	kips	195	KN
	fp jacking =	75%	fpu		
% prestressing stress after transfer with Max losses:	fpt =	67%	fpu		
% Effective prestressing stress considering Max losses:	fpe =	63%	fpu		
Effective prestressing strain considering Max losses:	ε_{po} =	0.00592			
Wedge seating:	Δ =	0.39	in	10	mm
Friction coefficient:	μ =	0.15	/rad	0.15	/rad
Wobble coefficient:	k =	0.000201	/ft	0.00066	/m
- Tendons:					
Duct diameter:	OD =	5.51	in	140	mm
Location of tendon CG with respect to duct CG	Z =	0.47	in	12	mm

2 Bearing location:

- Eccentricity between shaft and top of pier cap:	e1 =	39.17	in	995	mm

3 Loads:

- Case: Maximum Bearing: Vuy max

		PT + EQ + DL		PT + EQ + DL	
- *Loading effects at the section of maximum effect:*					
Axial (if <0 → Tension)	Pu =	35624	kips	159	MN
In-plane shear	Vuy =	19951	kips	89	MN
Out-of-plane shear	Vux =	1072	kips	5	MN
In-plane-bending moment: (if >0 → Top fiber in tension)	Mux =	224034	kip-ft	304	MN-m
Out-of-plane bending moment	Muy =	1819	kip-ft	2	MN-m
Torsion	Tu =	14051	kip-ft	19	MN-m

4 Pier cap section properties:

- Section No. 1
Location = Bearing

- *Dimensions:*						
Location from cap end	x =	16.59	ft		5058	mm
Width	B =	175.3	167	in	4453	mm
Depth	H =	236.2	228	in	6000	mm
Concrete cover:	Top =	4.33	in		110	mm
	Bottom =	4.33	in		110	mm
	Lateral =	4.33	in		110	mm

- *Section properties:*		Gross	Eff at ε_{cu}	
Area	A =	41414	37924	in2
Centroid (with respect to left bottom corner)	yc =	118.110	114	in
	xc =	88	83	in
Moment of inertia	Ix =	192573235	163652090	in4
	Iy =	106074258	87775676	in4
Section modulus	Sx =	1630453	1438326	in3
	Sy =	1210084	1053377	in3

- *Longitudinal tendon locations:*

*from section centerline
**from bottom of gross section

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	ε (rad)	x (mm)	y (mm)
1	B6.1	-54.09	22.60	36	7.81	0.25	-1374	492
2	B6.2	-25.75	22.60	37	8.03	0.25	-654	492
3	B6.3	18.03	22.60	37	8.03	0.25	458	492
4	B6.4	61.81	22.60	36	7.81	0.25	1570	492

5 Check for axial and flexural force effects:

- *Resistance factor:* A.5.10.11.4.1b ϕ = 0.90

- *Loading effects:*

1 Prestressing loading effects:	Pu =	32218	kips	143	MN
	Mux =	-3171	kip-ft	-4	MN-m
	Muy =	-1551	kip-ft	-2	MN-m
2 Net loading effects:	Pu =	3406	kips	15	MN
	Mux =	220863	kip-ft	300	MN-m
	Muy =	268	kip-ft	0	MN-m
3 Total loading stresses:	Pu/A =	0.860	ksi	5.9	MPa
	Mux/Sx =	1.649	ksi	11.4	MPa
	Muy/Sy =	0.018	ksi	0.1	MPa
	Max =	2.527	ksi	17.4	MPa
		32%	f'c	32%	
	Min =	-0.807	ksi	-5.6	MPa
		10%	f'c	10%	

- *Analysis approach:* *Strain compatibility*

- *Concrete behavior assumptions:* *Unconfined*

Stress distribution:	A.5.7.2.2		*Rectangular*				
		α_1 =	0.85				
		β_1 =	0.65				
Compressive strength:		f'c =	8	ksi			
Ultimate compression strain:		ε_{cu} =	0.004	*			
Modulus of Elasticity:	A.5.4.2.4-1	E =	5422	ksi			
Effective section properties:		b =	167	in		4233	mm
		h =	228	in		5780	mm

* Seismic Design of Reinforced Concrete and Masonry Buildings, Paulay and Priestly, pag. 98
Ignore concrete cover for calculations at this strain level

- *Major-axis bending resistance: Pnx, Mnx*

Eccentricity:	ey =	778.19	in
Neutral axis depth:	c =	79.41	in
	a =	51.61	in
Concrete forces:	Pc =	58492	kips
	Mc =	428805	kip-ft

Tendon forces:
Effective initial prestressing strain: ε_{po} = 0.00592

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B6.1	18.27	0.00284	81.01	633	-96	-5037
2	B6.2	18.27	0.00284	81.01	650	-96	-5177
3	B6.3	18.27	0.00284	81.01	650	-96	-5177
4	B6.4	18.27	0.00284	81.01	633	-96	-5037
5	B5.1	23.86	0.00313	89.03	696	-90	-5212
6	B5.2	23.86	0.00313	89.03	715	-90	-5356
7	B5.3	23.86	0.00313	89.03	715	-90	-5356
8	B5.4	23.86	0.00313	89.03	696	-90	-5212
9	B4.1	31.26	0.00350	99.65	800	-83	-5502
10	B4.2	31.26	0.00350	99.65	800	-83	-5502
11	B4.3	31.26	0.00350	99.65	800	-83	-5502
12	B4.4	31.26	0.00350	99.65	800	-83	-5502
13	T3.1	201.69	0.01208	261.38	2042	88	14959

14	T3.2	201.69	0.01208	261.38	2099	88	15375
15	T3.3	201.69	0.01208	261.38	2099	88	15375
16	T3.4	201.69	0.01208	261.38	2042	88	14959
17	T2.1	207.28	0.01237	261.79	2102	94	16378
18	T2.2	207.28	0.01237	261.79	2102	94	16378
19	T2.3	207.28	0.01237	261.79	2102	94	16378
20	T2.4	207.28	0.01237	261.79	2102	94	16378
21	T1.1	207.28	0.01237	261.79	2045	94	15936
22	T1.2	207.28	0.01237	261.79	2102	94	16378
23	T1.3	207.28	0.01237	261.79	2102	94	16378
24	T1.4	207.28	0.01237	261.79	2045	94	15936
					33571		127238

Reinforcement forces:

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B1	2.11	-0.00389	-60.00	-3797	-112	35335
2	B2	7.98	-0.00360	-60.00	-3322	-106	29291
3	B3	13.86	-0.00330	-60.00	-3322	-100	27664
4	B4	19.74	-0.00301	-60.00	-1424	-94	11159
5	B5	25.62	-0.00271	-60.00	-1424	-88	10461
6	B6	31.49	-0.00241	-60.00	-1424	-82	9764
7	B7	37.37	-0.00212	-60.00	-1424	-76	9066
8	B8	43.25	-0.00182	-52.82	-2925	-71	17191
9	B9	49.13	-0.00153	-44.23	-2449	-65	13196
10	B10	55.00	-0.00123	-35.65	-1974	-59	9668
11	B11	60.88	-0.00093	-27.06	-1498	-53	6605
12	M1	66.76	-0.00064	-18.47	-250	-47	978
13	M2	72.64	-0.00034	-9.89	-134	-41	458
14	M3	78.51	-0.00004	-1.30	-18	-35	52
15	M4	84.39	0.00025	7.28	98	-29	-241
16	M5	90.27	0.00055	15.87	214	-24	-420
17	M6	96.15	0.00084	24.46	330	-18	-485
18	M7	102.02	0.00114	33.04	446	-12	-437
19	M8	107.90	0.00144	41.63	562	-6	-275
20	M9	113.78	0.00173	50.22	678	0	0
21	M10	119.66	0.00203	58.80	794	6	389
22	M11	125.53	0.00232	60.00	810	12	794
23	M12	131.41	0.00262	60.00	810	18	1191
24	M13	137.29	0.00292	60.00	810	24	1588
25	M14	143.17	0.00321	60.00	810	29	1984
26	M15	149.04	0.00351	60.00	810	35	2381
27	M16	154.92	0.00380	60.00	810	41	2778
28	T1	160.80	0.00410	60.00	3085	47	12088
29	T2	166.68	0.00440	60.00	1187	53	5231
30	M17	172.55	0.00469	60.00	810	59	3969
31	M18	178.43	0.00499	60.00	810	65	4366
32	M19	184.31	0.00528	60.00	810	71	4763
33	M20	190.19	0.00558	60.00	810	76	5160
34	M21	196.07	0.00588	60.00	810	82	5557
35	M22	201.94	0.00617	60.00	810	88	5953
36	T3	207.82	0.00647	60.00	4746	94	37195
37	T4	213.70	0.00676	60.00	4746	100	39520
38	T5	219.58	0.00706	60.00	4509	106	39752
39	T6	225.45	0.00736	60.00	4509	112	41961
					10246		395649

Resultant forces:

Pnx =	14675	kips	65	MN
Mnx =	951693	kip-ft	1291	MN-m
ey =	778.19	in		
ey req/ ey res =	1.00		1.00	

6 Check for shear and torsional force effects:

- *Resistance factor:* A.5.5.4.2 — ϕ = 0.90

- *Torsional effects:*

Total area enclosed by outside perimeter of concrete:	Acp =	41414	in2		
Outside perimeter of the concrete section:	pc =	823	in		
Diameter of exterior stirrups:	d =	0.98	in	25	mm
Spacing of exterior stirrups:	s =	5.91	in	150	mm
Area enclosed by the shear path:	Ao =	37537	in2		
Perimeter of centerline of exterior stirrups:	ph =	784	in		
Torsional cracking moment:	Tcr =	109830	kip-ft	149	MN-m
	0.25ϕTcr=	24712	kip-ft	34	MN-m
	Tu =	14051	kip-ft	19	MN-m

Torsional effects DO NOT need to be considered for shear checking

Torsional resistance:	ϕTn=	69443	kip-ft	94	MN-m
	Tu/ϕTn =	0.20	OK!	0.20	

- *In-plane shear:*

Factored load:	Vuy =	19951	Kips	89	MN
Prestressing force in direction of Vu:	Vp =	2405	Kips	11	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2		
Effective width	bv =	175	in		
Minimum Shear depth:	dv =	189	in		
Net tensile Strains:	$\varepsilon_{s\ average}$ =	0.00088			
Crack angle:	θ =	32.1			
Parameter β assuming min. transverse reinforcement:	β=	2.892			
In-plane concrete shear resistance:	Vc =	8555	Kips	38	MN
In-plane steel shear resistance:	Vs =	18626	Kips	83	MN
Inclination of reinforcement:	α=	90	deg		
Diameter of stirrups:	d =	0.98	in		
Spacing of stirrups:	s =	5.91	in		
No. of reinforcement legs:	n =	8			
	ϕVn =	24463	Kips	109	MN
	Vu/ϕVn =	0.82	OK!	0.82	

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- *Out-of-plane shear:*

Factored load:	Vux =	1072	Kips	5	MN
Prestressing force in direction of Vu:	Vp =	0	Kips	0	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2		
Effective width	bv =	236	in		
Minimum Shear depth:	dv =	126	in		
Net tensile Strains:	$\varepsilon_{s\ average}$ =	0.00000			
Parameter β assuming min. transverse reinforcement:	β=	4.800			
In-plane concrete shear resistance:	Vc =	12792	Kips	57	MN
	ϕVc =	11513	Kips	51	MN
	Vu/ϕVc =	0.08	OK!	0.08	

- *Minor-axis bending resistance: Pny, Mny*

Eccentricity:	ex =	0.94	in
Neutral axis depth:	c =	255.56	in
	a =	166.11	in
Concrete forces:	Pc =	257042	kips
	Mc =	5825	kip-ft

Tendon forces:

Effective initial prestressing strain: ε_{po}= 0.00592

No.	Name	xeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B6.1	137.42	0.00408	116.06	907	54	4087
2	B6.2	109.08	0.00363	103.42	830	26	1782
3	B6.3	65.30	0.00295	83.91	674	-18	-1012
4	B6.4	21.52	0.00226	64.39	503	-62	-2591
5	B5.1	136.16	0.00406	115.50	902	53	3973
6	B5.2	107.82	0.00361	102.86	826	24	1685
7	B5.3	66.56	0.00297	84.47	678	-17	-948
8	B5.4	22.78	0.00228	64.96	507	-61	-2560
9	B4.1	136.79	0.00407	115.78	930	53	4142
10	B4.2	108.45	0.00362	103.14	828	25	1733
11	B4.3	65.93	0.00296	84.19	676	-17	-980
12	B4.4	22.15	0.00227	64.68	519	-61	-2648
13	T3.1	137.42	0.00408	116.06	907	54	4087
14	T3.2	109.08	0.00363	103.42	830	26	1782
15	T3.3	79.47	0.00317	90.23	724	-4	-233
16	T3.4	20.22	0.00224	63.82	499	-63	-2622
17	T2.1	122.62	0.00384	109.46	879	39	2878
18	T2.2	94.27	0.00340	96.82	777	11	709
19	T2.3	65.93	0.00296	84.19	676	-17	-980
20	T2.4	37.58	0.00251	71.55	575	-46	-2190
21	T1.1	136.16	0.00406	115.50	902	53	3973
22	T1.2	107.82	0.00361	102.86	826	24	1685
23	T1.3	80.73	0.00319	90.79	729	-3	-158
24	T1.4	21.48	0.00226	64.38	503	-62	-2592
					17607		13000

Resultant forces:	Pny =	239435	kips	1065	MN
	Mny =	18826	kip-ft	26	MN-m
	ex =	0.94	in		
	ex req/ ex res =	1.00		1.00	

- *Biaxial flexure check:* A.5.7.4.5

Minimum axial load to be considered as a column:	0.1ϕf'cAg =	29818	kips	133	MN
Factored axial load:	Pu =	3406	kips	15	MN
Nominal axial resistance:	Po =	246120	kips	1095	MN
Method to check biaxial flexure:		Load Contour Method			
Reciprocal Load Method:	Pr =	NA	Kips		
	Pu/Pr =	NA	NA	NA	
Load Contour Method:	Mux/Mrx =	0.26			
	Muy/Mry =	0.02			
	IE =	0.27	OK!	0.27	

0 Specifications: AASHTO LRFD Design Specs. 2012

1 Material properties:				**Metric**	
- Concrete density:	Wc =	0.15	kcf	24	KN/m3
- Concrete 28-day compressive strength:	f'c =	8	ksi	55	MPa
- Reinforcement strength:	fy =	60	ksi	414	MPa
- Prestressing steel strand:	Diameter =	0.60	in	15	mm
Strand cross-sectional area:	A =	0.217	in2	140	mm2
Modulus of Elasticity:	Ep =	28478	ksi	196500	MPa
Ultimate tensile strength:	Fpu =	58.43	kips	260	KN
	fpu =	269	ksi	1858	MPa
Minimum yield strength:	Fpy =	52.58	kips	234	KN
	fpy =	90%	fpu		
Maximum stressing force per strand:	Fp max =	43.82	kips	195	KN
	fp jacking =	75%	fpu		
% prestressing stress after transfer with Max losses:	fpt =	67%	fpu		
% Effective prestressing stress considering Max losses:	fpe =	63%	fpu		
Effective prestressing strain considering Max losses:	ε_{po} =	0.00592			
Wedge seating:	Δ =	0.39	in	10	mm
Friction coefficient:	μ =	0.15	/rad	0.15	/rad
Wobble coefficient:	k =	0.000201	/ft	0.00066	/m
- Tendons:					
Duct diameter:	OD =	5.51	in	140	mm
Location of tendon CG with respect to duct CG	Z =	0.47	in	12	mm

2 Bearing location:

- Eccentricity between shaft and top of pier cap:	e1 =	39.17	in	995	mm

3 Loads:

- Case: Maximum Shear Key: Mux max

- *Loading effects at the section of maximum effect:*		PT + EQ + DL		PT + EQ + DL	
Axial (if <0 → Tension)	Pu =	41614	kips	185	MN
In-plane shear	Vuy =	20054	kips	89	MN
Out-of-plane shear	Vux =	1840	kips	8	MN
In-plane-bending moment: (if >0 → Top fiber in tension)	Mux =	906959	kip-ft	1232	MN-m
Out-of-plane bending moment	Muy =	23576	kip-ft	32	MN-m
Torsion	Tu =	24111	kip-ft	33	MN-m

4 Pier cap section properties:

- Section No.		1					
	Location =	SK					
- *Dimensions:*							
Location from cap end	x =	30.51	ft			9298	mm
Width	B =	216.5		208	in	5500	mm
Depth	H =	236.2		228	in	6000	mm
Concrete cover:	Top =	4.33	in			110	mm
	Bottom =	4.33	in			110	mm
	Lateral =	4.33	in			110	mm
- *Section properties:*		Gross		Eff at ε_{cu}			
Area	A =	51150		47304	in2		
Centroid (with respect to left bottom corner)	yc =	118.110		114	in		
	xc =	108		104	in		
Moment of inertia	Ix =	237848451		204127410	in4		
	Iy =	199858768		170338765	in4		
Section modulus	Sx =	2013784		1794061	in3		
	Sy =	1845968		1638865	in3		

- *Longitudinal tendon locations:*
  - *from section centerline
  - **from bottom of gross section

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	θ (rad)	x (mm)	y (mm)
1	B6.1	-54.09	22.60	36	7.81	0.25	-1374	492
2	B6.2	-25.75	22.60	37	8.03	0.25	-654	492
3	B6.3	18.03	22.60	37	8.03	0.25	458	492
4	B6.4	61.81	22.60	36	7.81	0.25	1570	492

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	θ (rad)	x (mm)	y (mm)
5	B5.1	-52.83	28.19	36	7.81	0.43	-1342	634
6	B5.2	-24.49	28.19	37	8.03	0.43	-622	634
7	B5.3	16.77	28.19	37	8.03	0.43	426	634
8	B5.4	60.55	28.19	36	7.81	0.43	1538	634
9	B4.1	-53.46	35.59	37	8.03	0.48	-1358	822
10	B4.2	-25.12	35.59	37	8.03	0.48	-638	822
11	B4.3	17.40	35.59	37	8.03	0.48	442	822
12	B4.4	61.18	35.59	37	8.03	0.48	1554	822
13	T3.1	-54.09	206.02	36	7.81	0.21	-1374	5175
14	T3.2	-25.75	206.02	37	8.03	0.21	-654	5175
15	T3.3	3.86	206.02	37	8.03	0.21	98	5175
16	T3.4	63.11	206.02	36	7.81	0.21	1603	5175
17	T2.1	-39.29	211.61	37	8.03	0.12	-998	5317
18	T2.2	-10.94	211.61	37	8.03	0.12	-278	5317
19	T2.3	17.40	211.61	37	8.03	0.12	442	5317
20	T2.4	45.75	211.61	37	8.03	0.12	1162	5317
21	T1.1	-52.83	211.61	36	7.81	0.00	-1342	5317
22	T1.2	-24.49	211.61	37	8.03	0.00	-622	5317
23	T1.3	2.60	211.61	37	8.03	0.00	66	5317
24	T1.4	61.85	211.61	36	7.81	0.00	1571	5317
					190.96	0.25		
	Centroids =	-0.58	119.29	in				
	Eccentricity =	-0.58	1.18	in				

- *Longitudinal reinforcement locations:*
  - **from bottom of gross section

No.	Name	db (in)	y**(in)	Qty	Area (in2)	db (mm)	y (mm)
1	B1	2.24	6.44	20	79.10	57	164
2	B2	2.24	12.31	20	79.10	57	313
3	B3	2.24	18.19	20	79.10	57	462
4	B4	2.24	24.07	12	47.46	57	611
5	B5	2.24	29.95	12	47.46	57	761
6	B6	2.24	35.82	12	47.46	57	910
7	B7	2.24	41.70	12	47.46	57	1059
8	B8	2.24	47.58	14	55.37	57	1209
9	B9	2.24	53.46	14	55.37	57	1358
10	B10	2.24	59.33	14	55.37	57	1507
11	B11	2.24	65.21	14	55.37	57	1656
12	M1	1.69	71.09	6	13.51	43	1806
13	M2	1.69	76.97	6	13.51	43	1955
14	M3	1.69	82.85	6	13.51	43	2104
15	M4	1.69	88.72	6	13.51	43	2254
16	M5	1.69	94.60	6	13.51	43	2403
17	M6	1.69	100.48	6	13.51	43	2552
18	M7	1.69	106.36	6	13.51	43	2701
19	M8	1.69	112.23	6	13.51	43	2851
20	M9	1.69	118.11	6	13.51	43	3000
21	M10	1.69	123.99	6	13.51	43	3149
22	M11	1.69	129.87	6	13.51	43	3299
23	M12	1.69	135.74	6	13.51	43	3448
24	M13	1.69	141.62	6	13.51	43	3597
25	M14	1.69	147.50	6	13.51	43	3746
26	M15	1.69	153.38	6	13.51	43	3896
27	M16	1.69	159.25	6	13.51	43	4045
28	T1	2.24	165.13	13	51.42	57	4194
29	T2	2.24	171.01	5	19.78	57	4344
30	M17	1.69	176.89	6	13.51	43	4493
31	M18	1.69	182.76	6	13.51	43	4642
32	M19	1.69	188.64	6	13.51	43	4791
33	M20	1.69	194.52	6	13.51	43	4941
34	M21	1.69	200.40	6	13.51	43	5090
35	M22	1.69	206.27	6	13.51	43	5239
36	T3	2.24	212.15	36	142.39	57	5389
37	T4	2.24	218.03	28	110.75	57	5538
38	T5	2.24	223.91	27	106.79	57	5687
39	T6	2.24	229.78	27	106.79	57	5837
					1483.69		

5 Check for axial and flexural force effects:

- *Resistance factor:* A.5.10.11.4.1b ϕ = 0.90

- *Loading effects:*

1 Prestressing loading effects:	Pu =	32218	kips	143	MN
	Mux =	-3171	kip-ft	-4	MN-m
	Muy =	-1551	kip-ft	-2	MN-m
2 Net loading effects:	Pu =	9395	kips	42	MN
	Mux =	903788	kip-ft	1226	MN-m
	Muy =	22025	kip-ft	30	MN-m
3 Total loading stresses:	Pu/A =	0.814	ksi	5.6	MPa
	Mux/Sx =	5.405	ksi	37.3	MPa
	Muy/Sy =	0.153	ksi	1.1	MPa
	Max =	6.371	ksi	44.0	MPa
		80%	f'c	80%	
	Min =	-4.744	ksi	-32.7	MPa
		59%	f'c	59%	

- *Analysis approach:* *Strain compatibility*

- *Concrete behavior assumptions:* *Unconfined*

Stress distribution: A.5.7.2.2 *Rectangular*

α = 0.85

β = 0.65

Compressive strength: f'c = 8 ksi

Ultimate compression strain: ε_{cu} = 0.004 *

* Seismic Design of Reinforced Concrete and Masonry Buildings, Paulay and Priestly, pag. 98

Ignore concrete cover for calculations at this strain level

Modulus of Elasticity: A.5.4.2.4-1 E = 5422 ksi

Effective section properties: b = 208 in 5280 mm

h = 228 in 5780 mm

- *Major-axis bending resistance: Pnx, Mnx*

Eccentricity: ey = 1154.32 in

Neutral axis depth: c = 67.56 in

a = 43.92 in

Concrete forces: Pc = 62078 kips

Mc = 475008 kip-ft

Tendon forces:

Effective initial prestressing strain: ε_{po} = 0.00592

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B6.1	18.27	0.00301	85.60	669	-96	-5323
2	B6.2	18.27	0.00301	85.60	687	-96	-5470
3	B6.3	18.27	0.00301	85.60	687	-96	-5470
4	B6.4	18.27	0.00301	85.60	669	-96	-5323
5	B5.1	23.86	0.00334	95.03	742	-90	-5563
6	B5.2	23.86	0.00334	95.03	763	-90	-5717
7	B5.3	23.86	0.00334	95.03	763	-90	-5717
8	B5.4	23.86	0.00334	95.03	742	-90	-5563
9	B4.1	31.26	0.00378	107.51	863	-83	-5936
10	B4.2	31.26	0.00378	107.51	863	-83	-5936
11	B4.3	31.26	0.00378	107.51	863	-83	-5936
12	B4.4	31.26	0.00378	107.51	863	-83	-5936
13	T3.1	201.69	0.01387	263.42	2058	88	15076

14	T3.2	201.69	0.01387	263.42	2115	88	15495
15	T3.3	201.69	0.01387	263.42	2115	88	15495
16	T3.4	201.69	0.01387	263.42	2058	88	15076
17	T2.1	207.28	0.01420	263.69	2117	94	16497
18	T2.2	207.28	0.01420	263.69	2117	94	16497
19	T2.3	207.28	0.01420	263.69	2117	94	16497
20	T2.4	207.28	0.01420	263.69	2117	94	16497
21	T1.1	207.28	0.01420	263.69	2060	94	16051
22	T1.2	207.28	0.01420	263.69	2117	94	16497
23	T1.3	207.28	0.01420	263.69	2117	94	16497
24	T1.4	207.28	0.01420	263.69	2060	94	16051
					34344		124336

Reinforcement forces:

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B1	2.11	-0.00388	-60.00	-4746	-112	44169
2	B2	7.98	-0.00353	-60.00	-4746	-106	41845
3	B3	13.86	-0.00318	-60.00	-4746	-100	39520
4	B4	19.74	-0.00283	-60.00	-2848	-94	22317
5	B5	25.62	-0.00248	-60.00	-2848	-88	20922
6	B6	31.49	-0.00214	-60.00	-2848	-82	19528
7	B7	37.37	-0.00179	-51.84	-2460	-76	15666
8	B8	43.25	-0.00144	-41.75	-2312	-71	13587
9	B9	49.13	-0.00109	-31.66	-1753	-65	9444
10	B10	55.00	-0.00074	-21.56	-1194	-59	5849
11	B11	60.88	-0.00040	-11.47	-635	-53	2801
12	M1	66.76	-0.00005	-1.38	-19	-47	73
13	M2	72.64	0.00030	8.71	118	-41	-403
14	M3	78.51	0.00065	18.80	254	-35	-746
15	M4	84.39	0.00100	28.89	390	-29	-956
16	M5	90.27	0.00134	38.98	526	-24	-1031
17	M6	96.15	0.00169	49.07	663	-18	-974
18	M7	102.02	0.00204	59.16	799	-12	-783
19	M8	107.90	0.00239	60.00	810	-6	-397
20	M9	113.78	0.00274	60.00	810	0	0
21	M10	119.66	0.00308	60.00	810	6	397
22	M11	125.53	0.00343	60.00	810	12	794
23	M12	131.41	0.00378	60.00	810	18	1191
24	M13	137.29	0.00413	60.00	810	24	1588
25	M14	143.17	0.00448	60.00	810	29	1984
26	M15	149.04	0.00482	60.00	810	35	2381
27	M16	154.92	0.00517	60.00	810	41	2778
28	T1	160.80	0.00552	60.00	3085	47	12088
29	T2	166.68	0.00587	60.00	1187	53	5231
30	M17	172.55	0.00622	60.00	810	59	3969
31	M18	178.43	0.00656	60.00	810	65	4366
32	M19	184.31	0.00691	60.00	810	71	4763
33	M20	190.19	0.00726	60.00	810	76	5160
34	M21	196.07	0.00761	60.00	810	82	5557
35	M22	201.94	0.00796	60.00	810	88	5953
36	T3	207.82	0.00830	60.00	8543	94	66951
37	T4	213.70	0.00865	60.00	6645	100	55328
38	T5	219.58	0.00900	60.00	6407	106	56490
39	T6	225.45	0.00935	60.00	6407	112	59629
					16025		527027

Resultant forces:

Pnx =	11709	kips	52	MN
Mnx =	1126371	kip-ft	1528	MN-m
ey =	1154.32	in		
ey req/ ey res =	1.00		1.00	

6 Check for shear and torsional force effects:

- *Resistance factor:* A.5.5.4.2	ϕ =	0.90				
- *Torsional effects:*						
Total area enclosed by outside perimeter of concrete:	Acp =	51150	in2			
Outside perimeter of the concrete section:	pc =	906	in			
Diameter of exterior stirrups:	d =	0.98	in		25	mm
Spacing of exterior stirrups:	s =	12.60	in		320	mm
Area enclosed by the shear path:	Ao =	46876	in2			
Perimeter of centerline of exterior stirrups:	ph =	867	in			
Torsional cracking moment:	Tcr =	141976	kip-ft		193	MN-m
	0.25ϕTcr=	31945	kip-ft		43	MN-m
	Tu =	24111	kip-ft		33	MN-m
	Torsional effects DO NOT need to be considered for shear checking					
Torsional resistance:	ϕTn=	34007	kip-ft		46	MN-m
	Tu/ϕTn =	0.71	OK!		0.71	
- *In-plane shear:*						
Factored load:	Vuy =	20054	Kips		89	MN
Prestressing force in direction of Vu:	Vp =	2405	Kips		11	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2			
Effective width	bv =	217	in			
Minimum Shear depth:	dv =	189	in			
Net tensile Strains:	$\varepsilon_{average}$ =	0.00224				
Crack angle:	θ =	36.8				
Parameter β assuming min. transverse reinforcement:	β=	1.791				
In-plane concrete shear resistance:	Vc =	6543	Kips		29	MN
In-plane steel shear resistance:	Vs =	27211	Kips		121	MN
Inclination of reinforcement:	α=	90	deg			
Diameter of stirrups:	d =	1.69	in		43	mm
Spacing of stirrups:	s =	10.00	in		254	mm
No. of reinforcement legs:	n =	8				
	ϕVn =	30379	Kips		135	MN
	Vu/ϕVn =	0.66	OK!		0.66	
- *Out-of-plane shear:*						
Factored load:	Vux =	1840	Kips		8	MN
Prestressing force in direction of Vu:	Vp =	0	Kips		0	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2			
Effective width	bv =	236	in			
Minimum Shear depth:	dv =	156	in			
Net tensile Strains:	$\varepsilon_{average}$ =	0.00000				
Parameter β assuming min. transverse reinforcement:	β=	4.800				
In-plane concrete shear resistance:	Vc =	15800	Kips		70	MN
	ϕVc =	14220	Kips		63	MN
	Vu/ϕVc =	0.12	OK!		0.12	

- *Minor-axis bending resistance: Pny, Mny*

Eccentricity:	ex =	28.13	in
Neutral axis depth:	c =	242.39	in
	a =	157.55	in
Concrete forces:	Pc =	243794	kips
	Mc =	511190	kip-ft

Tendon forces:

Effective initial prestressing strain: ε_{po}= 0.00592

No.	Name	xeff in	ε_{si}	fsi ksi	Tsi kips	di in	Msi kip-ft
1	B6.1	158.03	0.00453	129.07	1008	54	4545
2	B6.2	129.69	0.00406	115.75	929	26	1994
3	B6.3	85.91	0.00334	95.18	764	-18	-1148
4	B6.4	42.13	0.00262	74.60	583	-62	-3002
5	B5.1	156.77	0.00451	128.48	1004	53	4419
6	B5.2	128.43	0.00404	115.16	925	24	1887
7	B5.3	87.17	0.00336	95.77	769	-17	-1075
8	B5.4	43.39	0.00264	75.19	587	-61	-2964
9	B4.1	157.40	0.00452	128.78	1034	53	4607
10	B4.2	129.06	0.00405	115.45	927	25	1940
11	B4.3	86.54	0.00335	95.47	767	-17	-1112
12	B4.4	42.76	0.00263	74.90	601	-61	-3066
13	T3.1	158.03	0.00453	129.07	1008	54	4545
14	T3.2	129.69	0.00406	115.75	929	26	1994
15	T3.3	100.08	0.00358	101.84	818	-4	-263
16	T3.4	40.83	0.00260	73.99	578	-63	-3040
17	T2.1	143.23	0.00429	122.12	980	39	3210
18	T2.2	114.88	0.00382	108.79	874	11	797
19	T2.3	86.54	0.00335	95.47	767	-17	-1112
20	T2.4	58.19	0.00288	82.15	660	-46	-2515
21	T1.1	156.77	0.00451	128.48	1004	53	4419
22	T1.2	128.43	0.00404	115.16	925	24	1887
23	T1.3	101.34	0.00360	102.43	822	-3	-178
24	T1.4	42.09	0.00262	74.58	583	-62	-3003
					19845		13769

Resultant forces:	Pny =	223949	kips		997	MN
	Mny =	524959	kip-ft		712	MN-m
	ex =	28.13	in			
	ex req/ ex res =	1.00			1.00	
- *Biaxial flexure check:* A.5.7.4.5						
Minimum axial load to be considered as a column:	0.10ϕf'cAg =	36828	kips		164	MN
Factored axial load:	Pu =	9395	kips		42	MN
Nominal axial resistance:	Po =	309901	kips		1379	MN
Method to check biaxial flexure:		Load Contour Method				
Reciprocal Load Method:	Pr =	NA	Kips			
	Pu/Pr =	NA	NA		NA	
Load Contour Method:	Mux/Mrx =	0.89				
	Muy/Mry =	0.05				
	IE =	0.94	OK!		0.94	

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	(rad)	x (mm)	y (mm)
5	B5.1	-52.83	28.19	36	7.81	0.43	-1342	634
6	B5.2	-24.49	28.19	37	8.03	0.43	-622	634
7	B5.3	16.77	28.19	37	8.03	0.43	426	634
8	B5.4	60.55	28.19	36	7.81	0.43	1538	634
9	B4.1	-53.46	35.59	37	8.03	0.48	-1358	822
10	B4.2	-25.12	35.59	37	8.03	0.48	-638	822
11	B4.3	17.40	35.59	37	8.03	0.48	442	822
12	B4.4	61.18	35.59	37	8.03	0.48	1554	822
13	T3.1	-54.09	206.02	36	7.81	0.21	-1374	5175
14	T3.2	-25.75	206.02	37	8.03	0.21	-654	5175
15	T3.3	3.86	206.02	37	8.03	0.21	98	5175
16	T3.4	63.11	206.02	36	7.81	0.21	1603	5175
17	T2.1	-39.29	211.61	37	8.03	0.12	-998	5317
18	T2.2	-10.94	211.61	37	8.03	0.12	-278	5317
19	T2.3	17.40	211.61	37	8.03	0.12	442	5317
20	T2.4	45.75	211.61	37	8.03	0.12	1162	5317
21	T1.1	-52.83	211.61	36	7.81	0.00	-1342	5317
22	T1.2	-24.49	211.61	37	8.03	0.00	-622	5317
23	T1.3	2.60	211.61	37	8.03	0.00	66	5317
24	T1.4	61.85	211.61	36	7.81	0.00	1571	5317
					190.96	0.25		

Centroids = -0.58 119.29 in
Eccentricity = -0.58 1.18 in

- *Longitudinal reinforcement locations:*

**from bottom of gross section

No.	Name	db (in)	y**(in)	Qty	Area (in2)	db (mm)	y (mm)
1	B1	2.24	6.44	20	79.10	57	164
2	B2	2.24	12.31	20	79.10	57	313
3	B3	2.24	18.19	20	79.10	57	462
4	B4	2.24	24.07	12	47.46	57	611
5	B5	2.24	29.95	12	47.46	57	761
6	B6	2.24	35.82	12	47.46	57	910
7	B7	2.24	41.70	12	47.46	57	1059
8	B8	2.24	47.58	14	55.37	57	1209
9	B9	2.24	53.46	14	55.37	57	1358
10	B10	2.24	59.33	14	55.37	57	1507
11	B11	2.24	65.21	14	55.37	57	1656
12	M1	1.69	71.09	6	13.51	43	1806
13	M2	1.69	76.97	6	13.51	43	1955
14	M3	1.69	82.85	6	13.51	43	2104
15	M4	1.69	88.72	6	13.51	43	2254
16	M5	1.69	94.60	6	13.51	43	2403
17	M6	1.69	100.48	6	13.51	43	2552
18	M7	1.69	106.36	6	13.51	43	2701
19	M8	1.69	112.23	6	13.51	43	2851
20	M9	1.69	118.11	6	13.51	43	3000
21	M10	1.69	123.99	6	13.51	43	3149
22	M11	1.69	129.87	6	13.51	43	3299
23	M12	1.69	135.74	6	13.51	43	3448
24	M13	1.69	141.62	6	13.51	43	3597
25	M14	1.69	147.50	6	13.51	43	3746
26	M15	1.69	153.38	6	13.51	43	3896
27	M16	1.69	159.25	6	13.51	43	4045
28	T1	2.24	165.13	13	51.42	57	4194
29	T2	2.24	171.01	5	19.78	57	4344
30	M17	1.69	176.89	6	13.51	43	4493
31	M18	1.69	182.76	6	13.51	43	4642
32	M19	1.69	188.64	6	13.51	43	4791
33	M20	1.69	194.52	6	13.51	43	4941
34	M21	1.69	200.40	6	13.51	43	5090
35	M22	1.69	206.27	6	13.51	43	5239
36	T3	2.24	212.15	36	142.39	57	5389
37	T4	2.24	218.03	28	110.75	57	5538
38	T5	2.24	223.91	27	106.79	57	5687
39	T6	2.24	229.78	27	106.79	57	5837
					1483.69		

0 Specifications: **AASHTO LRFD Design Specs. 2012**

1 Material properties:

		Value	Unit	Metric	Unit
- Concrete density:	Wc =	0.15	kcf	24	KN/m3
- Concrete 28-day compressive strength:	f'c =	8	ksi	55	MPa
- Reinforcement strength:	fy =	60	ksi	414	MPa
- Prestressing steel strand:	Diameter =	0.60	in	15	mm
Strand cross-sectional area:	A =	0.217	in2	140	mm2
Modulus of Elasticity:	Ep =	28478	ksi	196500	MPa
Ultimate tensile strength:	Fpu =	58.43	kips	260	KN
	fpu =	269	ksi	1858	MPa
Minimum yield strength:	Fpy =	52.58	kips	234	KN
	fpy =	90%	fpu		
Maximum stressing force per strand:	Fp max =	43.82	kips	195	KN
	fp jacking =	75%	fpu		
% prestressing stress after transfer with Max losses:	fpt =	67%	fpu		
% Effective prestressing stress considering Max losses:	fpe =	63%	fpu		
Effective prestressing strain considering Max losses:	ε_{po} =	0.00592			
Wedge seating:	Δ =	0.39	in	10	mm
Friction coefficient:	μ =	0.15	/rad	0.15	/rad
Wobble coefficient:	k =	0.000201	/ft	0.00066	/m
- Tendons:					
Duct diameter:	OD =	5.51	in	140	mm
Location of tendon CG with respect to duct CG	Z =	0.47	in	12	mm

2 Bearing location:

- Eccentricity between shaft and top of pier cap:	e1 =	39.17	in	995	mm

3 Loads:

- Case — Maximum Shear Key: Vuy max

- *Loading effects at the section of maximum effect:*

		PT + EQ + DL		PT + EQ + DL	
Axial (if <0 → Tension)	Pu =	35624	kips	159	MN
In-plane shear	Vuy =	20662	kips	92	MN
Out-of-plane shear	Vux =	1072	kips	5	MN
In-plane-bending moment: (if >0 → Top fiber in tension)	Mux =	506353	kip-ft	688	MN-m
Out-of-plane bending moment	Muy =	13092	kip-ft	18	MN-m
Torsion	Tu =	14049	kip-ft	19	MN-m

4 Pier cap section properties:

- Section No. 1 — Location = SK

- *Dimensions:*

Location from cap end	x =	30.51	ft			9298	mm
Width	B =	216.5		208	in	5500	mm
Depth	H =	236.2		228	in	6000	mm
Concrete cover:	Top =	4.33	in			110	mm
	Bottom =	4.33	in			110	mm
	Lateral =	4.33	in			110	mm

- *Section properties:*

		Gross	Eff at ε_{cu}	
Area	A =	51150	47304	in2
Centroid (with respect to left bottom corner)	yc =	118.110	114	in
	xc =	108	104	in
Moment of inertia	Ix =	237848451	204127410	in4
	Iy =	199858768	170338765	in4
Section modulus	Sx =	2013784	1794061	in3
	Sy =	1845968	1638865	in3

- *Longitudinal tendon locations:*

*from section centerline
**from bottom of gross section

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	ε_γ (rad)	x (mm)	y (mm)
1	B6.1	-54.09	22.60	36	7.81	0.25	-1374	492
2	B6.2	-25.75	22.60	37	8.03	0.25	-654	492
3	B6.3	18.03	22.60	37	8.03	0.25	458	492
4	B6.4	61.81	22.60	36	7.81	0.25	1570	492

5 Check for axial and flexural force effects:

- *Resistance factor:* A.5.10.11.4.1b ϕ = 0.90

- *Loading effects:*

1 Prestressing loading effects:

Pu =	32218	kips	143	MN
Mux =	-3171	kip-ft	-4	MN-m
Muy =	-1551	kip-ft	-2	MN-m

2 Net loading effects:

Pu =	3406	kips	15	MN
Mux =	503182	kip-ft	682	MN-m
Muy =	11541	kip-ft	16	MN-m

3 Total loading stresses:

Pu/A =	0.696	ksi	4.8	MPa
Mux/Sx =	3.017	ksi	20.8	MPa
Muy/Sy =	0.085	ksi	0.6	MPa
Max =	3.799	ksi	26.2	MPa
	47%	f'c	47%	
Min =	-2.406	ksi	-16.6	MPa
	30%	f'c	30%	

- *Analysis approach:* *Strain compatibility*

- *Concrete behavior assumptions:* *Unconfined*

Stress distribution: A.5.7.2.2 *Rectangular*

α = 0.85

β = 0.65

Compressive strength: f'c = 8 ksi

Ultimate compression strain: ε_{cu} = 0.004 *

* Seismic Design of Reinforced Concrete and Masonry Buildings, Paulay and Priestly, pag. 98

Ignore concrete cover for calculations at this strain level

Modulus of Elasticity: A.5.4.2.4-1 E = 5422 ksi

Effective section properties: b = 208 in 5280 mm

h = 228 in 5780 mm

- *Major-axis bending resistance: Pnx, Mnx*

Eccentricity: ey = 1772.91 in

Neutral axis depth: c = 64.80 in

a = 42.12 in

Concrete forces: Pc = 59538 kips

Mc = 460028 kip-ft

Tendon forces:

Effective initial prestressing strain: ε_{po} = 0.00592

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B6.1	18.27	0.00305	86.92	679	-96	-5404
2	B6.2	18.27	0.00305	86.92	698	-96	-5554
3	B6.3	18.27	0.00305	86.92	698	-96	-5554
4	B6.4	18.27	0.00305	86.92	679	-96	-5404
5	B5.1	23.86	0.00340	96.74	756	-90	-5663
6	B5.2	23.86	0.00340	96.74	777	-90	-5821
7	B5.3	23.86	0.00340	96.74	777	-90	-5821
8	B5.4	23.86	0.00340	96.74	756	-90	-5663
9	B4.1	31.26	0.00385	109.76	881	-83	-6060
10	B4.2	31.26	0.00385	109.76	881	-83	-6060
11	B4.3	31.26	0.00385	109.76	881	-83	-6060
12	B4.4	31.26	0.00385	109.76	881	-83	-6060
13	T3.1	201.69	0.01437	263.82	2061	88	15099

14	T3.2	201.69	0.01437	263.82	2118	88	15519
15	T3.3	201.69	0.01437	263.82	2118	88	15519
16	T3.4	201.69	0.01437	263.82	2061	88	15099
17	T2.1	207.28	0.01472	264.07	2120	94	16521
18	T2.2	207.28	0.01472	264.07	2120	94	16521
19	T2.3	207.28	0.01472	264.07	2120	94	16521
20	T2.4	207.28	0.01472	264.07	2120	94	16521
21	T1.1	207.28	0.01472	264.07	2063	94	16074
22	T1.2	207.28	0.01472	264.07	2120	94	16521
23	T1.3	207.28	0.01472	264.07	2120	94	16521
24	T1.4	207.28	0.01472	264.07	2063	94	16074
					34549		123382

Reinforcement forces:

No.	Name	yeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B1	2.11	-0.00387	-60.00	-4746	-112	44169
2	B2	7.98	-0.00351	-60.00	-4746	-106	41845
3	B3	13.86	-0.00314	-60.00	-4746	-100	39520
4	B4	19.74	-0.00278	-60.00	-2848	-94	22317
5	B5	25.62	-0.00242	-60.00	-2848	-88	20922
6	B6	31.49	-0.00206	-59.62	-2830	-82	19404
7	B7	37.37	-0.00169	-49.10	-2330	-76	14839
8	B8	43.25	-0.00133	-38.58	-2136	-71	12556
9	B9	49.13	-0.00097	-28.06	-1554	-65	8370
10	B10	55.00	-0.00060	-17.53	-971	-59	4756
11	B11	60.88	-0.00024	-7.01	-388	-53	1712
12	M1	66.76	0.00012	3.51	47	-47	-186
13	M2	72.64	0.00048	14.03	189	-41	-650
14	M3	78.51	0.00085	24.55	332	-35	-974
15	M4	84.39	0.00121	35.07	474	-29	-1160
16	M5	90.27	0.00157	45.59	616	-24	-1206
17	M6	96.15	0.00194	56.12	758	-18	-1114
18	M7	102.02	0.00230	60.00	810	-12	-794
19	M8	107.90	0.00266	60.00	810	-6	-397
20	M9	113.78	0.00302	60.00	810	0	0
21	M10	119.66	0.00339	60.00	810	6	397
22	M11	125.53	0.00375	60.00	810	12	794
23	M12	131.41	0.00411	60.00	810	18	1191
24	M13	137.29	0.00447	60.00	810	24	1588
25	M14	143.17	0.00484	60.00	810	29	1984
26	M15	149.04	0.00520	60.00	810	35	2381
27	M16	154.92	0.00556	60.00	810	41	2778
28	T1	160.80	0.00593	60.00	3085	47	12088
29	T2	166.68	0.00629	60.00	1187	53	5231
30	M17	172.55	0.00665	60.00	810	59	3969
31	M18	178.43	0.00701	60.00	810	65	4366
32	M19	184.31	0.00738	60.00	810	71	4763
33	M20	190.19	0.00774	60.00	810	76	5160
34	M21	196.07	0.00810	60.00	810	82	5557
35	M22	201.94	0.00847	60.00	810	88	5953
36	T3	207.82	0.00883	60.00	8543	94	66951
37	T4	213.70	0.00919	60.00	6645	100	55328
38	T5	219.58	0.00955	60.00	6407	106	56490
39	T6	225.45	0.00992	60.00	6407	112	59629
					17512		520527

Resultant forces:

Pnx =	7477	kips	33	MN
Mnx =	1103937	kip-ft	1497	MN-m
ey =	1771.80	in		
ey req/ ey res =	1.00		1.00	

6 Check for shear and torsional force effects:

- *Resistance factor:* A.5.5.4.2 ϕ = 0.90

- *Torsional effects:*

Total area enclosed by outside perimeter of concrete:	Acp =	51150	in2		
Outside perimeter of the concrete section:	pc =	906	in		
Diameter of exterior stirrups:	d =	0.98	in	25	mm
Spacing of exterior stirrups:	s =	12.60	in	320	mm
Area enclosed by the shear path:	Ao =	46876	in2		
Perimeter of centerline of exterior stirrups:	ph =	867	in		
Torsional cracking moment:	Tcr =	141976	kip-ft	193	MN-m
	0.25ϕTcr=	31945	kip-ft	43	MN-m
	Tu =	14049	kip-ft	19	MN-m

Torsional effects DO NOT need to be considered for shear checking

Torsional resistance:	ϕTn=	38227	kip-ft	52	MN-m
	Tu/ϕTn =	0.37	OK!	0.37	

- *In-plane shear:*

Factored load:	Vuy =	20662	Kips	92	MN
Prestressing force in direction of Vu:	Vp =	2405	Kips	11	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2		
Effective width	bv =	217	in		
Minimum Shear depth:	dv =	189	in		
Net tensile Strains:	$\varepsilon_{average}$ =	0.00134			
Crack angle:	θ =	33.7			
Parameter β assuming min. transverse reinforcement:	β=	2.396			
In-plane concrete shear resistance:	Vc =	8753	Kips	39	MN
In-plane steel shear resistance:	Vs =	30588	Kips	136	MN
Inclination of reinforcement:	α=	90	deg		
Diameter of stirrups:	d =	1.69	in	43	mm
Spacing of stirrups:	s =	10.00	in	254	mm
No. of reinforcement legs:	n =	8			
	ϕVn =	35407	Kips	158	MN
	Vu/ϕVn =	0.58	OK!	0.58	

- *Out-of-plane shear:*

Factored load:	Vux =	1072	Kips	5	MN
Prestressing force in direction of Vu:	Vp =	0	Kips	0	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2		
Effective width	bv =	236	in		
Minimum Shear depth:	dv =	156	in		
Net tensile Strains:	$\varepsilon_{average}$ =	0.00000			
Parameter β assuming min. transverse reinforcement:	β=	4.800			
In-plane concrete shear resistance:	Vc =	15800	Kips	70	MN
	ϕVc =	14220	Kips	63	MN
	Vu/ϕVc =	0.07	OK!	0.07	

- *Minor-axis bending resistance: Pny, Mny*

Eccentricity:	ex =	40.66	in
Neutral axis depth:	c =	210.06	in
	a =	136.54	in
Concrete forces:	Pc =	211279	kips
	Mc =	627992	kip-ft

Tendon forces:

Effective initial prestressing strain: ε_{po}= 0.00592

No.	Name	xeff (in)	ε_{si}	fsi (ksi)	Tsi (kips)	di (in)	Msi (kip-ft)
1	B6.1	158.03	0.00493	140.50	1098	54	4948
2	B6.2	129.69	0.00439	125.13	1005	26	2156
3	B6.3	85.91	0.00356	101.39	814	-18	-1223
4	B6.4	42.13	0.00273	77.65	607	-62	-3124
5	B5.1	156.77	0.00491	139.82	1092	53	4809
6	B5.2	128.43	0.00437	124.45	999	24	2039
7	B5.3	87.17	0.00358	102.07	820	-17	-1145
8	B5.4	43.39	0.00275	78.33	612	-61	-3088
9	B4.1	157.40	0.00492	140.16	1125	53	5014
10	B4.2	129.06	0.00438	124.79	1002	25	2097
11	B4.3	86.54	0.00357	101.73	817	-17	-1184
12	B4.4	42.76	0.00274	77.99	626	-61	-3193
13	T3.1	158.03	0.00493	140.50	1098	54	4948
14	T3.2	129.69	0.00439	125.13	1005	26	2156
15	T3.3	100.08	0.00383	109.08	876	-4	-282
16	T3.4	40.83	0.00270	76.94	601	-63	-3161
17	T2.1	143.23	0.00465	132.48	1064	39	3483
18	T2.2	114.88	0.00411	117.10	940	11	858
19	T2.3	86.54	0.00357	101.73	817	-17	-1184
20	T2.4	58.19	0.00303	86.36	693	-46	-2643
21	T1.1	156.77	0.00491	139.82	1092	53	4809
22	T1.2	128.43	0.00437	124.45	999	24	2039
23	T1.3	101.34	0.00385	109.76	881	-3	-191
24	T1.4	42.09	0.00273	77.63	606	-62	-3126
					21288		15810

Resultant forces:	Pny =	189991	kips	845	MN
	Mny =	643802	kip-ft	873	MN-m
	ex =	40.66	in		
	ex req/ ex res =	1.00		1.00	

- *Biaxial flexure check:* A.5.7.4.5

Minimum axial load to be considered as a column:	0.10ϕf'cAg =	36828	kips	164	MN
Factored axial load:	Pu =	3406	kips	15	MN
Nominal axial resistance:	Po =	309901	kips	1379	MN
Method to check biaxial flexure:		Load Contour Method			
Reciprocal Load Method:	Pr =	NA	Kips		
	Pu/Pr =	NA	NA	NA	
Load Contour Method:	Mux/Mrx =	0.51			
	Muy/Mry =	0.02			
	IE =	0.53	OK!	0.53	

0 Specifications: **AASHTO LRFD Design Specs. 2012**

1 Material properties:

				Metric	
- Concrete density:	Wc =	0.15	kcf	24	KN/m3
- Concrete 28-day compressive strength:	f'c =	8	ksi	55	MPa
- Reinforcement strength:	fy =	60	ksi	414	MPa
- Prestressing steel strand:	Diameter =	0.60	in	15	mm
Strand cross-sectional area:	A =	0.217	in2	140	mm2
Modulus of Elasticity:	Ep =	28478	ksi	196500	MPa
Ultimate tensile strength:	Fpu =	58.43	kips	260	KN
	fpu =	269	ksi	1858	MPa
Minimum yield strength:	Fpy =	52.58	kips	234	KN
	fpy =	90%	fpu		
Maximum stressing force per strand:	Fp max =	43.82	kips	195	KN
	fp jacking =	75%	fpu		
% prestressing stress after transfer with Max losses:	fpt =	67%	fpu		
% Effective prestressing stress considering Max losses:	fpe =	63%	fpu		
Effective prestressing strain considering Max losses:	ε_{po} =	0.00592			
Wedge seating:	Δ =	0.39	in	10	mm
Friction coefficient:	μ =	0.15	/rad	0.15	/rad
Wobble coefficient:	k =	0.000201	/ft	0.00066	/m
- Tendons:					
Duct diameter:	OD =	5.51	in	140	mm
Location of tendon CG with respect to duct CG	Z =	0.47	in	12	mm

2 Bearing location:

- Eccentricity between shaft and top of pier cap:	e1 =	39.17	in	995	mm

3 Loads:

- Case — Maximum Shear Key: Pu min

- *Loading effects at the section of maximum effect:*		PT + EQ + DL		PT + EQ + DL	
Axial (if <0 → Tension)	Pu =	46182	kips	206	MN
In-plane shear	Vuy =	20336	kips	90	MN
Out-of-plane shear	Vux =	1840	kips	8	MN
In-plane-bending moment: (if >0 → Top fiber in tension)	Mux =	866517	kip-ft	1177	MN-m
Out-of-plane bending moment	Muy =	23576	kip-ft	32	MN-m
Torsion	Tu =	24111	kip-ft	33	MN-m

4 Pier cap section properties:

- Section No. 1
- Location = SK

- *Dimensions:*

Location from cap end	x =	30.51	ft		9298	mm
Width	B =	216.5	208	in	5500	mm
Depth	H =	236.2	228	in	6000	mm
Concrete cover:	Top =	4.33	in		110	mm
	Bottom =	4.33	in		110	mm
	Lateral =	4.33	in		110	mm

- *Section properties:*

		Gross	Eff at ε_{pu}	
Area	A =	51150	47304	in2
Centroid (with respect to left bottom corner)	yc =	118.110	114	in
	xc =	108	104	in
Moment of inertia	Ix =	237848451	204127410	in4
	Iy =	199858768	170338765	in4
Section modulus	Sx =	2013784	1794061	in3
	Sy =	1845968	1638865	in3

- *Longitudinal tendon locations:*
  *from section centerline
  **from bottom of gross section

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	(rad)	x (mm)	y (mm)
1	B6.1	-54.09	22.60	36	7.81	0.25	-1374	492
2	B6.2	-25.75	22.60	37	8.03	0.25	-654	492
3	B6.3	18.03	22.60	37	8.03	0.25	458	492
4	B6.4	61.81	22.60	36	7.81	0.25	1570	492

No.	Name	x* (in)	y**(in)	Strands	Area (in2)	(rad)	x (mm)	y (mm)
5	B5.1	-52.83	28.19	36	7.81	0.43	-1342	634
6	B5.2	-24.49	28.19	37	8.03	0.43	-622	634
7	B5.3	16.77	28.19	37	8.03	0.43	426	634
8	B5.4	60.55	28.19	36	7.81	0.43	1538	634
9	B4.1	-53.46	35.59	37	8.03	0.48	-1358	822
10	B4.2	-25.12	35.59	37	8.03	0.48	-638	822
11	B4.3	17.40	35.59	37	8.03	0.48	442	822
12	B4.4	61.18	35.59	37	8.03	0.48	1554	822
13	T3.1	-54.09	206.02	36	7.81	0.21	-1374	5175
14	T3.2	-25.75	206.02	37	8.03	0.21	-654	5175
15	T3.3	3.86	206.02	37	8.03	0.21	98	5175
16	T3.4	63.11	206.02	36	7.81	0.21	1603	5175
17	T2.1	-39.29	211.61	37	8.03	0.12	-998	5317
18	T2.2	-10.94	211.61	37	8.03	0.12	-278	5317
19	T2.3	17.40	211.61	37	8.03	0.12	442	5317
20	T2.4	45.75	211.61	37	8.03	0.12	1162	5317
21	T1.1	-52.83	211.61	36	7.81	0.00	-1342	5317
22	T1.2	-24.49	211.61	37	8.03	0.00	-622	5317
23	T1.3	2.60	211.61	37	8.03	0.00	66	5317
24	T1.4	61.85	211.61	36	7.81	0.00	1571	5317
					190.96	0.25		
	Centroids =	-0.58	119.29 in					
	Eccentricity =	-0.58	1.18 in					

- *Longitudinal reinforcement locations:*
  **from bottom of gross section

No.	Name	db (in)	y**(in)	Qty	Area (in2)	db (mm)	y (mm)
1	B1	2.24	6.44	20	79.10	57	164
2	B2	2.24	12.31	20	79.10	57	313
3	B3	2.24	18.19	20	79.10	57	462
4	B4	2.24	24.07	12	47.46	57	611
5	B5	2.24	29.95	12	47.46	57	761
6	B6	2.24	35.82	12	47.46	57	910
7	B7	2.24	41.70	12	47.46	57	1059
8	B8	2.24	47.58	14	55.37	57	1209
9	B9	2.24	53.46	14	55.37	57	1358
10	B10	2.24	59.33	14	55.37	57	1507
11	B11	2.24	65.21	14	55.37	57	1656
12	M1	1.69	71.09	6	13.51	43	1806
13	M2	1.69	76.97	6	13.51	43	1955
14	M3	1.69	82.85	6	13.51	43	2104
15	M4	1.69	88.72	6	13.51	43	2254
16	M5	1.69	94.60	6	13.51	43	2403
17	M6	1.69	100.48	6	13.51	43	2552
18	M7	1.69	106.36	6	13.51	43	2701
19	M8	1.69	112.23	6	13.51	43	2851
20	M9	1.69	118.11	6	13.51	43	3000
21	M10	1.69	123.99	6	13.51	43	3149
22	M11	1.69	129.87	6	13.51	43	3299
23	M12	1.69	135.74	6	13.51	43	3448
24	M13	1.69	141.62	6	13.51	43	3597
25	M14	1.69	147.50	6	13.51	43	3746
26	M15	1.69	153.38	6	13.51	43	3896
27	M16	1.69	159.25	6	13.51	43	4045
28	T1	2.24	165.13	13	51.42	57	4194
29	T2	2.24	171.01	5	19.78	57	4344
30	M17	1.69	176.89	6	13.51	43	4493
31	M18	1.69	182.76	6	13.51	43	4642
32	M19	1.69	188.64	6	13.51	43	4791
33	M20	1.69	194.52	6	13.51	43	4941
34	M21	1.69	200.40	6	13.51	43	5090
35	M22	1.69	206.27	6	13.51	43	5239
36	T3	2.24	212.15	36	142.39	57	5389
37	T4	2.24	218.03	28	110.75	57	5538
38	T5	2.24	223.91	27	106.79	57	5687
39	T6	2.24	229.78	27	106.79	57	5837
					1483.69		

5 Check for axial and flexural force effects:

- *Resistance factor:* A.5.10.11.4.1b — ϕ = 0.90

- *Loading effects:*

1 Prestressing loading effects:	Pu = 32218	kips	143	MN
	Mux = -3171	kip-ft	-4	MN-m
	Muy = -1551	kip-ft	-2	MN-m
2 Net loading effects:	Pu = 13964	kips	62	MN
	Mux = 863346	kip-ft	1171	MN-m
	Muy = 22025	kip-ft	30	MN-m
3 Total loading stresses:	Pu/A = 0.903	ksi	6.2	MPa
	Mux/Sx = 5.164	ksi	35.6	MPa
	Muy/Sy = 0.153	ksi	1.1	MPa
	Max = 6.220	ksi	42.9	MPa
	78%	f'c	78%	
	Min = -4.414	ksi	-30.5	MPa
	55%	f'c	55%	

- *Analysis approach:* *Strain compatibility*

- *Concrete behavior assumptions:* *Unconfined*

Stress distribution: A.5.7.2.2 — *Rectangular*

α_1 = 0.85

β_1 = 0.65

Compressive strength: f'c = 8 ksi

Ultimate compression strain: ε_u = 0.004 *

* Seismic Design of Reinforced Concrete and Masonry Buildings, Paulay and Priestly, pag. 98
Ignore concrete cover for calculations at this strain level

Modulus of Elasticity: A.5.4.2.4-1 — E = 5422 ksi

Effective section properties: b = 208 in — 5280 mm; h = 228 in — 5780 mm

- *Major-axis bending resistance: Pnx, Mnx*

Eccentricity: ey = 741.91 in

Neutral axis depth: c = 72.29 in; a = 46.99 in

Concrete forces: Pc = 66418 kips; Mc = 499716 kip-ft

Tendon forces:
Effective initial prestressing strain: ε_{po} = 0.00592

No.	Name	yeff (in)	ε_{si}	fsi (ksi)	Tsi (kips)	di (in)	Msi (kip-ft)
1	B6.1	18.27	0.00294	83.59	653	-96	-5198
2	B6.2	18.27	0.00294	83.59	671	-96	-5342
3	B6.3	18.27	0.00294	83.59	671	-96	-5342
4	B6.4	18.27	0.00294	83.59	653	-96	-5198
5	B5.1	23.86	0.00324	92.40	722	-90	-5409
6	B5.2	23.86	0.00324	92.40	742	-90	-5559
7	B5.3	23.86	0.00324	92.40	742	-90	-5559
8	B5.4	23.86	0.00324	92.40	722	-90	-5409
9	B4.1	31.26	0.00365	104.06	836	-83	-5746
10	B4.2	31.26	0.00365	104.06	836	-83	-5746
11	B4.3	31.26	0.00365	104.06	836	-83	-5746
12	B4.4	31.26	0.00365	104.06	836	-83	-5746
13	T3.1	201.69	0.01309	262.67	2052	88	15033

14	T3.2	201.69	0.01309	262.67	2109	88	15451
15	T3.3	201.69	0.01309	262.67	2109	88	15451
16	T3.4	201.69	0.01309	262.67	2052	88	15033
17	T2.1	207.28	0.01339	262.99	2112	94	16453
18	T2.2	207.28	0.01339	262.99	2112	94	16453
19	T2.3	207.28	0.01339	262.99	2112	94	16453
20	T2.4	207.28	0.01339	262.99	2112	94	16453
21	T1.1	207.28	0.01339	262.99	2055	94	16009
22	T1.2	207.28	0.01339	262.99	2112	94	16453
23	T1.3	207.28	0.01339	262.99	2112	94	16453
24	T1.4	207.28	0.01339	262.99	2055	94	16009
					34018		125708

Reinforcement forces:

No.	Name	yeff (in)	ε_{si}	fsi (ksi)	Tsi (kips)	di (in)	Msi (kip-ft)
1	B1	2.11	-0.00388	-60.00	-4746	-112	44169
2	B2	7.98	-0.00356	-60.00	-4746	-106	41845
3	B3	13.86	-0.00323	-60.00	-4746	-100	39520
4	B4	19.74	-0.00291	-60.00	-2848	-94	22317
5	B5	25.62	-0.00258	-60.00	-2848	-88	20922
6	B6	31.49	-0.00226	-60.00	-2848	-82	19528
7	B7	37.37	-0.00193	-56.03	-2659	-76	16933
8	B8	43.25	-0.00161	-46.60	-2580	-71	15166
9	B9	49.13	-0.00128	-37.17	-2058	-65	11088
10	B10	55.00	-0.00096	-27.73	-1536	-59	7522
11	B11	60.88	-0.00063	-18.30	-1013	-53	4468
12	M1	66.76	-0.00031	-8.87	-120	-47	469
13	M2	72.64	0.00002	0.56	8	-41	-26
14	M3	78.51	0.00034	9.99	135	-35	-397
15	M4	84.39	0.00067	19.42	262	-29	-642
16	M5	90.27	0.00100	28.86	390	-24	-764
17	M6	96.15	0.00132	38.29	517	-18	-760
18	M7	102.02	0.00165	47.72	644	-12	-631
19	M8	107.90	0.00197	57.15	772	-6	-378
20	M9	113.78	0.00230	60.00	810	0	0
21	M10	119.66	0.00262	60.00	810	6	397
22	M11	125.53	0.00295	60.00	810	12	794
23	M12	131.41	0.00327	60.00	810	18	1191
24	M13	137.29	0.00360	60.00	810	24	1588
25	M14	143.17	0.00392	60.00	810	29	1984
26	M15	149.04	0.00425	60.00	810	35	2381
27	M16	154.92	0.00457	60.00	810	41	2778
28	T1	160.80	0.00490	60.00	3085	47	12088
29	T2	166.68	0.00522	60.00	1187	53	5231
30	M17	172.55	0.00555	60.00	810	59	3969
31	M18	178.43	0.00587	60.00	810	65	4366
32	M19	184.31	0.00620	60.00	810	71	4763
33	M20	190.19	0.00652	60.00	810	76	5160
34	M21	196.07	0.00685	60.00	810	82	5557
35	M22	201.94	0.00717	60.00	810	88	5953
36	T3	207.82	0.00750	60.00	8543	94	66951
37	T4	213.70	0.00782	60.00	6645	100	55328
38	T5	219.58	0.00815	60.00	6407	106	56490
39	T6	225.45	0.00848	60.00	6407	112	59629
					13599		536946

Resultant forces:

Pnx =	18801	kips	84	MN
Mnx =	1162370	kip-ft	1577	MN-m
ey =	741.92	in		
ey req/ ey res =	1.00		1.00	

6 Check for shear and torsional force effects:

- *Resistance factor:* A.5.5.4.2 — ϕ = 0.90

- *Torsional effects:*

Total area enclosed by outside perimeter of concrete:	Acp =	51150	in2		
Outside perimeter of the concrete section:	pc =	906	in		
Diameter of exterior stirrups:	d =	0.98	in	25	mm
Spacing of exterior stirrups:	s =	12.60	in	320	mm
Area enclosed by the shear path:	Ao =	46876	in2		
Perimeter of centerline of exterior stirrups:	ph =	867	in		
Torsional cracking moment:	Tcr =	141976	kip-ft	193	MN-m
	0.25ϕTcr=	31945	kip-ft	43	MN-m
	Tu =	24111	kip-ft	33	MN-m
	Torsional effects DO NOT need to be considered for shear checking				
Torsional resistance:	ϕTn=	34837	kip-ft	47	MN-m
	Tu/ϕTn =	0.69	OK!	0.69	

- *In-plane shear:*

Factored load:	Vuy =	20336	Kips	90	MN
Prestressing force in direction of Vu:	Vp =	2405	Kips	11	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2		
Effective width	bv =	217	in		
Minimum Shear depth:	dv =	189	in		
Net tensile Strains:	$\varepsilon_{average}$ =	0.00205			
Crack angle:	θ =	36.2			
Parameter β assuming min. transverse reinforcement:	β=	1.891			
In-plane concrete shear resistance:	Vc =	6908	Kips	31	MN
In-plane steel shear resistance:	Vs =	27875	Kips	124	MN
Inclination of reinforcement:	α=	90	deg		
Diameter of stirrups:	d =	1.69	in	43	mm
Spacing of stirrups:	s =	10.00	in	254	mm
No. of reinforcement legs:	n =	8			
	ϕVn =	31305	Kips	139	MN
	Vu/ϕVn =	0.65	OK!	0.65	

- *Out-of-plane shear:*

Factored load:	Vux =	1840	Kips	8	MN
Prestressing force in direction of Vu:	Vp =	0	Kips	0	MN
In-plane concrete shear resistance approach:	Method =	2	A.5.8.4.3.2		
Effective width	bv =	236	in		
Minimum Shear depth:	dv =	156	in		
Net tensile Strains:	$\varepsilon_{average}$ =	0.00000			
Parameter β assuming min. transverse reinforcement:	β=	4.800			
In-plane concrete shear resistance:	Vc =	15800	Kips	70	MN
	ϕVc =	14220	Kips	63	MN
	Vu/ϕVc =	0.12	OK!	0.12	

- *Minor-axis bending resistance: Pny, Mny*

Eccentricity:	ex =	18.93	in
Neutral axis depth:	c =	267.40	in
	a =	173.81	in
Concrete forces:	Pc =	268949	kips
	Mc =	381763	kip-ft

Tendon forces:

Effective initial prestressing strain: ε_{po}= 0.00592

No.	Name	xeff	ε_{si}	fsi	Tsi	di	Msi
		in		ksi	kips	in	kip-ft
1	B6.1	158.03	0.00429	122.13	954	54	4301
2	B6.2	129.69	0.00386	110.05	884	26	1896
3	B6.3	85.91	0.00321	91.40	734	-18	-1103
4	B6.4	42.13	0.00255	72.75	568	-62	-2927
5	B5.1	156.77	0.00427	121.59	950	53	4182
6	B5.2	128.43	0.00385	109.51	879	24	1794
7	B5.3	87.17	0.00323	91.94	738	-17	-1032
8	B5.4	43.39	0.00257	73.29	573	-61	-2889
9	B4.1	157.40	0.00428	121.86	978	53	4359
10	B4.2	129.06	0.00385	109.78	881	25	1845
11	B4.3	86.54	0.00322	91.67	736	-17	-1067
12	B4.4	42.76	0.00256	73.02	586	-61	-2989
13	T3.1	158.03	0.00429	122.13	954	54	4301
14	T3.2	129.69	0.00386	110.05	884	26	1896
15	T3.3	100.08	0.00342	97.44	782	-4	-252
16	T3.4	40.83	0.00254	72.20	564	-63	-2966
17	T2.1	143.23	0.00407	115.82	930	39	3045
18	T2.2	114.88	0.00364	103.74	833	11	760
19	T2.3	86.54	0.00322	91.67	736	-17	-1067
20	T2.4	58.19	0.00279	79.59	639	-46	-2436
21	T1.1	156.77	0.00427	121.59	950	53	4182
22	T1.2	128.43	0.00385	109.51	879	24	1794
23	T1.3	101.34	0.00344	97.97	787	-3	-170
24	T1.4	42.09	0.00255	72.73	568	-62	-2929
					18968		12528

Resultant forces:	Pny =	249982	kips	1112	MN
	Mny =	394291	kip-ft	535	MN-m
	ex =	18.93	in		
	ex req/ ex res =	1.00		1.00	

- *Biaxial flexure check:* A.5.7.4.5

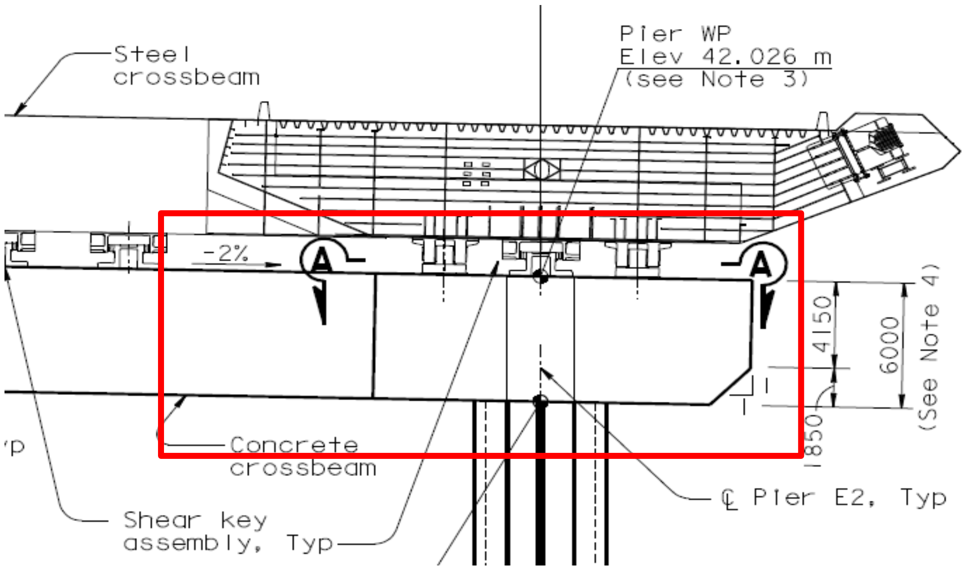
Minimum axial load to be considered as a column:	0.1ϕf'cAg =	36828	kips	164	MN
Factored axial load:	Pu =	13964	kips	62	MN
Nominal axial resistance:	Po =	309901	kips	1379	MN
Method to check biaxial flexure:		Load Contour Method			
Reciprocal Load Method:	Pr =	NA	Kips		
	Pu/Pr =	NA	NA	NA	
Load Contour Method:	Mux/Mrx =	0.83			
	Muy/Mry =	0.06			
	IE =	0.89	OK!	0.89	

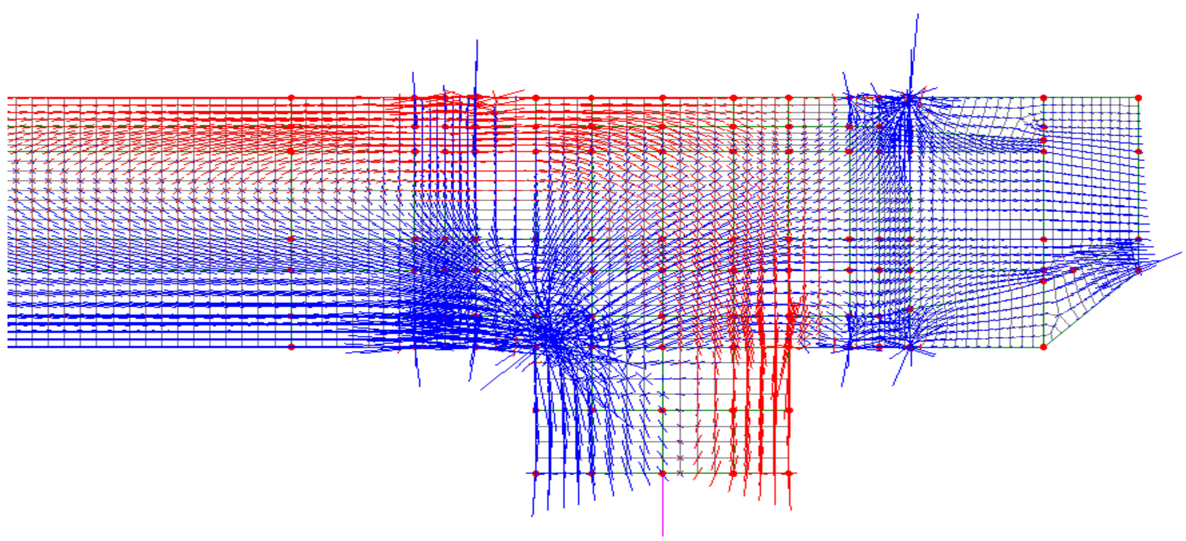
Appendix 5

Strut-and-Tie Model Calculations

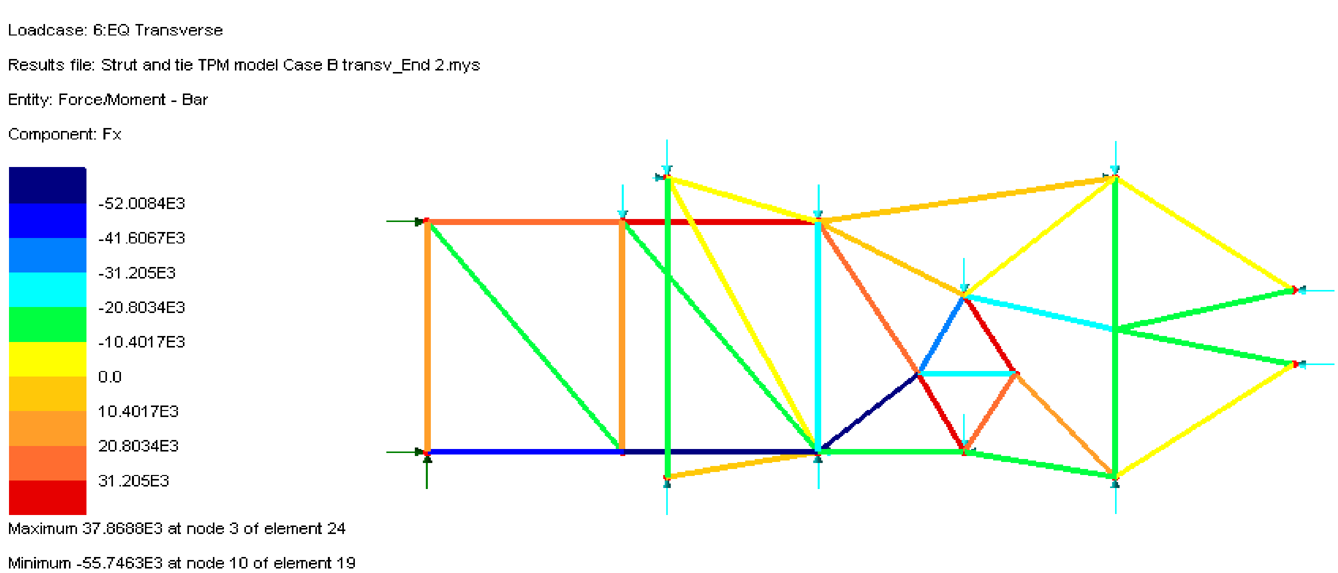
0 Specifications: AASHTO LRFD Design Specs. 2012

1 Principal Stress Directions from FEM





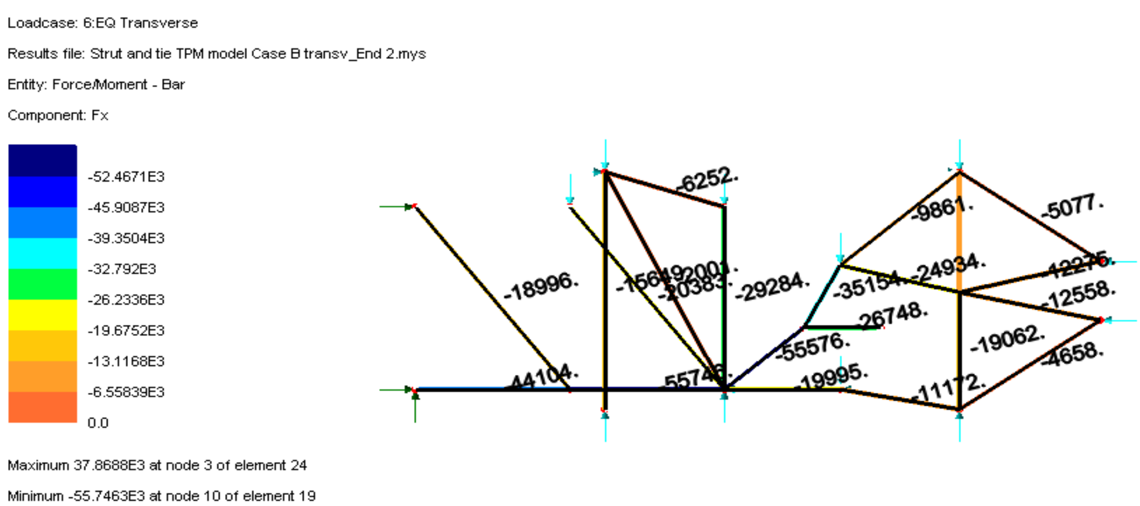
2 Strut-tie Model:

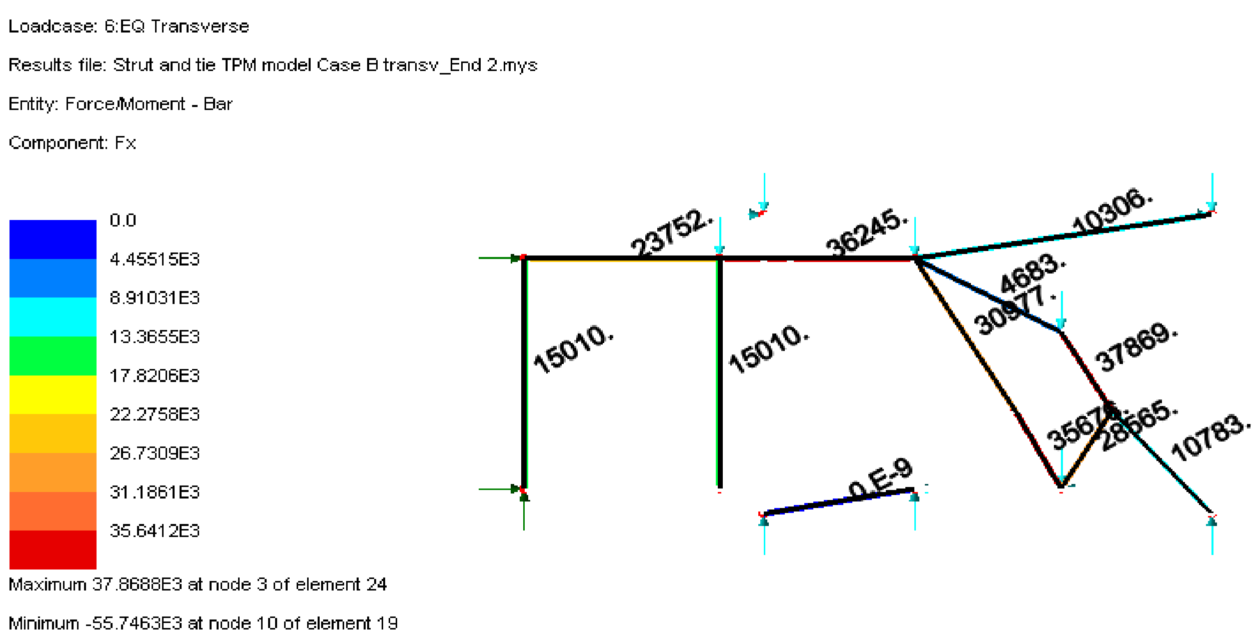


3 Boundary forces:

				US		Metric	
- Column:	Moment:			641000	kip-ft	870	MN-m
	Axial force:			25145	kips	112	MN
	Distance between resultant internal forces:			9.088	ft		
	Shear force:			13483	kips	60	MN
	Effective compression:			83105	kips	370	MN
	Effective Tension:			-57960	kips	-258	MN
- Longitudinal Post-tensioning forces:							
	Total postensiong force at service:			32121	kips	143	MN
	No. of points where the load is applied:			2			
	Force at points of application:			16061	kips	71	MN
- Bearing forces including PT forces:		Top	Hor. =	6853	kips	30	MN
			Vert. =	-19344	kips	-86	MN
		Bottom	Vert. =	15649	kips	70	MN

4 Model forces:





5 Checks:

- *Compressive struts:*

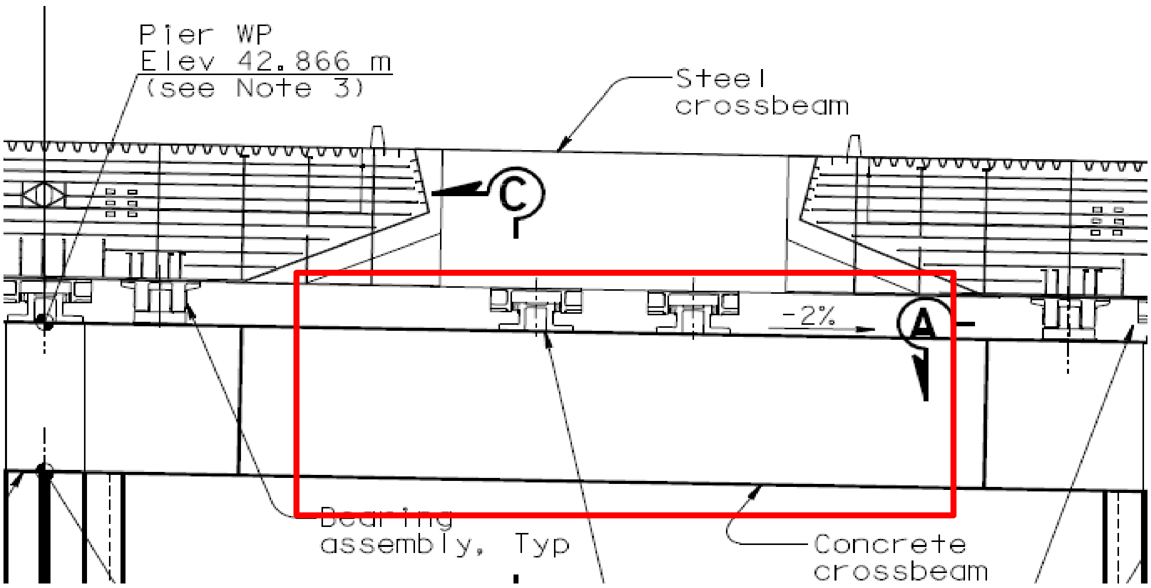
Effective cross sectional area of critical strut:	Acs =	13387.86	in2		
Diameter of the reinforcement that anchors the strut:	db =	2.24	in	57	mm
Distance between longitudinal bars in the column resisting compression:	s =	78.74	in	2000	mm
Horizontal projection of Width of strut:	la =	105.67	in	2684	mm
Inclination of strut:	θs =	40.29	deg		
Width of strut:	b1 =	68.33	in		
Depth of strut:					
Assume an average width of cap beam between column and bearings	d =	195.93	in	4976.5	mm
Limiting compressive stress in the strut:	fcu =	6.80	ksi		
Compressive concrete strength:	f'c =	8	ksi		
Angle between strut and adjoining tie:	α =	79.71	deg		
Tensile strain in the concrete in the direction of tie:					
Assume prestressing compression has not been exceeded	εs=	0.0000			
	εl=	6.59E-05			
Resistance Factor:	ϕ =	0.70			
Strut compressive resistance:	ϕPn =	63726	Kips	284	MN
Critical Compression:	Pu =	55576	Kips	247	MN
	D/C =	**0.87**	OK!		

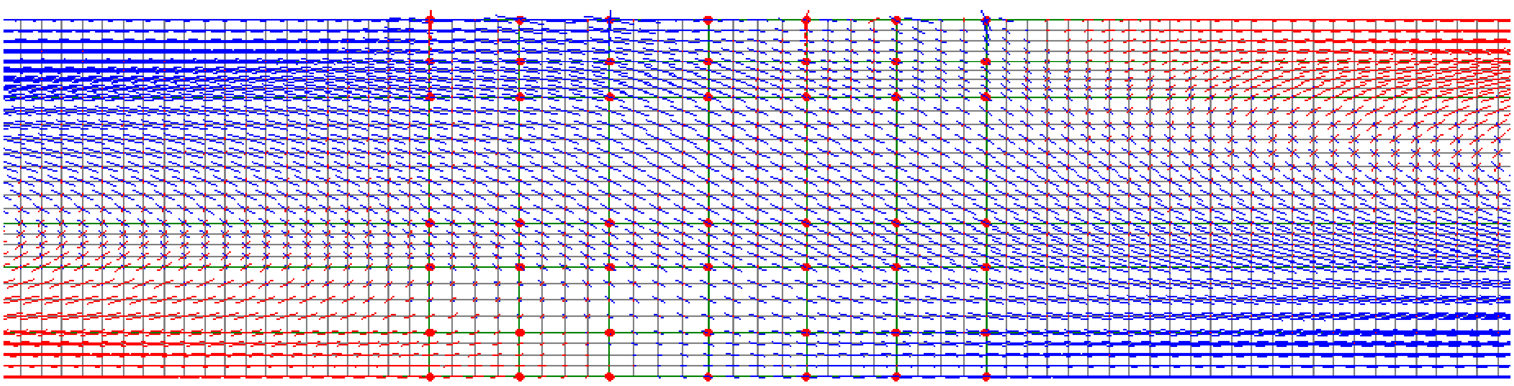
- *Tension ties:*

Area of longitudinal steel reinforcement in tie:		Ast =	852.90	in2	If uncut	
			677.97	in3	If all cut	
Column vertical reinforcement:	M bars:	db =	2.24	in	57	mm
		n =	96		If uncut	
			66		If all cut	
		db =	1.69	in	43	mm
		n =	72		If uncut	
			51		If all cut	
	N bars:	db =	1.69	in	43	mm
		n =	20		If uncut	
			16		If all cut	
Cap beam stirrups and bars:		db =	1.69	in	43	mm
		n =	102			
		db =	0.98	in	25	
		n =	48			
Yield strength of reinforcement:		fy =	60	ksi		
Resistance Factor:		ϕ =	0.90			
Tie tensile resistance:		ϕPn =	46057	Kips	205	MN
			36610	Kips	163	MN
Critical Tension:		Pu =	55634	Kips	248	MN
		D/C =	1.21	NG	If uncut	
		D/C =	1.52	NG	If all cut	

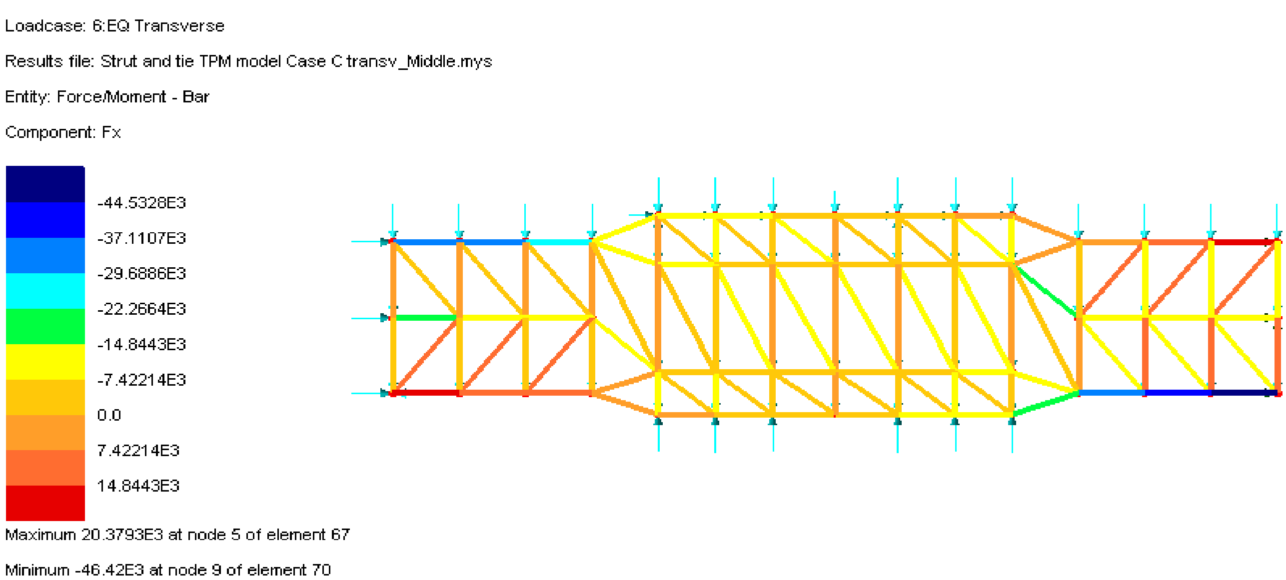
0 Specifications: AASHTO LRFD Design Specs. 2012

1 Principal Stress Directions from FEM





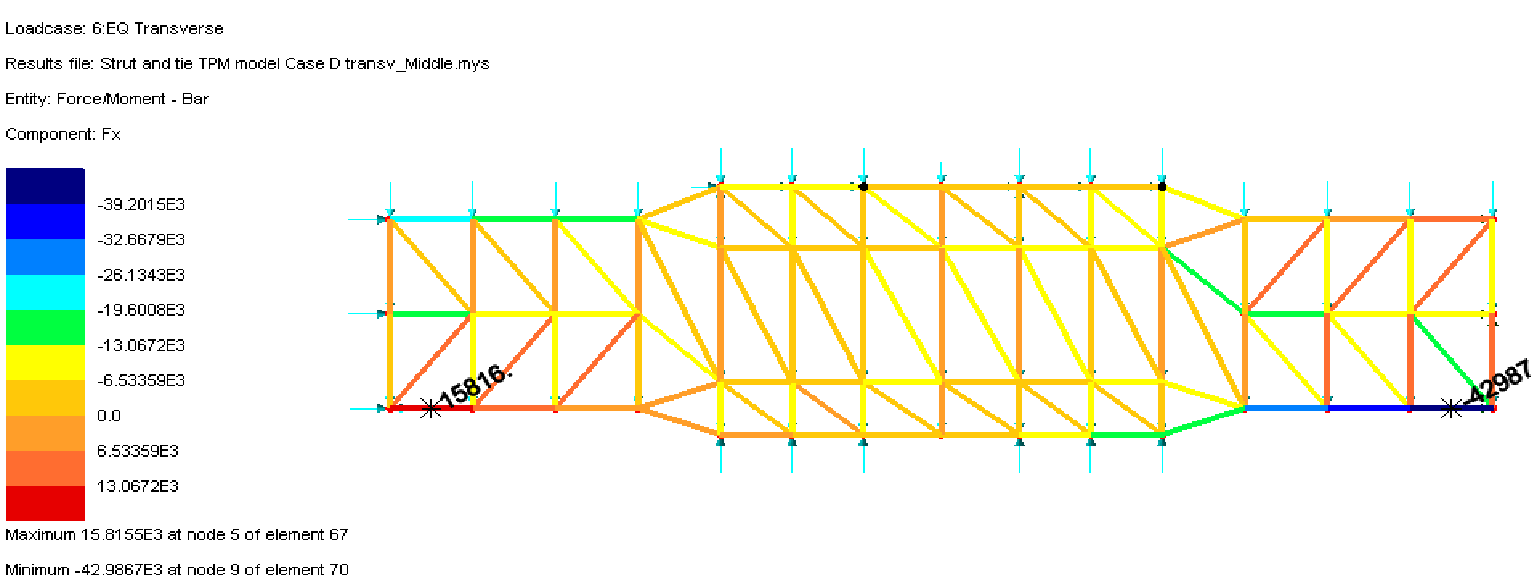
2 Strut-tie Model:



3 Boundary forces:

				US		Metric	
- Column:	Moment:			322754	kip-ft	438	MN-m
	Axial force:			13351	kips	59	MN
	Distance between resultant internal forces:			13.15	ft		
	Shear force:			12424.3	kips	55	MN
	Effective compression:	top		17868.5	kips	80	MN
	Effective Tension:	bottom		-31219.5	kips	-139	MN
- Longitudinal Post-tensioning forces:							
	Total postensiong force at service:			32121	kips	143	MN
	No. of points where the load is applied:			3			
	Midpoint:			16061	kips	71	MN
	End points:			8030	kips	36	MN
- Shear key forces including PT forces:		Midpoint:	Hor. =	2284	kips	10	MN
			Vert. =	0	kips	0	MN
		Lateral point a:	Hor. =	1142	kips	5	MN
			Vert. =	1641	kips	7	MN
		Lateral point b:	Hor. =	1142	kips	5	MN
			Vert. =	-1641	kips	-7	MN
- Shear key forces PT forces:			P/2 =	13538	kips	60	MN
			P/4 =	6769	kips	30	MN

4 Maximum Model Forces:



5 Checks:

- *Compressive struts:*

Critical Compression:	Pu =	42987	Kips	191	MN
Limiting compressive stress in the strut:	fcu =	6.80	ksi		
Compressive concrete strength:	f'c =	8	ksi		
Angle between strut and adjoining tie:	α =	90.00	deg		
Tensile strain in the concrete in the direction of tie:	εl=	7.5E-36			
Assume prestressing compression has not been exceeded	εs=	0.0000			
Resistance Factor:	ϕ =	0.70			
Reinforcement in direction of strut:	Ts =	17087	Kips		
	db =	2.24	in	57	mm
	N =	72			
	fy =	60.00	ksi		
Required cross sectional area of critical strut:	Acs =	5800.22	in2		
Depth of strut:	d =	154.72	in	3930	mm
Required width of strut:	b1 =	37.49	in	952	mm

Required width is similar to the distance over which the reinforcement is distributed ∴ OK!

- *Tension ties:*

Area of longitudinal steel reinforcement in tie:	Ast =	314.96	in2		
	db =	2.24	in	57	mm
	n =	72			
	db =	1.69	in	43	mm
	n =	8			
	db =	0.98	in	25	
	n =	16			
Yield strength of reinforcement:	fy =	60	ksi		
Area of prestressing steel in tie:	Aps =	95.48	in2		
Effective prestressing stress:	fpe =	169.47	ksi		
Resistance Factor:	ϕ =	0.90			
Tie tensile resistance:	ϕPn =	36727	Kips	163	MN
Critical Tension:	Pu =	15816	Kips	70	MN
	D/C =	0.43	OK!		