COMMONWEALTH EDISON COMPANY
CALCULATION REVISION PAGE

CALCULATION NO. 9200-EØ-S **PAGE NO.:** 0.2.56

REV: 3 **STATUS:** APPROVED **QA SERIAL NO. OR CHRON NO.** **DATE:**

PREPARED BY: S J Chhabra **DATE:** 4/4/96

REVISION SUMMARY:

CHECK FUNCTIONAL STATUS OF BEAM B4 AND CHEEK PLATE CONNECTION AT RIGHT END OF BEAM B10 IN QUAD CITIES, UNIT 1, SOUTHEAST (SE), RHR CORNER ROOM IN RESPONSE TO NCR QUESTIONS.

ADDED DCS PAGE 0.2.56

REVISED PAGES 89.9 - 89.13, 89.13.1
ADDED PAGES 89.18-89.33
ADDED PAGE " FOR REFERENCE ONLY " PAGES 89.33A1, 89.33A2

DO ANY ASSUMPTIONS IN THIS CALCULATION REQUIRE LATER VERIFICATION? YES ___ NO _X_

REVIEWED BY. [signature] **DATE:** 4/4/96

REVIEW METHOD: DETAILED **COMMENTS (C OR NC):** NC

APPROVED BY: Thomas J. Behringer **DATE:** 4/4/96

REV: **STATUS:** **QA SERIAL NO. OR CHRON NO.** **DATE:**

PREPARED BY: **DATE:**

REVISION SUMMARY:

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ELECTRONIC CALCULATION DATA FILES REVISED:
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REVIEW METHOD: **COMMENTS (C OR NC):**

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Purpose

Check functional status of Beam B4 in Quad Cities Unit 1 South-East (SE) corner room. See the background and methodology section for more detail.

References

1. AISC Manual 6th edition
2. S&L Dwg. B-273 Rev G Quad Cities Unit 1
3. ComEd calc No. QDC-0020-S-0055 Rev 0
4. AISC Manual 9 the edition
5. AISC LRFD Manual 2nd edition
6. ComEd Calc No. QDC-0020-S-0055 Rev 0 p. 9 of 10
7. LMS Run ID SQ1SE Dated 8/26/91 16:42
8. LMS Run Dated 04/03/96 10:58:58
9. Report entitled "Sargent & Lundy Structural Design Standard E5.0 Support for Increases in Allowable Stress Above Code Defined Limits", SDS E5.0 back up calculation

Background and Methodology

In Quad Cities Unit 1 (QC1) SE corner room LMS analysis (Ref 7) beam B4 has a maximum interaction coefficient (IC) of 2.09. Subsequent refined manual calculations (Ref. 3 p. 9 of 10) indicate that the beam IC can be reduced to 1.137 using plastic section modulus and by reducing the torsional warping stresses on the beam.

The original LMS analysis as well as the later manual calculations conservatively ignore the presence of a wide flange column at 8'-5" from the west end of the beam. This column will be included in the present assessment of the functional status of beam B4.

It will also be demonstrated, using Ref. 5, that the beam section can develop its plastic capacity.

Calculations

24WF76 Properties From AISC 6 th edition Manual (Ref 1):

$bf := 8.985 \cdot in$	$tf := 0.682 \cdot in$	
$d := 23.91 \cdot in$	$tw := 0.440 \cdot in$	
$A := 22.37 \cdot in^2$	$Ix := 2096.4 \cdot in^4$	$Sx := 175.4 \cdot in^3$
	$Iy := 76.5 \cdot in^4$	$Sy := 17 \cdot in^3$
	$ry := 1.85 \cdot in$	

$Fy := 36 \cdot ksi$ Yield Stress

From Ref. 5

$Zx := 200 \cdot in^3$ Plastic Modulus

Note that major axis properties of beam has not changed significantly between AISC 6th edition manual and LRFD 2nd edition manual.

From p. 4-18, Ref. 5

$Lr := 23.4 \cdot ft$ $Lp := 8. \cdot ft$

$Mr := \frac{343 \cdot kip \cdot ft}{0.9}$ Moment Resistance at unbraced length Lr

$Mp := Zx \cdot Fy$ $Mp = 600 \cdot kip \cdot ft$

$Lb := 13.19 \cdot ft$ Beam Unbraced Length (Ref 2)

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From table 4-1 of Ref 5, for beam B4 with lateral brace at the major load point:

$$Cb = 1.67$$

Using equation F1-2 of the LRFD Specification (Ref 5):

$$Mn1 = Cb \cdot \left[Mp - (Mp - Mr) \cdot \frac{Lb - Lp}{(Lr - Lp)} \right]$$

$Mn1 = 878.81 \cdot kip \cdot ft$ $\quad$ $Mp = 600 \cdot kip \cdot ft$

$Mn = \min\left(\begin{pmatrix} Mn1 \\ Mp \end{pmatrix}\right)$ $\quad$ $Mn = 600 \cdot kip \cdot ft$ $\quad$ Nominal moment strength of the beam about the major axis.

The above calculation demonstrates that beam B4 can develop full plastic capacity in major axis bending.

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To account for the presence of the column 8'-5" from west end of the beam, the column was included in the LMS model as column C1. A run was made with only the tank load of 94.8 kips (Ref. 8). No other loads were applied. The column reaction in this run is 35 kips.

Note that this column was added with the heat exchanger tank drained of 1620 gallons of water (Ref. 2) and will be effective in resisting this water weight as well as the seismic excitation loads of the tank. Therefore, the lower bound for the column reaction under SSE can be calculated as:

$Py\_C1\_Unit := \frac{35 \cdot kips}{94.8 \cdot kips}$ Reaction at column with unit tank load

$Py\_C1\_Unit = 0.37$

$gal = 0.13 \cdot ft^3$

$Wt\_Water := 1620 \cdot gal \cdot 62.4 \cdot \frac{lbf}{ft^3}$

$Wt\_Water = 13.51 \cdot kips$ Drained Water Weight

$Tank\_VSSE := 0.16 \cdot 94.8 \cdot kips$

$Tank\_VSSE = 15.17 \cdot kips$ Vertical SSE component of tank load

$Ry\_C1 := (Wt\_Water + Tank\_VSSE) \cdot Py\_C1\_Unit$

$Ry\_C1 = 10.59 \cdot kips$ Lower bound reaction of column C1 under SSE

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The major axis moment diagram for beam B4 from the LMS analysis without the column C1 is derived below (Ref 7) (critical load case WESTSSE). These stresses are at 21 equidistant points along the beam span:

$$fbx := \begin{bmatrix} 0 \\ 4.7 \\ 9.3 \\ 14.0 \\ 18.4 \\ 22.6 \\ 27 \\ 31.2 \\ 35.3 \\ 39 \\ 36.9 \\ 33.6 \\ 29.9 \\ 26.3 \\ 22.5 \\ 18.8 \\ 15.1 \\ 11.3 \\ 7.6 \\ 3.8 \\ 0 \end{bmatrix} \cdot ksi$$

$i := 1, 2 .. 21$ Stress fbx locations in LMS output

$L := 24.25 \cdot ft$ Beam Span Length

$zi_i := (i - 1) \cdot 0.05 \cdot L$

$Mx(z) := linterp(zi, fbx, z) \cdot Sx$

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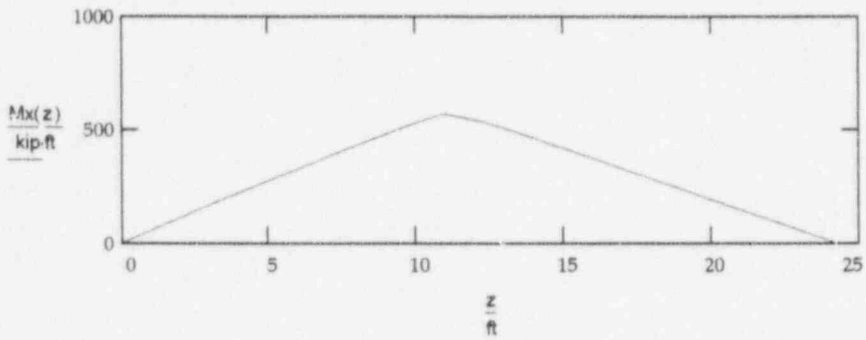
$z := 0 \cdot ft, 0.001 \cdot L .. L$ Left to Right (E to W)



$Mxmax := Mx(11 \cdot ft + 0.75 \cdot in)$

$Mxmax = 566.25 \cdot kip \cdot ft$

B4 Moment Mx Diagram without Column C1

Find the moment diagram of beam B4 due to column reaction:

$L_R := 8 \cdot ft + 5 \cdot in$ $L_L := L - L_R$ Load location; left and right

$P := Ry\_C1$

Reactions at left and right ends

$R_L := P \cdot \frac{L_R}{L}$ $R_R := P \cdot \frac{L_L}{L}$

$R_L = 3.68 \cdot kips$ $R_R = 6.91 \cdot kips$

$Mx\_C1(z) := if\left[z \le L_L, -R_L \cdot (z), -R_R \cdot (L - z)\right]$

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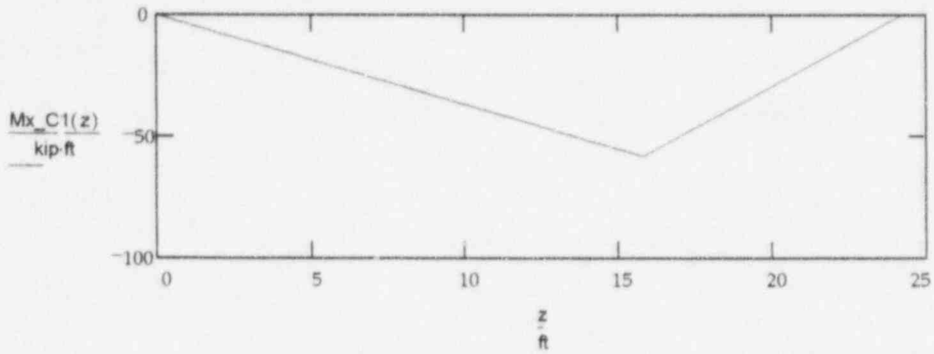
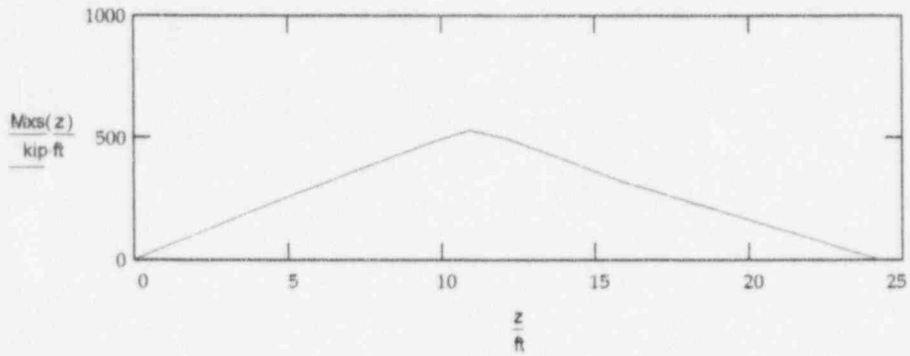


B4 Moment Mx Diagram due to Column C1 Reaction

The superimposed moment diagram for beam B4 with column C1 in place:

$$Mxs(z) := Mx(z) + Mx\_C1(z)$$



B4 Moment Mx Diagram with Column C1

$Mmax := Mxs(11 \cdot ft + 0.75 \cdot in)$ Max moment is at load location (where B8 frames into B4)

$Mmax = 525.59 \cdot kip \cdot ft$

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Revise the beam interaction calculation on p. 9 of 10 of Ref. 3 using the reduced major axis bending moment calculated (also add the direct axial load component from Ref. 7 result):

$$WSSEFIC := \frac{\frac{Mmax}{Zx}}{34.2 \cdot ksi} + \frac{(0.54 \cdot 11.7 \cdot ksi + 0.8 \cdot ksi) \cdot \frac{18.4}{1.5 \cdot 18.4}}{34.2 \cdot ksi} + \frac{0.4 \cdot ksi}{23.47 \cdot ksi}$$

$$WSSEFIC = 1.08$$

The conservatism in the calculation above are:

1. Use of 50 psf live load in LMS analysis
2. Loads based on a de-coupled seismic model of the heat exchanger tank and the piping.
3. Allowable stresses limited to 0.95Fy for bending and axial stresses
4. Specified minimum yield strength of the member is used.

Therefore, based on Ref 9, up to 10% increase in the allowable stress is permissible. Thus the 8% overstress calculated above is acceptable.

Conclusion

Beam B4 in Quad Cities Unit 1 South-East (SE) corner room is functional.

B10R Cheek Plate Connection

Purpose:

Determine stress interaction levels in cheek plate connection at right end of beam B10 using functional allowables. Use torsion at the connection based on the current hanger and gallery attachment loads. Connection ICs that are critical in LMS analysis will be addressed.

References:

1. LMS RUNID SQ2SE Dated 8/22/91 for Loads
2. Calc 8868-19-Q1-SE Rev 0 for derivation of lateral and torsional load capacity of the cheek plate.
3. AISC Manual 6th edition for beam properties
4. Vectra letter COE-348-001 Dated Dec 8, 1993 from Robert G. Carr to C. N. Petropoulos (ROL for Hanger M-1811-18)
5. Calc 8868-19-Q2-SE Rev 0 for gallery attachment loads.
6. Walkdown Info. on gallery attchements dated 4/1/96
7. Report entitled "Sargent & Lundy Structural Design Standard E5.0 Support for Increases in Allowable Stress Above Code Defined Limits", SDS E5.0 back up calculation

Methodology

Ref 2 SSE allowable equations modified to use the plastic section modulus will be used to generate the functional capacities. Torsional loads on the beam will be based upon the current data on hanger and gallery attachment loads.

Solution

$Fy := 36 \cdot ksi$ — Yield Stress

$Fb := 0.95 \cdot Fy$ — Allowable Bending stress

$L := 6.5 \cdot in$ — Cheek plate Length (Ref 2)

$D := 17 \cdot in$ — Cheek plate Depth (Ref 2)

$t := 0.375 \cdot in$ — Cheek plate Thickness (Ref 2)

$L1 := L - 0.875 \cdot in$ $\quad L1 = 5.62 \cdot in$ — Cheek plate length less beam setback

Functional Allowables based on 0.95 Mp (Mp refers to the plastic moment) by modifying old calc (see derivation in Ref 2):

Cheek Plate Functional Allowables:

Axes: x = WF major axis, y = WF minor axis; z = WF axial axis

$$Rxop := \frac{Fb \cdot D \cdot t^2}{6 \cdot (L - 0.5 \cdot L1)} \cdot 1.5 \qquad Rxop = 5.54 \cdot kips$$

$$Mzop := \frac{Fb \cdot D^2 \cdot t^2}{24 \cdot (L - 0.5 \cdot L1)} \cdot 1.5 \qquad Mzop = 1.96 \cdot kip \cdot ft$$

$$Ryop := \frac{2 \cdot Fb \cdot D^2 \cdot t}{6 \cdot (L - 0.5 \cdot L1)} \cdot 1.5$$

Similar to Rxop; based on max stress location at the same point as for Rx & Mz; use two times as both plates are effective

$Ryop = 502.57 \cdot kips$

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Under WESTSSE (critical load comb from LMS) the right reactions are:

$Rx := 2.8 \cdot kips$ $Ry := 40 \cdot kips$ $Rz := .0016 \cdot kips$

$Mx := 0. \cdot kip \cdot ft$ $My := 0 \cdot kip \cdot ft$ $Mz := 2.0 \cdot kip \cdot ft$

LMS data (ref 1) p. 46 indicates that a Ry load of 4.20 kip is applied on this beam from hanger M-1811-18. The revised hanger load data (refs 4) shows a max Ry load of 3.35 kips. Thus the Ry reaction at right support can be reduced by:

$$\delta Ry := \frac{(4.20 - 3.35) \cdot kips}{13.69 \cdot ft} \cdot 8.81 \cdot ft \qquad \delta Ry = 0.55 \cdot kips$$

$$Ry := Ry - \delta Ry \qquad Ry = 39.45 \cdot kips$$

$$Fvop := \frac{0.95 \cdot Fy}{\sqrt{3}}$$

Determine Reduced Torsion (use Ref 1 loads, except take M1811-18 loads from Ref 4)

Torsional Loads on the Beam:

Vertical Ry reaction from gallery attchemnets (Ref 5)	Eccentricity of gallery load wrt flange center line (Ref 6)	LMS ID
		M-GALL1
$Ryg := \begin{pmatrix} 0.28 \\ 0.32 \\ 0.35 \end{pmatrix} \cdot kips$	$ecc := \begin{pmatrix} -4 \\ -1 \\ 6 \end{pmatrix} \cdot in$	M-GALL2
		M-GALL4

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Torsion	LMS ID	Location of load from Left end
$Ryg_1 \cdot ecc_1$	M-GALL1	4.98
$Ryg_2 \cdot ecc_2$	M-GALL2	6.33
$Ryg_3 \cdot ecc_3$	M-GALL4	8.83
$0.063 \cdot kip \cdot ft$	M1811-18	8.81

$$Mza := \begin{bmatrix} Ryg_1 \cdot ecc_1 \\ Ryg_2 \cdot ecc_2 \\ Ryg_3 \cdot ecc_3 \\ 0.063 \cdot kip \cdot ft \end{bmatrix} \qquad Lc := \begin{bmatrix} 4.98 \\ 6.33 \\ 8.83 \\ 8.81 \end{bmatrix} \cdot ft$$

Torsional reaction at the right end of the beam:

$$Mzr := \sum_{i=1}^{3} Mza_i \cdot \frac{Lc_i}{13.69 \cdot ft} + \left| Mza_4 \cdot \frac{Lc_4}{13.69 \cdot ft} \right| \qquad Mzr = 0.11 \cdot kip \cdot ft$$

<u>Compute Cheek plate ICs</u>

$$IC\_CHK\_BND0 := \frac{Rx}{Rxop} + \frac{Mzr}{Mzop} + \frac{Ry}{Ryop} \qquad IC\_CHK\_BND0 = 0.64$$

$$Fvop := \frac{0.95 \cdot Fy}{\sqrt{3}}$$

$$IC\_CHK\_SHR0 := \frac{Rx}{Fvop \cdot D \cdot t} + \frac{Mzr}{\left(\frac{Fvop \cdot D^2 \cdot t}{4}\right)} + \frac{Ry}{Fvop \cdot D \cdot t \cdot 2} \qquad IC\_CHK\_SHR0 = 0.18$$

Web Functional Allowables:

21WF55 properties from AISC 6th edition

$tw := 0.375 \cdot in$ $k := 1.0625 \cdot in$

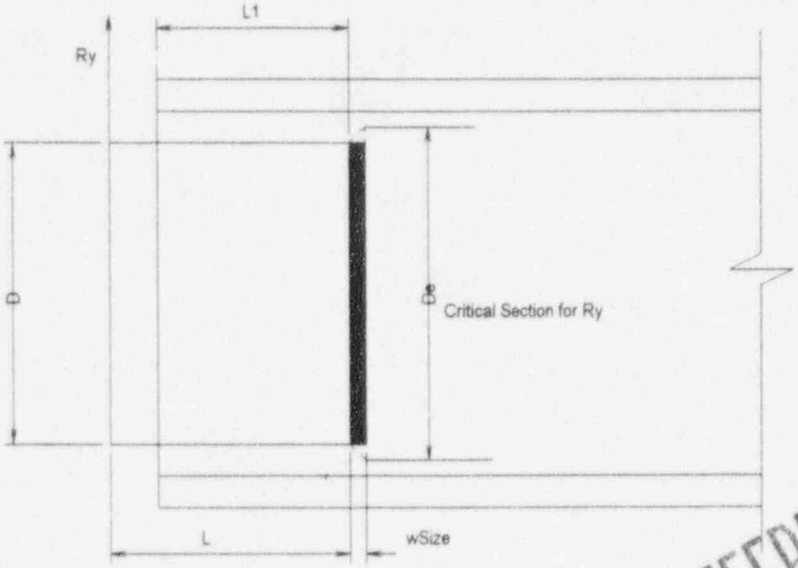
$d := 20.80 \cdot in$ $tf := 0.522 \cdot in$ $L1 = 5.62 \cdot in$

$$Mzwebop := 2 \cdot (L1 + 6 \cdot tw) \cdot \frac{tw^2}{6} \cdot Fb \cdot 1.5 \qquad Mzwebop = 1.58 \cdot kip \cdot ft$$

$$Rxwebop := \frac{Fb \cdot (L1 + 6 \cdot tw) \cdot \frac{tw^2}{6}}{\left(\frac{d - 2 \cdot k}{4} - \frac{D}{8}\right)} \cdot 1.5 \qquad Rxwebop = 3.72 \cdot kips$$

Note: Web bending span reduced from d-2tf to d-2k.

Assume that a web width of D+2*wSize can be mobilized to resist Ry. See sketch. Use Ry lever arm of L+wSize:



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$wSize := \frac{5}{16} \cdot in$

$De := D + 2 \cdot wSize \quad Le := L + wSize$

$Rywebop := \frac{\left(\frac{De^2 \cdot tw}{6}\right) \cdot Fb}{Le} \cdot 1.5 \qquad Rywebop = 146.2 \cdot kips$

$Fvop := \frac{0.95 \cdot Fy}{\sqrt{3}}$

Compute Web ICs $\qquad \frac{Rx}{Rxwebop} = 0.75 \qquad \frac{Ry}{Rywebop} = 0.27$

$IC\_WEB\_BND0 := \frac{Rx}{Rxwebop} + \frac{Mzr}{Mzwebop} + \frac{Ry}{Rywebop} \qquad IC\_WEB\_BND0 = 1.09$

$Fvop := \frac{0.95 \cdot Fy}{\sqrt{3}} \qquad \frac{Rx}{Rxwebop} = 0.75$

Approx...

$IC\_WEB\_SHR0 := \frac{Rx}{2 \cdot L1 \cdot Fvop \cdot tw} + \frac{Mzr}{(Fvop \cdot (d - 2 \cdot k) \cdot L1 \cdot tw)} + \frac{Ry}{Fvop \cdot D \cdot tw} \qquad IC\_WEB\_SHR0 = 0.35$

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To: S. CHHABRA

Extension: 6322

Location: 25

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CALC. No. 9200-ED-S
REV 3 DATE ____
PAGE 89.33A OF ____

From: R. Scoville

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Subject: ______________________

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Number of Pages Plus Cover Sheet: ______

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Conclusion

Following are the connection ICs for the two conditions investigated:

IC_CHK_BND0 = 0.64 IC_CHK_SHR0 = 0.18

IC_WEB_BND0 = 1.09 IC_WEB_SHR0 = 0.35

The conservatism in the calculation above are:

1. Use of 50 psf live load in LMS analysis
2. Loads based on a de-coupled seismic model of the heat exchanger tank and the piping.
3. Allowable stresses limited to 0.95Fy for bending and axial stresses
4. Specified minimum yield strength of the member is used.

Theretore, based on Ref ~~8~~ 7, up to 10% increase in allowables is permissible. Thus the 9% overstress shown above is acceptable.

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RWP 3007 106 10V1 SV
CLASS 2 RATE/DOSE

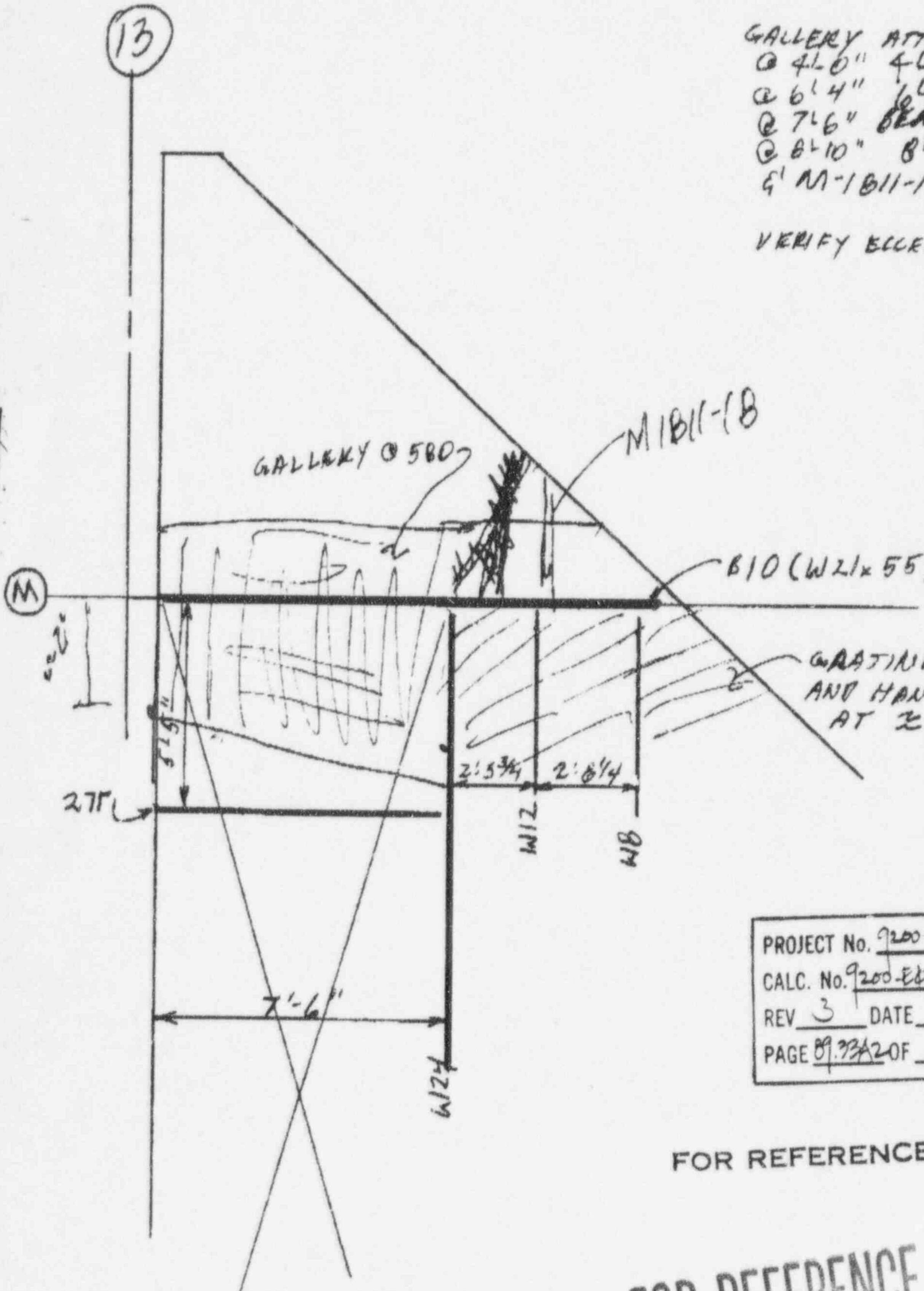
Construction Technology Laboratories, Inc.
5420 Old Orchard Road, Skokie, Illinois 60077-1030

Project ______ Sheet ______ of ______
Initials RDL Date 4-1-96
Checked ______ Date ______ Revised ______ Date ______

Title U-2 "B" RHR (SOUTHEAST)

GALLERY ATTACH FROM LEFT END
@ 4'-0" 4'11 3/4 @ 4" E
@ 6'4" 6'5" @ 1" E
@ 7'6" BEAM
@ 8'10" 8'10 W
& M-1811-18

VERIFY ECCENTRICITIES



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