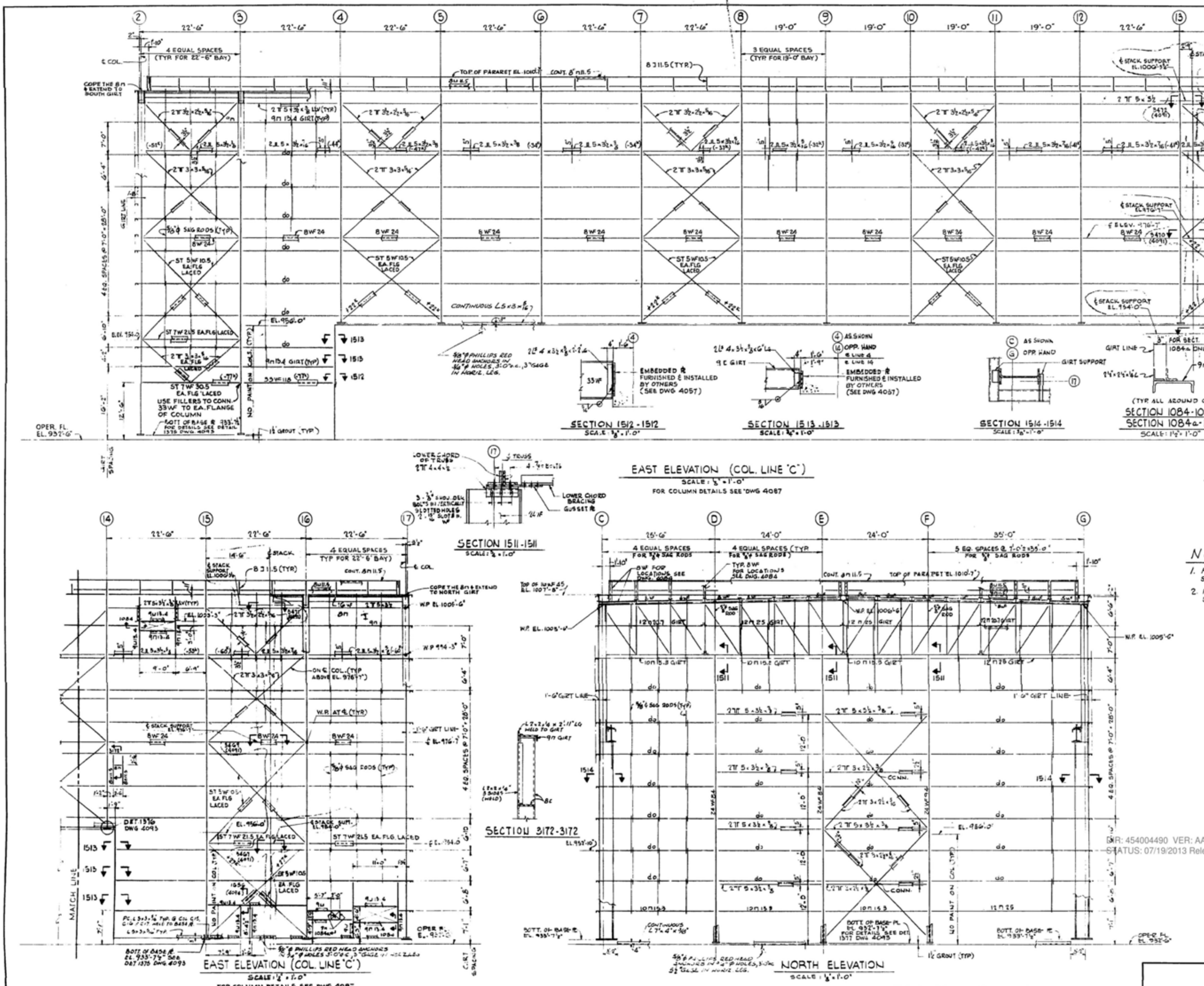
Enclosure 4

NPPD Drawings

(11 pages)

4088	Structural Turbine Generator Building Elevations Sheet No. 1
4089	Structural Turbine Generator Building Elevations Sheet No. 2
9150	Turbine Generating Building Sheet E101 Roof Framing Plan
9150	Turbine Generating Building Sheet E107 Elevation
9150	Turbine Generating Building Sheet E106 Elevations
9150	Turbine Generating Building Sheet 133 Girts
9150	Turbine Generating Building Sheet 139 Girts and Sag Rods
9150	Turbine Generating Building Sheet 156 Column Details
9150	Turbine Generating Building Sheet 157 Column Details
9150	Turbine Generating Building Sheet 158 Column Details
9150	Turbine Generating Building Sheet 159 Column Details



NOTE:
+ = TENSION
− = COMPRESSION

NOTES:
1. FOR GENERAL STEEL NOTES & MISC. STEEL DETAILS SEE DWG 4094.
2. FOR DETAILS OF LACED BRACING SEE DWG 4093.

RECEIVED
N.P.P.D. D.C.C.
S.N. 4490

FILMED

DIR: 454004490 VER: AA REV: 00 SIZE: E 34 x 44 - 36 x 48
STATUS: 07/19/2013 Release FIO

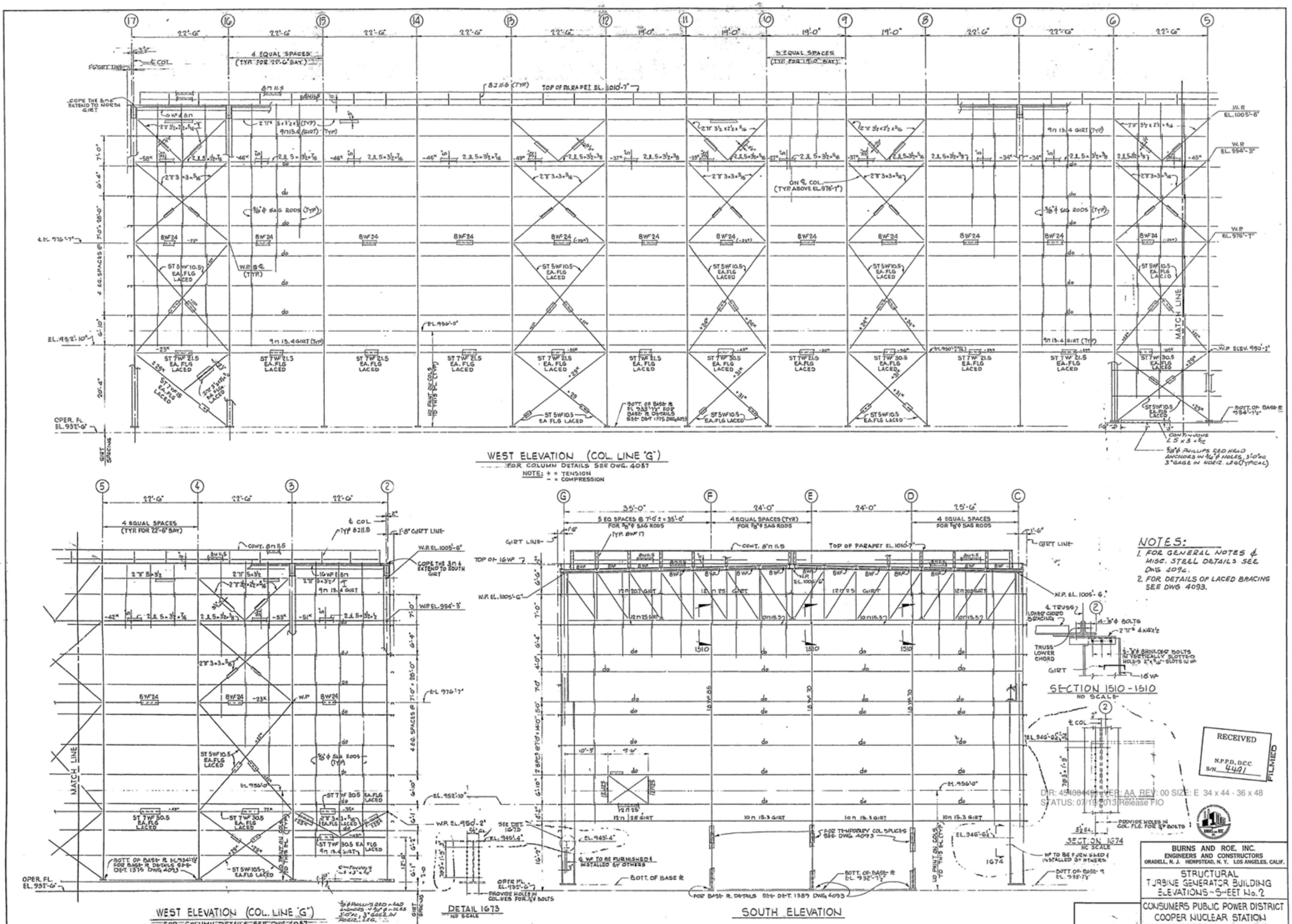
BURNS AND ROE, INC.
ENGINEERS AND CONSTRUCTORS
ORADELL, N. J. HEMPSTEAD, N. Y. LOS ANGELES, CALIF.

STRUCTURAL
TURBINE GENERATOR BUILDING
ELEVATIONS - SHEET Nº1

CONSUMERS PUBLIC POWER DISTRICT
COOPER NUCLEAR STATION

DRAWN BY C.M. & J.R. | DESIGNED BY R.C. WOOD
CHECKED BY TL/ESS | DATE 3/14/69 | SCALE AS SHOWN
APPROVED FOR CONSTRUCTION
CIVIL ENGINEER
W. O. 2520
DWG. 4086 | REV 8

REV. NO.	REVISION	BY	CHKD.	APP.	DATE
1	ADDED NOTE NO 1 & ANGLE WITH PHILLIPS RED HEAD; REVISED EAST ELEVATION COL. LINE "C" & NORTH ELEVATION; ADDED SECTIONS 1511, 1512, 1513 & 1514	J.R. H.E.	TL		
2	REVISED EAST ELEV (COL LINE "C") & NORTH ELEV	H.E	TL		
3	SECT. 1511, 1512, 1513 & 1514, ADDED NOTE 2. REVISED BASE PL. DIMS. & ELEVS. ON EAST ELEV. (COL. LINE C) & NORTH ELEV. ADDED SECT. 1084 & REMOVED VERTICAL SUPPORTS IN PARAPET ON EAST ELEV. (COL. LINE C) REVD. LADDED NOTE EAST ELEV. (COL. LINE C)	H.E K.R H.E	TL TL		
4	RELEASED APPROVED FOR CONSTRUCTION FOR E-65-15 ONLY; ADDED ℄ TO PARAPET FOR LADDER SUPPORT EAST ELEV (COL. LINE C). REVD. OPNG. EAST ELEV (COL. LINE C) ... ADDED DOOR OPNG TO EAST ELEV. (COL. LINE C)	Y.F. C.M	T.L. T.L.		
5	RELOCATED STRUTS, AND REVISED STRUT STRESS	LAB	HM	CFK	
6	ADDED SECT. 1084a	C.H	T.L.	CFK	
7	ADD FRMG. & SECT. SIZE FOR DUCT OPNG, REV. DIMENSION, CURB OUTLINE & ANGLES @ EAST ELEV., CLARIFIED SECT. 1513	WR	HM		
8	ADDED STACK ... TO ... ELEVATION (COL. LINE C)	T.L.	HM		



NOTES:

1. FOR GENERAL NOTES & MISC. STEEL DETAILS SEE DWG 4096.
2. FOR DETAILS OF LACED BRACING SEE DWG 4093.

ENG. DES.	ELEC. ENG.	MECH. ENG.	CIVIL ENG.	PROJ. ARCH.
CLEARED BY	[illegible]	[illegible]	C.O. [illegible]	[illegible]
DATE	8/5/69	8/8/69	8/8/69	8/8/69
REV. BY CH. DRAFTS	A.A. Villani / E.R. Ilman 8/7/69			

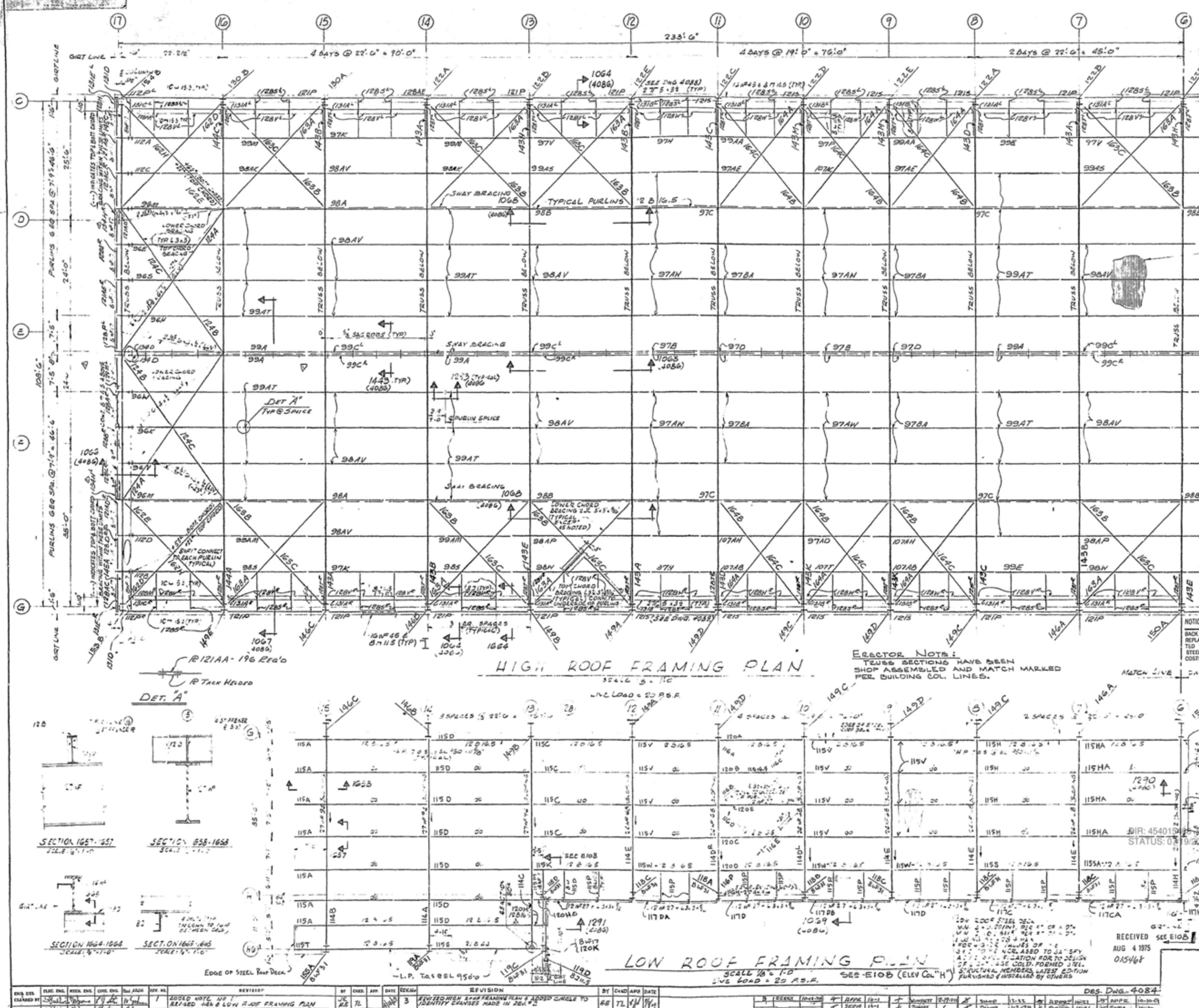
REV. NO.	REVISION	BY	CHKD.	APP.	DATE
1	ADDED NOTE NO. 1 & SECT. 1510-1510. REVD. WEST ELEVATIONS & SOUTH ELEVATION, ADDED & REVISED [illegible]	J.R. R.B.	TL		[illegible]
2	REVISED WEST ELEVATIONS, SOUTH ELEVATION & SECT. 1510. ADDED NOTE TO WEST ELEV. AND NOTE #2	E.B.	TL		[illegible]
3	REV'D WEST EL. (COL. LINE G) ADDED NOTE, REV. SOUTH EL. & ADDED DET. 1673, 1674 & CIRCLE TO IDENTIFY CHANGES MADE IN REV. #2. RELEASED APPROVED FOR CONSTRUCTION FOR E-69-19 ONLY	H.E.	TL	[illegible]	9/6/69
4	CLARIFIED GIRTS WEST ELEV. & SOUTH ELEV. AT COL. LINE 2 & G. REVISED DIM. WEST ELEV. (COL. LINE G) & SOUTH ELEV.	C.M.	G.A.	CFK	[illegible]
5	RELOCATED STRUTS AND REVISED STRUT STRESS	L.A.B.	H.M.	CFK	11/4/69
6	REVISED DETAIL 1673 & CHANGED DETAIL 1674 TO SECTION 1674 & REVISED SAME.	R.S.	WR	CFK	2-20-70

BURNS AND ROE, INC.
ENGINEERS AND CONSTRUCTORS
ORADELL, N. J. HEMPSTEAD, N. Y. LOS ANGELES, CALIF.

STRUCTURAL
TURBINE GENERATOR BUILDING
ELEVATIONS ~ SHEET No. 2

CONSUMERS PUBLIC POWER DISTRICT
COOPER NUCLEAR STATION

DRAWN BY C.M. & J.R.	SQUAD LDR. R.C. WOOD
CHECKED BY GGJ/TL	DATE 3/14/69 SCALE 1/8"=1'-0"
APPROVED FOR CONSTRUCTION [signature] 8/25/69 FOR CHIEF CIVIL ENGINEER	W. O. 2520
	DWG. 4089 REV. 6



NOTES:

1. FOR GENERAL STEEL NOTES & MISC. STEEL DETAILS SEE DES. DWG. 4094.
2. WHERE AXIAL LOADS ARE SHOWN ON MEMBERS:
   + = TENSION
   − = COMPRESSION
3. LOWER CHORD BRACING SHALL BE FRAMED FLUSH WITH BOTTOM CHORD OF TRUSS.
4. ALL STRUCTURAL STEEL TO BE A.S.T.M. – A36, UNLESS NOTED.

STEEL ROOF DECK FOR HIGH ROOF MIN. S = 0.116 IN³ PER FT. OF WIDTH MIN. I = 0.287 IN⁴ PER FT. OF WIDTH USING W/L = 28.4 MAX. FOR HIGHER VALUES OF W/L S & I TO BE INCREASED TO SATISFY A.I.S.I. SPECIFICATION FOR THE DESIGN OF LIGHT GAGE COLD-FORMED STEEL STRUCTURAL MEMBERS, LATEST EDITION. FURNISHED & INSTALLED BY OTHERS

BY OTHERS

FIELD ERECTION NOTES:

UNL. NOTED ON DWG'S ALL PRIMARY FIELD BOLTED CONNECTIONS SHALL BE MADE WITH 3/4" (A325) HIGH-STR. BOLTS W/THREADS IN A SHEAR PLANE. NO WASHER REQD. H.S. BOLTS TO BE TIGHTENED BY THE TURN OF THE METHOD.

UNL. NOTED ON DWG'S ALL FIELD BOLTED CONNECTIONS FOR PURLINS, GIRTS, AND SECONDARY FRAMING SHALL BE MADE WITH 3/4" MACH. BOLTS.

ERECTOR NOTE

ERECT MEMBERS WITH PIECE MARKS IN THE POSITION SHOWN ON PLAN. PIECE MARKS ARE PAINTED ON THE LEFT END OF PIECES AS DETAILED & ARE READ (UPRIGHT) FROM THE GROUND.
THE ABOVE DOES NOT APPLY TO COLUMNS.

NOTICE TO CONTRACTOR AND ERECTOR:
BACK CHARGES FOR CORRECTIVE WORK OR REPLACED MATERIALS WILL NOT BE ACCEPTED UNLESS AUTHORIZED BY CAPITOL STEEL & IRON CO. BEFORE ANY SUCH COSTS ARE INCURRED.

ERECTOR NOTE:
TRUSS SECTIONS HAVE BEEN SHOP ASSEMBLED AND MATCH MARKED PER BUILDING COL. LINES.

LOW ROOF STEEL DECK MIN. S = 0.201 IN³ PER FT. OF WIDTH ... FOR HIGHER VALUES OF ... TO BE INCREASED TO SATISFY A.I.S.I. SPECIFICATION FOR THE DESIGN OF LIGHT GAGE COLD-FORMED STEEL STRUCTURAL MEMBERS LATEST EDITION FURNISHED & INSTALLED BY OTHERS

RECEIVED AUG 4 1975
015468

FILMED AUG 7 1975

REV. 2-18-70 ADDED NOTE 4 PER APPL. DATED FEB. 6, 1970.
REV. 4-9-70 TO REVISE ... PER ...

CONTENTS THIS SHEET - ROOF FRAMING PLAN

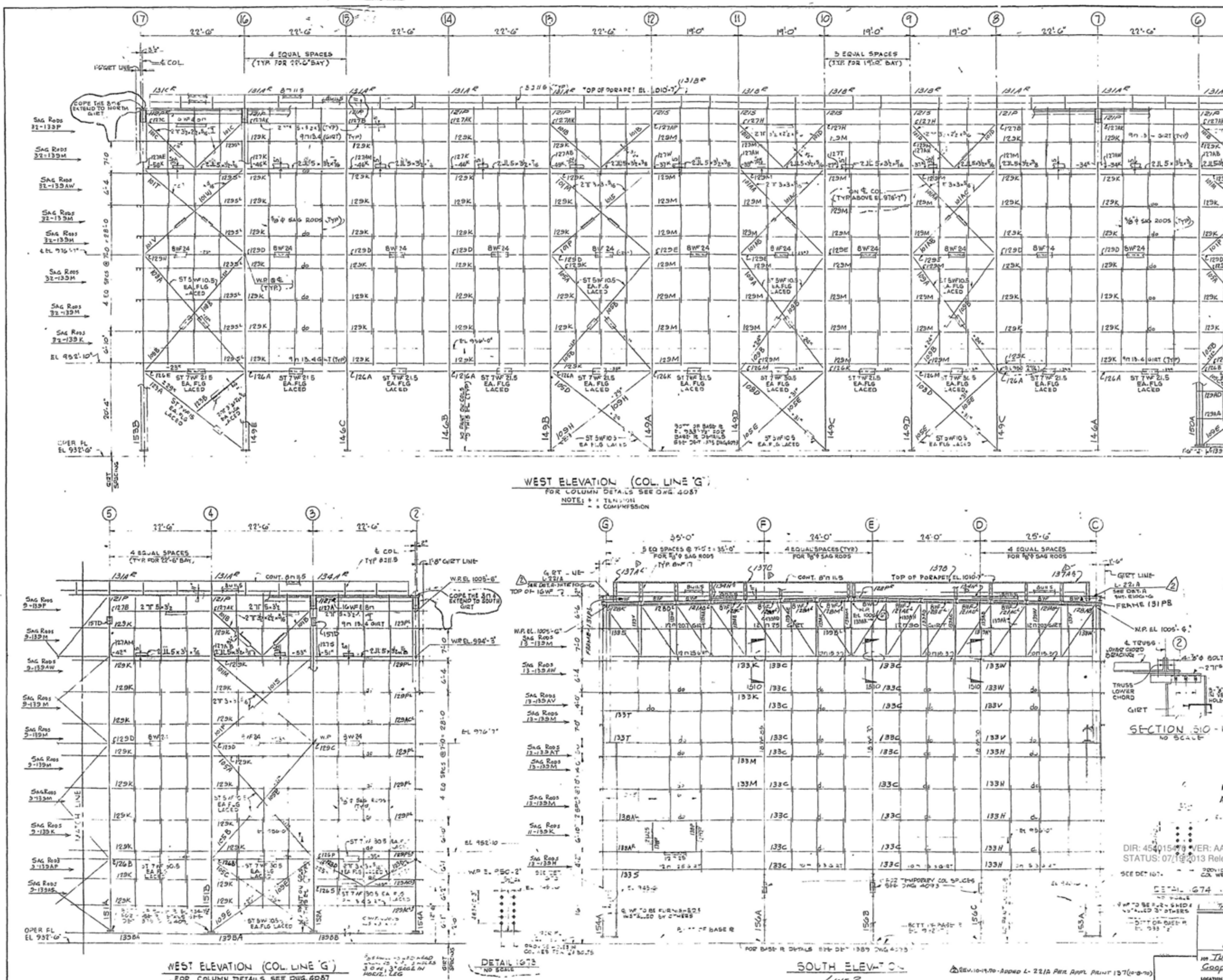
CAPITOL STEEL & IRON COMPANY
OKLAHOMA CITY — HOUSTON

JOB TURBINE GENERATING BUILDING
COOPER NUCLEAR STATION CONT. E69-15
LOCATION BROWNVILLE, NEBR.
CONTRACTOR CONSUMER PUBLIC POWER DISTRICT
Arch. or Engr. BURNS & ROE, INC.
Det. by WSS Chk. by SLH Supt. BUTENHOFF
CONTRACT 9150 SHEET E101 REV 4

DES. DWG. 4084

REV. NO.	REVISION	BY	CHKD.	APP.	DATE
1	ADDED NOTE NO. 1. REVISED HIGH & LOW ROOF FRAMING PLAN	JR	TL		
2	REVISED HIGH & LOW ROOF FRAMING PLAN. ADDED NOTES # 2 & 3, SECT. 1657–1658, 1664 & 1665	HS	TL		
3	REVISED HIGH ROOF FRAMING PLAN & ADDED CIRCLE TO IDENTIFY CHANGES MADE IN REV. # 3	GS	TL		

C1214A



ERECTOR NOTE

ERECT MEMBERS WITH PIECE MARKS IN THE POSITION SHOWN ON PLAN. PIECE MARKS ARE PAINTED ON THE LEFT END OF PIECES AS DETAILED & ARE READ (UPRIGHT) FROM THE GROUND.
THE ABOVE DOES NOT APPLY TO COLUMNS.

NOTES:

1. FOR GENERAL NOTES & MISC STEEL DETAILS SEE DWG 4094 & E101
2. FOR DETAILS OF LACED BRACING SEE DWG 4093.

NOTICE TO CONTRACTOR AND ERECTOR:

BACK CHARGES FOR CORRECTIVE WORK OR REPLACED MATERIALS WILL NOT BE ACCEPTED UNLESS AUTHORIZED BY CAPITOL STEEL & IRON CO. BEFORE ANY SUCH COSTS ARE INCURRED.

FILMED
AUG 7 1975

RECEIVED
AUG 4 1975

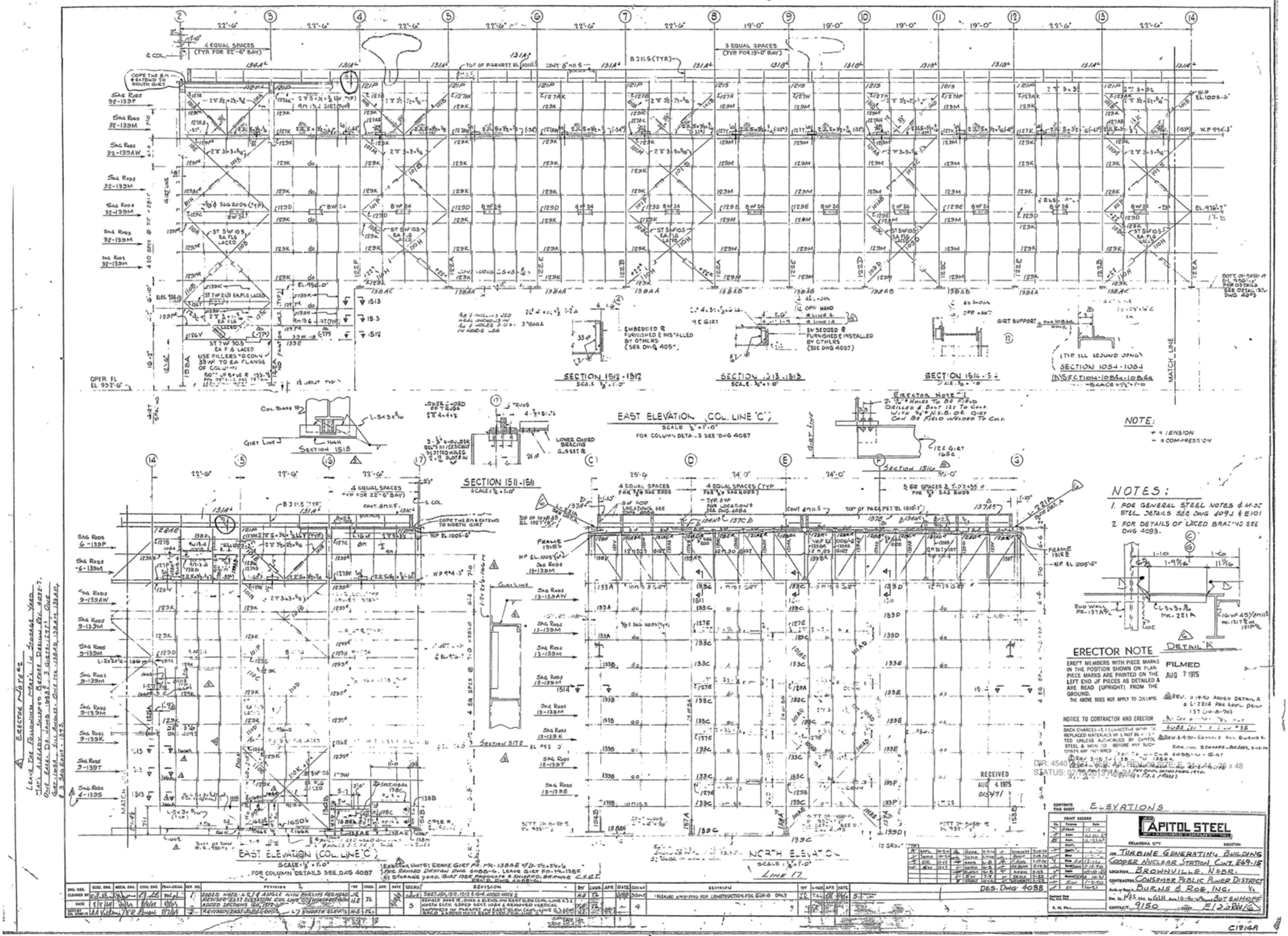
THIS SHEET - ELEVATION

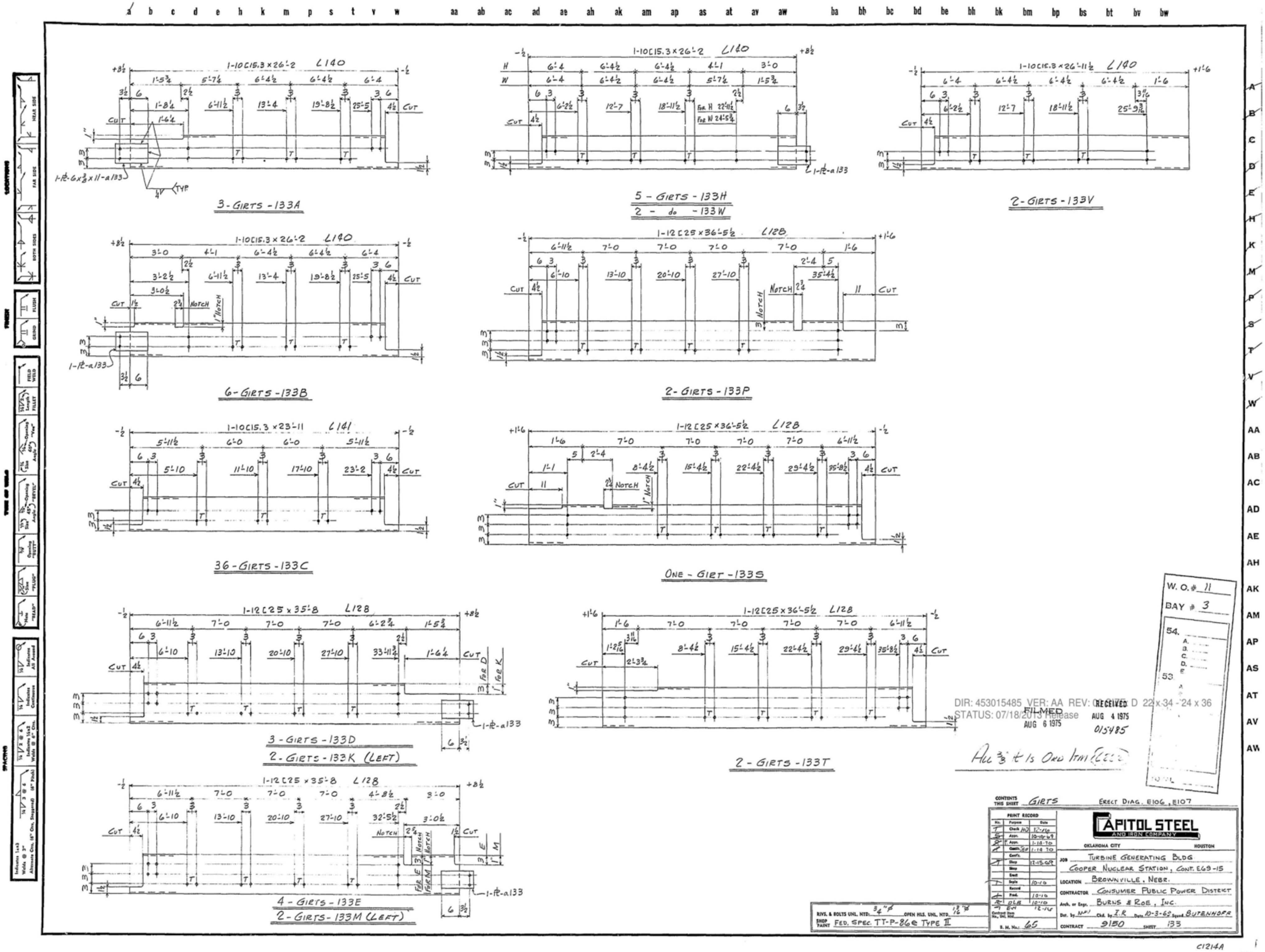
CAPITOL STEEL AND IRON COMPANY
OKLAHOMA CITY HOUSTON

JOB TURBINE GENERATING BUILDING
COOPER NUCLEAR STATION, CONT E69-15
LOCATION BROWNVILLE, NEBR.
CONTRACTOR CONSUMERS PUBLIC POWER DISTRICT
Arch. or Engr. BURNS & ROE, INC.
Dr. by W.S.S. Ckd. by G.L.H. Date 9-8-69 Appvd. BUTENHOFF
CONTRACT 9150 SHEET E107 Rev. 2

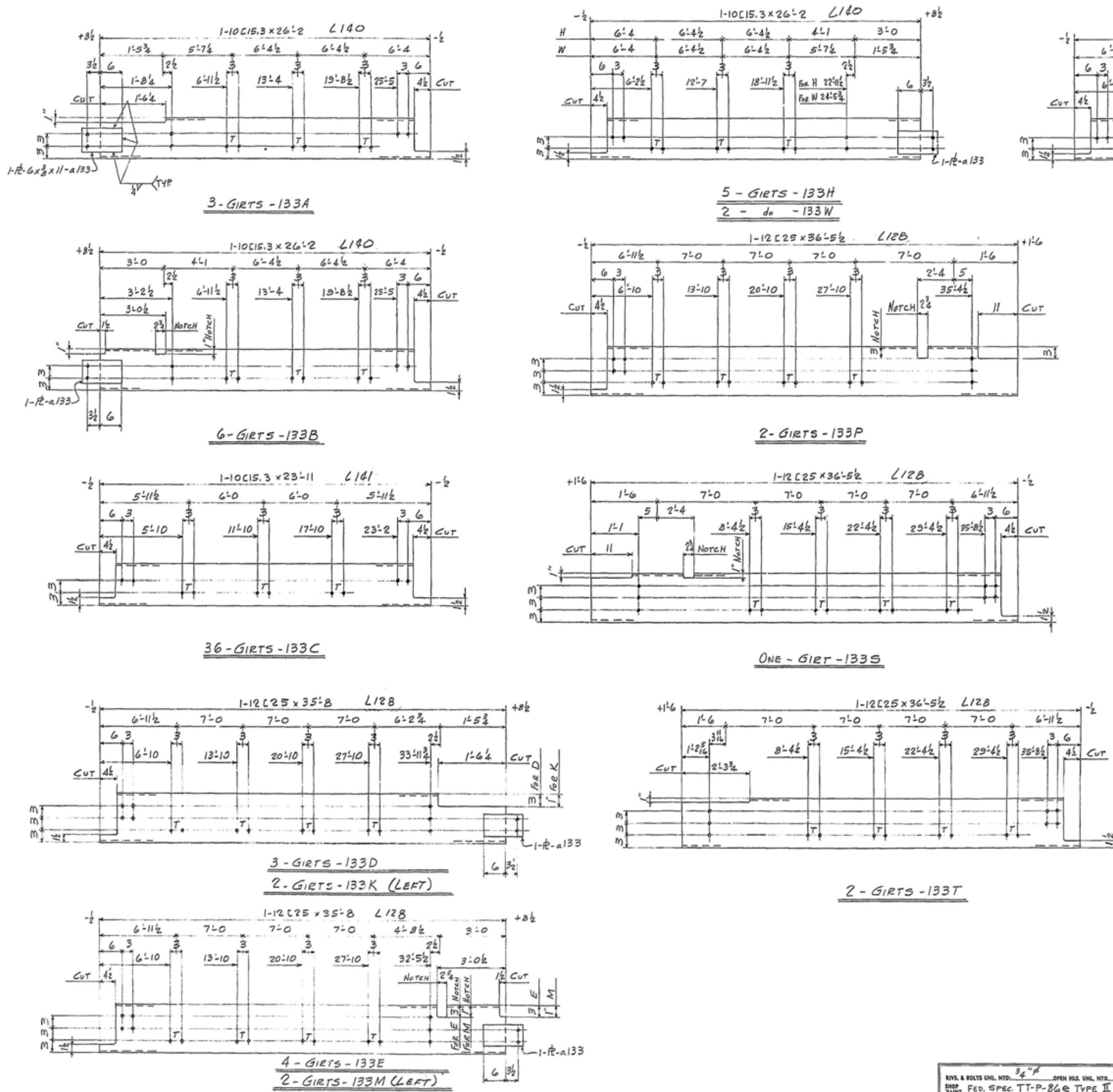
DES. DWG. 4089

REV. 1 – ADDED NOTE NO. 1 & SECT 1510-1510, REVISED WEST ELEVATIONS & SOUTH ELEVATION
REV. 2 – REVISED WEST ELEVATIONS, ADDED NOTE TO WEST ELEV. AND NOTE #2
REV. 3 – REV'D WEST EL (COL. LINE G) ADDED NOTE, REV. SOUTH EL.


EAST ELEVATION (COL. LINE 'C')
SCALE 1/8" = 1'-0"
FOR COLUMN DETAILS SEE DWG 4087
SECTION 1512-1512
SECTION 1513-1513
SECTION 1514-1514
SECTION 1084-1084
SECTION 1515
SECTION 1511-1511
SECTION 1516
EAST ELEVATION (COL. LINE 'C')
FOR COLUMN DETAILS SEE DWG 4087
NORTH ELEVATION
LINE 17
NOTE:
+ = TENSION
− = COMPRESSION
NOTES:
1. FOR GENERAL STEEL NOTES & MISC STEEL DETAILS SEE DWG 4093 & E101
2. FOR DETAILS OF LACED BRACING SEE DWG 4093.
DETAIL "A"
ERECTOR NOTE
ERECT MEMBERS WITH PIECE MARKS IN THE POSITION SHOWN ON PLAN. PIECE MARKS ARE PAINTED ON THE LEFT END OF PIECES AS DETAILED & ARE READ (UPRIGHT) FROM THE GROUND.
THE ABOVE DOES NOT APPLY TO COLUMNS
NOTICE TO CONTRACTOR AND ERECTOR
FILMED
AUG 7 1975
RECEIVED
AUG 4 1975
CONTENTS THIS SHEET ELEVATIONS
CAPITOL STEEL
OKLAHOMA CITY
HOUSTON
JOB TURBINE GENERATING BUILDING
COOPER NUCLEAR STATION, CONT. E69-15
LOCATION BROWNVILLE, NEBR.
CONTRACTOR CONSUMER PUBLIC POWER DISTRICT
BURNS & ROE, INC.
CONTRACT 9150
DES. DWG 4088
C1214A
DIR: 4540 VER: AA REV: 09 SIZE: E 34 x 44 - 36 x 48
STATUS: 07/19/2013



3-GIRTS-133A
1-10C15.3 x 26'-2 L140
5-GIRTS-133H
2-do-133W
1-10C15.3 x 26'-2 L140
2-GIRTS-133V
1-10C15.3 x 26'-11½ L140
6-GIRTS-133B
1-10C15.3 x 26'-2 L140
2-GIRTS-133P
1-12C25 x 36'-5½ L128
36-GIRTS-133C
1-10C15.3 x 23'-11 L141
ONE-GIRT-133S
1-12C25 x 36'-5½ L128
3-GIRTS-133D
2-GIRTS-133K (LEFT)
1-12C25 x 35'-8 L128
2-GIRTS-133T
1-12C25 x 36'-5½ L128
4-GIRTS-133E
2-GIRTS-133M (LEFT)
1-12C25 x 35'-8 L128
W.O. # 11
BAY # 3
DIR: 453015485 VER: AA REV: 00 SIZE: D 22 x 34 - 24 x 36
STATUS: 07/18/2013 Release
FILMED AUG 6 1975
RECEIVED AUG 4 1975
015485
CONTENTS THIS SHEET GIRTS
ERECT DIAG. E106, E107
CAPITOL STEEL AND IRON COMPANY
OKLAHOMA CITY HOUSTON
JOB TURBINE GENERATING BLDG
COOPER NUCLEAR STATION, CONT. E69-15
LOCATION BROWNVILLE, NEBR.
CONTRACTOR CONSUMER PUBLIC POWER DISTRICT
Arch. or Engr. BURNS & ROE, INC.
CONTRACT 9150 SHEET 133
SHOP PAINT FED. SPEC. TT-P-86e TYPE II
C1214A



W.O. # 11
BAY # 3

DIR: 453015485 VER: AA REV: 00 SIZE: D 22 x 34 - 24 x 36
STATUS: 07/18/2013 Release

FILMED AUG 6 1975

RECEIVED AUG 4 1975
015485

All ¾" ⌀ Is Ord Itm (Ecc)

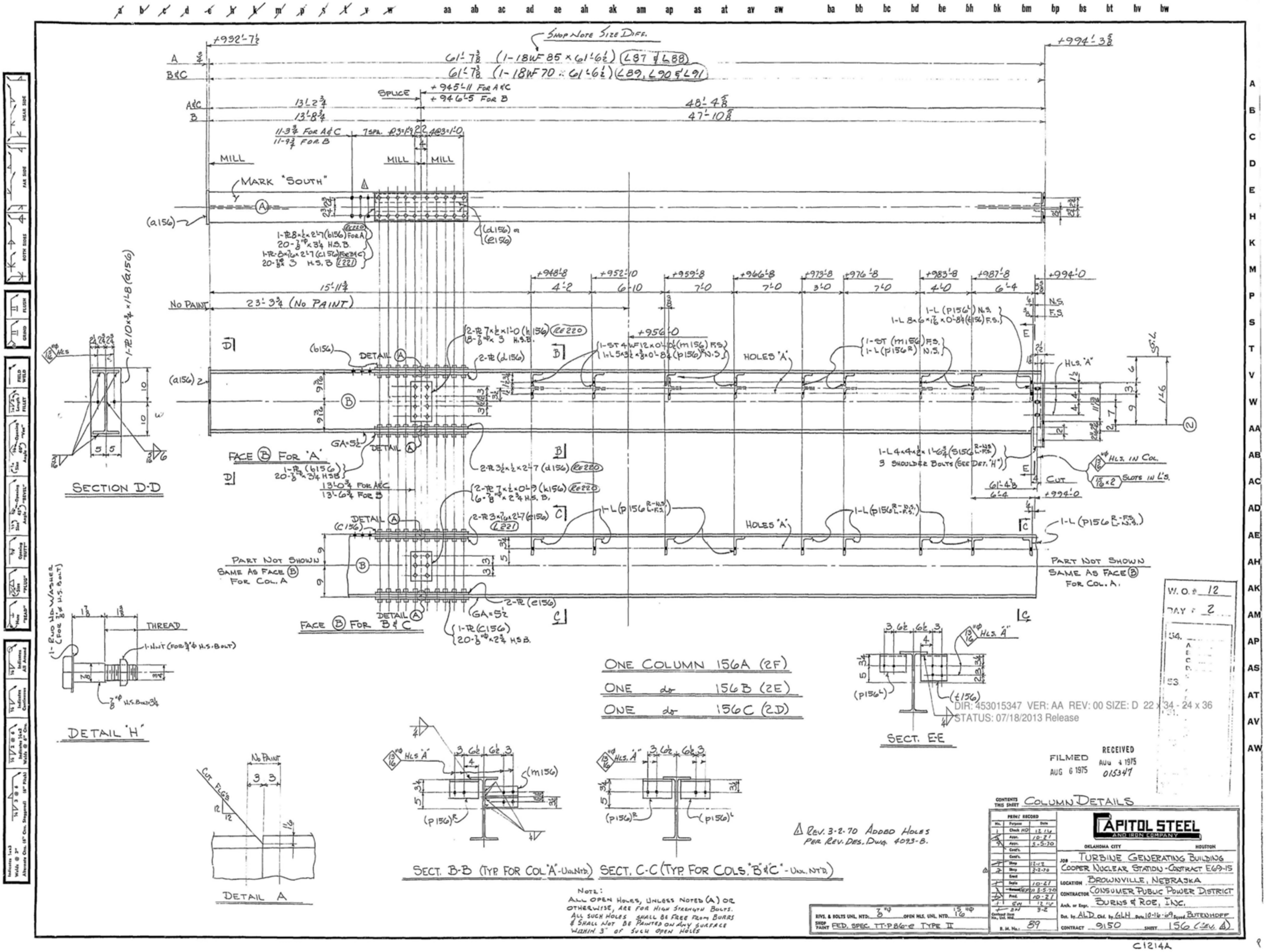
CONTENTS THIS SHEET: GIRTS — ERECT DIAG. E106, E107

CAPITOL STEEL AND IRON COMPANY
OKLAHOMA CITY — HOUSTON

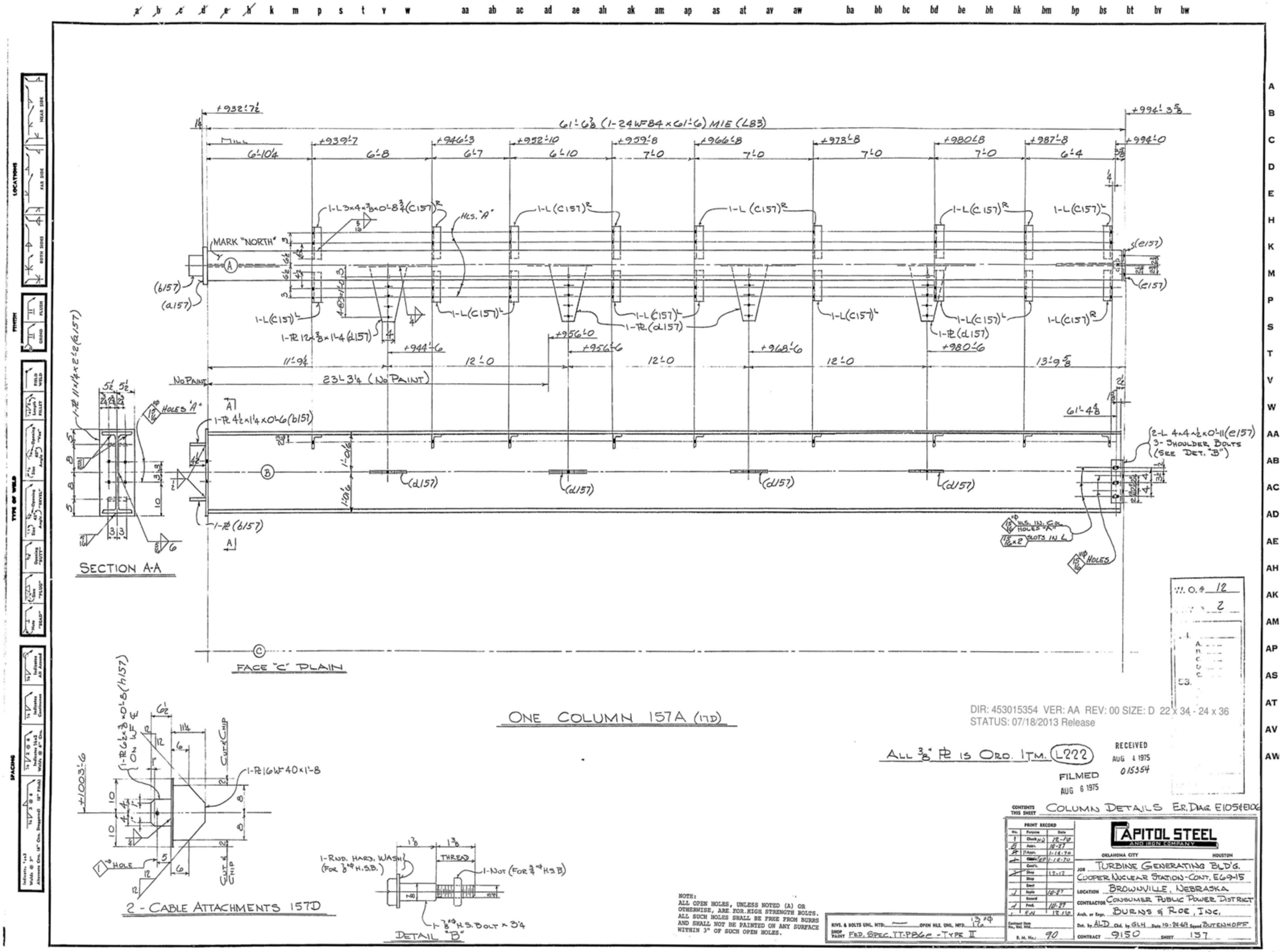
JOB: TURBINE GENERATING BLDG, COOPER NUCLEAR STATION, CONT. E69-15
LOCATION: BROWNVILLE, NEBR.
CONTRACTOR: CONSUMER PUBLIC POWER DISTRICT
Arch. or Engr.: BURNS & ROE, INC.
Det. by: NMJ Chk. by: I.R Date: 10-3-69 Squad: BUTENHOFF
CONTRACT: 9150 SHEET: 133

RIVS. & BOLTS UNL. NTD: ¾" ⌀ OPEN HLS. UNL. NTD: 13/16"
SHOP PAINT: FED. SPEC. TT-P-86e TYPE II
B. M. No.: 65

C1214A



SHOP NOTE SIZE DIFF.
1-18WF 85 × 61'-6½ (L87 & L88)
1-18WF 70 × 61'-6½ (L89, L90 & L91)
SPLICE
+945'-11 FOR A&C
+946'-5 FOR B
MILL
MARK "SOUTH"
23'-3¾ (NO PAINT)
NO PAINT
DETAIL A
HOLES "A"
FACE B FOR "A"
FACE B FOR B & C
PART NOT SHOWN SAME AS FACE B FOR COL. A
SECTION D-D
DETAIL "H"
THREAD
SECT. E-E
SECT. B-B (TYP. FOR COL "A" - UNL. NT'D)
SECT. C-C (TYP. FOR COLS. "B" & "C" - UNL. NT'D)
DETAIL A
ONE COLUMN 156A (2F)
ONE do 156B (2E)
ONE do 156C (2D)
Rev. 3-2-70 ADDED HOLES PER REV. DES. DWG. 4093-B.
NOTE: ALL OPEN HOLES, UNLESS NOTED (A) OR OTHERWISE, ARE FOR HIGH STRENGTH BOLTS. ALL SUCH HOLES SHALL BE FREE FROM BURRS & SHALL NOT BE PAINTED ON ANY SURFACE WITHIN 3" OF SUCH OPEN HOLES
DIR: 453015347 VER: AA REV: 00 SIZE: D 22 x 34 - 24 x 36
STATUS: 07/18/2013 Release
FILMED AUG 6 1975
RECEIVED AUG 1975 015347
W.O. # 12
CONTENTS THIS SHEET COLUMN DETAILS
CAPITOL STEEL AND IRON COMPANY
OKLAHOMA CITY HOUSTON
JOB TURBINE GENERATING BUILDING COOPER NUCLEAR STATION - CONTRACT E69-15
LOCATION BROWNVILLE, NEBRASKA
CONTRACTOR CONSUMER PUBLIC POWER DISTRICT
BURNS & ROE, INC.
Det. by ALD Chk. by GLH Date 10-16-69
CONTRACT 9150 SHEET 156 (Rev. A)
SHOP PAINT FED. SPEC. TT-P86-e TYPE II
C1214A



DIR: 453015354 VER: AA REV: 00 SIZE: D 22 x 34 - 24 x 36
STATUS: 07/18/2013 Release

W.O.# 12

RECEIVED
AUG 4 1975
015354

FILMED
AUG 6 1975

NOTE:
ALL OPEN HOLES, UNLESS NOTED (A) OR OTHERWISE, ARE FOR HIGH STRENGTH BOLTS. ALL SUCH HOLES SHALL BE FREE FROM BURRS AND SHALL NOT BE PAINTED ON ANY SURFACE WITHIN 3" OF SUCH OPEN HOLES.

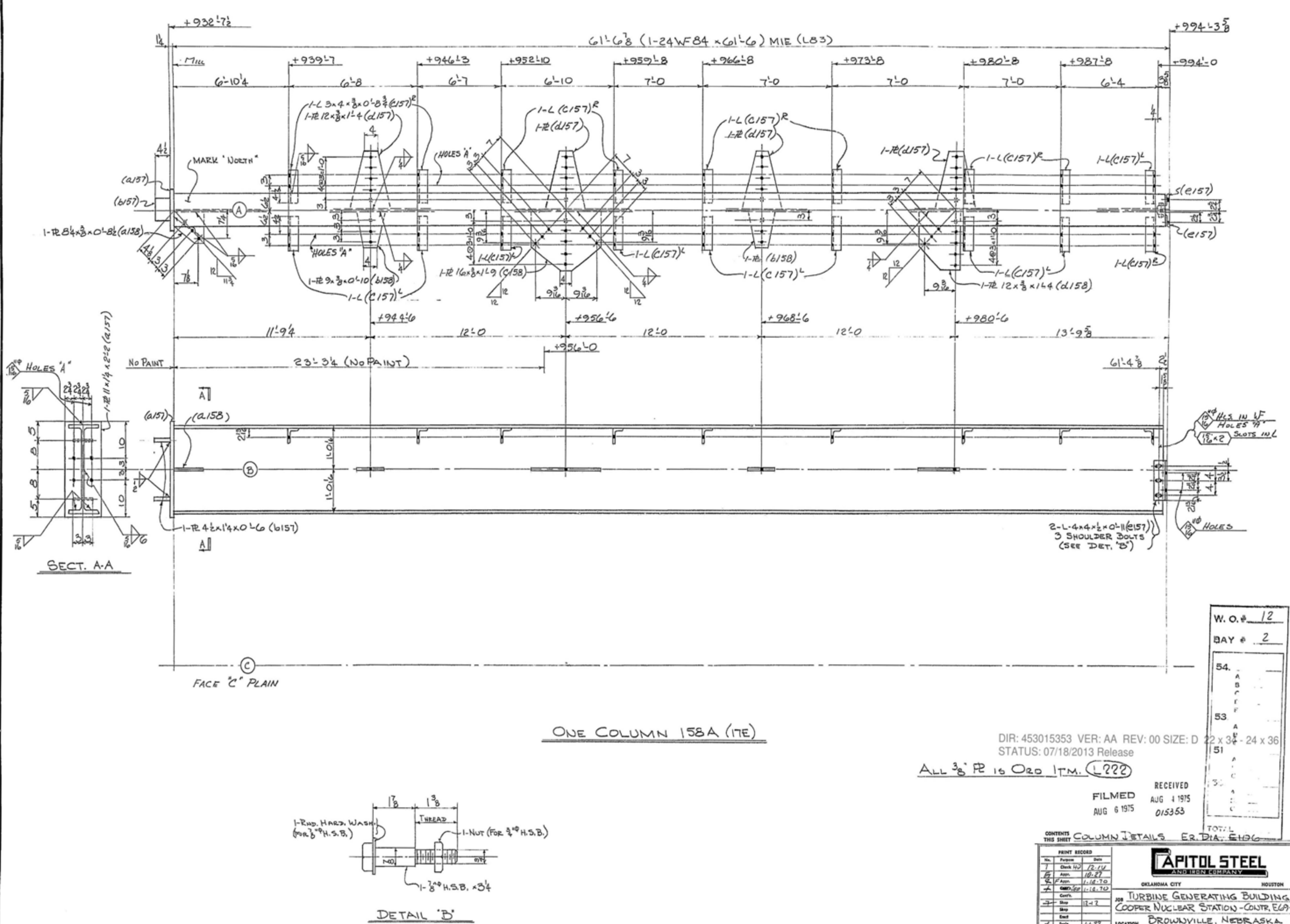
RIVS. & BOLTS UNL. NTD. — OPEN HLS. UNL. NTD. 13/16"φ
SHOP PAINT FED. SPEC. TT-P86e - TYPE II

CONTENTS THIS SHEET: COLUMN DETAILS ER. DIAG. E105 & E106

CAPITOL STEEL AND IRON COMPANY
OKLAHOMA CITY — HOUSTON

JOB: TURBINE GENERATING BLD'G.
COOPER NUCLEAR STATION - CONT. E69-15
LOCATION: BROWNVILLE, NEBRASKA
CONTRACTOR: CONSUMER PUBLIC POWER DISTRICT
Arch. or Engr.: BURNS & ROE, INC.
Dft. by ALD Chk. by GLH Date 10-24-69 Squad BUTENHOFF
CONTRACT 9150 SHEET 157
E.M. No. 90

C1214A



ONE COLUMN 158A (1TE)

ALL 3/8" ℞ IS ORD ITM. (L222)

W. O. # 12

BAY # 2

FILMED AUG 6 1975

RECEIVED AUG 4 1975 015353

NOTE:
ALL OPEN HOLES, UNLESS NOTED (A) OR OTHERWISE, ARE FOR HIGH STRENGTH BOLTS. ALL SUCH HOLES SHALL BE FREE FROM BURRS AND SHALL NOT BE PAINTED ON ANY SURFACE WITHIN 3" OF SUCH OPEN HOLES.

RIVS. & BOLTS UNL. NTD: ~ OPEN HLS. UNL. NTD: 13/16"φ

SHOP PAINT: FED. SPEC. TT-P86c-TYPE II

CONTENTS THIS SHEET: COLUMN DETAILS ER. DIA. E196

CAPITOL STEEL AND IRON COMPANY
OKLAHOMA CITY — HOUSTON

JOB: TURBINE GENERATING BUILDING, COOPER NUCLEAR STATION - CONTR. E69-15

LOCATION: BROWNVILLE, NEBRASKA

CONTRACTOR: CONSUMER PUBLIC POWER DISTRICT

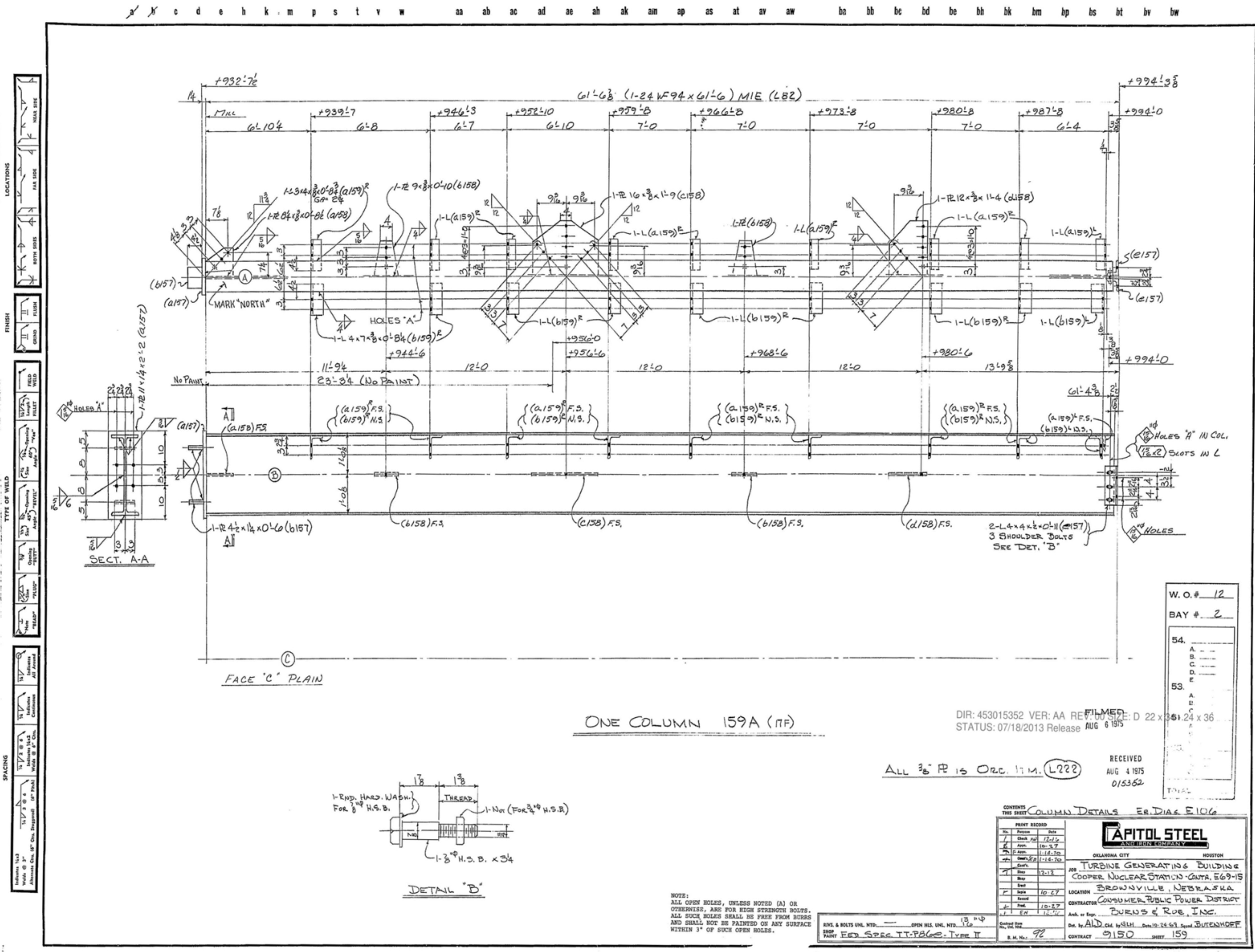
Arch. or Engr.: BURNS & ROE, INC.

Det. by ALD Chk. by GLH Date 10-24-69 Squad BUTENHOFF

CONTRACT 9150 SHEET 158

E. M. No. 91

C1214A


61'-6 3/8 (1-24 WF 94 x 61'-6) MIE (L82)
MARK "NORTH"
HOLES "A"
SECT. A-A
FACE "C" PLAIN
ONE COLUMN 159A (πF)
ALL 3/8" ℞ IS ORC. IT M. (L222)
DETAIL "B"
1-RND. HARD. WASH. FOR 3/4"φ H.S.B.
1-NUT (FOR 3/4"φ H.S.B.)
THREAD
1-3/4"φ H.S.B. x 3 1/4
2-L4x4x1/2x0'-11 (e157) 3 SHOULDER BOLTS SEE DET. "B"
HOLES "A" IN COL.
SLOTS IN L
23'-3 1/4 (NO PAINT)
NO PAINT
W.O.# 12
BAY # 2
DIR: 453015352 VER: AA REV: 00 SIZE: D 22 x 34 24 x 36
STATUS: 07/18/2013 Release
FILMED AUG 6 1975
RECEIVED AUG 4 1975 015352
NOTE: ALL OPEN HOLES, UNLESS NOTED (A) OR OTHERWISE, ARE FOR HIGH STRENGTH BOLTS. ALL SUCH HOLES SHALL BE FREE FROM BURRS AND SHALL NOT BE PAINTED ON ANY SURFACE WITHIN 3" OF SUCH OPEN HOLES.
CONTENTS THIS SHEET COLUMN DETAILS ER. DIAG. E106
CAPITOL STEEL AND IRON COMPANY
OKLAHOMA CITY HOUSTON
JOB TURBINE GENERATING BUILDING COOPER NUCLEAR STATION - CONTR. E69-15
LOCATION BROWNVILLE, NEBRASKA
CONTRACTOR CONSUMER PUBLIC POWER DISTRICT
Arch. or Engr. BURNS & ROE, INC.
Det. by ALD Chk. by GLH Date 10-24-69 Squad BUTENHOFF
CONTRACT 9150 SHEET 159
RIVS. & BOLTS UNL. NTD. OPEN HLS. UNL. NTD. 13/16"φ
SHOP PAINT FED SPEC TT-P86e - TYPE II
B.M. No. 92
PRINT RECORD
LOCATIONS
FINISH
TYPE OF WELD
SPACING

Enclosure 1

NPPD Calculation NEDC 16-003, Rev. 0, "Structural Evaluation of the Turbine Building Blowout Panels Steel Supports"

(100 pages)

CALCULATION COVER PAGE	(5) CALCULATION NO: 16-003 (6) REVISION/Change Notice No: 0	(20) Effective Date: 3/23/16 (2) Page 1 of 4
(1) EC #: (EE) 13-041	(7) Title: Structural Evaluation of the Turbine Building Blowout Panels Steel Supports	
(3) Design Basis Calc: ☒ YES ☐ NO	(9) System(s)/Structure: Turbine Building Panel Siding	(10) Discipline: Civil/Structural
(11) Safety Class: ☒ Quality Related ☐ Non-Quality Related	(12) Component/Equipment/Structure: Turbine Building	
(18) Proprietary: ☐ YES ☒ NO		
(4) Superseded: ☐ YES ☒ NO		

(14) Keywords (Description/Topical Codes):

Turbine Building, Siding, Panel, Blowout, FSAR Amendment 25, HELB, Finite Element Analysis, ANSYS, Owner Acceptance Calculation, LPI (Lucius Pitkin)

(8) **Calculation Description:**

The purpose of this calculation is to determine the Turbine Building pressure from a High Energy Line Break (HELB), which would cause failure of the siding structural support system, which would in turn result in failure (blowout) of the siding. The CNS Updated Safety Analysis Report (USAR) Amendment 25 states that the building siding will blowout at 0.5 psig thereby venting released steam (HELB) completely, relieving pressure in the Turbine Building to the atmosphere. This vendor prepared calculation utilizes ANSYS to analyze the panel support systems in the Turbine Building north and south walls.

(13) **Conclusion/Recommendations:**

The steel girts supporting the blowout panels on the north and south Turbine Building walls were analyzed to determine if failure would occur at a HELB pressure of 0.5 psi. The analysis concludes at a HELB pressure of 0.45 psi, failure of the panel support system (girts and connections) would occur and result in the loss (blowout) of 5,362 ft^2 of total panel area on the north and south Turbine Building walls.

REVIEWS

(15) Name/Signature/Date	(16) Name/Signature/Date	(17) Name/Signature/Date
Vendor LPI, Inc. (Lucius Pitkin) 2-4-16 *signatures on calc cover* **Responsible Engineer**	Taylor Sutton Taylor Sutton 2/8/16 ☐ **Design Verifier** ☒ **Technical Reviewer** ☒ Comments Attached	Alan L. Able Alan Able 3-23-16 **Supervisor/Approval** ☐ Comments Attached

CALCULATION REFERENCE SHEET	CALCULATION NO: 16-003 REVISION/Change Notice: 0

I.a. Change Notices Incorporated
List: None

I.b. Change Notices NOT Incorporated
List: None

II.	**Relationships**:	Sht.	Rev.	Input Doc	Pending Changes	Output Doc	Impact Y/N	Tracking No.
1.	DWG. 9150	133	00/AA	☒	None	☐	No	N/A
2.	DWG. 9150	156	00/AA	☒	None	☐	No	N/A
3.	DWG. 9150	157	00/AA	☒	None	☐	No	N/A
4.	DWG. 9150	158	00/AA	☒	None	☐	No	N/A
5.	DWG. 9150	159	00/AA	☒	None	☐	No	N/A
6.	DWG. 9150	E101	00/AA	☒	None	☐	No	N/A
7.	DWG. 9150	E106	00/AA	☒	None	☐	No	N/A
8.	DWG. 9150	E107	00/AA	☒	None	☐	No	N/A
9.	USAR Amendment 25	-	-	☒	None	☐	No	N/A
10.	Stearns-Roger, CMTRs [Media 09031-2021 thru 2107]	-	-	☒	None	☐	No	N/A
11.	Burns & Roe, Computer Analysis of Turbine Building Pressure History [Media (1) 8317-1718, (2) 64158-1094]	-	-	☒	None	☐	No	N/A
12.	EC # (EE) 13-041	-	3	☐	None	☒	Yes	N/A

III. REFERENCES:

1. NPPD Purchase Order 4200002638, Including Amendment 1
2. AISC Manual of Steel Construction, Sixth Edition
3. R.E. Peterson, "Stress Concentration Factors," John Wiley & Sons, 1974
4. Blodgett, O.W. "Design of Welded Structures", Cleveland, James F. Lincoln Arc Welding Foundation, 1966
5. Shigley, J.E., Mechanical Engineering Design, McGraw-Hill Book Co., 1972, 2nd Edition (unless otherwise indicated)
6. E. Oberg, et.al., Machinery's Handbook, Industrial Press, 27th Edition (unless otherwise indicated)
7. MATHCAD, Version 14.0, Parametric Technology Corp., 2007
8. ASTM SA-307, "Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength"
9. NUREG/CR-2137, "Realistic Seismic Design Margins of Pumps, Valves, and Piping", June 1981
10. Inryco Wall Systems Technical Data, L10 Series Liner Panel (obtained from vendor)
11. Inryco Job No.49054, "Vacuum Load Test L10 Steel/IW21A Aluminum 1820 Gage Wall Panel Cooper Nuclear Power Station, Nebraska", March 1973

12. National Aerospace Standards Committee NASM1312-13, :Fastener Test Method 13 - Double Shear Test", 2013
13. Baumeister T., et.al., "Mark's Standard Handbook for Mechanical Engineers", 8th Edition, McGraw-Hill
14. AISC Shapes Database v13.1 Historic.xls
15. Email Correspondence K. Tom (NPPD/CNS) to B. Elaidi (LPI), "Hardness of Supplied Bolts", November 28th, 2015
16. CNS Construction Contract No.E69-15, "Structural Steel for Turbine Generator and Reactor Buildings and Intake Structure", Revision 11, Dated 7/11/69
17 CNS Calculation NEDC 13-028, Revision 1, "Ultimate Internal Pressure of Turbine Building Blowout Panels and Metal Wall System"
18. ANSYS References:
    18.A ANSYS General Purpose Finite Element Analysis Software Code, Version 14, ANSYS Inc., LPI Report No. V&V-ANSYS-14, Rev.0, "Verification and Validation of ANSYS Software Program"
    18.B LPI Quality Assurance Procedure No. 4.1, Revision 5, "Software Control"

IV. SOFTWARE USED:

Title: ANSYS Version/Release: 14 MSI No.: N/A

Title: MATHCAD Version/Release: 14.0 MSI No.: N/A

V. DISK/CDS INCLUDED: None

Description of Contents: ______________________

VI. OTHER CHANGES: None

ATTACHMENT 9.4 RECORD OF REVISION

Sheet 1 of 1

Revision	Record of Revision
0	Initial issue.

ATTACHMENT 9.7 DESIGN VERIFICATION COMMENT SHEET

REVIEWER COMMENT/RESOLUTION RECORD

Document: LPI Calculation No. A15406-C-001, Rev.1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
1	Editorial comments have been provided in the calculation markup.	TES 1-14-16	All editorial comments are incorporated	BME 2-8-16	TES 2-8-16	
2	Section 1.0 Purpose (Page 7): Both the north and south walls are said to be analyzed but the analysis only considers the connection angle on the north wall, which is smaller than the connection angle of the south wall. Why wasn't the south wall analyzed specifically? Does the north wall analysis bound the south wall? If so, add statement to the calculation (maybe best placed in the methodology section?).	TES 11-23-15	The analysis shows that failure is predicted in the weld and bolts. The angle as a component is not part of the failure modes described. The only contribution is from the resulting size of weld along the vertical leg. The 4x3 angle was selected to be consistent with the girt spacing of the North wall that is used in the analysis. As shown in Figures 4.2-4 and 4.2-8, the maximum weld stresses for the 4x3 angle are concentrated at the lower ends of the welds and decreases' significantly toward the upper end of the weld. This characteristic of weld stress distribution remains true for the larger size angle (5x3.5) and the peak stress will drop slightly. As shown in Figure 4.2-5, maximum weld stresses for the 4x3 angle significantly exceed the failure limit. Thus, any slight drop in weld stress will not invalidate the conclusion of weld failure. A discussion will be added for the South wall configuration.	BME 11-23-15	TES 12-1-15	

REVIEWER COMMENT/RESOLUTION RECORD

Document: LPI Calculation No. A15406-C-001, Rev.1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
3	Section 2.2.4 Assumptions (Page 9): Does this create sufficient blow area to vent the Turbine Building per FSAR Amendment 25? Or do we need to consider smaller spacings?	TES 11-23-15	LPI will document the corresponding blowout siding area in the calculation.	BME 11-23-15	TES 1-12-15	
4	Section 2.2.6 Assumptions (Page 10): Why is the bolt hardness field test data not used?	TES 11-23-15	This assumption is related to the yield strength of the bolt material which is not known from hardness testing. Due to the short length of the bolts, this assumption is not critical. This was confirmed by initial trial computer runs. The bolt double shear test results will be included into the report.	BME 11-23-15	TES 12-1-15	
5	Section 4.1 Nonlinear Load Step Analysis (Page 20): Is there a reference document for the statement "weld failure in general is postulated at some stress between these two limits"?	TES 11-23-15	Stress field in fillet welds is a complex combination of axial, shear, and bending stress. Therefore, failure stress of fillet welds falls within the range of tensile, bending, and shear failure stresses. The failure stress range for carbon steel is 0.75Fu (for shear) to 1.0Fu (for tension). In the present analysis, the upper bound failure stress is used to predict conservative higher failure loads. Additional words will be added to clarify this point.	BME 11-23-15	TES 12-1-15	

REVIEWER COMMENT/RESOLUTION RECORD

Document: LPI Calculation No. A15406-C-001, Rev.1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
6	Section 4.3 Failure Mechanism (Page 29): Does the weld failure mechanism ensure the panel assemblies will be released from the Turbine Building? The connection angles are not affected by the weld failure, and are still bolted to the girt. How do the angles release from the outer flange of the structural steel columns after the welds fail?	TES 11-23-15	The complete failure of the weld is followed by excessive rotation of the girt and bending stresses that are significantly over the ultimate strength. This causes failure of the girt midspan section in bending and subsequent sliding of the girt ends off the column flanges. Additional explanation is added into the analysis section to expand upon this point.	BME 11-23-15	TES 12-1-15	
7	Section 5.0 Summary & Conclusions (Page 32): Failure mode 3 occurs after the welds failure (failure mode 2). Failure modes 2 and 3 should be combined.	TES 11-23-15	The conclusions will be rewarded to clarify.	BME 11-23-15	TES 12-1-15	
8	Section 5.0 Summary & Conclusions (Page 32): State the pressure value that causes the stated failure mechanisms.	TES 11-23-15	Will be included.	BME 11-23-15	TES 12-1-15	

REVIEWER COMMENT/RESOLUTION RECORD

Document: LPI Calculation No. A15406-C-001, Rev.1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
9	Section 5.0 Summary & Conclusions (Page 32): Can a more definitive conclusion be drawn from the analysis? That is, can the controlling failure mechanism be determined?	TES 11-23-15	The analysis identifies two possible failure mechanisms that are predicted to occur at nominally same amount of loading. Noting the variations in material properties and stress concentration at bolt threads, ranking one mechanism over the other is purely a theoretical issue and would not be of much practical value. However, a discussion on the potential additional strength from the noted field weld could push the failure to more likely the bolts, for those connections that have the additional field weld. Additional explanation will be added to discuss this point.	BME 11-23-15	TES 12-1-15	
10	Section 5.0 Summary & Conclusions (Page 45): Both failure modes state that the panels will fail but there is no definition of what panel failure implies. Define panel failure as the physical deformation or blowout of panels that releases the internal HELB pressure to the atmosphere.	TES 12-1-15	Panel failure will be defined in the report as blowout of the panels as a result of failure of the panels structural support system.	BME 1/15/16	TES 2-4-16	
11	Section 6.0 References (Page 46): Is Reference 7c a typo? It should be sheet E106.	TES 12-1-15	The sheet number on the drawing is not legible. The sheet number will be changed to E106.	BME 1/15/16	TES 2-4-16	

REVIEWER COMMENT/RESOLUTION RECORD

Document: LPI Calculation No. A15406-C-001, Rev.1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
12	Should this analysis be treated as LPI proprietary information?	TES 1-12-16	The LPI report is not proprietary. However, LPI requests that NPPD limit the distribution of the report in accordance with the purpose it is intended for. The ANSYS files should be shared only in connection with investigation of the TB blowout panels.	BME 1/15/16	TES 2-4-16	
13	Appendix B.5 Deadweight and HELB Pressure Loadings (Pages B3 and B4): In regards to the preceding comment, to satisfy curiosity, were both the panel deadweight twisting moment (-10.06 lbf-in) and the HELB twisting moment (14.12 lbf-in) applied to the girt, or was the difference between the two (4.06 lbf-in) applied? Either method would generate the same result.	TES 1-12-16	The dead weight was applied in the first load step and then the HELB loads were applied in a subsequent load step. In nonlinear analysis, the order of load application can be important since the structural response is history dependent. This is the correct sequence of loading the girts.	BME 1/15/16	TES 2-4-16	

REVIEWER COMMENT/RESOLUTION RECORD

Document: LPI Calculation No. A15406-C-001, Rev.1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
14	Section 3.1 Analysis Methodology (Page 20): In the Model 4 paragraph, please provide further detail explaining how not accounting for dead load in Model 4, in contrast to Model 1 where dead load is accounted for, bounds the cases. It isn't entirely clear how accounting for dead load in one model and not accounting for dead load in another model (two different models with two different inputs) bounds the analysis.	TES 1-12-16	The amount of dead weight transferred from the girt to the end connections is dependent on the sag rod tightness (preload) and stiffness. The effect of the dead weight is to apply vertical load as well as twisting moment due to the eccentric weight of the panels. The dead weight twisting moments oppose the twisting moment from the HELB pressure. Also, note that the failure modes of the girt are controlled by failure of the end connections. In Model 1, the dead weight reactions at the connections include HELB lateral reaction force, HELB torsional moment that is reduced by panel dead weight and vertical dead weight reaction force. The torsional moment and lateral force reactions have the greatest effect on the connection. In Model 4, excluding the dead weight increases the torsional moment reaction at the connection and eliminates the vertical reaction. Based on the condition of the sag rods, the amount of the dead weight transferred to the end connections is between the two limits simulated in the Model 1 and Model 4 analyses and therefore the two models bounds possible scenarios for dead weight effects.	BME 1/15/16	TES 2-4-16	

REVIEWER COMMENT/RESOLUTION RECORD

Document: LPI Calculation No. A15406-C-001, Rev.1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
15	Section 4.4 Girt Between Column Lines C&D (Page 43): The cantilever span that is calculated is for girt 133B. Girt 133A has a longer span (see drawing 9150 Sht133). The span length from column line D to the near flange of column line C (or angle bolts) for girt 133A is (6'-4")+(6'-4-1/2")+(6'-4-1/2")+(5'-7-1/4")-(2-1/2") = 24'-5-3/4". It is conservative to use the span length for girt 133B for the entire bay, however it should be stated that girt 133A is longer.	TES 1-12-16	It will be stated in the input section that for panels between column lines C and D, girt 133B is conservatively used in the analysis since it is shorter than girt 133A.	BME 1/15/16	TES 2-4-16	
16	Section 4.7 Siding Blowout Area (Page 49 and 50): The distance between column lines C and D should be conservatively equal to the cantilever span for girt 133B, which is 22'-11-1/2". Update the total blowout area accordingly.	TES 1-12-16	The panel blowout area will be updated to use the shorter span.	BME 1/15/16	TES 2-4-16	
17	Section 5.0 Summary & Conclusions (Page 50): Ensure the language is clear concerning which column lines on the north and south walls were analyzed. The second paragraph needs to be corrected.	TES 1-12-16	A statement will be added in the description of Model 4 in Section 3.1 that says:"The girts between column lines C and D on the south wall are types 133 H, V, and W. The analysis of girt type 133B on the north wall bounds those types. Calculation of the blowout area between column lines C and D does not however consider the south wall for conservatism.	BME 1/15/16	TES 2-4-16	
18	Section 6.0 References (Page 51): Add the structural steel CMTRs to the reference list.	TES 1-13-16	The CMTR will be added to the list of references.	BME 1/15/16	TES 2-4-16	

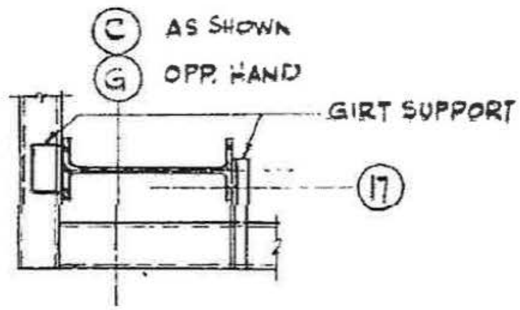
REVIEWER COMMENT/RESOLUTION RECORD

Document: LPI Calculation No. A15406-C-001, Rev.1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
19	Section 6.0 References (Page 51): Add Contract No. E-69-15, "Structural Steel for Turbine Generator and Reactor Buildings and Intake Structure", Revision 11, Dated 7/11/69. This should be the reference for the bolts (ASTM A307) on page 12.	TES 1-13-16	Reference will be added.	BME 1/15/16	TES 2-4-16	
20	Section 6.0 References (Page 51): After a discussion with the knowledgeable mechanical engineer on HELB, the reference to the EDS Report (reference 3) should be removed. In its place, please reference Burns & Roe Inc. document "Computer Analysis of CNS Multi-Compartment Pressure History: Main Steam, Feedwater, and Extraction Line Breaks in Turbine Building", Date:10-25-73 [Media (1) 08317-1718, (2) 64158-1094]	TES 1-13-16	The EDS report is cited because it includes time history plots of the HELB pressure demonstrating the vibratory characteristic nature of the HELB pressure. No design input is used from the EDS report. The Burns & Roe calculation will also be added as a reference document, but the EDS calculation should remain as a reference.	BME 1/15/16	TES 2-4-16	

REVIEWER COMMENT/RESOLUTION RECORD

Document: LPI Calculation No. A15406-C-001, Rev.1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
21	Section 2.2 Assumptions (Page 15): Add the following figure (from DWG.4088 or DWG.9150 Sht.E106) to this page to help show the connection of the girt at column line C on the north wall.  SECTION 1514-1514 SCALE : 3/8"=1'-0"	TES 1-14-16	Figure will be added.	BME 1/15/16	TES 2-4-16	

REVIEWER COMMENT/RESOLUTION RECORD

Document: LPI Calculation No. A15406-C-001, Rev.1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
22	Section 2.1.5 Input (Page 12): Is there a justification (standard) for accepting the testing of non-quality controlled material (the bolts) that comes from a different batch than what is installed, even though both are of the same specification?	TES 1-14-16	The bolts in question are ASTM A-307 Grade A. The applicable strength and chemistry requirements have not changed significantly since the construction of the plant. The A-307 Grade A bolts are standard items that do not require special processes or expertise for production. The published ASTM minimum strength for these bolts has not changed since the date of installation to current date. As such, the bolts obtained from the warehouse at CNS for testing are reasonably representative of A307 bolts. Thus it is reasonable to consider the tested "lot" would not differ substantially from the installed bolts.	BME 1/15/16	TES 2-4-16	

[1] See Step 5.6[6] for code definitions.

LPI, Inc. Consulting Engineers

Advanced Analysis & Fitness for Service
Failure & Materials Evaluation
Nondestructive Engineering

STRUCTURAL EVALUATION OF THE TURBINE BUILDING BLOWOUT PANELS STEEL SUPPORT

Calculation No. A15406-C-001, Rev. 1

February 2016

Prepared For

NEBRASKA PUBLIC POWER DISTRICT
Cooper Nuclear Station

Prepared By

LPI, Inc.

Lucius Pitkin – established 1885
36 Main Street • Amesbury, MA 01913 • 978.517.3100
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Ensuring the integrity of today's critical infrastructure for tomorrow's world.

DOCUMENT RECORD

Document Type:		☒ **Calculation** ☐ **Report** ☐ **Procedure**			
Document No:		A15406-C-001			
Document Title:		Structural Evaluation of the Turbine Building Blowout Panels Steel Support			
Client:		Nebraska Public Power District			
Client Facility:		Cooper Nuclear Station			
Client PO No:		4200002638 including amendment 1			
Quality Assurance:		**Nuclear Safety Related?** ☐ **No** ☒ **Yes**			
Computer Software Used:		☐**No**[1] ☒ **Yes**[2]	1. Check NO when EXCEL, MathCAD and/or similar programs are used since algorithms are explicitly displayed. 2. Include Software Record for each computer program utilized.		
Instrument Used:		☐**No** ☒ **Yes**[3]	3. Include Document Instrument Record.		
Revision	**Approval Date**	**Preparer**[5]	**Checker**[5]	**Design Verification**[5]	**Approver**[4, 5]
0	11/25/2015	B. Elaidi	R. Chen	A. Smyth	P. Bruck
1	2/4/2016	B. Elaidi	R. Chen	R. Chen	P. Bruck

[4] The Approver of this document attests that all project examinations, inspections, tests and analysis (as applicable) have been conducted using approved LPI Procedures and are in conformance to the contract/purchase order.
[5] Electronic signatures may be used only with prior concurrence.

Page	**2**	**of**	**85**	**Total Pages**	(include any Title Sheet and Attachments in page count. Document Back Cover, if utilized, not included in page count)

DESIGN VERIFICATION CHECKLIST

Document No(s)[1]:	A15406-C-001	Rev.:	1

Review Method:	X	Document Review		Alternate Calculation		Test

	Criteria	DV[2]
1	Were the inputs correctly selected and incorporated into design?	RC
2	Are assumptions necessary to perform the design activity adequately described and reasonable? Where necessary, are the assumptions identified for subsequent re-verifications when the detailed design activities are completed? If applicable, has an as built verification been performed and reconciled?	RC
3	Are the appropriate quality and quality assurance requirements specified?	RC
4	Are the applicable codes, standards and regulatory requirements including issue and addenda properly identified and are their requirements for design met?	RC
5	Have applicable construction and operating experience been considered, including operation procedures?	n/a
6	Have the design interface requirements been satisfied?	RC
7	Was an appropriate design method used?	RC
8	Is the output reasonable compared to inputs?	RC
9	Are the specified parts, equipment, and processes suitable for the required application?	n/a
10	Are the specified materials compatible with each other and the design environmental conditions to which the material will be exposed?	n/a
11	Have adequate maintenance features and requirements been specified?	n/a
12	Are accessibility and other design provisions adequate for performance of needed maintenance and repair?	n/a
13	Has adequate accessibility been provided to perform the in-service inspection expected to be required during the plant life?	n/a
14	Has the design properly considered radiation exposure to the public and plant personnel?	n/a
15	Are the acceptance criteria incorporated in the design documents sufficient to allow verification that design requirements have been satisfactorily accomplished?	RC
16	Have adequate pre-operational and subsequent periodic test requirements been appropriately specified?	n/a
17	Are adequate handling, storage, cleaning and shipping requirements specified?	n/a
18	If software was used, have the computer type and operating system been properly identified? Is use of the software, hardware and O/S appropriate for the conditions, components evaluated? Has a V&V been performed?	RC
19	Are requirements for identification, record preparation review, approval, retention, etc., adequately specified?	RC
20	Has an internal design review been performed for applicable design projects? Have comments from the Internal Design Review been appropriately considered/addressed?	n/a

(1) Include any drawings developed from reviewed documents, or include separate checklist sheet for drawings
(2) Design Verifier shall initial indicating review and mark N/A where not applicable

	Printed Name	Signature	Date
DV Completed By:	R. Chen	Chen	2/2/16

Page 1 of 1 Total Pages Include DV Checklist and Comment Resolution sheets in page count

Form: LPI-3.1-Rev-8-Fig-5-5

Document Software Record[1]

(include separate sheet for each software package used)

1	**Computer Software Used** (Code/Version)	ANSYS Version 14.0 [5a]
2	**Software Supplier**	ANSYS, Inc.
3	**Software Update Review**	☒ Error notices; describe: Reviewed error reports for elements and options used postdate in [5b] ☐ Other; describe:
4	**Nuclear Safety Related Software**	☐ NO ☒ YES[2] [2] If YES, complete the following: Computer type: Desktop T-7500 Computer S/N: HC74VR1 Computer O/S: Windows 7 Professional SP1 V & V (include as ref): [5b]
5	**Bases for Application**	Identify the bases that support use of this software for the application herein; may be separately discussed elsewhere in this document (indicate section) and/or may be addressed in the V&V (identify reference number): — See Section 3
6	**Input Listing/Summary**[3]	☒ Input listing and/or summary: supplied separately: ☐ Not attached; identify File/Disc ID:
7	**Output Data/Identifier(s)**[3]	☒ Output results attached: selected output included ☐ Not attached; identify File/Disc ID: Span North3.txt 11-14-2015 8:51pm Span North4.txt 11-17-2015 8:15am Span North5.txt 1-20-2016 7:54am Span North qb.txt 1-20-2016 9:31am Span North t with one bolt.txt 01-20-2016 10:36am
	[3]e.g., run date/time; use for reference, as appropriate, within body of calculation	
8	**Comments**	None
9	**Keywords**[4]	SOLID186, MPC184, BEAM189, CONTA170, CONTA176, TARGE170, MISO, static analysis
	[4]For use in describing software features used in this calculation; use common terms based on software user manual and/or help files.	
10	**Project Manager Name:**	Bahaa Elaidi

[1] If computer software is used on project, complete form with required information.
Update the LPI Computer Software Use List per LPI Procedure 13.1 requirements.

Form: LPI-3.1-Rev-8-Fig-5-8

Document Instrument Record				
Instrument Used		**Instrument Description**	**Serial No.**	**Calibration Due Date**
1	☐	Tensile Testing Machine (50 kips)	Baldwin Emery 50-SR4-36	
2	☒	Tensile Testing Machine (120 kips)	Baldwin 372005	3/11/2016
3	☐	Instron Tensile Machine	8800R/141	
4	☒	Micrometer	Mitutoyo 05062457	2/12/2016
5	☐			
6	☐			
7	☐			
8	☐			
9	☐			
10	☐			
11	☐			
12	☐			
13	☐			
14	☐			

For instruments used (as indicated above), include an accuracy statement using one of the following methods (identify as applicable):

☐	Include discussion of accuracy statement for instruments and results within body of output document.
☒	Include calibration records of instruments within output document. (most recent calibration statement is filed and maintained in LPI QA records)
☐	Include instrument accuracy values within output document.

Project Manager Name:	Bahaa Elaidi

If instrument(s) was used on project, identify instrument, include the instrument calibration due date.
Update the LPI Instrument Use List per LPI Procedure 13.1 requirements.

Form: LPI-3.1-Rev-8-Fig-5-10

RECORD OF REVISION

Revision No.	Date	Description of Change	Reason
0	11-25-2015	Original issue	
1	*	Revised the entire calculation except Appendices A through C for editorial changes and inserted description of column lines C-D girt model and analysis results. Added Appendix D.	Revised to include analysis of girts between column lines C and D, incorporate review comments and make minor editorial changes.

*See Document Record page

TABLE OF CONTENTS

Page No.

Appendices (30 pp total) TOTAL PAGES

1.0 Purpose/Scope

This calculation documents the structural analysis of the steel components supporting the blowout panels in the Cooper Nuclear Station (CNS) Turbine Building for sustained dead weight and predicted accident pressure loading resulting from a High Energy Line Break (HELB) within the Turbine Building. This document is generated in accordance with Nebraska Public Power District (NPPD) Purchase Order 4200002638 [1][1].

The CNS Updated Safety Analysis Report (USAR) Amendment 25 [2] states that the building siding will blowout at 0.5 psig internal pressure, thereby limiting the resulting accident pressure within the Turbine Building. The blowout panels considered in this analysis are located on the north and south walls of the upper Turbine Building above elevation 932'. The steel structure supporting the panels consists of horizontal C10x15.3 steel channels extending between building wide flange steel columns. The channels (girts) are oriented so that the web is horizontal and the siding panels are fastened with screws to the channel flange. The girts are attached to the building columns via steel angles that are bolted to the channel web with two A307 Grade A 3/4" bolts and welded to the building column flange with two fillet line welds (Figure 1-1)[2]. The girt vertical spacing varies in the north and south sides of the Turbine Building (Figures 1-2 and 1-3). The girt weight is also supported with the use of sag rods, as can be seen in Figure 1-2.

The purpose of this calculation is to determine the Turbine Building pressure from a HELB that would cause failure of the siding structural support system, which would in turn result in failure (blowout) of the siding.

The scope of this calculation is the Turbine Building siding structural support system on the north and south walls of the building. Evaluation of the siding and attachment to the structural support system was previously assessed by CNS [14], and found not to be limiting.

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[1] Numbers in [x] refer to numbered references listed in Section 6.0.

[2] The two vertical line welds are per the design drawings. Additional horizontal weld was observed on several of the supports, including the one in Figure 1-1.

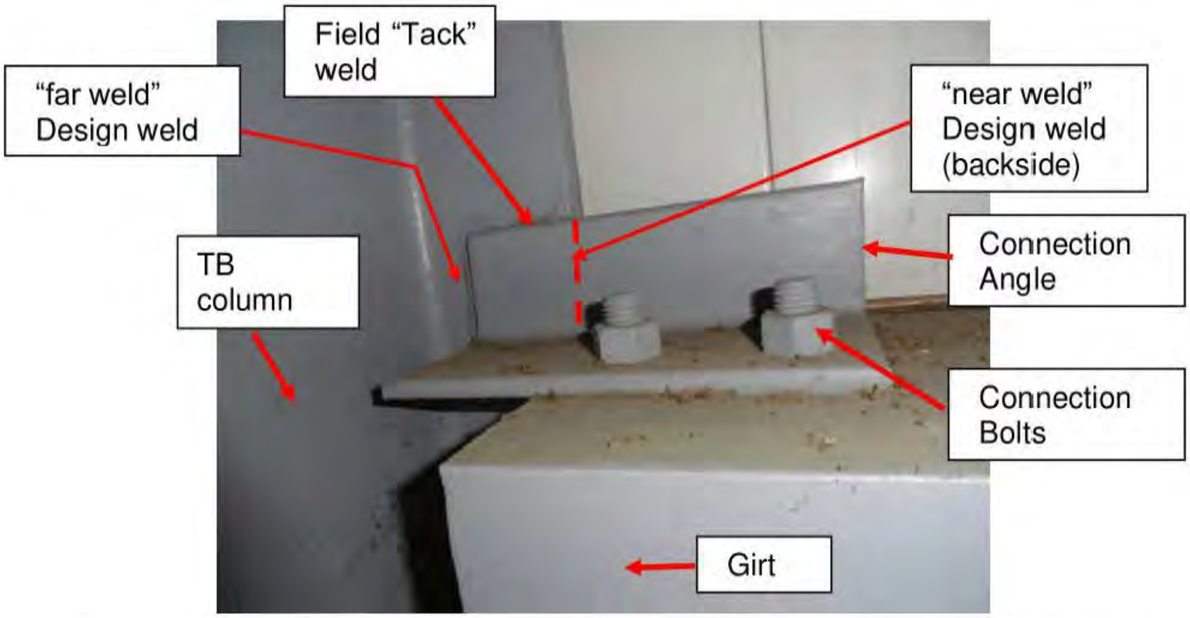

Figure 1-1: General View of the Blowout Panel Girt Attachment to Building Column

Figure 1-2: General View of the Turbine Building North Wall

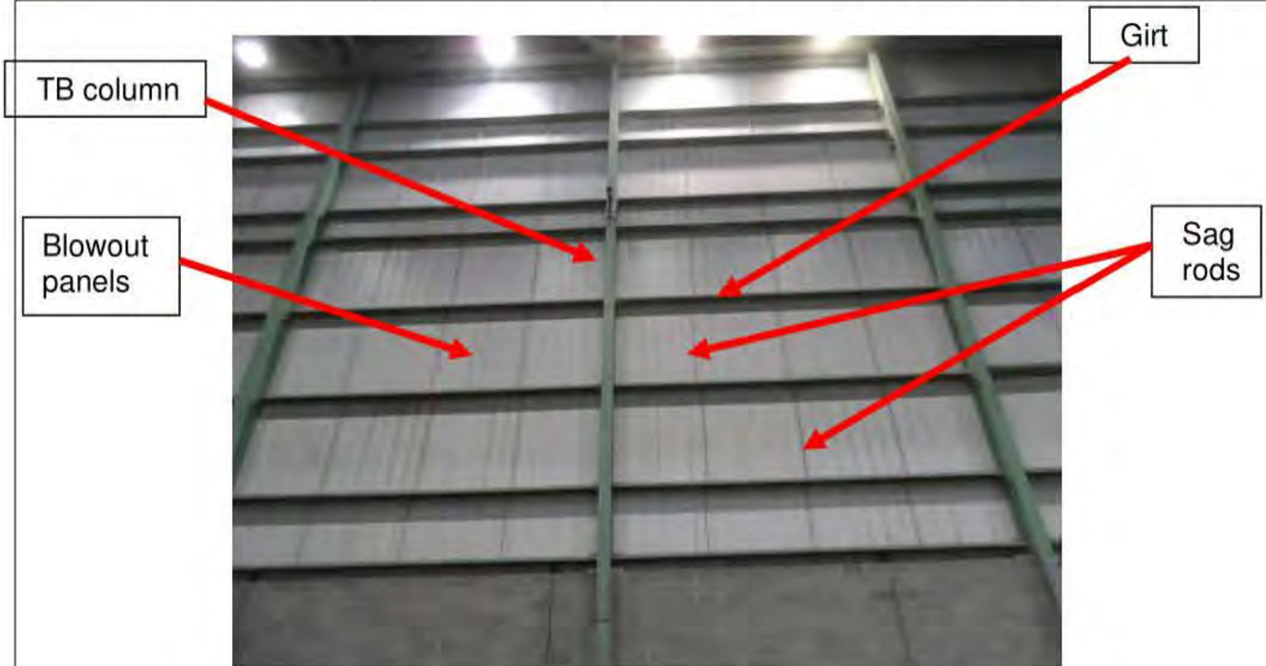

Figure 1-3: General View of the Turbine Building South Wall

2.0 Input and Assumptions

2.1 Input

1. The peak pressure included in this analysis is 0.5 psig dynamic pressure. Per USAR Amendment 25 [2], this is the maximum HELB pressure that can occur inside the Turbine Building with the postulated venting area in the siding (i.e., the minimum siding "blowout" area achieved).

2. Dimensions and geometry of the steel structure are per the Turbine Building structural drawings [7] listed in the reference section, together with walkdown photographs included within the body of this calculation. Some of the key input parameters include:

    a. The blowout panels on the North and South walls are located between column lines C and F. The column spacing in this area is 24 ft. on center except for the spacing between column lines C and D which is 25.5 ft.
    b. Girts supporting the blowout panels on the North and South Walls between column lines C and F are identified as parts 133A, 133B, 133C, 133H, 133V, and 133W. These part IDs are for a steel channel section C10x15.3.
    c. Range of girt spacing on North Wall: 6'-4" to 7'-0"
    d. Range of girt spacing on South Wall: 3'-0" to 7'-0"
    e. 7'-0" girt spacing is the dominant spacing on both walls and is considered as a reference spacing value in the present analysis. Effect of the various spacing values is discussed within the calculation.
    f. Connection angle size is 4x3x3/8 (part C157) on the North wall and 5x3.5x3/8 (part C156) on the South wall. Effects of the two angle sizes are addressed herein.
    g. The girt connection design at column lines D, E, and F for the girts identified above is identical (see Figure 1-1).
    h. The North wall girt connection at column line C is a different design as depicted in Figure 2-1 and shown in the photograph in Figure 2-2 (contrasted to the connection shown in Figure 1-1). The girt channel is double bolted to a steel angle that is welded to the flange of the column. The angle is oriented normal to the channel and the weld length and layout provide for stronger welds. The girt extends eastward beyond the angle where it is bolted to the corresponding perpendicular girt on the adjacent East wall. Because of the angle orientation, the angle transfers the HELB pressure from the girt to the column in bending instead of torsion (at column lines D through F connections). The arrangement

of the bolts with the bolt line normal to the channel also reduces the resulting bending on the bolts.

3. Details of the blowout siding are per vendor information included in Appendix A.

4. Material properties for the structural steel are per the original construction Certified Material Test Reports (CMTRs) included in Appendix A. Based on those records, the enveloped yield and tensile (ultimate) strength of carbon steel are 47,200 psi and 78,400 psi, respectively. For a failure analysis, use of actual material strength values vs. minimum specification values is important to better predict failure. As such, enveloped material properties from the CMTRs are used.

5. The bolts used in the channel girt connection to the Turbine Building columns are specified as ASTM A307 Grade A [19]. Based on hardness testing for a group of 7 bolts retrieved from stock at CNS (see Appendix A), the hardness values (Rockwell B (HRB)) were determined, and converted into approximate tensile strength values. The average of the group tested was 75,800 psi, with a low of 73,600 psi and a high of 78,900 psi. The ASTM standard for A307 Grade A [12] lists a minimum value of 60 ksi. Shear failure testing of the supplied bolts was performed as outlined in Section 3.0. The actual failure capability of the bolts will be derived from the test data and utilized in this evaluation.

6. Section properties for the structural elements are derived from the AISC Manual [4]. See Appendix B for input parameters used.

7. Based on testing performed by the siding manufacturer, Inryco [14], the screws attaching the siding to the channel girts are considered adequate to carry the applied load, beyond the HELB pressure of 0.5 psi.

8. Material stiffness properties for steel used in the analysis are [4]:

   E, modulus of elasticity = 29,000 ksi
   Poisson's ratio = 0.3

2.2 Assumptions

1. The steel structure supporting the panel siding is analyzed statically for the peak HELB pressure of 0.5 psig using material strength appropriately derived from static type load tests (i.e., not fast acting dynamic load tests). The dynamic nature of the HELB load (see for example figure 3-28 in [3]) would result in dynamic amplification of the applied load to the steel structure, with corresponding dynamic material strength

being exhibited. Such dynamic material strength would exceed the corresponding static strength. As such, the application of the HELB load in a static manner is considered acceptable, since the resulting increase in the material strength is offset and/or bounded by the resulting dynamic amplification of the load.

2. The TB siding blowout panels are considered fastened to the girt flange at mid flange width. Based on review of walkdown photographs (Figures 2-3 and 2-4), the location of the attachment points vary and tend to be toward the toe of the flange. This assumption (of fasteners in the center of the channel's flange) results in lower eccentricity of the blowout pressure on the girt cross section and, it is therefore conservative for this failure analysis relative to the prediction of upper bound strength.

3. The sag rods supporting the channel girts are flexible in bending because of the relative large span (between girts) and small cross section, resulting in flexibility and low bending resistance. Therefore the sag rods do not provide significant resistance to twisting of the girt channel. The relative stiffness of the blowout panels is likewise small compared to the stiffness of the girt channel and will not significantly affect the twisting of the girt channel.

4. The analysis is based on girt vertical spacing of 7 ft. Other smaller spacings are not modeled (see Figures 1-2 and 1-3). The use of the girt spacing will be justified and other spacing values will be discussed based on the results derived from the analysis.

5. The fillet weld material is considered as strong as the parent metal. This is a reasonable assumption for evaluation of the weld, where failure will occur either through the root of the weld, or along the weld to base metal fusion line. The use of CMTR material properties further supports this assumption.

6. The bolt hardness (Appendix A) and shear testing (Appendix C) do not provide yield strength values for the bolt material. The nominal bolt material yield strength used in the analytical modeling is considered to be 33% higher than the minimum specified values for similar A36 material. This is considered a reasonable upper bound estimate of actual strength (see for instance [13] where it is shown in Table A1 that the increase over minimum specified yield strength for comparable carbon steel materials ranges from 19% to 31%). Failure criteria of the bolts will be based on the mechanical testing results of a sample.

7. The weight of the blowout panel is based on the weight supplied by the vendor for the inside 18 gage steel lining, an assumed similar weight for the outside cover, and assumed 1.65 lb/ft^3 density for the insulation sheets. Combining the weights of the

steel and insulation results in total unit panel weight of approximately 6 lb per square foot. Based on the presence of the sag rods and integral manner the panels are positioned, the panel and girt self-weight effect on the girt is minimal. However, as shown in Appendix B, the twisting moment resulting in the girt due to eccentricity of the HELB pressure load is opposed by the twisting moment from the panel weight. As a result, for added conservatism with respect to applied torsion, the deadweight and associated torsional loading are applied to the girt.

8. Some of the welded connections (see for example Figure 1-1) include an additional field weld that was added for installation of the angles, typically referred to as a "tack" weld. These field welds are approximately 1" to 1.5" long and of smaller leg size than the structural vertical design welds. Based on the relative size of the tack welds, and their location, these welds provide minimal increase in the angle weld strength. On this basis, and consistent with typical structural design methods, the tack welds are not credited as a load carrying component.

9. Friction between steel surfaces is credited to transfer load between the angle and channel and between the angle and column flanges following weld failure. A range of friction for carbon steels is provided in various text (see [16] as an example). A dry mild steel on mild steel value of 0.78 is listed under static conditions, with a sliding value around 0.42. A value of 0.35 has been commonly used as a lower bound friction value for most steel on steel surface conditions. Since selecting a low value is considered conservative, a value of 0.25 is utilized herein.

10. As shown in Figure 1-2, the duct running in the E-W direction adjacent to the North wall is supported off the girts above and below the duct and off the building steel columns. This imparts additional deadweight on the lower two elevation girts. This additional deadweight is not considered in the analysis. This is acceptable because the duct support points on the girts are located in-between the sag rods and far from the girt supports. As such, most of the duct weight is picked up by the sag rods and the impact on the girt supports is small and negligible.

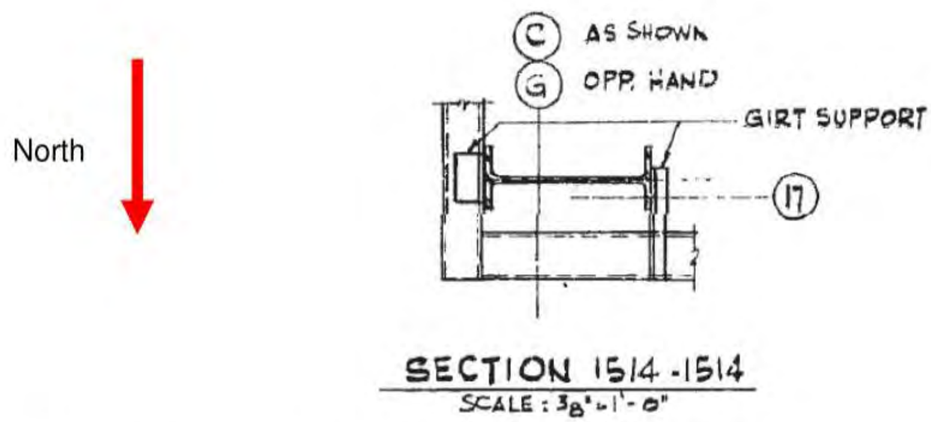

Figure 2-1: Drawing of North Wall Girt Connection at Column Lines C&G [7c]

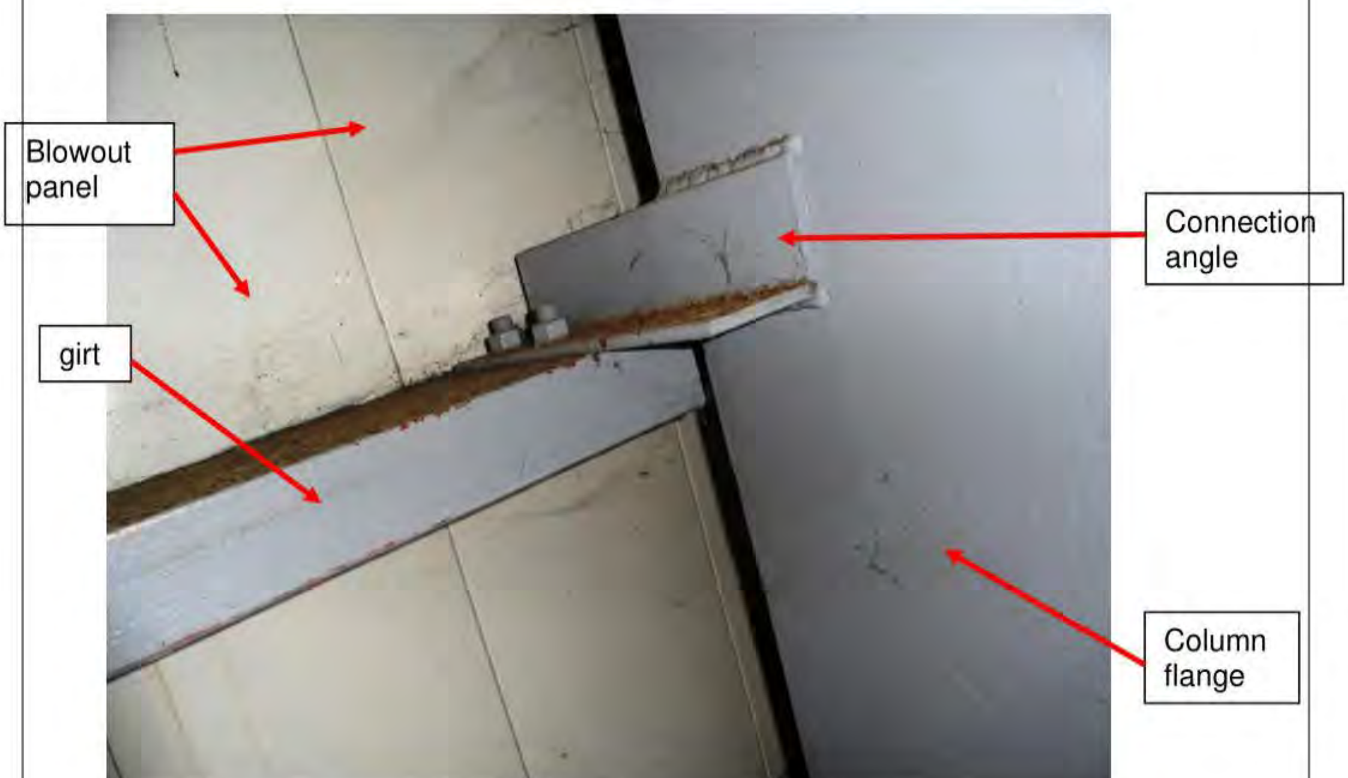

Figure 2-2: North Wall Girt Connection at Column Line C

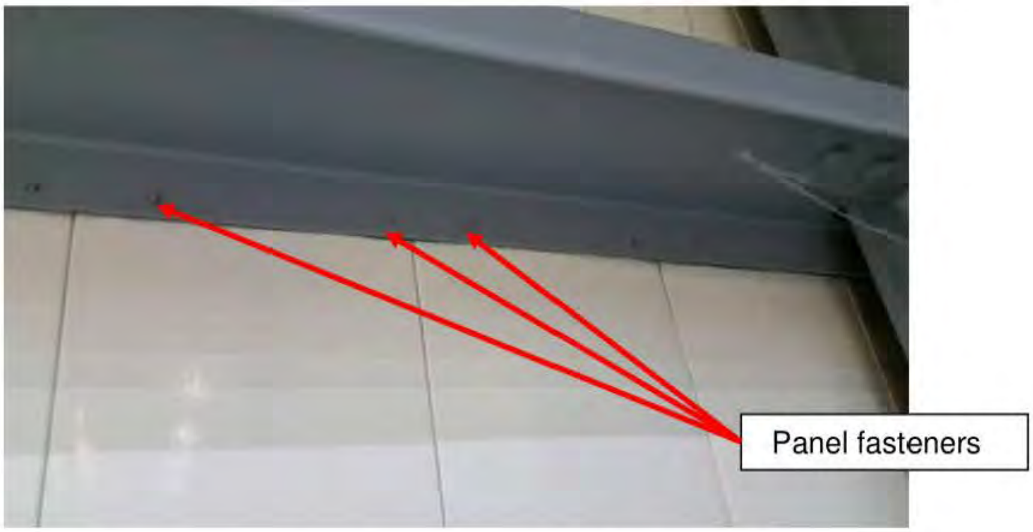

Figure 2-3: General View of Blowout Panel Fasteners to Girt Flange

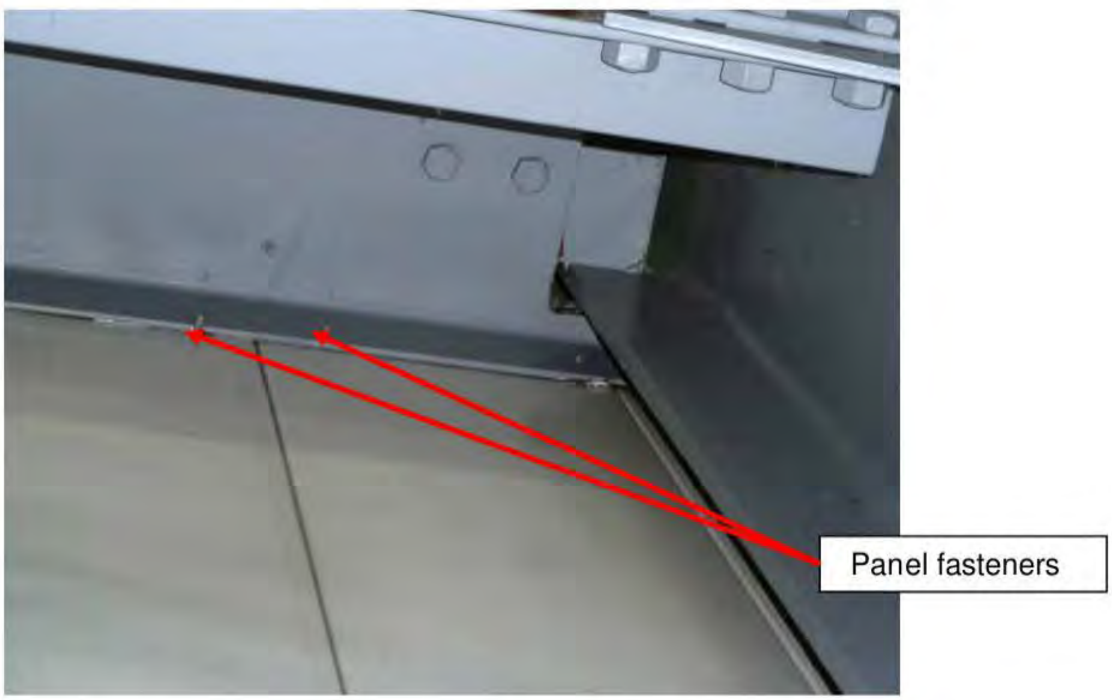

Figure 2-4: General View of Blowout Panel Fastener to Girt Flange

3.0 Methodology and Failure Criteria

3.1 Analysis Methodology

As discussed in Section 1.0, the purpose of this analysis is to predict the failure pressure of the structural supporting system of the siding with respect to the stated value of 0.5 psi in CNS USAR [2]. Prior evaluations [14, 20] have determined the fasteners attaching the siding to the girts and siding panels have sufficient strength to preclude their failure below 0.5 psi. As such the structural support system is the focus of this evaluation performed herein.

For failure to initiate in the support system, the load path in the girt channel beam and its corresponding end connections needs to be investigated. The connector angles (except at column line C), under HELB Turbine Building pressure bear against the flanges of the vertical Turbine Building columns. The angles are welded to the building columns, and bolted to the girts (refer to Figure 3-1). The failure mechanism of this system is investigated as described herein. The girt connection at column line C is a stronger design, is not postulated to fail, as such analysis of column line C connection is not performed.

The analysis of the girt channel, connecting angles, bolts, and welds is performed by means of computer model simulation using the finite element analysis (FEA) method[3]. Figure 3-1 depicts the structural components that constitute the structural support of the blow-out panel siding. The finite element analysis is performed by application of the pressure loading in a static manner. Material properties of the structural elements are included in the model in a non-linear fashion, crediting the materials yield point, and the material behavior following the onset of yielding. To perform this analysis, the load is applied in a load step manner, where the load is increased with each subsequent load step.

Using this approach it is then possible to develop the response history of the girt and its connections, considering the non-linear response of the material as the connection is subjected to the applied loads. A manual (i.e., "hand") calculation is performed and documented in Appendix B, to determine the deadweight and HELB pressure load acting on the girt and the end connections.

The HELB pressure acts horizontally and is reacted by the girt by bending about its strong axis. The deadweight includes the girt channel self-weight and blowout panel weight. Both the lateral HELB pressure and vertical deadweight of the attached siding are eccentric with

[3] The FEA method is a numerical technique for finding solutions to boundary value problems for partial differential equations. It uses subdivision of a whole problem domain into simpler parts, called finite elements. Analogous to the idea that connecting many tiny straight lines can approximate a larger circle, FEA encompasses methods for connecting many simple element equations over many small subdomains, named finite elements, to approximate a more complex equation over a larger domain.

respect to the channel section and, therefore, torsional moments are introduced on the girts. The inclusion of the torsional effects of the deadweight is appropriate, since they reduce the torsional effect of the lateral HELB pressure. Details of the load calculation are provided in Appendix B.

Geometric properties of the steel shapes of the channel and connection angles are obtained from the AISC Manual of Steel Construction [4]. The girt loads are applied as they occur on the channel and resulting load eccentricities are accounted for.

The ANSYS [5a] computer software code was used to perform the finite element analysis. The following analytical models were developed to investigate different parts of the support system.

1. For investigation of the response of the connections to the Turbine Building columns, the analytical model included detailed modeling of the girt end connection and used line beam elements for the remainder of the girt span. This model is half symmetry[4] as shown in Figure 3-2 with symmetry boundary conditions applied at the midspan of the girt.

2. To derive bolt loads, the above model was modified by removing the 3D solid elements used to model the bolts, and replacing them with equivalent beam elements. This modification simplifies post processing of the shear forces and bending moments in the bolts.

3. To investigate the behavior of the girt, following predicted failure of the end connection welds, a study was performed using a variation of the model described above that included explicit modelling of the girt along its complete length with detailed solid element modeling. This model replaced the detailed solid model of the connection angle with line to line contact to represent bearing and sliding of the end connection angles, against the edge of the column flange.

4. Additional simulation is performed for girts between column lines C&D due to the different connection design at column line C. This simulation includes detailed modeling of the connection at column line D and fixed boundary condition at column line C (the connection at column line C is not modeled in detail consistent with the evident strength of the connection as discussed above). This model includes

[4] Half symmetry refers to a modelling technique, where only one half of a symmetric system is modeled and the software develops the structural equations and matrices considering the reflective symmetry without requiring a full model. Since the girt system is considered symmetric about the girt mid-span, use of a half symmetry model and symmetry boundary condition is appropriate.

application of the 0.5 psi HELB pressure and assessment of the welding and bolting stresses at column line D.

The ANSYS software program is a well-established structural/mechanical analysis code that is widely used in both the commercial and nuclear industry for analysis of structural and mechanical systems, subjected to a wide range of static and dynamic loading. The program includes capabilities to incorporate various material constitutive modeling, contact problems, large deformation and large strain. This software is verified and validated (V&V) [5b] in accordance with LPI quality assurance program procedures [5c], applicable for safety related work applications.

Model 1 and Model 2:

These finite element models consist of the 3-D quadratic beam element BEAM189 for the majority of the girt channel with three translation, three rotation, and one warping degrees of freedom at each node. Near the end support a detailed model is used (Figure 3-3) where the girt is modeled with the 3-D 20-node solid element SOLID186. Three elements are used across the thickness of the web and thickness of the flanges of the girt channel. With the mid-side node feature, 7 nodes are used through all thicknesses. Similar modeling is used for the connection angle. The bolts are modeled with solid elements[5]. Rigid connectors (MPC184[6]) are used to transition the girt's section modeled with BEAM189 with the girt's section modeled with SOLID186 elements. The mixed modeling of the girt is adequate for this analysis (Model 1, Model 2) since the intent of this analysis was to capture the detailed behavior at the connection.

Contact element pairs CONTA174 and TARGE170 are used between the angle and channel, and between the solid model bolt shank and angle/channel hole surface. The welds are simulated by restraining the boundary condition of the nodes along the two weld lines.

Self-weight of the steel girt is included by specifying unit weight of steel of 0.284 lb/in^3 [4] for SOLID186 elements and BEAM189 elements. Weight of the panels was applied as external forces and twisting moments on the channel model.

The material behavior of the girt channel and connecting angle is simulated with multilinear isotropic hardening material using the typical shape of the stress-strain relationship for carbon steel material. The stress-strain relationship is scaled to achieve at least the yield

[5] Model 2 was similar, but this model was based on BEAM189 elements used to model the bolts for ease of obtaining the bolt shear forces and bending moments.
[6] These are short rigid elements that are used to effect coupling of connected nodes. The element size is small and is not shown graphically for clarity.

and ultimate stresses recorded in the CMTRs (Appendix A), see item #4 in Section 2.1. The scaled relationship is then converted to true stress-true strain relation using the following:

$$\sigma_{true} = \sigma_{eng}\,(1+\varepsilon_{eng})$$

$$\varepsilon_{true} = \ln\,(1+\varepsilon_{eng})$$

Where:
σ = stress
ε = strain

A graph of the true stress-strain curve is shown in Figure 3-4. The bolts are modeled with a bilinear isotropic hardening law using a yield strength of 47.3 ksi[7] and post yield stiffness of 2%[8] of the elastic modulus of steel of 29,000 ksi. A Poisson's ratio of 0.3 is used for all steel components.

Model 3:

Post weld failure behavior was studied using a variation of the model described above that included detailed solid modeling of the entire span and BEAM189 elements at the support. The model included line to line contact to represent bearing and sliding of BEAM189 against the edge of the column flange.

Model 4:

This model is used for the girts between column lines C and D. It is similar to Model 1 but includes solid modeling of the full length of the girt channel with SOLID186 elements (Figure 3-5). The North wall girts between column lines C and D are 133A and 133B. The shorter span for 133B is conservatively used in the analysis. The girt finite element nodes at column line C are fully restrained. The analysis of Model 4 does not include deadweight of the channel or the panels assuming that the sag rods are effectively supporting the deadweight. This contrasts with Model 1 where the full deadweight is applied to the girt. The analysis of both models therefore bounds possible cases of deadweight sharing by the sag rods. Though the South wall girts are similar, Model 4 is used for analysis of the North wall girts.

Potential failure modes that are investigated by the analysis include the following:

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[7] The yield stress is estimated as a value approximating that of A36, but increased by 33%, as outlined in Section 2.2. This value is conservative by comparison to the increase in ultimate strength demonstrated by bolt testing results in Section 4.0.
[8] A post yield stiffness of 2% is considered as reasonable based on carbon steel stress-strain behavior.

1. Global bending failure of the girt channel at midspan. This failure mode would initiate by the formation of a plastic hinge at midspan which would then lead to large strains, material strain hardening, and rotation of the girt at the hinge. This would lead to failure of the support system (i.e., The beam would be unable to carry any additional load).

2. Bending failure of the connecting angle at end supports. This mechanism would release and limit the bending at the end condition, resulting in the releasing of end moment and increasing the mid-span moment in the girt (see # 1 above). The limited bending at the angle would be associated with strain hardening that is based on the rotational slope of the girt at the support. This failure leads to failure scenario "1" and/ or "3" below.

3. Lateral torsional buckling instability of the girt channel. This is difficult to calculate by manual methods with adequate accuracy due to lack of full symmetry of the girt channel cross-section. ANSYS finite element analysis is utilized to estimate this load limit. At the torsional buckling instability point, the girt is unable to carry additional load because of buckling of the compression flange. This is also associated with rotation of the girt, causing the resulting stress from the girt bending moment to increase as a result of decreased bending resistance (i.e., the plastic section modulus is decreased from a maximum with bending about the strong axis to a minimum with rotation to the weak axis). This results in formation of a hinge similar to "1" above.

4. Torsional failure of the angle. The resulting torsional shear stresses are expected to be large because of the low torsional resistance of single angle sections. However, due to the short length of the angle, the torsional response will be complex and would involve bending and twisting of the angle legs. This is investigated by review of the deformed shape and strain field of the angle. Failure of the angle would lead to scenario "1" or "3" above.

5. Tearing of channel web at bolted connection. This tendency results due to the transfer of the twisting moment from the girt to the angle. If the bolts should tear through the channel's web, the support system has failed.

6. Shear and or bending failure of bolts. The two bolts are subjected to single shear mechanism and bending due to twisting of the girt. Failure of the bolts would result in failure of the support system.

7. Failure of weld. Considering the vertical two lines of weld between the end angles and the column flange, the outer weld can potentially fail due to the bending and twisting of

the angle. Subsequent to failure of the outer weld, the stresses would increase on the inner weld and can potentially reach the failure limit. Failure of the weld alone, will not result in failure of the support system, but could result in additional loads being transferred to other components of the support system, leading to complete failure of the girt (I.e., under "1" and/or "3" above).

8. The fastening of the panels to the girt was evaluated by CNS [14] and found to be not a credible failure mode at the given pressure load for 7 ft girt spacing. That evaluation investigated tensile failure, pullout, and pull-over of the connecting screws and concluded that 0.5 psi pressure would not fail the panel connection to the channel. The evaluation did not consider the twisting of the girt channel which would cause additional forces on the screws and panel sheet metal. This however is a difficult mechanism to evaluate without detailed modeling of the panel sheet metal. At this time, additional failure mechanisms involving the panel fasteners were not investigated.

The above potential failure modes were investigated by review of the results of the FEA. The girt connection at column line C is of different design that is considered to be more rugged. Therefore, the above connection failure mechanisms are concerned with the girt connections at column lines D, E, and F.

The results of the analysis are discussed in Section 4.0

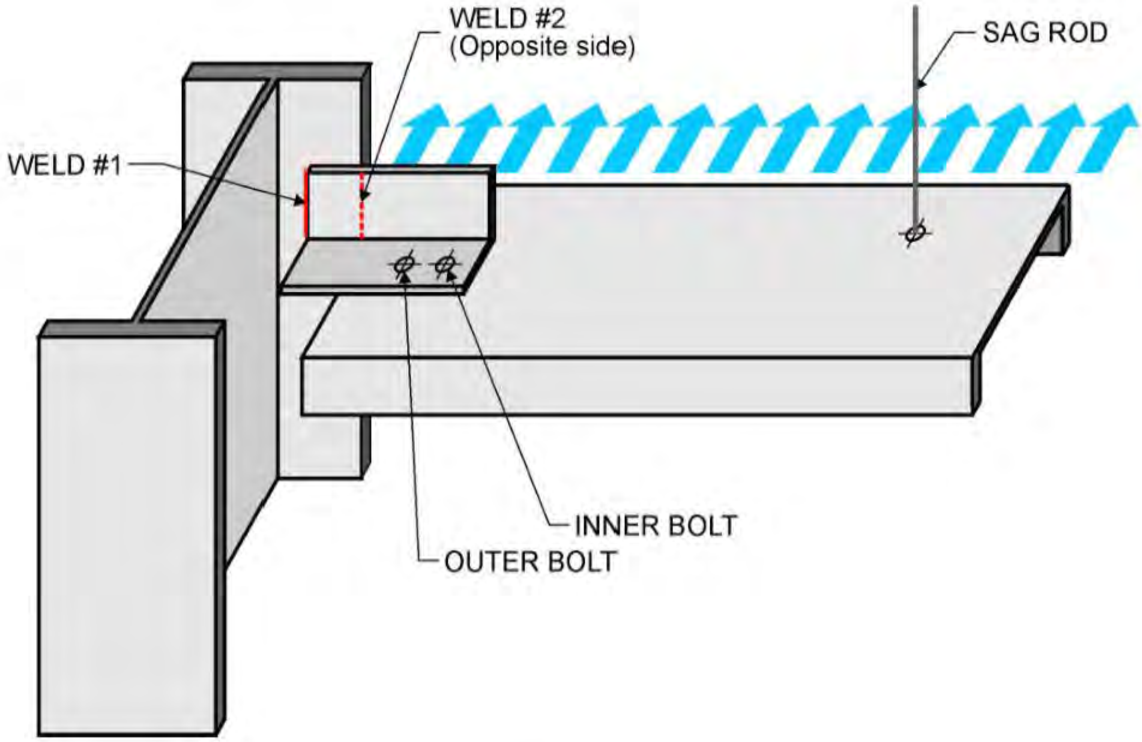
3.2 Bolt Failure Testing

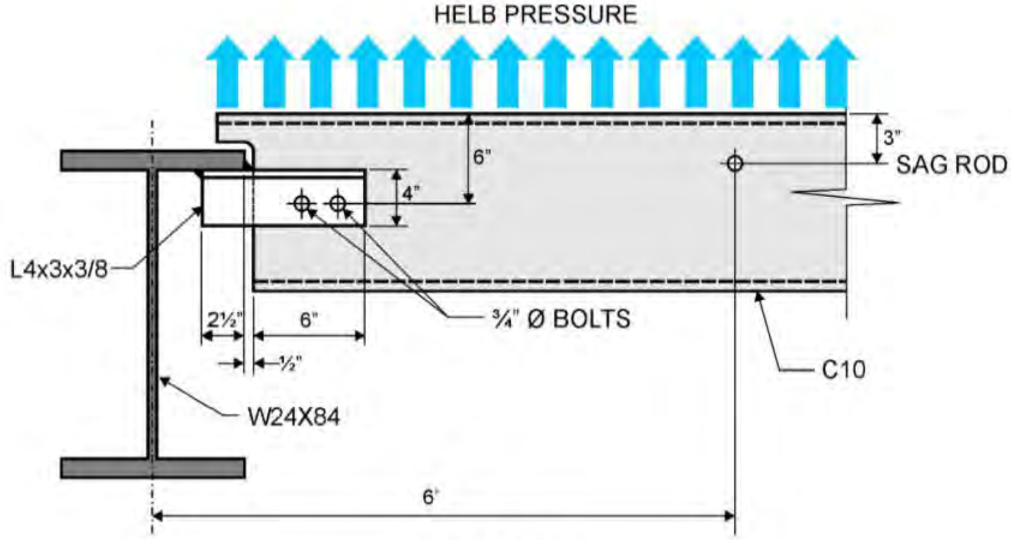
CNS provided LPI a sample of A307 bolting from stock for failure testing. The bolts were hardness tested by CNS, with obtained results listed in Appendix A. The Rockwell B (HRB) hardness testing results were converted to approximate tensile strength values, with an average tensile strength of 75,800 psi.

Shear testing of the supplied bolts was performed by LPI per the National Aerospace Standard NASM1312-13 test method [15], as outlined in Appendix C. The purpose of the testing was to demonstrate the actual shear failure stress for the bolts and to derive the ultimate tensile strength of the bolt material. The bolts were tested in double shear configuration (consistent with the standard). The configuration of the bolts in the connection are in single shear. The double shear test results provide a basis for determination of the absolute strength of the bolt.

3.3 Failure Criteria

Failure criteria of the evaluated elements are generally based on component stresses exceeding the material tensile limit (Fu) under tensile load, 75%Fu for shear loading, the formation of plastic hinges (i.e., predicted moment exceeds the plastic moment capacity of the evaluated section), and the formation of instability through buckling (i.e., compression flange failure and resulting incipient buckling of the section). Fillet welds contain complex state of stresses that involve tension, shear, compression, and bearing. As such, a conservative limit for fillet weld failure to take place was postulated to occur between 0.75%Fu (shear failure) and Fu (tensile failure). The shear failure limit of 0.75%Fu is based on guidance in Table 13 of reference [10].





TOP VIEW

Not to Scale

Figure 3-1: General View of the Girt Channel and End Connection.
(lower: plan view on connection)

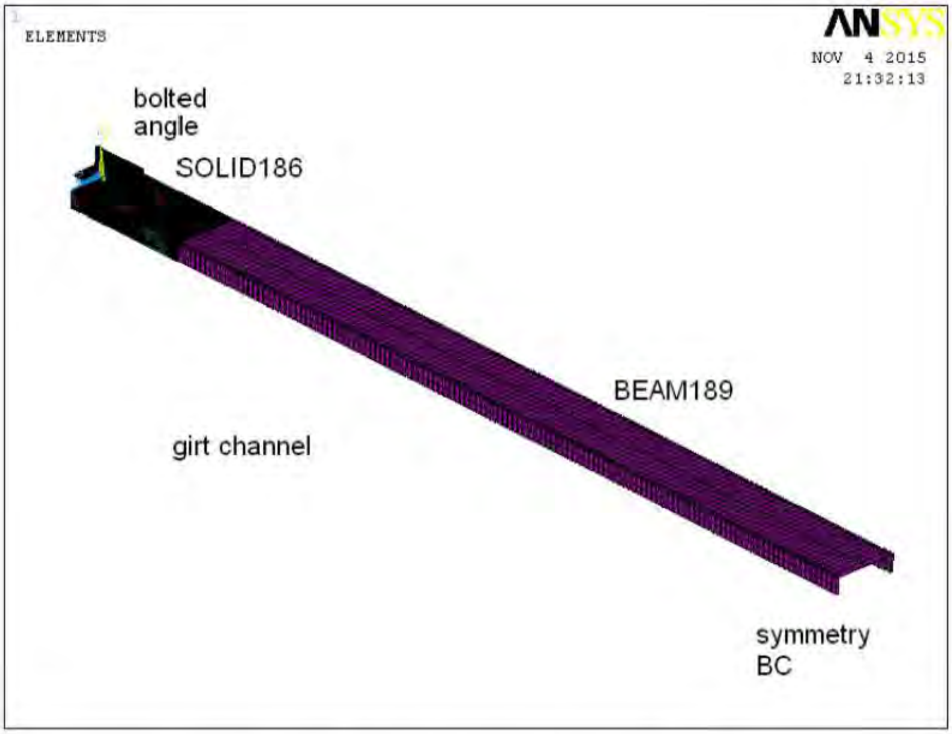


Figure 3-2: General View of the Finite Element Model – Half Symmetry Boundary Condition (BC)
(Symmetry boundary condition at mid-span point)

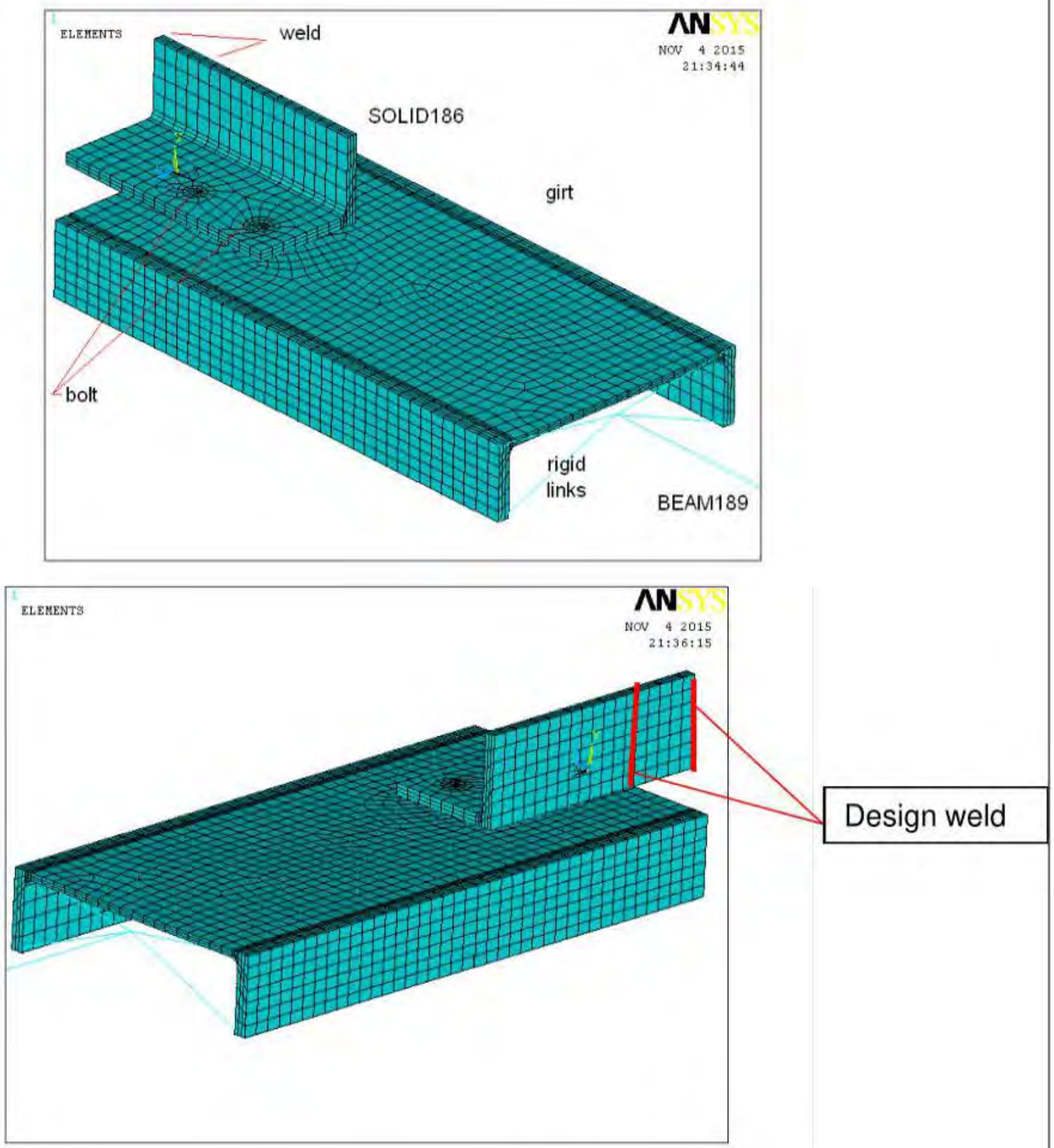


Figure 3-3: Detailed FE Model at Connection

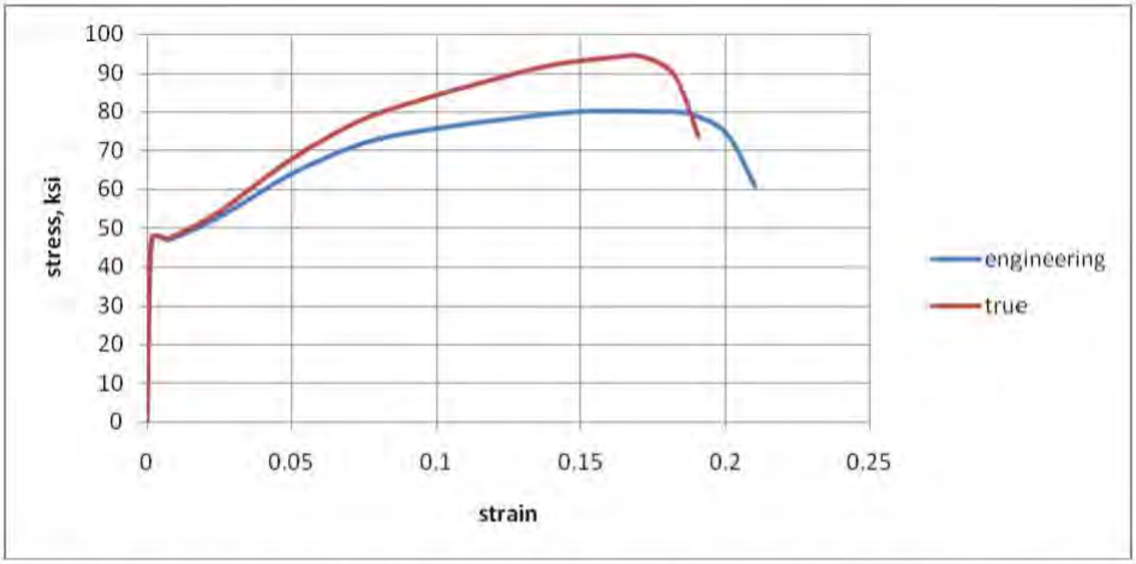
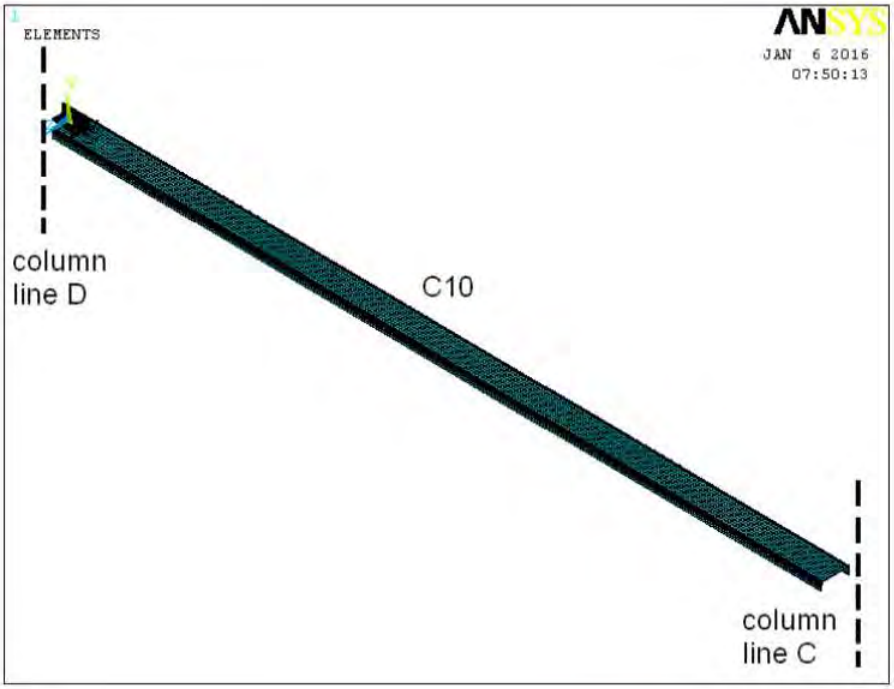


Figure 3-4: True Stress Strain Curve Used for the Girt Channel and Angle Finite Element Analysis



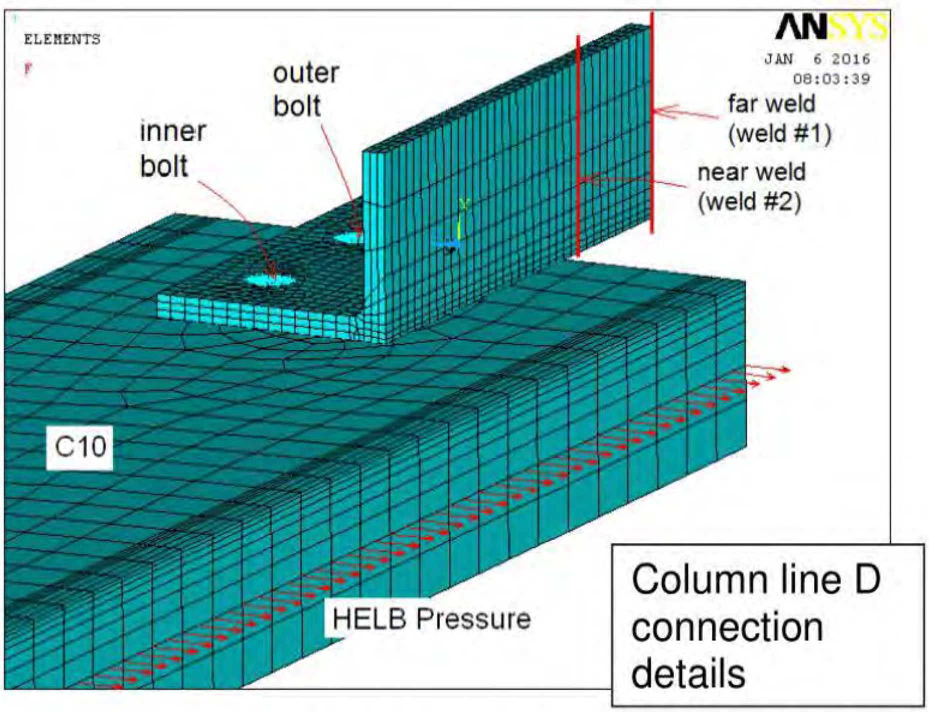


Figure 3-5: General View of FEA Model 4 of Girt Between Column Lines C&D.

4.0 Analysis Results

The results of bolt testing and the finite element analyses described in Section 3.0 are presented in this section.

4.1 Bolt Testing

Bolt testing was performed, as outlined in Section 3.0 with results presented in Appendix C. Bolt failure results are summarized in Table 4-1 below. The average derived ultimate strength value is about 10% higher than the minimum code specified value of 60 ksi [12]. This indicates that the 33% increase assumed for the yield strength (see Assumption number 6 in Section 2.2) is conservative relative to driving an upper bound failure limit and is thus acceptable.

Table 4-1: Bolt Test Results

Bolt Number	Peak Double Shear Force, kips	Single Shear Failure Load, kips	Derived Ultimate Strength of Bolt Material, Ksi
1	31.726	15.863	70
2	28.425	14.213	63
3	29.858	14.929	66
4	28.267	14.134	62
5	28.877	14.439	64
6(1)	33.860	16.930	75
7	30.432	15.216	67
Average (excluding minimum and maximum values – 5 bolts)			66

Notes:

(1) Results for bolt #6 are presented for information only. The results were biased upward due to binding in the test fixture that caused the measured load to be higher than the actual force in the bolt.

(2) The derived ultimate strength is based on bolt root area of 0.302 $inch^2$ [4] and shear failure stress of 0.75 of the ultimate strength for carbon steel materials [10].

4.2 Nonlinear Load Step Analysis

The analytical model (Model 1) incorporated elastic-plastic material properties to simulate the behavior of the girt support system under increasing load. The model included solid modeling of the angle and a short piece of the channel, the bolting, and the contact between the angle and channel. The vertical leg of the angle is restrained by the two vertical weld lines. The HELB pressure load is applied by increasing the load with each subsequent load step.

Figure 4-1 and Figure 4-2 show the vertical and lateral displacement contours, respectively. The vertical displacement shows significant twisting of the girt. The maximum lateral displacement at mid-span is 5.2 inches. Figure 4-3 and Figure 4-4 show total mechanical strain intensity contours in the angle. Significant strain is obtained around the bolt holes, along the weld lines and in the angle fillet. Twisting of the angle is also noticeable.

Bolt forces and moments are obtained from the analysis model (Model 2) with the bolts modeled with the beam elements. The bolt section corresponds to the tensile area of 0.334 $inch^2$ [4]. The results at 0.5 psi HELB pressure loading and with both fillet welds in place, show maximum bolt tensile stress of 62.4 ksi at the nut (dominated by bending) and shear stress of 11.7 ksi in the two orthogonal direction (resultant shear of 16.5 ksi). The shear stress is well within the failure threshold of 0.75 Fu (i.e., 16.5 ksi < 0.75 x 66.0 ksi = 49.5 ksi)[9]. The tensile stress is within the ultimate strength of the bolt material (i.e., 62.4 ksi < 66.0 ksi). However, since this stress is dominated by bending and is applied to the threaded section of the bolt, the true tensile stress in the threads is larger because of geometric discontinuity of the threaded bolt. Per reference [6], 65% of bolt failures occur in the threads at the nut face. Though the discussion in reference [6] is related more to fatigue loading which is not present in this case, bending and stress concentrations are detrimental to bolts. Per reference [6], stress concentration factors (SCF) in bolts vary. Based on ranges of values in reference [6], a value of 2.5 is considered a conservative lower bound. This indicates that the bolts would fail before the pressure reaches 0.5 psi. Using Model 2 results to derive the history of the maximum bolt tension, this evaluation indicates a maximum un-intensified bolt tensile stress of 34 ksi at 0.25 psi HELB pressure. With the 2.5 SCF, the bolt stress can reach the ultimate strength (i.e., 2.5 x 34 ksi = 85 ksi > 66.0 ksi failure strength)[10].

The weld forces are obtained by square-root-sum-of-squares (SRSS) combination of the orthogonal reactions. This is an appropriate method, but consistent with typical design practice, to assess stresses in the weld elements. The weld stress is computed by dividing the weld force by the weld area along the throat of the 0.25" leg size of the fillet welds (the throat thickness is typically considered as 0.707 x leg length, or 0.177 in). As shown in Figure 4-4, weld stresses are highest at the lower length of the weld (near the "crotch" of the angle), because of the diaphragm (stiffening) action of the horizontal leg of the angle. A graph of the weld stress vs. load step is shown in Figure 4-5. Weld stress shown in the

[9] Where Fu was derived in Table 4-1, as 66.0 ksi.
[10] Resulting failure could be approximated at a pressure of 66/85 x 0.25 = 0.194 psi, conservatively use 0.25 psi.

graph is the maximum stress in the line weld which occurs at the lower end. The graph shows the maximum weld stress for both the near and far welds[11]. The initial value of stress at the beginning of the graph is due to deadweight loading. The horizontal axis indicates the fraction of the HELB pressure load of 0.5 psi. It is shown that as the weld stresses increase with the increase in the HELB pressure load, the weld stress at some point exceeds the failure limit[12]. The graph also shows the near weld stress is higher than the far weld stress. This is due to significant bearing of the angle against the column at the near weld. Due to the higher strength in bearing, the bearing stress component is not postulated to induce failure for this configuration. Therefore, the stress in the near weld is recalculated excluding the bearing stress component. This is shown in the graph as "near weld exclude bearing".

The failure of the far weld is postulated to occur at approximately 50% of the 0.5 psi pressure load (i.e., at 0.25 psi) where the far weld stress is well above 75% Fu and slightly below Fu. With the failure of the far weld, the stress in the near weld shows a sharp increase and follows a new curve labeled as "near weld; far weld fails at 0.25 psi.

It is postulated that weld failure for both the near and far welds begins at the lower edge of the weld leg. Once failure initiates, it will propagate upward because the lower end of the remaining weld ligament will always have stresses exceeding the failure limit.

Following the curve of the near weld stress (with far weld failed), the stress increases as the pressure continues to ramp up and exceed the failure limit at about 55% to 75% of 0.5 psi pressure load (i.e., 0.275 to 0.38 psi). Thus, it is postulated that the welds will fail at or below 0.5 psi.

Results of the analysis with near weld in place, far weld failed and removed, for pressure at 0.5 psi are shown in Figure 4-6 through Figure 4-9. In these figures, the contour values have been adjusted so that more detail of high strain distribution can be displayed. Gray areas indicate strains outside the range of the shown contour values (typically higher than the maximum value shown). The plots show that the strain is well into the plastic range and concentrated at the bolt holes, angle fillet, on the angle corner edge bearing against the channel web, and on the back of the angle along weld lines. High strain is also shown at the location of the removed far weld. This is due to residual strain remaining in the back of the angle after unloading associated with failure of the weld.

[11] The terminology of near and far is with respect to the girt channel (see also Figure 1-1).

[12] The failure of the weld is dependent on the state of stresses and the dominant stress in the weld. Shear failure is set at 0.75% of ultimate strength (Fu) and tensile failure is set at Fu. Because of the complex state of stresses in fillet welds, fillet weld failure would occur at some stress between these two limits.

By review of the strain intensity contours at 0.5 psi pressure with far weld failed in those figures, failure of the angle and/or channel web at the bolt location due to plate tearing is not likely. The high strain intensity is localized at the bolt hole and within limited distance from the bolt hole and is dominated by bearing action against the bolts. Note that the bolt head and nut are not modeled and therefore, this strain limit is likely overestimated. Since the strain field in the angle and channel web does not approach failure strains for carbon steel, plate tearing around the bolts is not a credible failure mechanism.

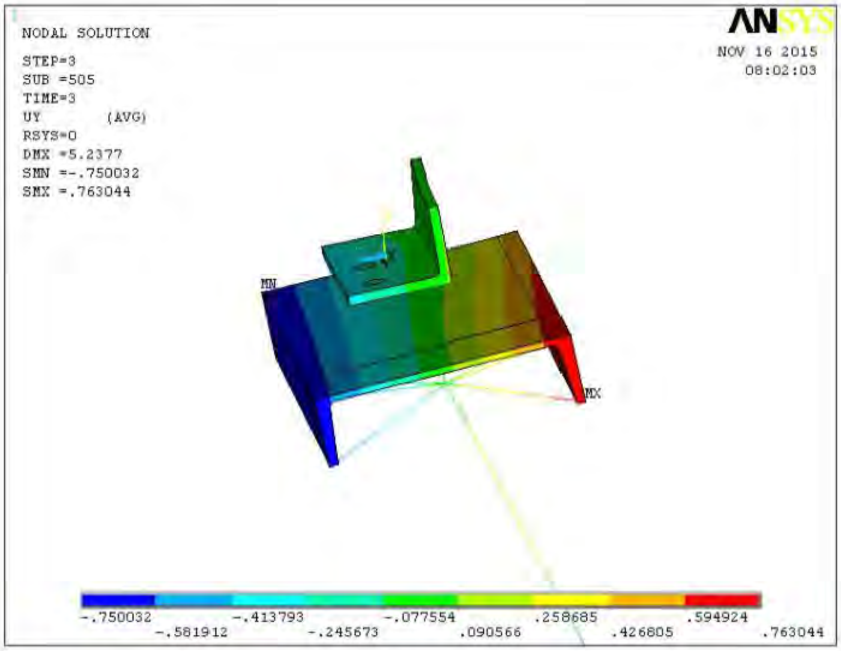


Figure 4-1: Contours of Vertical Displacement – 0.5 psi

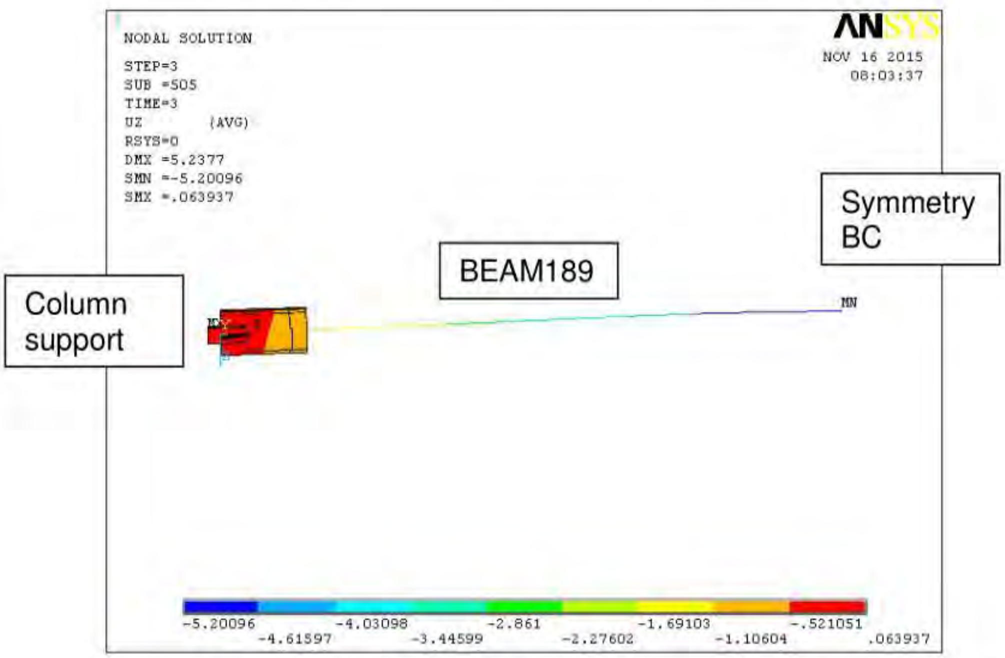


Figure 4-2: Contours of Lateral Displacement – 0.5 psi

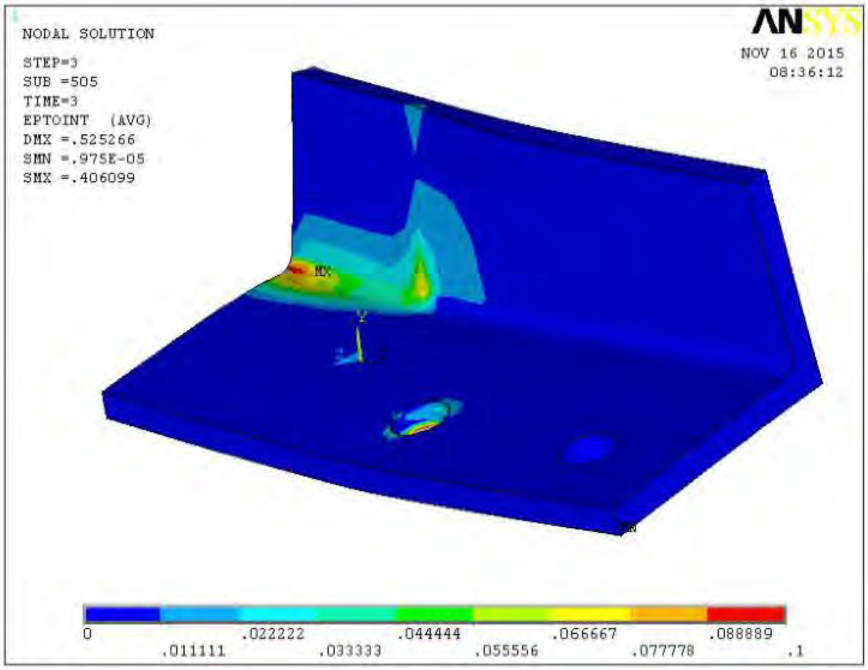


Figure 4-3: Contours of Strain Intensity in the Angle – 0.5 psi

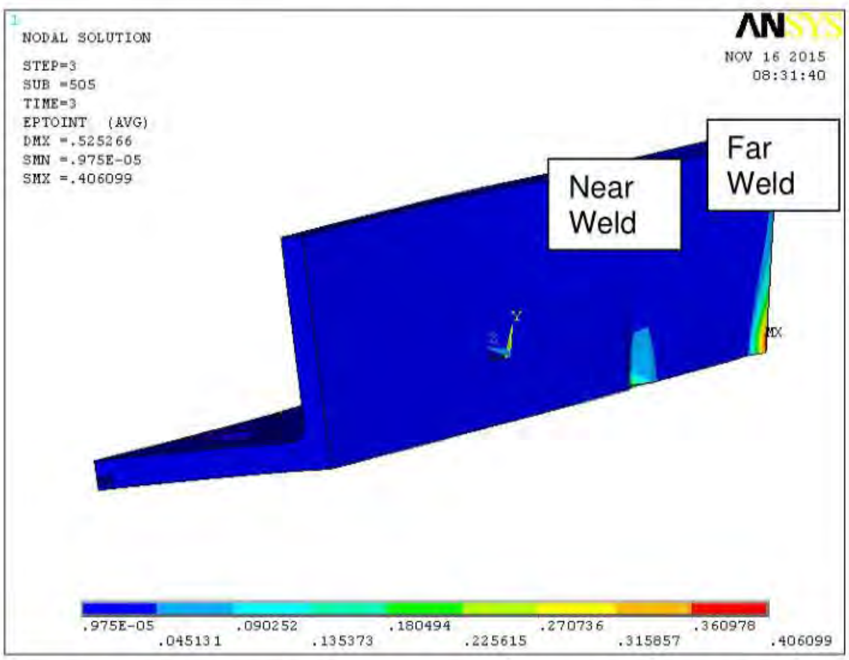


Figure 4-4: Contours of Strain Intensity in the Back of Angle – 0.5 psi

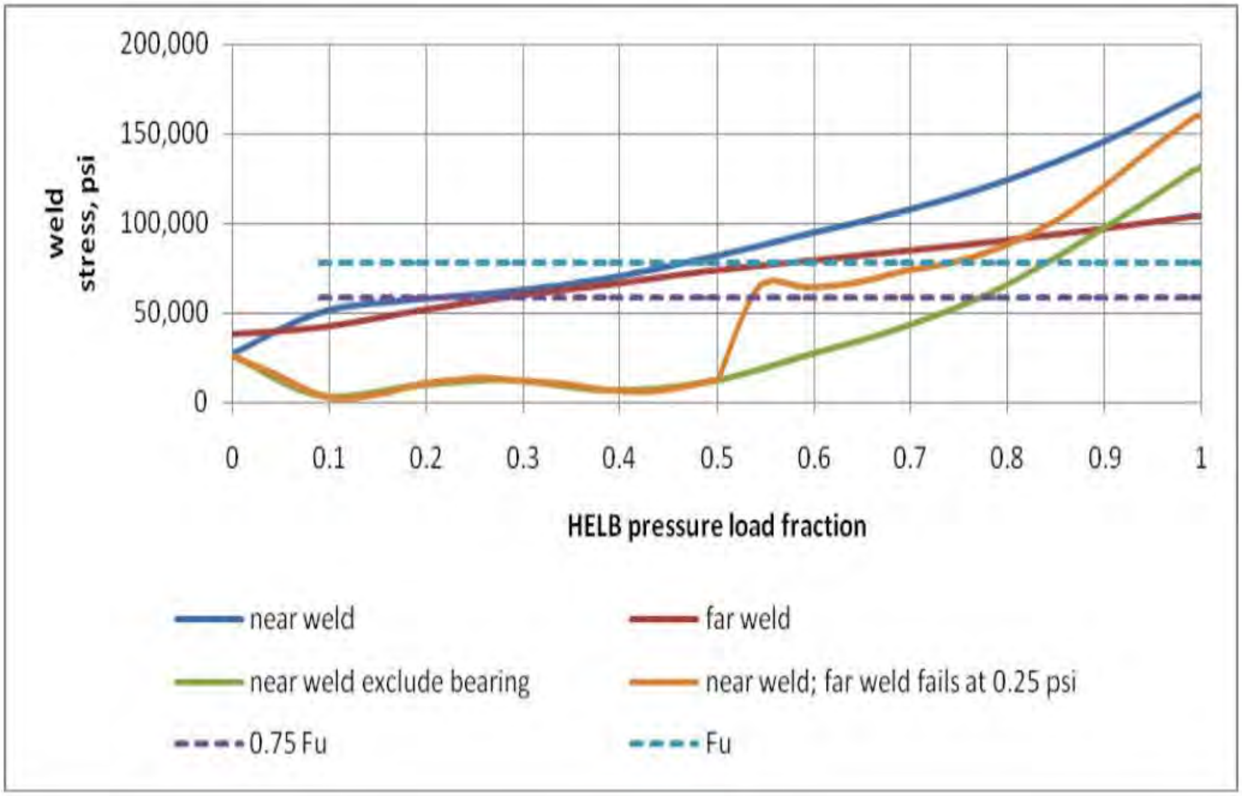


Figure 4-5: History of Maximum Weld Stress
(Horizontal axis represents the fraction of HELB pressure of 0.5 psi)

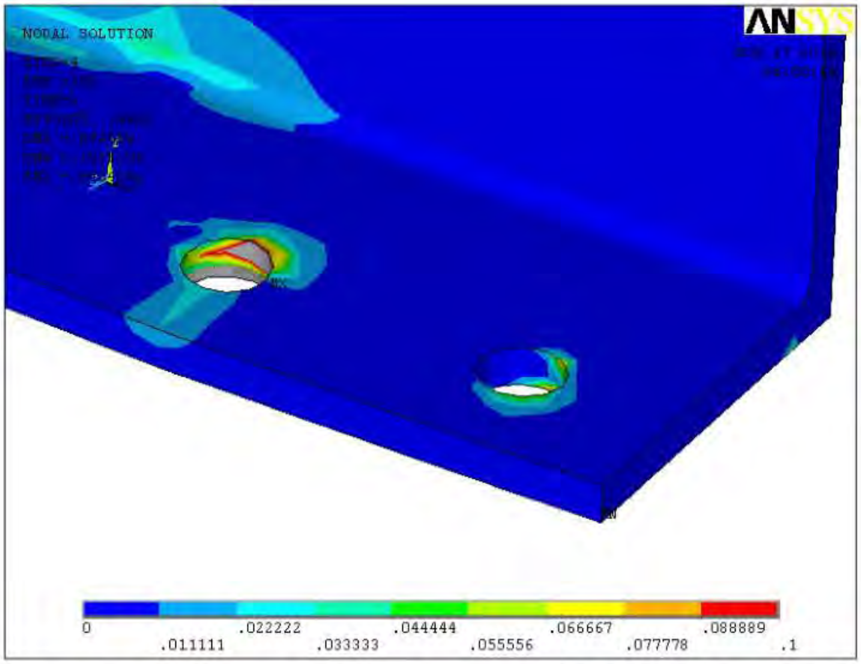


Figure 4-6: Strain Intensity Contours in the Angle – 0.5 psi, far weld removed

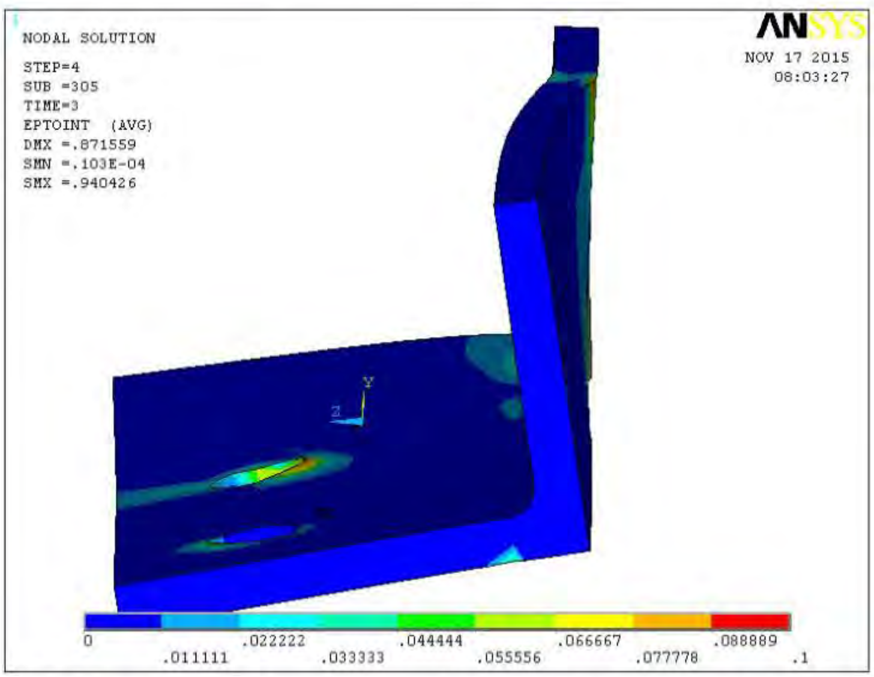


Figure 4-7: Strain Intensity Contours in the Angle – 0.5 psi, far weld removed

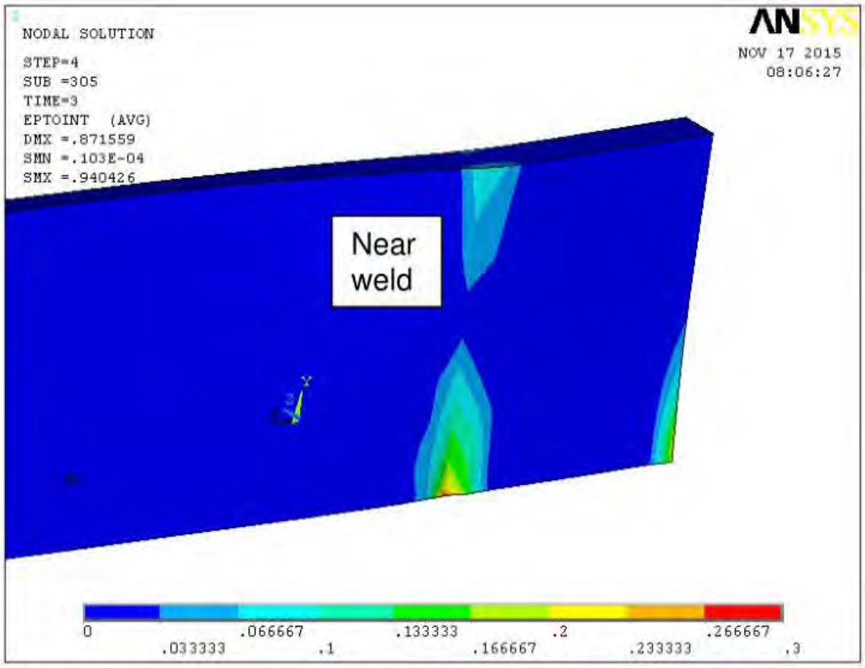


Figure 4-8: Strain Intensity Contours in the Back of Angle – 0.5 psi, far weld removed

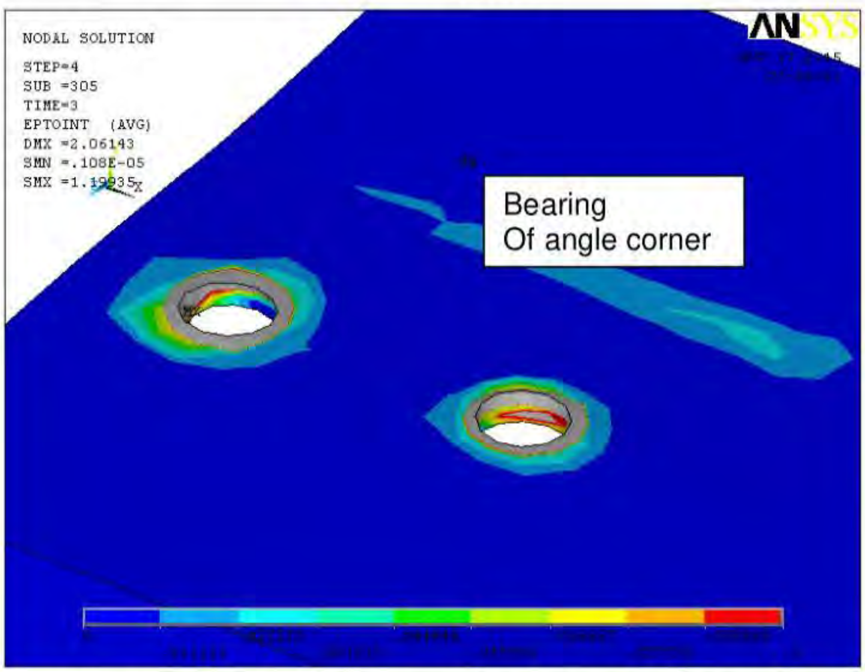


Figure 4-9: Strain Intensity Contours in the back of Channel – 0.5 psi, far weld removed

4.3 Post Weld Failure Analysis

Post weld failure, the girt angles bear against the column flanges due to the effect of the lateral pressure (see configuration in Figure 3-1 as an example). The deadweight of the girt and panel will be shared by the sag rods and by friction at the support point of the angle bearing against the Turbine Building column flange. Based on assessment below, this friction is adequate to carry the applied deadweight:

Reaction from HELB pressure, R_pr = HELB_pressure x Trib. Width x 1/2 Span[13]

$$= 0.5 \text{ psi} \times 7 \text{ ft} \times 12 \text{ ft} \times 144 \text{ (inch/ft)}^2 = 6{,}048 \text{ lbs}$$

This reaction force is applied at each column flange

Maximum friction force (using conservative lower bound coefficient of friction of 0.25 – See Section 2.2):

$$R\_fr = 6{,}048 \text{ lbs} \times 0.25 = 1{,}512 \text{ lbs}$$

Deadweight reaction, R_wt = Girt Wt x 1/2 Span + Siding Wt x Trib. Width x 1/2 Span

$$= (15.3 \text{ lb/ft} \times 12 \text{ ft}) + (6 \text{ lb/ft}^2 \times 7 \text{ ft} \times 12 \text{ ft}) = 687.6 \text{ lbs}$$

Thus, the full deadweight reaction[14] demand R_wt (687.6 lbs) is well below the friction limit capability R_fr (1,512 lbs).

The girt will continue to bend laterally and twist until it reaches instability due to lateral torsional buckling. This is simulated using a similar model (Model 3) that excludes the welds and the detailed end connection model and includes detailed modeling of the girt at mid-span. This model will accurately capture the deformed shape of the twisting, bending and lateral torsional buckling. The model included line to line friction at the end support to represent sliding of the girt angle against the edge of the column flange. The remaining part of the girt is modeled using BEAM189 elements which extend to the support. A symmetry boundary condition is applied at the mid-span end of the model. Figure 4-12 shows a general view of the model. The analysis is simplified by not modeling the angle and excluding the deadweight effect. This analytical approach is acceptable, since the lateral torsional buckling is highly influenced by the lateral pressure.

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[13] Distance between Turbine Building columns on North and South walls is 24 ft. Thus, half span of the girt is 12 ft.

[14] This is conservative, since most of this weight could potentially remain supported by the sag-rod, if still functional following failure of the girt connection welds, which would reduce the demand on the friction support.

The nonlinear material, load step analysis of the 0.5 psi pressure was applied using a refined load step until a converged solution could not be obtained. A non-converged solution is an indication of instability in the model, as the analysis predicts buckling. Results of the last converged solution are shown in Figure 4-13. It is shown that the maximum strain at mid-span of the girt indicates rotation of the girt and yielding at the extremities of the cross section; however, there exists potential for additional strength due to yielding and strain hardening. The strain profile along the mid-span cross section indicates also significant rotation of the cross section.

Figure 4-14 shows a graph of the deflection history at mid-span. The deflections are obtained at two nodes at the top and bottom flanges of the girt. The deflections show significant rotation of the girt at mid-span. It is also shown that at approximately 85% of the 0.5 psi pressure (0.425 psi), the behavior becomes unstable and indicates lateral torsional buckling mode of response. For conservatism, the load step at which the solution did not converge is taken to be the critical load for lateral instability. At this point, the channel in the buckled configuration cannot support the applied load.

At low level of HELB pressure and prior to any rotation of the channel, the lateral load is reacted entirely by the strong axis moment of inertia of the channel. The rotation of the channel causes the lateral load reacted by the strong axis moment of inertia to decrease while the component affecting the weak axis builds up[15]. Using the illustration below in Figure 4-10, this is represented mathematically as follows:

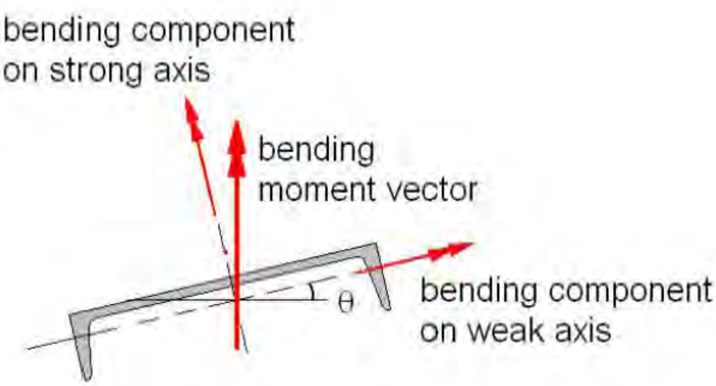

Figure 4-10: Configuration of Channel during Rotation under Load

[15] In the limit when the rotation is theoretically 90 degrees, the full pressure load would be reacted by the weak axis moment of inertia.

$Z_{xx\_c10} := 15.8 \cdot in^3$ plastic bending section modulus about strong axis [17]

$Z_{yy\_c10} := 2.35 \cdot in^3$ plastic bending section modulus about weak axis [17]

$M_x := w_{Pr\_girt} \cdot \frac{span_{girt}^2}{8}$ girt mid-span bending moment for simply supported beam, load density is obtained from Appendix B

$M_{x\_c10}(\theta) := M_x \cdot \cos(\theta)$ component of bending moment about strong axis

$M_{y\_c10}(\theta) := M_x \cdot \sin(\theta)$ component of bending moment about weak axis

$\sigma_z(\theta) := \frac{M_{x\_c10}(\theta)}{Z_{xx\_c10}} + \frac{M_{y\_c10}(\theta)}{Z_{yy\_c10}}$ combined bending stress

$\sigma_{ult} := 78.4 \cdot ksi$ ultimate stress of channel steel material (Section 2.1)

The results of this evaluation are illustrated below in Figure 4-11.

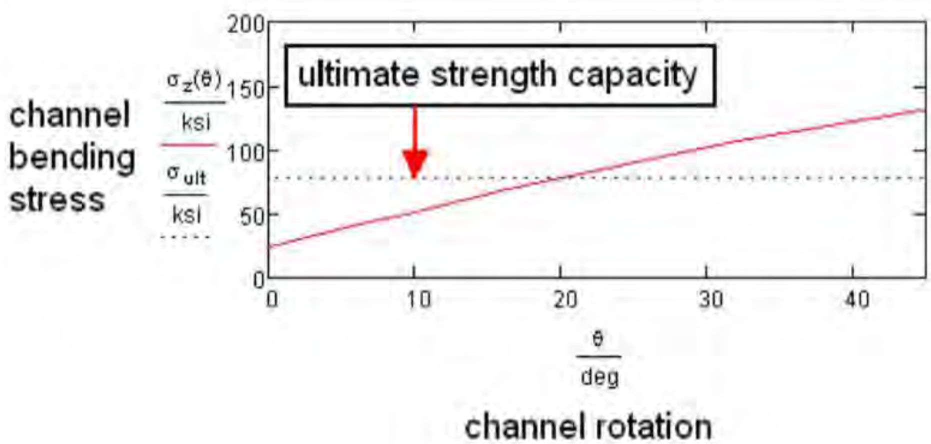

Figure 4-11: Bending Stress Variation with Channel Rotation

Figure 4-11 shows that the bending stresses in the channel increase significantly with small rotation of the channel due to the very low moment of inertia about the weak axis. Therefore, rotation of the channel due to torsion and lateral buckling is detrimental to the state of axial (longitudinal) strains and would quickly lead to complete failure of the mid-span section by literally forming a plastic hinge in the girt. Based on the displacement results in Figure 4-14, the rotation of the channel at mid-span exceeds 20 degrees and therefore, bending failure at the mid-span section is predicted. Since the HELB pressure is applied in the FEA model as a surface pressure, the direction of the pressure remains normal to the channel flange surface as the channel section rotates. This is conservative in the evaluation model since this reduces the effect of loading the channel about its weak axis.

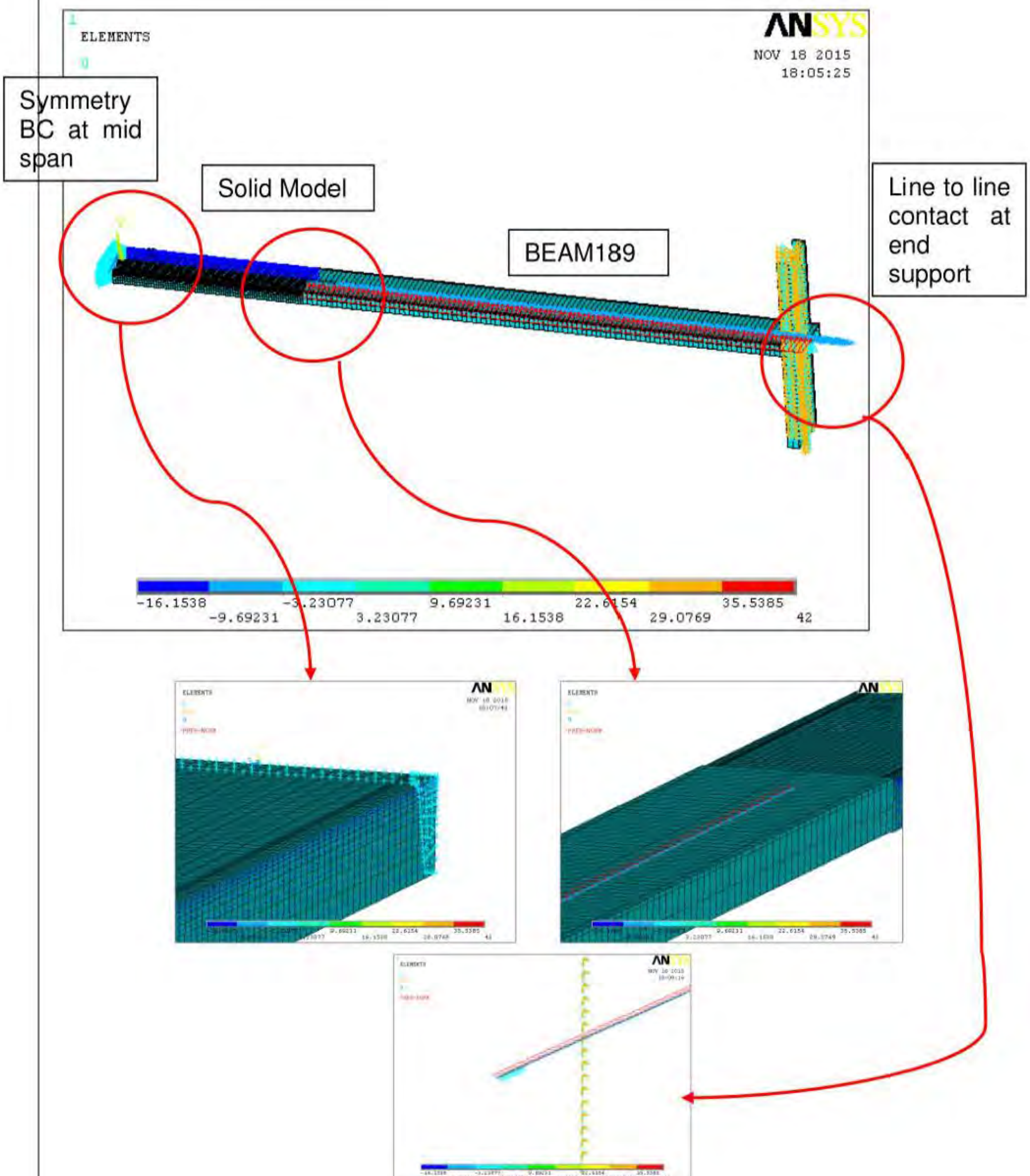

Figure 4-12: General View of the FEA Model

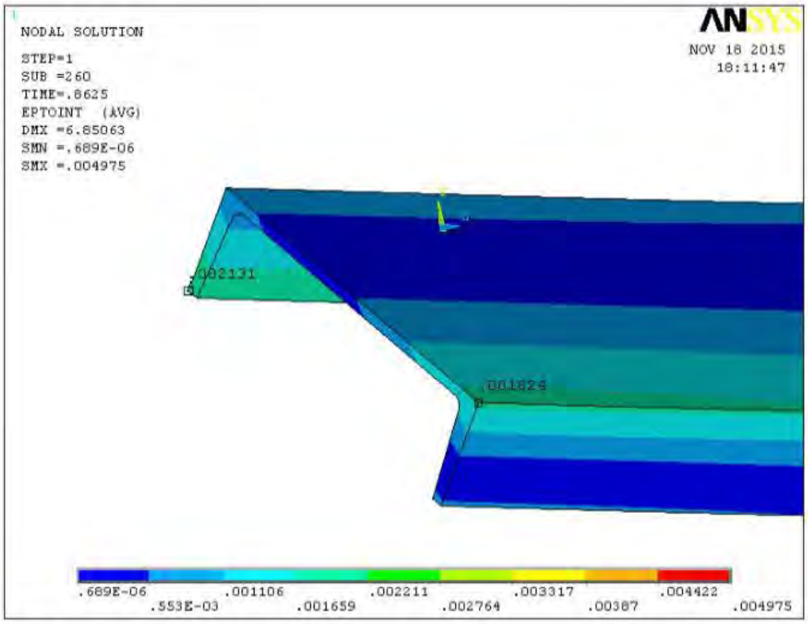

Figure 4-13: Total Strain Intensity Contours at Mid-Span – 0.86 x 0.5 psi

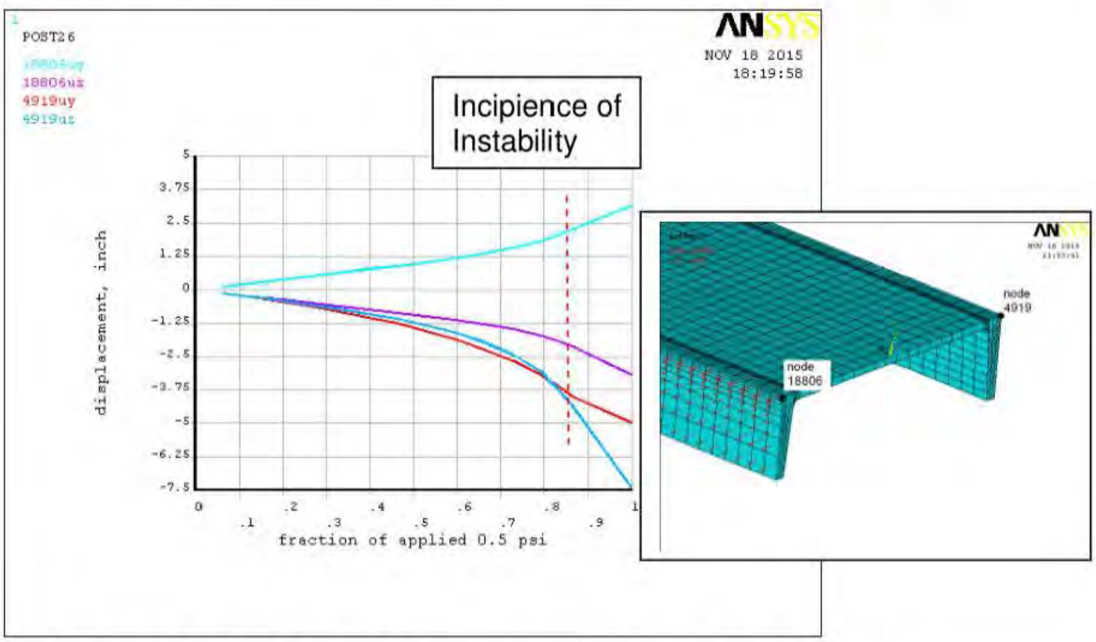

Figure 4-14: History of Displacements at Mid-Span Section

4.4 Girt Between Column Lines C&D

Model 4 was subjected to 0.5 psi HELB pressure which results in 42 lb/in load on the girt for 7 ft girt spacing. Investigation of the weld and bolt stresses (at column line D connection) indicated that failure of the bolts and/or the welds would occur below 0.5 psi HELB pressure as shown in Figure 4-15 and 4-16. Figure 4-15 shows the stress history of the maximum bending stress in the two bolts in the connection. At 78% of the 0.5 psi HELB pressure, the far bolt is predicted to fail and is removed from the model. This is followed closely with failure of the near bolt at approximately 79% of the HELB pressure.

Figure 4-16 show the history of the maximum stresses in the two line welds. As shown, weld failure is predicted at loads between 66% and 86% of the 0.5 psi HELB pressure load.

Allowing for uncertainty in weld strength and bolt strength, the failure pressure limits for the bolts and welds can practically be treated as the same (i.e., failure of the girt will be initiated by either failure of the bolts or the welds). This is similar to the results discussed earlier for the girts between column lines D and F.

Failure of the welds will disconnect the girt at column line D and cause it to slide over the edge of the column flange under the HELB pressure. As the girt slides, bending and twisting increase leading to significant stresses and instability similar to the results of Model 3 described in Section 4.3.

Failure of both bolts causes the girt to react the HELB pressure loads in a cantilever mode of response (anchored at column line C connection). The calculation below demonstrates that the channel section at the fixed end of the cantilever would fail due to bending stresses about the channel strong axis. It is to be noted that the channel section is stronger than the connection angle and likely the bolts and welds. Therefore, the evaluation below demonstrates total failure of the girt at column C connection.

Cantilever span = (6'-4")+(6'-4.5")+(6'-4.5")+(4'-1")-2.5" = 275.5 inches

Cantilever moment = 42 lb/in x $(275.5 \text{ in})^2 / 2$ = 1,594 kip.in

Plastic section modulus of C10x15.3 about strong axis = 15.8 in^3

Moment capacity at ultimate strength = 15.8 in^3 x 78,400 psi = 1,239 kip.in

Thus, the cantilever moment is well above the failure limit, with the bolts failed at column line D, a plastic hinge would form at column line C connection[16] and the attached siding would fail.

[16] As explained earlier, in the cantilever mode of response, failure of column line C connection (i.e., bolts, angle, and/or welds) can potentially occur prior to formation of the plastic hinge in the channel. Complete failure of the panel supporting steel would occur irrespective of the failure mode at column line C being in the connection or in the channel.

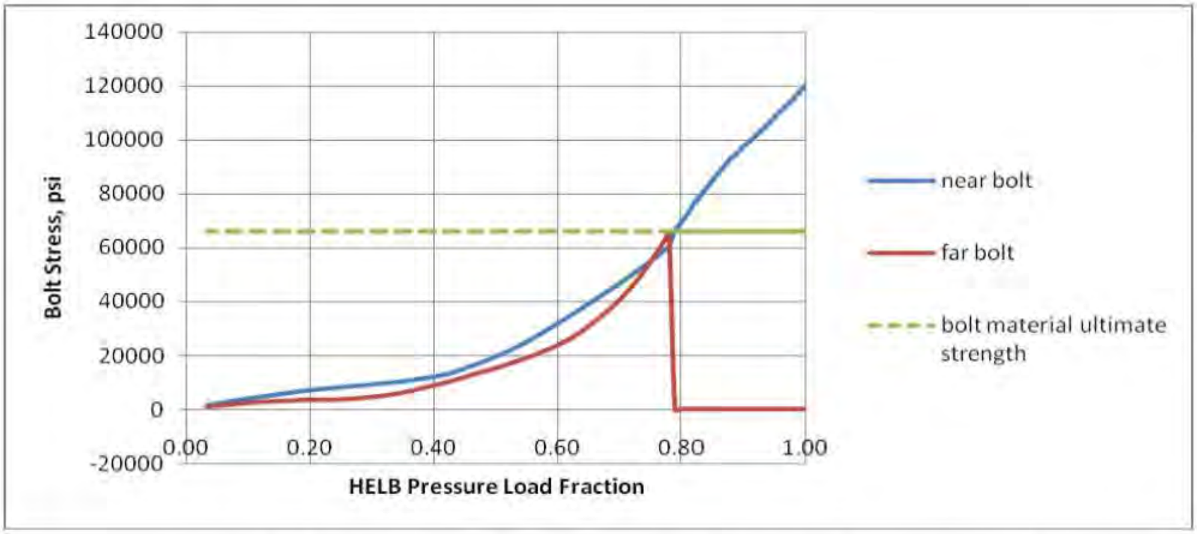

Figure 4-15: History of Maximum Bolt Stress at Column Line D (Horizontal axis represents the fraction of HELB pressure of 0.5 psi)

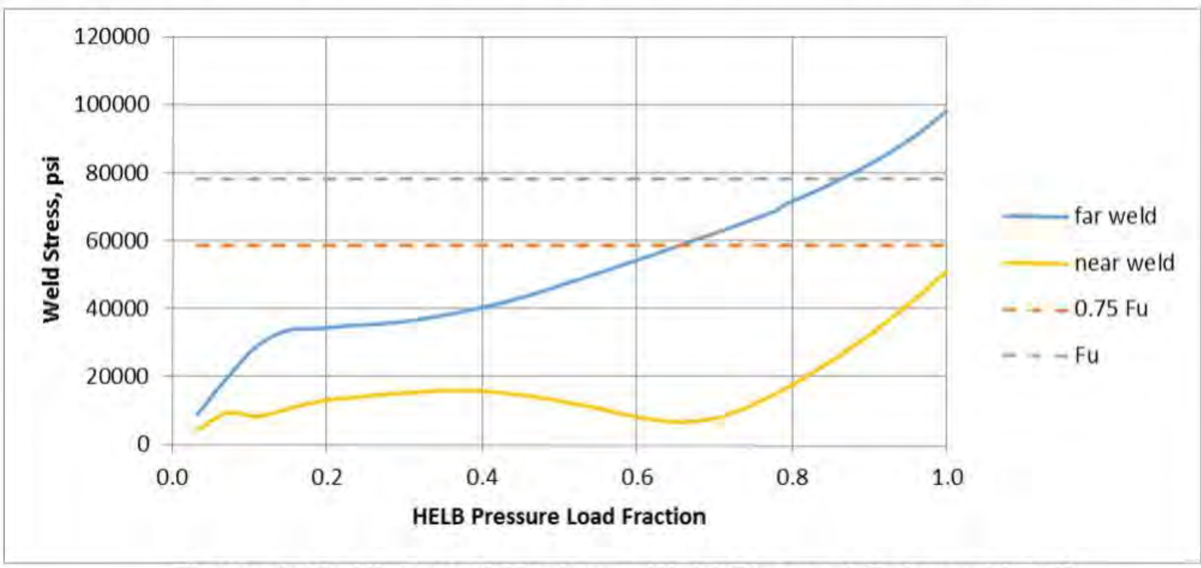

Figure 4-16: History of Maximum Weld Stress at Column Line D (Horizontal axis represents the fraction of HELB pressure of 0.5 psi)

4.5 Failure Mechanism

Based on the analysis results described in Section 4.0, significant deformation and failure is anticipated in the blowout panel steel support structure. The failure is anticipated to occur at a HELB pressure below 0.5 psi. Based on the results of the analysis, two possible failure mechanisms will take place and both can initiate at about the same HELB pressure load level. During an event, only one of the two mechanisms will occur, leading to failure.

1. Failure of the weld

   This failure mechanism begins with failure of the fillet weld farthest from the girt at about 0.25 psi. The other (near) fillet weld will follow at about 0.38 psi. The response of the girt following complete weld failure involves significant twisting of the channel and buckling in a lateral torsional buckling mode slightly below 0.5 psi (approximately 0.425 psi). This mechanism is illustrated schematically in Figure 4-17. Rotation of the channel also causes significant reduction in its load resistance due to reacting more of the pressure load by bending about its weak axis. With the failure of the girt angle weld to the TB column flange and rotation of the channel, the bending strains at mid-span increase significantly and reach failure limits, causing complete collapse of the channel.

2. Failure of the bolts

   This mechanism involves failure of the outer bolt at about 0.25 psi due to bending stresses that result from rotation of the channel. This would be rapidly followed by failure of the second (inner) bolt in a similar manner, which is subjected to rise in the bolt bending stress after failure of the outer bolt. Figure 4-18 provides an illustration of this mechanism. Once failure of the bolts occurs, the support steel channel becomes unsupported and blowout under the lateral pressure load.

The failure mechanism for girts between column lines C and D is similar to the above mechanism where failure initiates at the connection at column line D by bolt failure and/or weld failure. The connection at column line C is considered rugged. Post connection failure is similar to above except that for the case of bolt failure the girt reacts the pressure load as a cantilever and fails in bending at column line C as demonstrated in Figure 4-19.

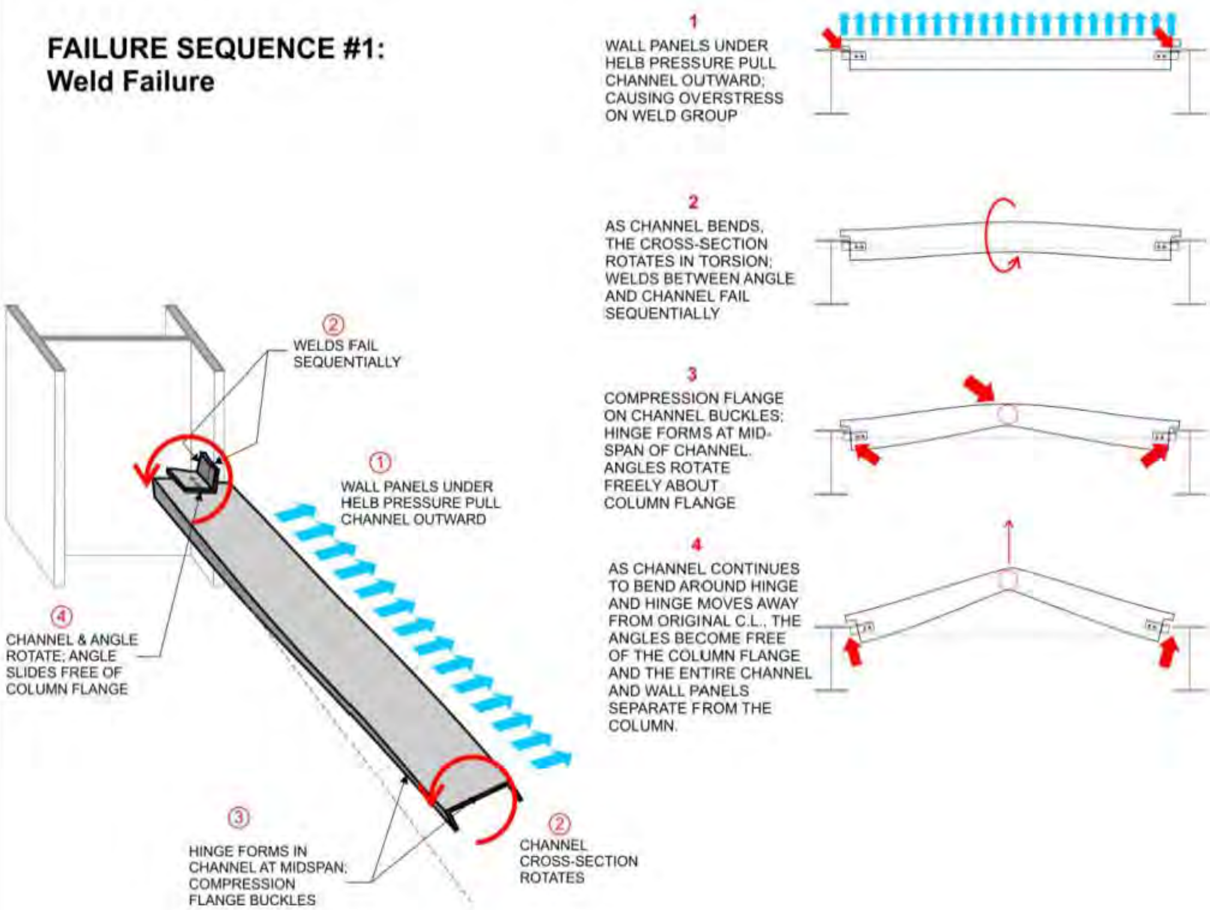

Figure 4-17: Sequence of Failure Mechanism 1 – "Failure of the Weld" Leading to Girt Failure and Structural Collapse

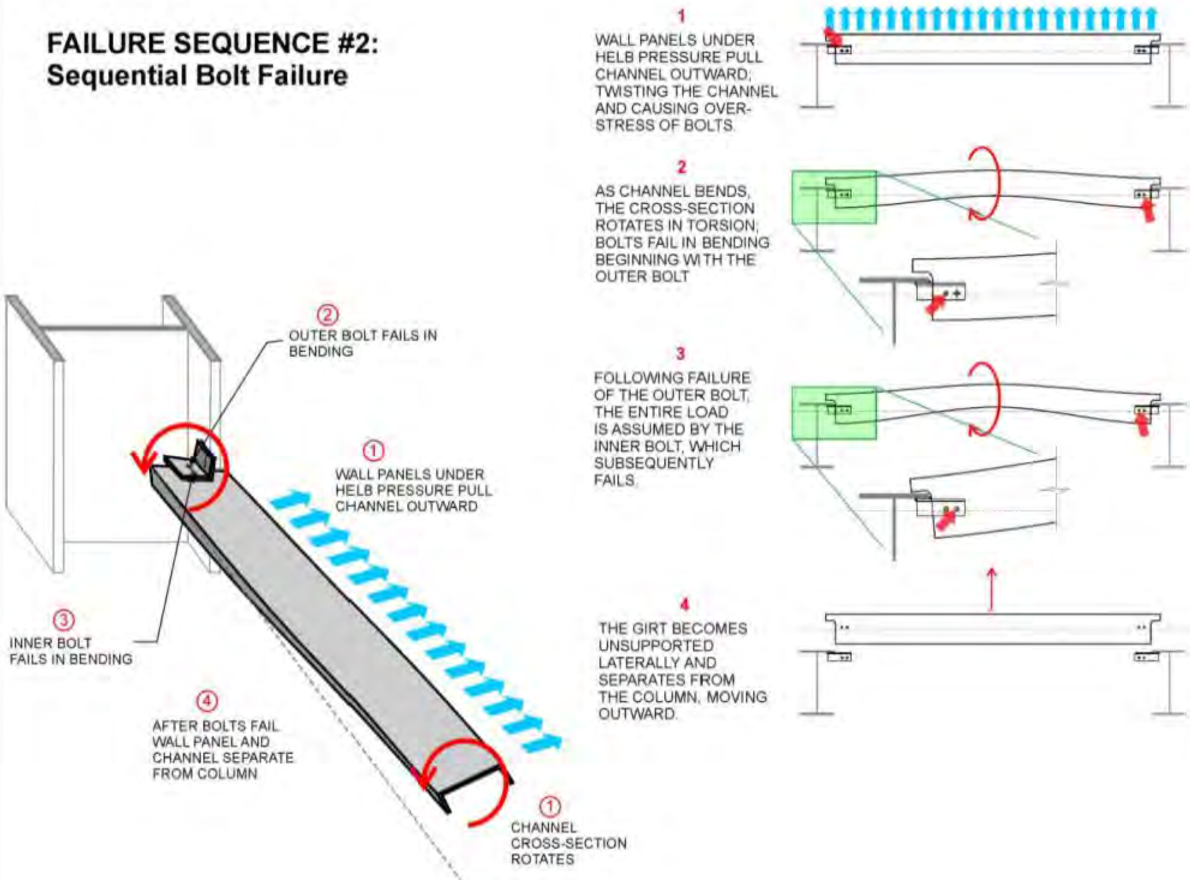

Figure 4-18: Sequence of Failure Mechanism 2 – "Failure of the Bolts" Leading to Loss of Structural Integrity

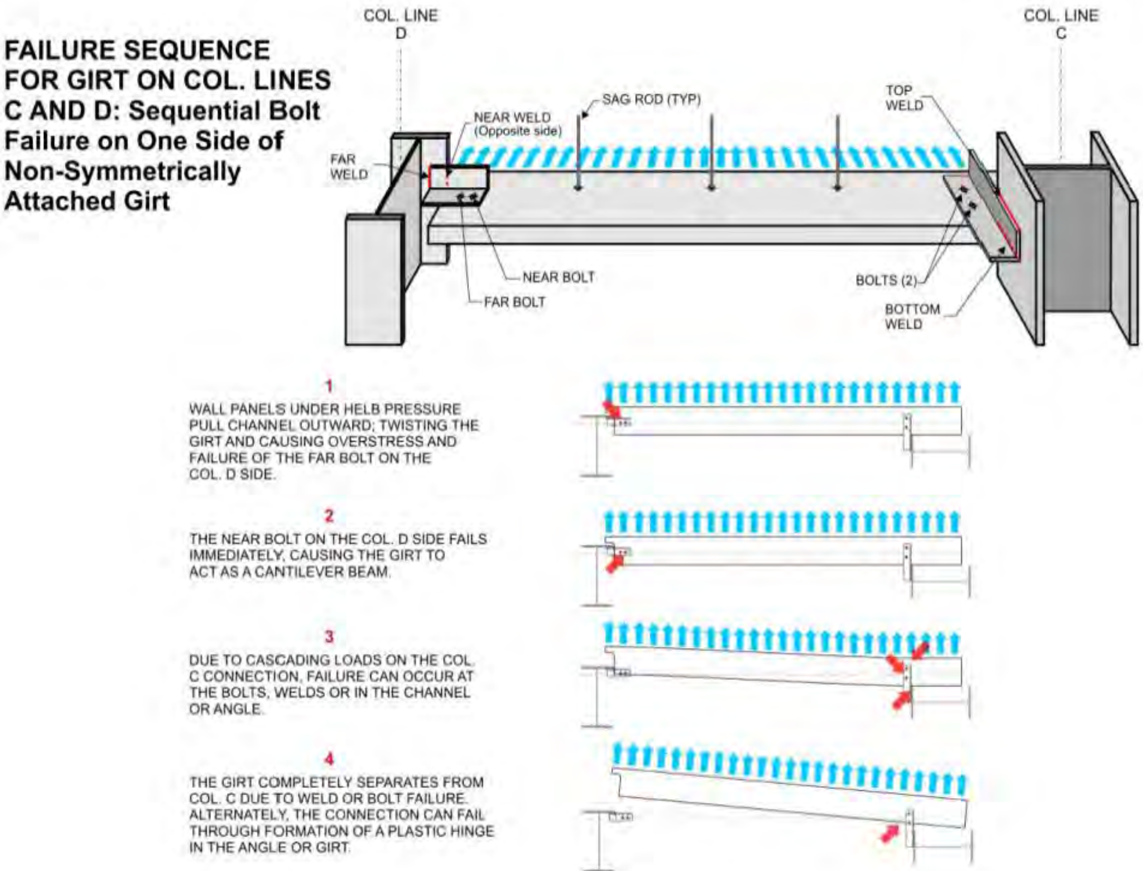

Figure 4-19: Sequence of Failure of Girts Between Column Lines C and D
(Shown is scenario of bolt failure at column line D. See Figure 4-17 for mechanism for failure initiated by weld failure at column line D)

4.6 Angle Size Effect

The finite element models considered the 4x3x3/8 angle installed in the North wall girt connections. The girt connections on the South wall use 5x3.5x3/8 angles. The larger angle is associated with larger weld length (3.5 inch vs 3 inch).

The use of the smaller angle in the analysis is justified because the weld stresses are concentrated at the lower end of the line welds as seen in the finite element strain contour plot in Figure 4-4. The progression of the weld failure beginning from the lower end will always have weld stress concentration at the lower end of the remaining weld ligament and this stress concentration is the driver that fails the weld. The strains in the angle itself are shown to be high but the angle is not predicted to fail as part of the failure mechanisms predicted herein. The larger angle would also have an un-conservative effect since it will initially draw more end moment at the support because it is stiffer than the slightly smaller evaluated angle. The stress concentration at the lower end of the weld would also be higher for the larger angle because the diaphragm action of the horizontal leg that causes the stress concentration. This would result in increased stress for the stiffer larger angle.

In conclusion, the larger angle is not considered to change the failure mechanism and failure pressures developed herein, considering the L4x3x3/8 used in the analytical models.

4.7 Siding Blowout Area

The finite element analysis was performed using a HELB pressure load that corresponds to a nominal 7 ft girt spacing. The analysis for girts between column lines D and F predicted failure could occur at a pressure load as high as 0.425 psi when failure is initiated at the welds (weld failure at approximately 0.38 psi, leading to lateral torsional buckling of the channel at 0.425 psi). This indicates that for a girt spacing smaller than 7 ft, a girt can still fail at or below 0.5 psi. The limiting girt spacing for failure to occur at 0.5 psi is obtained using a conservative lower failure pressure of 0.45 psi, which gives a limiting spacing of 79 inches[17] (6' - 7"). This limit spacing is the tributary spacing which is the average of the two adjacent spacings above and below the girt.

Based on the limiting spacings above for the wall area between column lines D and F, all girts on the North wall would fail at or below 0.5 psi. On the South wall, only two consecutive levels of girts bounded by spacing of 6'-10" and 7'-0" would fail at or below 0.5 psi.

Thus, the resulting blowout area between column lines D and F would be:

[17] The limiting spacing for 0.45 psi failure pressure is calculated as 0.425 psi x 7 ft / 0.45 psi = 6.611 ft = 6' – 7" = 79 inches

Area_blowout_North = 24' x 2 x (994'-932'-6.25") = 2,951 ft^2

Area_blowout_South = 24' x 2 x (7' x 2 + 6' +10") = 1,000 ft^2

For the area between column lines C and D, only the North wall is considered and the additional area in the South wall is not included. The area associated with panels between column lines C and D is:

Area_blowout_North = (22'+11.5") x (994'-932'-6.25") = 1,411 ft^2

Total blowout area = 5,362 ft^2

5.0 Summary & Conclusions

The steel girts supporting the blowout panels in the Turbine Building were analyzed to determine if the steel support system would fail due to HELB pressure of 0.5 psi. The analysis was based on the use of finite element analyses methods, with consideration of material and geometric nonlinearities. Material properties included the use of Certified Material Test Report (CMTR) properties of actual yield and ultimate stress, together with bolt strength derived from test data, such that accurate upper bound structural failure capacity could be simulated.

The analysis included girts located between column lines D and F on the North and South walls and girts between column lines C and D on the North wall of the operating floor. The analysis indicates that one or more of the following failure modes will be reached under application of the 0.5 psi HELB pressure:

1. Failure of the connection welds at pressures of approximately 0.38 psi. This leads to lateral torsional buckling of the girt channel at a pressure of approximately 0.425 psi. Once lateral torsional buckling of the girt channel occurs, the mid-span strain will increase significantly, the channel will be incapable of carrying load, and the support system is lost, and the panels will blowout.

2. Failure of the 3/4" bolts in bending. The elastic bending stress obtained at 0.25 psi HELB pressure after application of thread stress concentration factors exceeding the bolt material ultimate strength. Once failure of bolting at one or both ends of a girt occurs, the structural support system is lost and the panels will blowout. Failure in this scenario appears to occur at a lower pressure than "Scenario 1" above.

3. The girts between column lines C and D would fail similar to above by failure of the welds and/or bolts in the west end connection (column line D). The connection at column line C is considered rugged. Subsequent to a bolt failure at column line D, the structural system of the girt transforms to a cantilever fixed at column line C which results in immediate failure of the girt at the east connection. The support system for the siding is then lost leading to its blowout.

Since the above failure pressures are below 0.5 psi and correspond to girt spacing of 7 ft., girts at spacings down to 6'-7" would fail at pressures below 0.5 psi. Using a HELB pressure of 0.45 psi, it is predicted failure of the panel support system at this pressure will result in a loss (blowout) of total panel area of 5,362 ft^2.

6.0 References

1. NPPD Purchase Order 4200002638, Including Amendment 1.
2. NPPD Cooper Nuclear Station (CNS) Updated Safety Analysis Report (USAR) Amendment 25.
3. EDS Nuclear Inc., Report 01-0840-1115, Revision 0, "Cooper Nuclear Station - Environmental Effects Due to Pipe Rupture," October 1988.
4. AISC Manual of Steel Construction, Sixth Edition.
5. ANSYS References:
   a. ANSYS General Purpose Finite Element Analysis Software Code, Version 14, Ansys Inc.
   b. ANSYS General Purpose Finite Element Analysis Software Code, Version 14, ANSYS Inc., LPI Report No. V&V-ANSYS-14, Rev. 0, "Verification and Validation of ANSYS Software Program."
   c. LPI Quality Assurance Procedure No. 4.1, Revision 5, "Software Control".
6. R.E. Peterson, "Stress Concentration Factors," John Wiley & Sons, 1974.
7. Turbine Building Structure Steel Drawings:
   7a. Capitol Steel and Iron Contract 9150, sheet 133
   7b. Capitol Steel and Iron Contract 9150, sheet E101
   7c. Capitol Steel and Iron Contract 9150, sheet E106
   7d. Capitol Steel and Iron Contract 9150, sheet E107
   7e. Capitol Steel and Iron Contract 9150, sheet 158
   7f. Capitol Steel and Iron Contract 9150, sheet 156
   7g. Capitol Steel and Iron Contract 9150, sheet 157
   7h. Capitol Steel and Iron Contract 9150, sheet 159
8. Blodgett, O.W., "Design of Welded Structures," Cleveland, James F. Lincoln Arc Welding Foundation, 1966.
9. Shigley, J.E., Mechanical Engineering Design, McGraw-Hill Book Co., 1972 (2nd ed. unless otherwise indicated).
10. E. Oberg, et. al., Machinery's Handbook, Industrial Press (27th ed. unless otherwise indicated).
11. MATHCAD, Version 14.0, Parametric Technology Corp., 2007.
12. ASTM A-307 "Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength"
13. NUREG/CR-2137 "Realistic Seismic Design Margins of Pumps, Valves, and Piping," June 1981.

14. Inryco Job. N0. 49054 "Vacuum Load Test L10 Steel/IW21A Aluminum 1820 Gage Wall Panel Cooper Nuclear Power Station, Nebraska," Dated March 1973.
15. National Aerospace Standards Committee NASM1312-13, "Fastener Test Method 13 – Double Shear Test," 2013.
16. Baumeister T, et.al. "Mark's Standard Handbook for Mechanical Engineers", 8th Edition, McGraw-Hill.
17. AISC Shapes Database v13.1 Historic.xls
18. Email Correspondence K. Tom (– NPPD/CNS) to B. Elaidi (-LPI) "Hardness of Supplied Bolts", Dated 10/28/2015 (see Appendix A for Data).
19. Contract No. E-69-15, "Structural Steel for Turbine Generator and Reactor Buildings and Intake Structure," Revision 11, Dated 7/11/69.
20. NPPD Calculation NEDC 13-028, Revision 1, "Ultimate Internal Pressure of Turbine Building Blowout Panels and Metal Wall System."
21. Burns and Roe Inc. Document, "Computer Analysis of CNS Multi-Compartment Pressure History: Main Steam, Feedwater, and Extraction Line Breaks in Turbine Building,"10-25-73 [Media (1) 08317-1718, (2) 64158-1094].
22. Stearns-Roger Quality Assurance Mill Test Reports; Dated March 31,1970 "Turbine Generator Building."

APPENDIX A

Vendor Information and Miscellaneous Input

C Satir
w/a

DOCUMENT (SET) START

Stearns-Roger
CORPORATION

ENGINEERS • CONSTRUCTORS
ESTABLISHED 1885

660 BANNOCK STREET
PHONE 303-222-8484

P. O. BOX 5888, DENVER, COLORADO 80217
TWX 910-931-0453 TELEX 045-540

March 31, 1970

Capitol Steel and Iron Company
P. O. Box 26487
Oklahoma City, Oklahoma 73126

Attention: Mr. E. H. Wimsett

Gentlemen:

Stearns-Roger quality assurance has reviewed the mill test reports as forwarded by your letter of March 19, 1970. Following are Stearns-Roger's comments:

1. Intake Structure

a. The following are hereby returned "Not Approved". It is requested that the indicated deficiencies be corrected and the MTR resubmitted:

(1) United States Steel Corporation heat number 73T341. In copying the certification the column indicating results of the bend tests was cut off. Please submit a copy showing this column.

(2) Armco Steel Corporation heat number 13276, the first yield point reading was illegible. Please submit a legible copy.

(3) Jones and Laughlin Steel Company varies heats with 10 Ga, 11 Ga, 12 Ga, 14 Ga and 16 Ga sheet. The material specification (ASTM No.) is not indicated.

b. The following are hereby returned "Approved": Inland heats 35292, 18559, 24371, 36688, 36687; U. S. Steel heat 38T816; Northwestern heats 49709, 49720, 57240; Inland heats 18660, 27360; Northwestern heats 40143, 57715; Inland heat 26584; CF&I heats W6425, W6424; Northwestern heats 57801, 48929, 57675; Armco heats 18664, 65751, 93141, 66131, 31218, 24732, 31933, 11716, 31596; Northwestern heats 60124, 41605; CF&I heats W6598, W5634, W5636; Armco heats 93141, 93149, 93150, 66492; Northwestern heats 40250, 60073; U. S. Steel heat

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0903120221

Stearns-Roger

PAGE 2

Capitol Steel and Iron Company
Oklahoma City, Oklahoma
Attn: Mr. E. H. Wimsett
March 31, 1970

E64106; CF&I heat X-4554; Armco heat 12760; CF&I heat 16-104; Armco heats 26297, 64182; U. S. Steel heat 38P408; Armco heats 18605, 53601, 15649; CF&I heat W-7425; U. S. Steel heats D64290, 58T294; Armco heats 72055, 72109, 18688, 62175, 62159, 80170; CF&I heats 11-526, 11-524, 8-259; Northwestern heats 58039, 49753; CF&I heats X-6584, 17-482; U. S. Steel heat 36P671, 28R081, 37R354; Northwestern heats 56868, 56994, 57055, 57067, 57216, 54548, 57348, 56941, 57690; CF&I heats W-4920, 17-886, 9-954, X-5183, X-6425, W-1026; Armco heats 17145, 51314 13942, 14041; Bethlehem Steel heats 517M1729, 531M0273, 492M4761, 532M0368, 516M1615, 518M1760, 517M1819, 516M1760, 517M1819, 517M1733, 516M1763, 516M1772; U. S. Steel heats H81156, H80938; Northwestern heats 47601, 47645, 47805, 66132, 77025, 41858, 61116, 41937, 42110, 61307, 41733, 59318, 42061, 58224, 61250, U. S. Steel heat 37T636, CF&I heat 17-564; Armco heats 76598, 26672, 81023, 58053, 26835, 75962, 66656, 76730, 26824; U. S. Steel heat 33R406; Armco heat 80170; Northwestern heat 48878.

2. Turbine Generator Building

a. The following are hereby returned "Not Approved".

(1) Jones and Laughlin for various heats with 10 Ga, 11 Ga, 12 Ga, 14 Ga and 16 Ga sheet. The material specification (ASTM No.) is not indicated.

(2) Jones and Laughlin heat numbers 957360 and 95767, no material specification.

(3) U. S. Steel heat 30T742 - bend test results not visible on our copy.

b. The following are hereby returned "Approved": Northwestern heat 57936; U. S. Steel heats 29T505, 33T550, 27T445, 36T512; CF&I heats W-4920, X-6743, W-2366, W-5016, 17-157, X-4135; Northwestern heats 58047, 40527; Armco heats 12399, 12714, 31218, 24732, 31932, 31933, 23837, 31934, 12244, 19636, 24752, 11716, 11378, 31564, A270924, 12291, 23406, 31596, 89927, 31722, 12417, 31101, 66131; U. S. Steel heats 39P500, 36P368, 38P669, 30P370; Armco heats 18664, 65751, 93141; CF&I heats W-4920, 17-886, 9-954, X-5183, X-6425, W-1026; 17-883, X-1640, W-2467, X-4051; Armco

Stearns-Roger PAGE 3

Capitol Steel and Iron Company
Oklahoma City, Oklahoma
Attn: Mr. E. H. Wimsett
March 31, 1970

heats 76598, 26672, 12663, 72268; Northwestern heats 41605, 60861; CF&I heats X-4554, W-3836, W-3835, W-3828, X-5099; Armco heats 64561, 25196, 12515, 26297, 65201; Northwestern heats 57777, 57846, 40315, 57875, 40889, 41249, 60919, 60904, 48802, 48634, 56211; U. S. Steel heats H80669, J80499, K80119; Northwestern heats 77025, 41858, 61116, 41937, 42110, 61307, 41733, 59318, 42061, 58224, 42272; CF&I heat W-8651; Northwestern heat 48842; Armco heat 75086; Northwestern heat 60636; U. S. Steel heats 31P418, 29P474, 36P446; Armco heats 65371, 56545; Northwestern heat 61058; CF&I heats W-0636, W-0640, 11-545, W-6279; Northwestern heat 60636.

Very truly yours,

STEARNS-ROGER CORPORATION

T. S. Frost

T. S. Frost
Project Manager, QA

TSF:alb

cc: R. E. Reder
H. J. Dager
A. J. Divis
D. Teschner
I. Gabel w/1
W. E. Gubb w/1
ECR w/1
TSF w/1
Gen. File

09031 2093

TURB GEN BLDG

Armco Steel Corporation
7000 Roberts, Kansas City, Mo. 64125

09031 2090

REPORT OF CHEMICAL ANALYSES AND PHYSICAL TESTS

Date Jan. 9, 19[illegible]

Customer Capitol Steel & Iron Company Address Oklahoma City, Oklahoma

Shipped To Same Car No. PLE-17006

Armco Order No. K-3624 Customer Order No. 11208 Shipping Date 1/8/69

FORM K-2018 REV. 12/68

Heat No.	Description	YIELD Lbs. Per Sq. In.	TENSILE Lbs. Per Sq. In.	Elongation % 8"	Reduction of Area %	Bend Test	Car.	Mang.	Phos.	Sul.	Cu	Si	Mo.	Cr.	Ni.
	HOT ROLLED CARBON STEEL STRUCTURAL ANGLES ASTM A36-67														
12399	4 x 4 x 3/8	47330	67800	30.47		Ok	.25	.50	.010	.024					
12714	4 x 4 x 1/4	45500	67150	30.47		Ok	.23	.49	.010	.025					
31218	4 x 3-1/2 x 3/8	40430	64440	33.59		Ok	.18	.57	.010	.030					
24732	4 x 3-1/2 x 5/16	44130	66730	27.34		Ok	.21	.55	.010	.029					
[illegible]	4 x 3-1/2 x 1/4	[illegible]	[illegible]	[illegible]		Ok	.19	.49	.010	.025					
31933	4 x 3 x 3/8	43170	68100	31.25		Ok	.24	.52	.010	.025					
[illegible]	4 x 3 x 5/16	[illegible]	[illegible]	30.47		Ok	.21	.51	.010	.029					
31934	4 x 3 x 1/4	43200	64270	26.56		Ok	.19	.49	.010	.023					
12244	3-1/2 x 3-1/2 x 5/16	42200	64760	27.34		Ok	.21	.49	.010	.021					
19636	3-1/2 x 3 x 5/16	44460	67740	28.90		Ok	.21	.50	.010	.024					
24752	3-1/2 x 3 x 1/4	44830	64990	28.90		Ok	.19	.49	.010	.023					
11716	3 x 3 x 3/8	41700	65070	31.25		Ok	.21	.53	.010	.026					
11376	3 x 3 x 5/16	43550	66240	28.12		Ok	.22	.49	.010	.041					
31564	3 x 3 x 1/4	44220	66140	29.69		Ok	.18	.52	.010	.032					
A270924	3 x 2 x 3/8	43020	69080	30.48		Ok	.26	.52	.007	.023					
12291	3 x 2 x 1/4	46070	67320	30.47		Ok	.19	.53	.012	.029					
	HOT ROLLED CARBON STEEL ANGLES ASTM A36-67														
23406	2-1/2 x 2 x 3/16	43450	64470	29.69		Ok	.21	.48	.010	.029					
31596	2 x 2 x 1/4	45720	68100	30.47		Ok	.20	.56	.010	.039					
69927	2 x 2 x 3/16	NO TENSILE REQUIRED				Ok	.17	.80	.014	.028					
	HOT ROLLED CARBON STEEL STRUCTURAL CHANNELS ASTM A36-67														
31722	4" x 5.4#	44560	65720	25.00		Ok	.21	.50	.010	.025					
12417	3" x 6.0#	44330	68240	31.25		Ok	.22	.57	.010	.038					
	HOT ROLLED CARBON STEEL PLATE ASTM A36-67														
31101	2 x 1/4	42950	60880	34.38		Ok	.16	.64	.011	.085					

APPROVED
MAR 31 1970
Stearns-Roger

Subscribed and sworn to before me this 9th day of Jan. 19 69 Lois K. Motley My Commission expires Feb. 26, 19 72

THIS CERTIFIED TEST REPORT HAS BEEN DELIVERED TO A CONSIGNEE OF MATERIAL PURCHASED FROM ARMCO STEEL CORPORATION. TO AVOID THE POSSIBILITY OF ITS MISUSE, ON THE REDELIVERY OF THIS REPORT TO A THIRD PARTY IT MUST BE RE-

The Chemical, Physical, or Mechanical Tests reported above are correct as contained in the records of the corporation.

Signed R. S. Roslinger

FORM 2799 (COPY)

FORM 2138(R2)

CF&I STEEL CORPORATION

RECEIVED

INVOICE

CUSTOMER'S ORDER NO. AND DATE: 9999 8-20-68

CONTRACT NO. OCT 17 7 40 AM 1968

CF&I SALES ORDER NO. AND DATE: 763-16-597 8-21-68

750-203 CAPITOL STEEL & IRON COMPANY

183680 OKLAHOMA CITY, OKLA.

CAPITOL STEEL AND IRON COMPANY
P.O. BOX 26487
OKLAHOMA CITY, OKLAHOMA 73126

CAPITOL STEEL AND IRON COMPANY
OKLAHOMA CITY, OKLAHOMA

TERMS:
CASH DISCOUNT OF ___ %
BASED ON ___ % OF ___ WILL BE ALLOWED FOR PAYMENT ___ DAYS FROM DATE OF INVOICE
DUE NET ___ DAYS

NO DISCOUNT WILL BE ALLOWED ON ANY PART OF TRANSPORTATION CHARGES, OR ON TAXES AND ASSESSMENTS CHARGEABLE HEREUNDER.
CHECKS AND/OR DRAFTS ACCEPTED SUBJECT TO FINAL CASH PROCEEDS BEING RECEIVED.

MILL: X OTHER: | SHIPMENT: XXXX | DATE SHIPPED: 9-30-68 | CAR INITIAL & NO.: SP-340202

ITEM DESCRIPTION	ITEM NO.	UNIT PRICE	SHIPPED QUANTITY		SHIPPED POUNDS		AMOUNT
STRUCTURAL ANGLES ASTM A-36(21.9.4)			BDLS	BARS			
4 X 4 X 3/8 60 FT	1		1	(10)	5880		
HT. NO. W-4920							
3-1/2 X 2-1/2 X 3/8 40 FT	2		1	(20)	5760		
HT. NO. 17-886							
3-1/2 X 3 X 3/8 60 FT	3		2	(14)	6636		
HT. NO. 9-954							
4 X 3-1/2 X 3/8 60 FT	4		1	(10)	5460		
HT. NO. X-5183							
5 X 3-1/2 X 3/8 60 FT	5		1	(15)	9360		
HT. NO. X-6425, W-1026		1					

TEST REPORT

APPROVED
MAR 31 1970
Stearns-Roger

CF&I FORM 2876(R1) CERTIFICATE OF INSPECTION

TEST REPORT DATE OCT 15 1968

HEAT NO.	CHEMICAL ANALYSIS C.	MN.	PHOS.	SUL.			YIELD POINT	TENSILE STRENGTH	ELONG % IN 8"	RED. AREA %	BENDS
W-4920	21	53	018	019			41300	61800	28.0		ok
17-886	20	53	010	044			40000	65700	28.0		ok
9-954	23	58	016	038			37900	63800	28.0		ok
[illegible]	[illegible]	85	013	[illegible]			[illegible]	[illegible]	[illegible]		ok
X-6425	23	60	018	023			41500	63800	29.0		ok
W-1026	24	85	028	023			42700	78400	25.0		ok

Signed and sworn to before me this 15 day of OCT 1968 I CERTIFY THE ABOVE RESULTS TO BE CORRECT AS CONTAINED IN THE RECORDS OF THE CORPORATION.

[signature] Notary Public

[signature] QUALITY CONTROL DEPARTMENT

My Commission Expires Nov. 30, 1969

09003 2107

NORTHWESTERN STEEL AND WIRE COMPANY

STERLING, ILLINOIS 61081

METALLURGICAL DEPARTMENT - CERTIFIED MILL TEST REPORT

NEW NUMBER	SHIP DATE	B/L NO.	CUSTOMER P.O. NO.	METHOD	CAR OR TRUCK NUMBERS	SHIP LOC	[illegible]	CM	NOT	CORR	PAGE
57327	09-20-69	75566	14591 9-19	RAIL	LV 31236 CNW 48335	24	12		X	34	1

SHIP TO:
CAPITOL STEEL AND IRON CO
PO BOX 26487
1726 S AGNEW
OKLAHOMA CITY OKLA 73126

SOLD TO:
CAPITOL STEEL AND IRON CO
PO BOX 26487
1726 S AGNEW
OKLAHOMA CITY OKLA 73126

WE HEREBY CERTIFY THAT THE FOREGOING DATA IS A TRUE COPY OF THE MILL TEST REPORTS RESULTING FROM TESTS IN OUR LABORATORY.

NORTHWESTERN STEEL AND WIRE COMPANY

BY M.B. Middleton

THE FOLLOWING TESTS CONFORM TO THE REQUIREMENTS OF THE SPECIFICATIONS [illegible]

LINE NO.	SECTION	SPEC.	LENGTH	PIECES	WEIGHT	HEAT #	YIELD POINT PSI	TENSILE STRENGTH PSI	ELONG % IN 8"	BEND TEST	CAR	MN	PHOS	SUL
1	8 X 11,5	STR CHAN A-36	601	10	6900	77025	45000	66700	28,1	OK	25	53	020	030
2	7 X 9,8	STR CHAN A-36	601	7	4120	41858	44200	61500	28,9	OK	22	73	017	047
3	5 X 6,7	STR CHAN A-36	601	10	4020	61116	48100	66000	22,7	OK	18	66	019	031
4	4 X 5,4	STR CHAN A-36	601	13	4210	41937	45000	66000	25,0	OK	23	66	009	022
5	10 X 15,3	STR CHAN A-36	601	6	5510	42110	47200	73700	22,7	OK	23	55	005	020
6	15 X 33,9	STR CHAN A-36	601	2	4070	[illegible]	[illegible]	[illegible]	[illegible]	OK	[illegible]	[illegible]	[illegible]	[illegible]
7	8 X 31	WF BEAM A-36	601	3	5580	41733	45200	67700	27,3	OK	23	58	010	034
8	10 X 49	WF BEAM A-36	601	2	5880	59318	42300	64500	25,0	OK	23	60	013	030
9	10 X 39	WF BEAM A-36	601	3	7020	42061	45200	67800	25,6	OK	20	55	009	031
10	14 X 30	WF BEAM A-36	601	10	18000	58224	46700	67500	28,1	OK	22	63	010	031

Ralph A. [illegible] A NOTARY PUBLIC IN AND FOR THE COUNTY OF WHITESIDE IN THE STATE OF ILLINOIS DO HEREBY CERTIFY THAT THIS AFFIDAVIT WAS SUBSCRIBED AND SWORN TO BEFORE ME BY M.B. MIDDLETON A DULY AUTHORIZED AGENT OF NORTHWESTERN STEEL AND WIRE COMPANY. GIVEN UNDER MY HAND AND NOTARIAL SEAL THIS 7 DAY OF OCT. 19 69

Ralph A. [illegible]

MY COMMISSION EXPIRES 4-20-73

REMARKS:

APPROVED
MAR 31 1970
Stearns-Roger

Inryco Wall Systems Technical Data

Field-insulated Walls

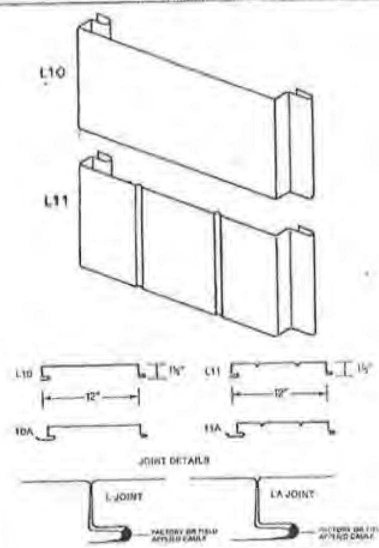
Inryco
an Inland Steel company

L10 Series

Liner Panel

Available in two surface textures, L10 and L11 panels are designed for use as interior liners in combination with any Inryco exterior panel or other suitable facing system in cavity wall construction or on interior partition systems. Standard as G90 steel, stucco embossed, with a two coat polyester (Duoprimer®) on both sides. Other metals and coatings available on a special inquiry basis.

The steel panel with caulked joints, performs as the vapor barrier. Acts with the exterior panel in supporting applied loads.



Accessories:

Available for use with the L10 Interior Wall Panel Series:

1. Batt or roll type insulation
2. Top, base, and corner trim
3. Sub-girt selection
4. Fastener type selection
5. Brake-formed trim for special conditions
6. Sealant-factory/or field-applied

Panel properties L10 series

Panel thickness ....1½"
Panel width ....12"
Joint configuration ....Continuous caulked interlocking side.
U-Factor ....135 (steel face panel, 1½" glass fiber insulation).
Base material ....G90 galvanized steel, 33,000 psi yield.

Finishes
Exposed surface ....Duoprimer
Inner surface ....Duoprimer

Availability

Panel Lengths

The following gages and lengths are recommended to facilitate erection, minimize handling damage and surface aberrations:

Profiles	Gages	Max. Length*
L10 or 10A	22	22'-0"
	20	24'-0"
	18	28'-0"
L11 or 11A	24	20'-0"
	22	22'-0"
	20	24'-0"
	18	28'-0"

*Lengths available to 38'-0" on special request.

Special Applications:

Assistance on special or unusual applications of LW panels are available from your Inryco Sales Engineer: Helpful information on fire-wall ratings, air and water infiltration criteria, corrosive exposures, missile-wall applications, unusual environmental conditions, special material or finish requirements, and many other pertinent subjects.

Performance Features

Thermal properties
U-value of .135 BTU/in./sq. ft./°F when corrected to a 15 mph wind condition.

Air infiltration
No air leakage, per square foot of surface, greater than .06 cfm at 1.56 psf air pressure differential.

Water infiltration
No uncontrolled water leakage at 4 psf* air pressure differential.

Acoustical properties
LW 13 available perforated for use as an acoustical liner with an NRC of .90 and an STC of 34.

*Data is based on interior panel tested with exterior panel in place.

Design Tables Maximum Spans — L10 Liner Panel Series

L10 Series Liner Panels

Span Type		24 Gage			22 Gage		
		S	D	T	S	D	T
Load		MAXIMUM SPAN LENGTHS IN FEET					
6.4 PSF*	f	13.23	13.23	14.79	15.55	15.55	17.39
15 PSF	f	8.64	8.64	9.66	10.16	10.16	11.35
	Δ	6.88	9.23	8.51	7.62	10.22	9.42
20 PSF	f	7.48	7.48	8.37	8.79	8.79	9.83
	Δ	6.26	8.38	7.73	6.93	9.28	8.56
25 PSF	f	6.69	6.69	7.48	7.87	7.87	8.79
	Δ	5.81	7.78	7.18	6.43	8.62	7.94
30 PSF	f	6.11	6.11	6.83	7.18	7.18	8.03
	Δ	5.46	7.32	6.75	6.05	8.11	7.48
35 PSF	f	5.66	5.66	6.32	6.65	6.65	7.43
	Δ	5.19	6.95	6.41	5.75	7.70	7.10
40 PSF	f	5.29	5.29	5.92	6.22	6.22	6.95
	Δ	4.96	6.66	6.14	5.50	7.37	6.79
45 PSF	f	4.99	4.99	5.58	5.86	5.86	6.55
	Δ	4.77	6.40	5.90	5.29	7.08	6.53
50 PSF	f	4.73	4.73	5.29	5.56	5.56	6.22
	Δ	4.61	6.18	5.70	5.10	6.84	6.31

L10 Series Liner Panels

Span Type		20 Gage			18 Gage		
		S	D	T	S	D	T
Load		MAXIMUM SPAN LENGTHS IN FEET					
6.4 PSF*	f	17.80	17.80	19.90	20.88	20.88	23.35
15 PSF	f	11.62	11.62	13.00	13.64	13.64	15.25
	Δ	8.44	11.31	10.42	9.58	12.85	11.84
20 PSF	f	10.07	10.07	11.26	11.81	11.81	13.21
	Δ	7.66	10.27	9.47	8.71	11.67	10.76
25 PSF	f	9.00	9.00	10.07	10.57	10.57	11.81
	Δ	7.11	9.54	8.79	8.08	10.83	9.99
30 PSF	f	8.22	8.22	9.19	9.65	9.65	10.78
	Δ	6.70	8.97	8.27	7.61	10.20	9.40
35 PSF	f	7.61	7.61	8.51	8.93	8.93	9.98
	Δ	6.36	8.63	7.86	7.22	9.68	8.93
40 PSF	f	7.12	7.12	7.96	8.35	8.35	9.34
	Δ	6.08	8.15	7.52	6.91	9.26	8.54
45 PSF	f	6.71	6.71	7.50	7.88	7.88	8.81
	Δ	5.85	7.84	7.23	6.64	8.91	8.21
50 PSF	f	6.37	6.37	7.12	7.47	7.47	8.35
	Δ	5.65	7.57	6.98	6.41	8.60	7.93

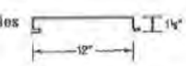
* For use when liner is to be temporarily left with no face panel: determined by using a 6.4 (50 mph) wind load, no deflection limit.

Form 22-32 10M 12/79

Explanatory Notes for Design Tables

1. Panel spanning conditions
   S — Simple Span; D — Double Span; T — Triple Span
2. Numbers in the tables indicate distance between adjacent structural supports (girts).
3. Span length limitation factors
   f = Stress factor limitation, using (0.6 (Fy)) as design stress, increased 33% for wind loading.
   Δ = L/180 as the maximum allowable deflection (For L/120, use: ((L Table) x (1.145)))
4. Static load in relation to wind velocity:
   PSF = (0.00256) (MPH)^2
5. Shaded areas indicate that if the panel were to be used at those span lengths and number of spans, the panel would exceed max. length recommended.

L10 Series



L10 Engineering Properties**

Nominal Gage	Thickness (mm.)	Weight (lbs./ft²)	+S in.³/ft.	−S in.³/ft.	+I in.⁴/ft.	−I in.⁴/ft.
24	.60	1.44	.063	.097	.056	.115
22	.74	1.78	.087	.119	.076	.141
20	.90	2.16	.114	.144	.103	.171
18	1.17	2.81	.157	.186	.151	.223

**Section properties and load carrying capacity of L10 and L11 are identical. (Positive designates top surface of panel in compression.) For carrying capacity with face panel see appropriate face panel data sheet.

Inryco
an Inland Steel company

INRYCO, Inc. (General Offices, Melrose Park, Illinois)
BUILDING PANELS DIVISION
P. O. Box 393, Milwaukee, Wisconsin 53201
Phone 414/383-4030

Printed in U.S.A.

CNS Hardness Testing of Bolts Sample [18]:

Bolt #1	83.2	1st	Discarded due to irregular strike
Shank	81.8	2nd	
	82.2	3rd	
	80.4	4th	Discarded due to irregular strike
	81.9	5th	
	246	**Total**	
	81.96667	**Average**	
	82	HRB value	
	75	Approximate Tensile Strength (ksi)	Interpolated value from LTI Rockwell B Hardness Conversion Chart
	76	Approximate Tensile Strength (ksi)	Interpolated value from "Tensile Strength to Hardness Conversion Chart"

Bolt #2	82.8	1st	
Shank	83.1	2nd	
	82.3	3rd	
	248	**Total**	
	82.73333	**Average**	
	82.7	HRB value	
	76.4	Approximate Tensile Strength (ksi)	Interpolated value from LTI Rockwell B Hardness Conversion Chart
	77.4	Approximate Tensile Strength (ksi)	Interpolated value from "Tensile Strength to Hardness Conversion Chart"

Bolt #3	81.2	1st	
Shank	79.9	2nd	
	80	3rd	
	241	**Total**	
	80.36667	**Average**	
	80.4	HRB value	
	72.8	Approximate Tensile Strength (ksi)	Interpolated value from LTI Rockwell B Hardness Conversion Chart
	74.4	Approximate Tensile Strength (ksi)	Interpolated value from "Tensile Strength to Hardness Conversion Chart"

Bolt #4	83.4	1st	
Shank	80.5	2nd	
	82.7	3rd	
	247	**Total**	
	82.2	**Average**	
	82.2	HRB value	
	75.4	Approximate Tensile Strength (ksi)	Interpolated value from LTI Rockwell B Hardness Conversion Chart
	76.4	Approximate Tensile Strength (ksi)	Interpolated value from "Tensile Strength to Hardness Conversion Chart"

Bolt #5	80.1	1st	
Shank	81.3	2nd	
	78.1	3rd	Discarded due to irregular strike
	80.9	4th	
	242	**Total**	
	80.76667	**Average**	
	80.8	HRB value	
	73.6	Approximate Tensile Strength (ksi)	Interpolated value from LTI Rockwell B Hardness Conversion Chart
	74.8	Approximate Tensile Strength (ksi)	Interpolated value from "Tensile Strength to Hardness Conversion Chart"

Bolt #6	83.9	1st	
Shank	78.2	2nd	Discarded due to irregular strike
	84.8	3rd	
	83.1	4th	
	252	**Total**	
	83.93333	**Average**	
	83.9	HRB value	
	77.9	Approximate Tensile Strength (ksi)	Interpolated value from LTI Rockwell B Hardness Conversion Chart
	79.8	Approximate Tensile Strength (ksi)	Interpolated value from "Tensile Strength to Hardness Conversion Chart"

Bolt #7	81.3	1st	
Shank	82.7	2nd	
	82.6	3rd	
	82.1	4th	
	329	**Total**	
	82.175	**Average**	
	82.2	HRB value	
	75.4	Approximate Tensile Strength (ksi)	Interpolated value from LTI Rockwell B Hardness Conversion Chart
	76.4	Approximate Tensile Strength (ksi)	Interpolated value from "Tensile Strength to Hardness Conversion Chart"

Appendix B
Analysis Parameters and Hand Calculations

This Appendix documents the geometric properties of the girt channel, angle, bolt, and welds; and deadweight and HELB pressure loading. Section parameters for the girt C10 and connection steel angle are obtained from [4]. These data provide input into the finite element analysis.

B.1 Input Parameters of C10x15.3

$A_{c10} := 4.47 \cdot in^2$	cross section area
$b_{f\_c10} := 2.625 \cdot in$	flange width
$t_{f\_c10} := \frac{7}{16} \cdot in$	average flange thickness
$d_{c10} := 8.125 \cdot in + 2 \cdot \frac{15}{16} \cdot in$	depth
$d_{c10} = 10 \cdot in$	
$t_{w\_c10} := 0.25 \cdot in$	web thickness
$e_{c10} := 0.796 \cdot in$	shear center eccentricity
$x_{c10} := 0.64 \cdot in$	geometric center eccentricity
$I_{xx\_c10} := 66.9 \cdot in^4$	area moment of inertia about strong axis
$S_{xx\_c10} := 13.4 \cdot in^3$	area bending modulus about strong axis
$I_{yy\_c10} := 2.3 \cdot in^4$	area moment of inertia about weak axis
$S_{yy\_c10} := 1.16 \cdot in^3$	area bending modulus about weak axis
$J_{c10} := 0.21 \cdot in^4$	torsional constant
$wt_{c10} := 15.3 \cdot \frac{lbf}{ft}$	selfweight

B.2 Input Parameters of L 4x3 x3/8 (North Wall)

The angle size used is for the North Wall girt connections. See discussion within the body of the report for effects of the larger angle on the South Wall connections.

$A_{a4x3} := 2.48 \cdot in^2$ — cross section area

$S_{xx\_a4x3} := 1.46 \cdot in^3$ — bending modulus about strong axis

$Z_{xx\_a4x3} := 2.64 \cdot in^3$ — plastic bending modulus about strong axis

$S_{yy\_a4x3} := 0.866 \cdot in^3$ — bending modulus about weak axis

$Z_{yy\_a4x3} := 1.56 \cdot in^3$ — plastic bending modulus about weak axis

$J_{a4x3} := 0.116 \cdot in^4$ — torsional constant

B.3 Other Geometric Parameters

$d_{bolt\_hole} := \frac{13}{16} \cdot in$ — bolt hole diameter (1/16" larger than bolt nominal size per [7])

$span_{girt} := 23 \cdot ft + 11 \cdot in - 2 \cdot (6 \cdot in + 3 \cdot in)$ — effective span of the girt between two inside bolts using the fabrication length of girt ID 133C.

$span_{girt} = 22.42 \cdot ft$

$girt\_spacing := 7 \cdot ft$ — nominal vertical spacing of girts. The north wall has 4 equal spaces of 7ft. The south wall has 2 adjacent spaces of 7 ft. The spacing determines the tributary pressure load acting on each girt.

$D_n := \frac{3}{4} \cdot in$ — nominal diameter of bolt

$A_t := 0.334 \cdot in^2$ — tensile area of bolts

$A_s := 0.302 \cdot in^2$ — shear area of bolts

B.4 Steel Properties

$E := 29 \cdot 10^6 \cdot psi$ — elastic modulus

$\nu := 0.3$ — Poisson's Ratio

$G := \frac{E}{2 \cdot (1 + \nu)}$ — Shear modulus

B.5 Deadweight and HELB Pressure Loadings

Deadweight includes the girt channel weight and the blowout panel weight.

$$Wt_{pl} := \left(1.65 \cdot \frac{lbf}{ft^3} \cdot 1.5 \cdot in\right) + 2 \cdot 2.81 \cdot \frac{lbf}{ft^2}$$

panel weight based on assumed insulation density and thickness and double 18 gage lining sheet obtained from vendor data sheet (Attachment A)

$$Wt_{pl} = 5.83 \cdot \frac{lbf}{ft^2} \quad \text{use} \quad Wt_{pl} := 6 \cdot \frac{lbf}{ft^2}$$

$$wt_{pl\_girt} := Wt_{pl} \cdot girt\_spacing$$

panel weight distribution on girt

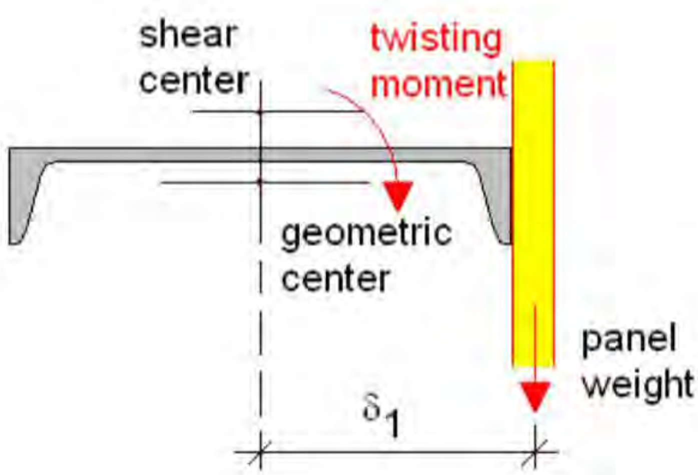
$$wt_{pl\_girt} = 3.5 \cdot \frac{lbf}{in}$$

The girt finite element (FE) beam model is located at the geometric center of the C10. Thus all weight/force eccentricities are calculated with respect to the geometric center of C10. The panel weight is eccentric with respect to the geometric center of the girt channel as shown in the sketch below. This creates twisting moment that is computed per FE node as follows:

$$M_{Wtp} := -wt_{pl\_girt} \cdot \left(\frac{d_{c10}}{2} + \frac{1.5 \cdot in}{2}\right) \cdot 0.5 \cdot in$$

$$M_{Wtp} = -10.06 \cdot lbf \cdot in$$

panel weight torsional load distributed to BEAM189 nodes using node spacing of 0.5 inch. This twisting moment is applied in the FE at each nodal point along the girt.



The steel structure includes sag rods that support the weight of the panels. Since the twisting moment of the panel weight opposes the twisting moment of the HELB pressure calculated below, including the panel weight is conservative and accounts for possible loosening in the sag rods.

$$\mu_{girt} := wt_{c10} + wt_{pl\_girt}$$

total deadweight distributed along girt

HELB pressure load is transferred to the girt at midpoint of the channel flange. This force is thus eccentric with respect to the geometric center of the channel will therefore be associated with a twisting moment distributed along the channel length and defined per FE node.

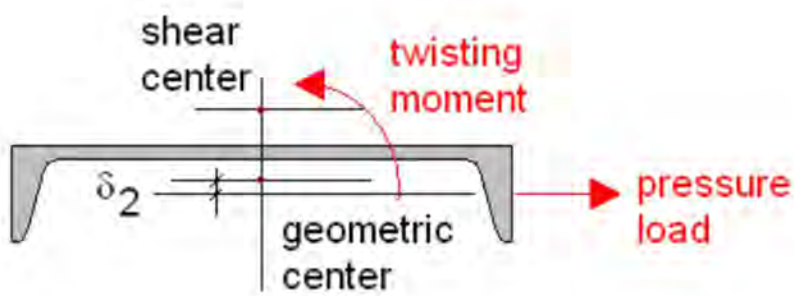
$Pr := 0.5 \cdot psi$ HELB pressure on siding

$w_{Pr\_girt} := Pr \cdot girt\_spacing$ HELB pressure load distribution on girt

$w_{Pr\_girt} = 42 \cdot \frac{lbf}{in}$

$M_{Pr} := w_{Pr\_girt} \cdot \left(\frac{b_{f\_c10}}{2} - x_{c10}\right) \cdot 0.5 \cdot in$ HELB pressure torsional load distributed to BEAM189 nodes using node spacing of 0.5 inch. This twisting moment is applied in the FE at each nodal point along the girt.

$M_{Pr} = 14.12 \cdot lbf \cdot in$



APPENDIX C

Bolt Testing

A sample of the girt angle bolts obtained from stock at CNS was submitted to LPI[18] for testing in double shear at LPI laboratories in NYC. Hardness testing of the bolt sample was conducted by CNS [18] and the results are included in Appendix A. The testing was performed in accordance with [15] that describes the test fixture and testing procedure. A fixture was fabricated in the laboratory. Some of the dimensions of the fixture specified in [15] were modified to ensure appropriate fit-up. This modification did not affect the manner the bolts were loaded, tested and results derived. The fixture was heated and oil quenched to achieve hardness significantly higher than that of the bolts, to ensure failure in the bolts was not influences by deformation in the test fixture.

Testing was conducted on 11/19 and 11/20/2015. The test sample included 7 bolts that were all tested. The rate of loading was significantly less than the rate specified in [15] thus insuring that the obtained results do not include any increase in strength that would be associated with dynamic rate-of-loading.

The testing was conducted on a 120 kip Baldwin Test Equipment made by Sensotec and shown in Figure C-1. The bolts were loaded until complete failure of the bolts occurred. Figure C-2 shows testing of bolt # 1 and Figure C-3 shows the broken bolt after the test (as an example). All bolts showed similar failure.

The failure load, based on the double shear test performed) was divided by 2 to obtain the failure load for single shear, consistent with the arrangement of the evaluated connection). Considering that shear failure for carbon steel is at 0.75 of ultimate strength [10], the ultimate strength of the bolt material was calculated using the root area of 0.302 in^2. The results are summarized in Table C-1.

[18] Refer to Visual Exam – sheet C-6

Table C-1: Bolt Test Results

Bolt Number	Peak Double Shear Force, kips	Single Shear Failure Load, kips	Derived Ultimate Strength of Bolt Material, Ksi
1	31.726	15.863	70
2	28.425	14.213	63
3	29.858	14.929	66
4	28.267	14.134	62
5	28.877	14.439	64
6(1)	33.860	16.930	75
7	30.432	15.216	67
Average (excluding minimum and maximum values – 5 bolts)			66

Notes:

1. Results for bolt #6 are presented for information. The results are biased upward due to binding in the test fixture that caused the measured load to be higher than the actual force in the bolt.
2. The derived ultimate strength is based on bolt root area of 0.302 $inch^2$ and shear failure stress of 0.75 of the ultimate strength for carbon steel [10]

Figure C-1: General View of Load Frame and Test Fixture

Figure C-2: Testing of Bolt #1
(as an example)

Figure C-3: Broken Bolt After Testing
(Bolt # 1 as an example)

VISUAL EXAMINATION

Exam. Instruction

1. Ensure you are qualified to perform this examination. Verify qualifications at www.lpicert.com.
2. Ensure you have read and understand any project specific Procedure(s) and/or Instruction(s) that may apply to this examination.
3. Locate, read and understand examination protocol as specified in industry Standard(s) applicable for this examination (for example ASTM or NA), identify in Examination Standard Box.
4. Ensure ALL Test & Measuring Equipment (T&ME) used for this examination is calibrated. Record ALL T&ME serial numbers and calibration information on this form. Identify if calibration is per use.
5. If test temperature is other than room temperature, identify in Notes section.

Check Box ✓ when complete ☑

Project No:	A15406	Client/Facility:	NPPD – COOPER NUCLEAR	
Project:	TB STRUCT. EVAL	Test/Exam Standard(s):	VISUAL	☑

Sketches / Data / Information

7 Bolts RECEIVED FROM CNS FOR SHEAR TESTING WERE RECEIVED IN LPI AMESBURY MA OFFICE. BOLTS WERE VISUALLY INSPECTED THEN SENT TO LPI NYC OFFICE FOR SHEAR TESTING. ALL BOLTS MARKED ON HEADS AS FOLLOWS

WAS 1 THRU 7 MARKED IN "SHARPIE" PEN.

BOLTS DIA MEASURED AS FOLLOWS:

BOLT	1	2	3	4	5	6	7
DIA (IN)	0.74	0.745	0.7496	0.75	0.74	0.739	0.751

☑

Notes:

ALL BOLTS NEW & IN GOOD CONDITION

☑

TEST EQUIPMENT DOCUMENTATION & CERTIFICATION

Instrument Used:	Inst. Serial No.	Cal. Due Date
MITUTOYO MICROMETER	0506247	2/12/2016

☑

Ensure all instructions have been met, form complete, T&ME information entered, Complete Check Boxes

Performed By:	Date:	Reviewed By:	Date:
[signature]	11/18/2015	[signature]	11/18/2015

APPENDIX D

Instrument Calibration Records

See Document Instrument Record on page 5 for list of instruments

1. LPI Micrometer/Dial Gauge/Caliper Calibration Record
2. Instron Certificate of Calibration

LPI MICROMETER/DIAL GAUGE/CALIPER CALIBRATION RECORD

Serial No.: 05062457 Instrument Location: Amesbury

Manufacturer: Mitutoyo Calibrated by: Nicholas Firicano Date: 2/12/2015

Calibration Range: 0-1" Reviewed by: Jared Russell Date: 2/12/2015

Range	Standard No.	Standard Length (in.)	Reading (in.)	Error*	After Adjustment Reading (in.)	After Adjustment Error
Minimum Range	NA	0	0.00000	0.00000	-	-
Low Range	C0858	0.200	0.20005	0.00005	-	-
Mid Range	C2306	0.500	0.50005	0.00005	-	-
High Range	C2306, C1812	0.800	0.80005	0.00005	-	-
Full Range	C2802	1.000	1.00005	0.00005	-	-

Calibration Due Date: 2/12/16

Form: LPI-12.1-FIG-6-1-Rev-5

*Reading errors are within the measurement accuracy of the caliper (+/-0.00005")

Equipment used for calibration:
Gage Blocks from Fowler Gage Block Set, Serial No. 23130:
0.2" Block, Serial No. C0858
0.3" Block, Serial No. C1812
0.5" Block, Serial No. C2306
1.0" Block, Serial No. C2802

CERTIFICATE OF CALIBRATION

ISSUED BY: INSTRON CALIBRATION LABORATORY

DATE OF ISSUE: 11-Mar-15 CERTIFICATE NUMBER: 445031115102251

NVLAP Lab Code 200301-0

Instron
825 University Avenue
Norwood, MA 02062-2643
Telephone: (800) 473-7838
Fax: (781) 575-5750
Email: service_requests@instron.com

APPROVED SIGNATORY

Brian Leary

Type of Calibration: Force
Relevant Standard: ASTM E4-14
Date of Calibration: 11-Mar-15

Customer Requested Due Date: 11-Mar-16

Customer

Name: LPI Inc
Address: 304 Hudson St
New York, NY 10013
PAli@luciuspitkin.com
P.O./Contract No.: PO#4281
Contact: Pats Ali
Customer Asset No.: CONTRACT#CALP04528_4

Machine

Manufacturer: SATEC
Serial Number: 372005
System ID: R55120BTEC372005
Range Type: Single

Transducer

Manufacturer: SATEC
Transducer ID: 372005
Capacity: 120000 lbf
Type: Compression

Classification

1. Digital Readout - PASSED**

Certification Statement

This certifies that the forces verified with machine indicator(s) (listed above) that passed are WITHIN ±1% accuracy, 1% repeatability, and zero return tolerance.
All machine indicators were verified on-site at customer location by Instron in accordance with ASTM E4.
The certification is based on runs 1 and 2 only. A third run is taken to satisfy uncertainty requirements according to ISO 17025 specifications.
The verification and equipment used conform to a controlled Quality Assurance program which meets the specifications outlined in ANSI/NCSL Z540-1, ISO 10012, ISO 9001:2008 and ISO/IEC 17025:2005.

**** within ±0.5% accuracy and 0.5% repeatability.**

Method

The testing machine was verified in the 'as found' condition with no adjustments carried out.

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
445031115102251

Page 2 of 4 pages

Summary of Results

Temperature at start of verification: 74.10 °F.

Indicator 1. - Digital Readout (lbf)

Range Full Scale (%)	Tested Force Range (lbf)	Mode	ASTM E4 Max Error (%)	ASTM E4 Max Repeat Error (%)	Zero Return	Resolution (lbf)	ASTM E4 Lower Limit (lbf)
100	-1234.15 to -119753	C	-0.30	0.11	Pass	1	200

Temperature at end of verification: 74.70 °F.

Data Point Summary - Indicator 1. - Digital Readout (lbf)

COMPRESSION

% of Range	Run 1 Error (%)	Run 2 Error (%)	Run 3 Error (%)	ASTM E4 Repeat Error (%)	Relative Uncertainty* (%)	Uncertainty of Measurement* (± lbf)
100% Range (Full Scale: -119753 lbf)						
1	-0.17	-0.18	-0.03	0.01	0.17	2.1
2	-0.11	-0.10	-0.14	0.01	0.13	3.2
4	-0.08	-0.07	-0.11	0.01	0.13	6.3
7	-0.30	-0.19	-0.16	0.11	0.15	13
10	-0.26	-0.24	-0.23	0.02	0.13	16
20	-0.25	-0.25	-0.25	0.00	0.13	31
40	-0.20	-0.10	-0.11	0.10	0.14	69
70	-0.03	-0.08	-0.01	0.05	0.13	113
100	0.04	0.03	0.04	0.01	0.13	153

** The reported expanded uncertainty is based on a standard uncertainty multiplied by a coverage factor k = 2, providing a level of confidence of approximately 95%.*

Data - Indicator 1. - Digital Readout (lbf)

COMPRESSION

% of Range	Run 1 Indicated (lbf)	Run 1 Applied (lbf)	Run 2 Indicated (lbf)	Run 2 Applied (lbf)	Run 3 Indicated (lbf)	Run 3 Applied (lbf)
100% Range (Full Scale: -119753 lbf)						
0 Return	1		0		0	
1	-1232	-1234.15	-1246	-1248.2	-1220	-1220.4
2	-2400	-2402.75	-2430	-2432.4	-2400	-2403.35
4	-4808	-4811.85	-4820	-4823.25	-4800	-4805.05
7	-8400	-8424.96	-8400	-8416	-8400	-8413.44
10	-12000	-12030.72	-12000	-12029.44	-12000	-12027.52
20	-24003	-24062.08	-24000	-24060.16	-24020	-24079.36
40	-48000	-48094.08	-48000	-48049.92	-48000	-48053.12
70	-84000	-84026.88	-84000	-84064	-84000	-84011.52
100	-119800	-119752.96	-119200	-119158.4	-119500	-119457.92

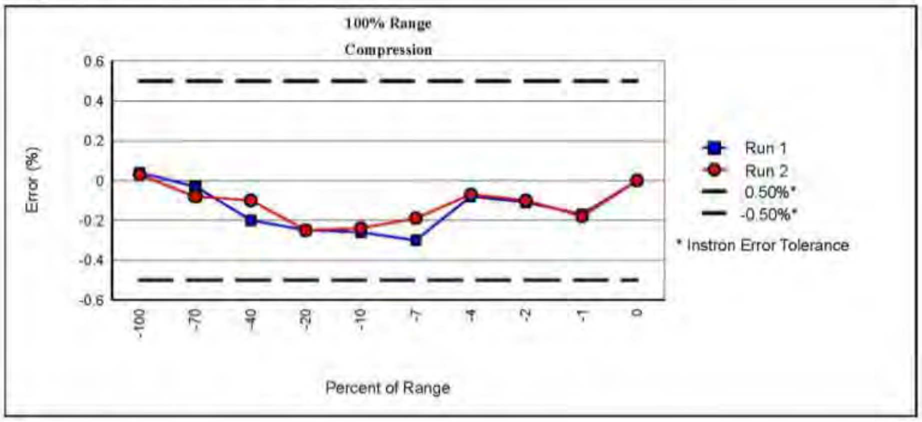
CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER: 445031115102251

The Return to Zero tolerance is ± the indicator resolution, 0.1% of the maximum force verified in the range, or 1% of the lowest force verified in the range, whichever is greater.

Graphical Data - Indicator 1. - Digital Readout (lbf)



Verification Equipment

Make/Model	Serial Number	Description	Calibration Agency	Capacity	Cal Date	Cal Due
Extech 445580	1036629	temp. indicator	Masy Systems Inc.	NA	22-Aug-14	22-Aug-16
Flintec 10KFC7	198916	load cell	Instron	12000 lbf	11-Apr-14	11-Apr-16
Interface 9840	93029	force indicator	Instron	NA	06-Jan-15	06-Jan-16
Tovey 112637A	112637A	load cell	Instron	142000 lbf	28-May-14	28-May-15

Verification Equipment Usage

Range Full Scale (%)	Mode	Standard Serial Number	Percent(s) of Range	Lower Limit for Standard (lbf)	Accuracy (+/-)
100	C	198916	1/ 2/ 4	Class A1: 200	0.1% of reading
		112637A	7/ 10/ 20/ 40/ 70/ 100	Class A1: 5000	0.1% of reading
All	C	1036629	All	NA	2 °F

CERTIFICATE OF CALIBRATION

NVLAP ACCREDITED CALIBRATION LABORATORY No. 200301-0

CERTIFICATE NUMBER:
445031115102251

Page 4 of 4 pages

Instron standards are traceable to the SI (The International System of Units) through standards maintained by the National Institute of Standards and Technology (NIST) or other internationally recognized National Metrology Institutes (NMIs).

The standard Class A lower limit is used for systems with an accuracy of +/-1.0% and the standard Class A1 lower limit is used for systems with an accuracy of +/-0.5%.

The accuracy of the force indicator used with elastic devices is incorporated into the devices stated accuracy.

Standard forces have been temperature compensated as necessary.

The accuracy of the verification equipment used was equal to or better than the accuracy indicated in the table above.

Comments

Verified by: Brian Leary
Field Service Engineer

NOTE: Clause 19 of ASTM E4 states; It is recommended that testing machines be verified annually or more frequently if required. In no case shall the time interval between verifications exceed 18 months (except for machines in which long term test runs beyond the 18 month period). Testing machines shall be verified immediately after repairs that may in any way affect the operation of the weighing system or values displayed. Verification is required immediately after a testing machine is relocated and where there is a reason to doubt the accuracy of the force indicating system, regardless of the time interval since the last verification.

LPI, Inc. *Consulting Engineers*

Advanced Analysis & Fitness for Service
Failure & Materials Evaluation
Nondestructive Engineering

Est. 1885

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Amesbury, MA 01913
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Form: LPI-3.1-Rev-8-Fig-5-11