FOIA/PA NO: 2016-0129

RECORDS BEING RELEASED IN THEIR ENTIRETY

Braisted, Jonathan

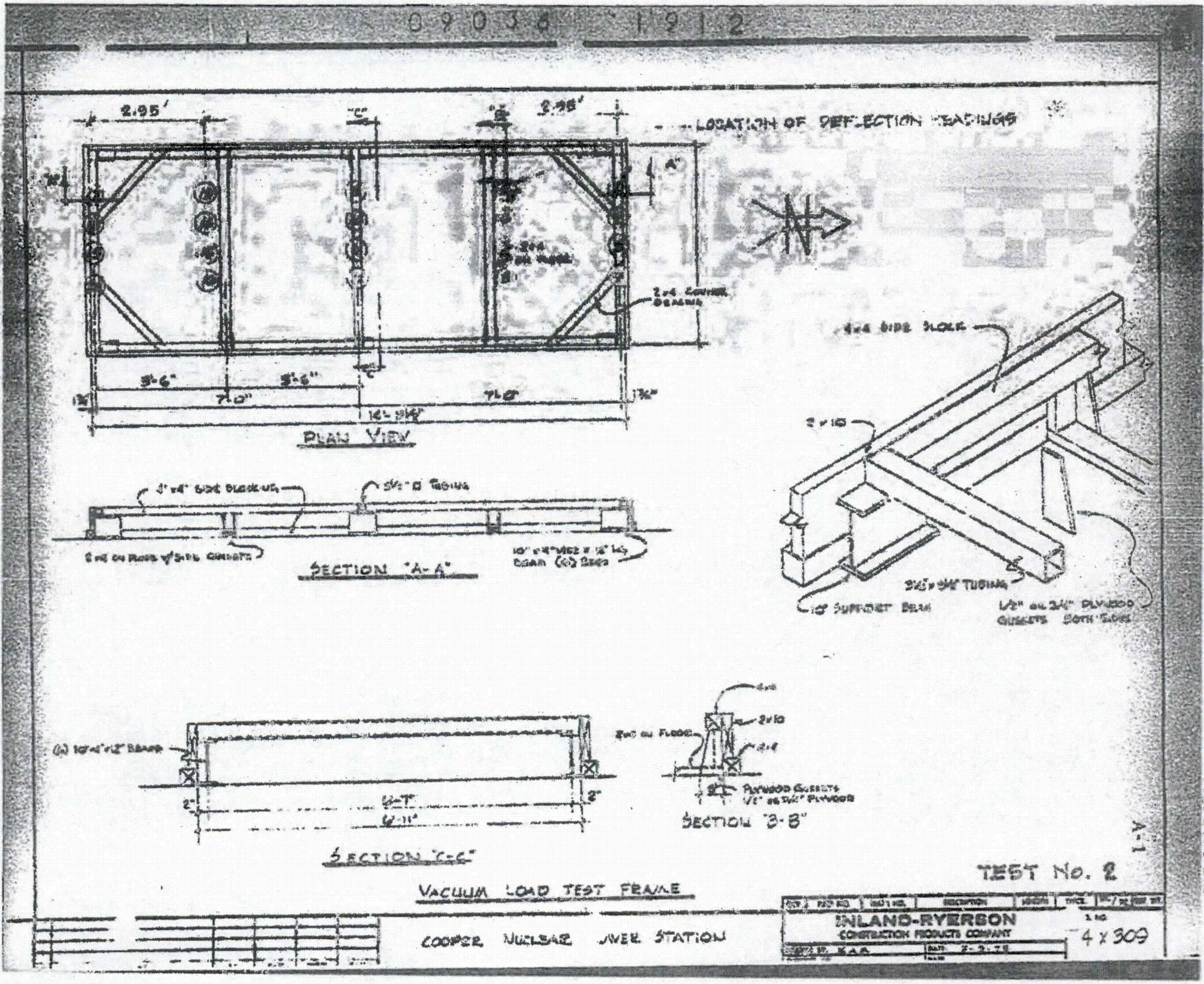
From: Sutton, Taylor E. <tesutto@nppd.com>
Sent: Tuesday, August 04, 2015 2:21 PM
To: Reinert, Dustin; Braisted, Jonathan
Cc: Flaherty, James R.
Subject: [External_Sender] Turbine Building Blow-Out Panel Calculation
Attachments: NEDC 13-028 Rev.1 Approved (email size p2).pdf

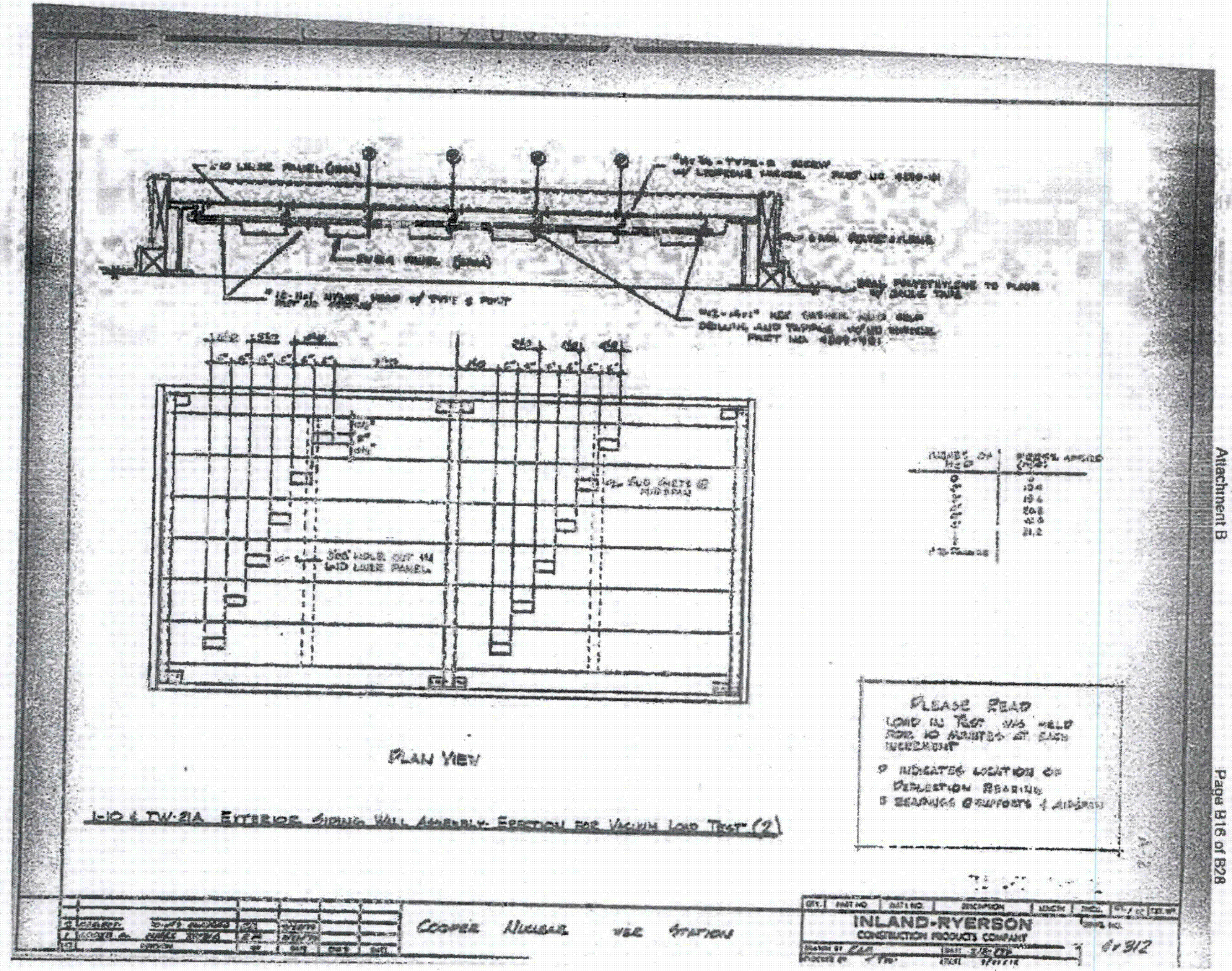
All,

Attached is the second half of NEDC 13-028 Revision 1.

TAYLOR SUTTON
DESIGN CIVIL ENGINEERING DEPARTMENT
COOPER NUCLEAR STATION – NPPD
TESUTTO@NPPD.COM
O. (402) 825-2706

APPENDIX


09038 1912
LOCATION OF DEFLECTION READINGS
2.95'
2.95'
"C"
"B"
"A"
2x4 CORNER BRACING
3'-6"
3'-6"
7'-0"
7'-0"
PLAN VIEW
SECTION "A-A"
4x4 SIDE BLOCK
2x10
10" SUPPORT BEAM
1/2" or 3/4" PLYWOOD GUSSETS BOTH SIDES
SECTION "B-B"
2x10
PLYWOOD GUSSETS
SECTION "C-C"
VACUUM LOAD TEST FRAME
TEST No. 2
INLAND-RYERSON
CONSTRUCTION PRODUCTS COMPANY
COOPER NUCLEAR STATION
4 x 309
A-1


PLAN VIEW
I-10 & TW-21A EXTERIOR SIDING WALL ASSEMBLY- ERECTION FOR VACUUM LOAD TEST (2)
PLEASE READ
INDICATES LOCATION OF DEFLECTION READING
COOPER NUCLEAR STATION
INLAND-RYERSON
CONSTRUCTION PRODUCTS COMPANY

CONSTRUCTION PRODUCTS COMPANY [redacted] A-3

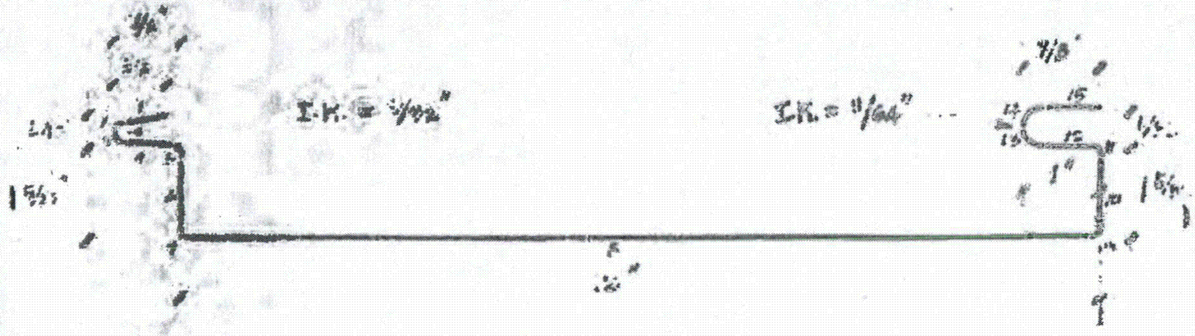
JOB NO. ______

SHEET NO. ______ OF ______

JOB NAME ... HIGLEAM POWER STATION BY ______ DATE ______

SUBJECT L-10 ... 18GA (t = .048 ...) CHKD. BY ______ DATE ______

NOTE: ALL I.R. = 1/16" ± 1/8" UNLESS NOTED.



L-10 SECTION

PROPERTIES:

$E_s = 29.5 \times 10^6$ PSI

$I_{F_s} = 0.2891$ IN.^{4}/FT.

Form 318

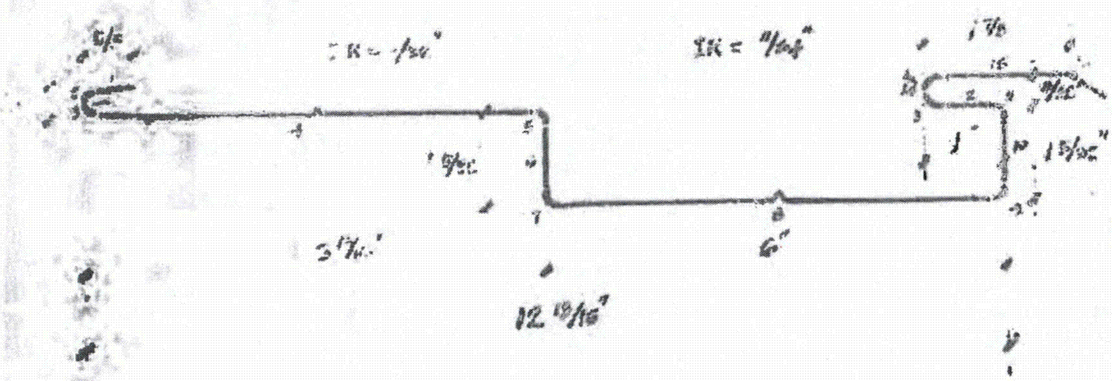
... PRODUCTS COMPANY

JOB NO. ______

SHEET NO. ____ OF ____

JOB NAME ... BAR POWER STATION BY ______ DATE ______

SUBJECT ... 20 GA t = .032 IN.) CHKD. BY ______ DATE ______



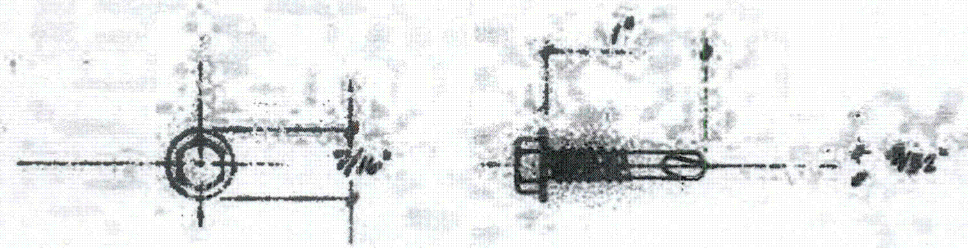
NOTE: ALL I.K = 1/16" $\pm$ 1/16 UNLESS NOTED

PROPERTIES:

$E_A = 10 \times 10^6$ PSI

$I_{D_A} = 0.0803$ $IN.^4/FT.$

Form



PART NO: 4209-951 (HWH TEKS/2, #12 - 14 x 1", Hex. Washer Head, Self-Drilling and Tapping Sheet Metal Screw).

MATERIAL: #410 Stainless Steel, Cadmium Plated.

PACKAGING: 1. 1000 per waterproof carton.

2. Suppliers label to include part no., description and quantity.

APPROVED SUPPLIER: Buildex Manufacturing - Buy through Mutual Sales.

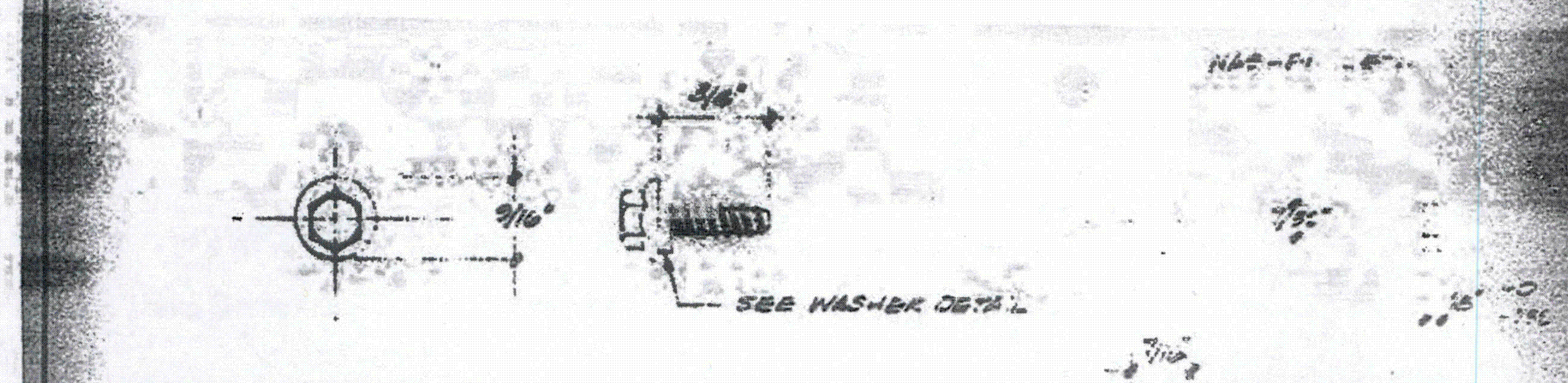
NO	REVISION	BY	DATE
0	ISSUED	[illegible]	[illegible]

HWH TEKS/2
W/ NEOPRENE WASHER

INLAND-RYERSON
CONSTRUCTION PRODUCTS CO.

DRAWN: [illegible] DATE: [illegible]
CHK'D: [illegible] DATE: [illegible]
NO [illegible]

A-5



PART NO: 4290-101 (#14 x 3/4" - Type B, Undercut Hex. Washer Head with Neoprene Washer, Self-Tapping Sheet Metal Screw). (Stocked Item).

MATERIAL: 1. Screw - #410 Stainless Steel, Cadmium Plated (Min. Torque of 100 inch pounds).

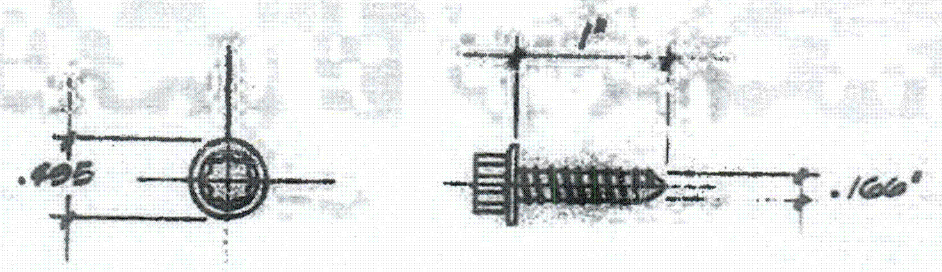
2. Neoprene Washer - Manufactured from Neoprene Composition, Carbon Black, Softeners, Conditioners and Accelerators which include Anti Oxydents and Anti-Ozonents.

PACKAGING: 1. 1000 per waterproof carton.

2. Suppliers label to include part no., description and quantity.

APPROVED SUPPLIER: Champion Screw, Chicago, Illinois

NO	REVISION	BY	DATE		
1	DESIGN CHANGE	RB	5-97	SELF TAPPING HEX HD. TYPE B w/ NEOPRENE WASH.	INLAND-RYERSON CONSTRUCTION PRODUCTS CO. A-6 DRAWN: [illegible] DATE: [illegible] NO: 4290-101 CHK'D: RB DATE: 5-9-71



PART NO: 4290-121 (#12 - 11 x 1" Nylon Head with Type S point, self tapping and sealing sheet metal screw).

MATERIAL:
1. Screw - Carbon Steel, Cadmium Plated.
2. Screw Head - Panel match TM matched weatherproof Dupont Zytel nylon resin which provides resistance to climatic and chemical corrosion, discoloration and embrittlement.
3. Standard Colors -
   - 041 (White)
   - 442 (Stormy Blue)
   - 446 (Bamboo Tan)
   - 450 (Surf Green)
   - 454 (Fieldstone Brown)
   - 456 (Antique Gold)
   - 460 (Dusty Green)

PACKAGING:
1. 1000 per waterproof carton.
2. Supplier's label to include part number, color code number, color part description and quantity.

APPROVED SUPPLIER: Buildex - Itasca, Illinois

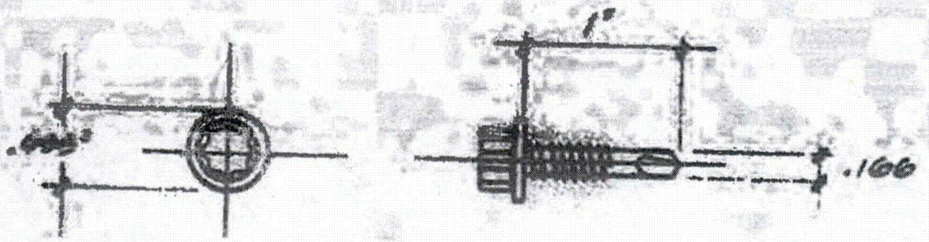
NON-STOCK STANDARD

NO	REVISION	BY	DATE
2	NON-STK. NOTE ADDED	EB	12-4-72
1	COLOR NO. CODE REV.	EB	[illegible]
0	ISSUED	DLG	5-1-72

TYPE-S POINT
NYLON HD.

INLAND-RYERSON
CONSTRUCTION PRODUCTS CO.

DRAWN: DLG	DATE: 5-1-72	N
CHK'D: EB	DATE: [illegible]	4290-121



PART NO: 4290-122 (#12 - 14 x 1" Nylon head, self drilling, tapping and sealing, sheet metal screw).

MATERIAL:

1. Screw - Carbon Steel, Cadmium Plated.
2. Screw Head - Panel match™ matched weatherproof Dupont Zytel nylon resin which provides resistance to climatic and chemical corrosion, discoloration and embrittlement.
3. Standard Colors - 041 (White) | 442 (Stormy Blue) | 448 (Bamboo Tan) | 450 (Surf Green) | 454 (Fieldstone Brown) | 456 (Antique Gold) | 460 (Dusty Green)

PACKAGING:

1. 1000 per waterproof carton.
2. Supplier's label to include part number, color code number, color, part description and quantity.

APPROVED SUPPLIER: Buildex - Itasca, Illinois

NO	REVISION	BY	DATE
2	NON STK. NOTE ADDED	RB	[illegible]
1	COLOR No. CODE REV	RB	[illegible]
0	ISSUED	DLG	[illegible]

TEK/12
NYLON HEAD
([illegible])

NON STOCK STANDARD

INLAND-RYERSON
CONSTRUCTION PRODUCTS CO.

DRAWN: [illegible] DATE: [illegible] 72
CHK'D: [illegible] DATE: [illegible]
NO. [illegible]90-122

A-8

[redacted] | [illegible] CALCULATION | PAGE A-9
DATE

TEST # 2

AISI → [illegible] E = 10×10^6 [illegible]
I = [illegible]

DEFLECTION [illegible]:

$\Delta = \frac{5}{384}\frac{wL^4}{EI}$ = [illegible] $wL^4 (144)$ [illegible]

$\Delta = \frac{[illegible]\,(7)^4\,(144)}{[illegible]}$

$\Delta = [illegible] \times 10^{-3}$ [illegible]

NORTH SPAN:

TEST RESULTS → [illegible] $w = 40$ PSF [illegible]

$\Delta = [illegible]$

CALCULATION: $w = 40$ PSF

$\Delta = 3.0[illegible]7 \times [illegible] (40) = 0.1215''$

SOUTH SPAN:

TEST RESULTS → [illegible] GRAPH. $w = 84$ PSF [illegible]

$\Delta = 0.110''$

CALCULATION: $w = 84$ PSF

$\Delta = 3.037 \times 10^{-[illegible]} (84) = 0.1084''$

SHEAR (V):

$V_u = 1.25\,w\,L = 1.25(78)(7) = 682.5$ #/FT.

$V_u = 682.5/2 = 341.25$ #/FT. PER SCREW

$V_{DES} = 5244\ t_2 = (5244)(.032) = 167.8$ #/FT /SCREW

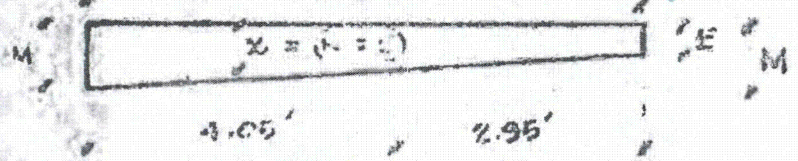
BY | CHK'D

Form 318 (Rev. 1-71)

INLAND RYERSON

TEST NO. 2
REDUCED DATA (LOAD-DEFLECT'N) PAGE A-10
[illegible] NUCLEAR POWER STATION
[illegible] ALUM [illegible] PANELS
DATE

HEAD (IN.)	PSF	DEFLECTION (Δ), INCH NORTH SPAN (N)	[illegible]	(M)	Δ	SOUTH SPAN (5)	(4)	(4)-(5)	Δ
1	ZERO	—	—	—	—	—	—	—	—
2	10.4	[illegible]	1.75	1.7830	.0181	[illegible]	2.00	[illegible]	[illegible]
3	15.6	[illegible]	2.75	1.5125	.0375	1.6554	3.25	1.7542	[illegible]
4	20.8	5.0255	[illegible]	1.7205	.0244	[illegible]	4.50	2.5517	[illegible]
5	[illegible]	6.3250	4.75	[illegible]	.0532	[illegible]	5.50	[illegible]	[illegible]
6	31.2	1.3736	4.75	2.4464	.0666	[illegible]	5.50	3.7675	[illegible]
7	36.4	6.0250	5.75	3.5220	.0871	[illegible]	6.75	4.0411	[illegible]
8	41.6	6.4136	[illegible]	4.3824	.1095	[illegible]	7.75	4.2457	[illegible]
9	46.8	[illegible]	[illegible]	[illegible]	.1365	[illegible]	9.00	[illegible]	[illegible]
10	52.0	3.2571	9.25	5.8429	.1478	5.0357	10.85	[illegible]	[illegible]
11	57.2	3.7125	10.25	5.5913	.1634	4.2011	12.00	7.7081	[illegible]
12	62.4	4.9364	11.50	6.5968	.1644	5.2123	13.25	8.0375	[illegible]
13	67.6	4.7786	13.00	8.2214	.2055	5.0678	14.00	[illegible]	[illegible]
14	72.8	5.0786	13.50	[illegible]	.1957	7.5586	15.60	7.2714	[illegible]
15	78.0	5.7786	15.25	9.4714	.2368	5.0[illegible]3	16.50	10.9681	[illegible]



LET $\bar{x}$ = STRAIGHT LINE AVERAGE DEFLECTION OF SUPPORTS @ NORTH AND SOUTH SPANS.

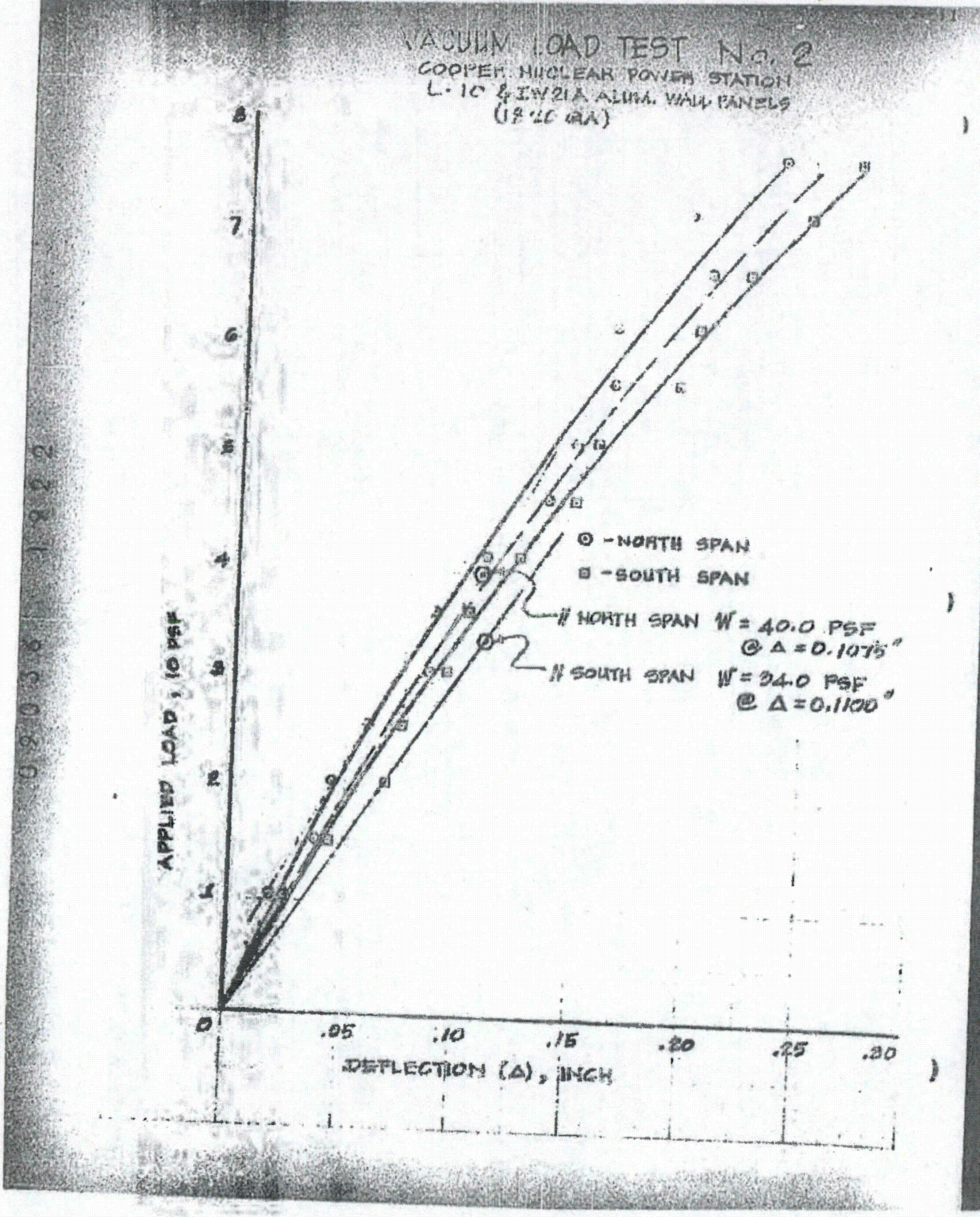
$\bar{x} = [2.95\ M + 4.05\ E] / 7.0$ WHERE M & E ARE THE AVE. @ MID-SUPPORT & END SUPPORT RESPECTIVELY

$\Delta = .04[\ldots]$ AND $\Delta = .04[(4) - (5)]$

NOTE: MAXIMUM DEFLECTION IS AT ".4126 L" FROM BOTH END.

BY | CHK'D.

Form 318 (Rev.1-71)


VACUUM LOAD TEST No. 2
COOPER NUCLEAR POWER STATION
L-10 & IW21A ALUM. WALL PANELS
O - NORTH SPAN
□ - SOUTH SPAN
NORTH SPAN W = 40.0 PSF
@ Δ = 0.1075"
SOUTH SPAN W = 34.0 PSF
@ Δ = 0.1100"
APPLIED LOAD, 10 PSF
DEFLECTION (Δ), INCH
0
.05
.10
.15
.20
.25
.30
1
2
3
4
5
6
7
8

TEST No. 2 VACUUM LOAD TEST

COOPER NUCLEAR POWER STATION
L-10 / IW21A ALUM. WALL PANELS

LOAD		NORTH END SUPPORT DEFLECTION ①						NORTH MID-SPAN DEFLECTION ②						MID-SUPPORT DEFLECTION ③					
INCR OF LOAD	PRESS (PSF)	1	2	3	4	"Δ"	[redacted]	5	6	7	8	"Δ"	[redacted]	9	10	11	12	"Δ"	[redacted]
0	APPLIED	362	364	365	367	-	364.5	349	350	352	355	-	351.50	338	338	340	342	-	339.50
2	10.4	362	363	365	366	0.50	364.0	347	348	350	354	1.75	349.75	336	336	338	341	1.75	337.75
3	15.6	362	363	365	[illegible]	0.50	364.0	346	347	349	353	2.75	348.75	335	336	338	340	2.25	337.25
4	20.8	360	362	364	[illegible]	1.50	363.0	345	346	347	352	4.00	347.50	334	335	337	339	3.25	336.25
5	26.0	360	362	[illegible]	[illegible]	1.75	362.75	344	345	347	351	4.75	346.75	334	335	337	339	3.25	336.25
6	31.2	362	363	365	[illegible]	0.25	364.25	343	345	347	352	4.75	346.75	335	336	337	340	2.75	336.75
7	36.4	362	363	364	366	0.75	363.75	343	344	346	350	5.75	345.75	333	334	336	338	4.25	335.25
8	41.6	361	363	363	366	1.25	363.25	341	343	345	349	7.00	344.50	333	334	336	337	4.50	335.00
9	46.8	362	363	363	366	1.00	363.50	341	342	343	347	8.25	343.25	332	333	335	337	5.25	334.25
10	52.0	361	362	[illegible]	366	1.25	363.25	340	340	343	346	9.25	342.25	332	332	333	336	6.25	333.25
11	[illegible]	[illegible]	362	[illegible]	[illegible]	1.75	363.00	338	340	342	345	10.25	341.25	331	332	332	336	6.25	332.75
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2.50	362.00	337	[illegible]	[illegible]	[illegible]	11.50	340.00	329	[illegible]	332	[illegible]	8.25	331.25
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		2.25	362.25	[illegible]	337	339	343	13.00	338.50	330	330	331	334	8.25	331.25
[illegible]	[illegible]	359	360	[illegible]		3.25	361.25	[illegible]	[illegible]	[illegible]	[illegible]	13.50	338.00	330	329	330	333	9.00	330.50
[illegible]	[illegible]	359	360	362		3.25	361.25	[illegible]	[illegible]	[illegible]	[illegible]	15.25	336.25	330	328	330	333	9.25	330.25

NOTE: AN APPLIED LOAD OF 7[illegible].0 PSF WAS HELD 10 MINUTES BEFORE THE SPECIMEN FAILED & FAILURE OCCURRED IN APPLYING THE NEXT LOADING INCREMENT.

REMARKS: ALL LOAD INCREMENTS HELD @ 10 MINUTES EACH. 2 POPS ON WEST SIDE OF [illegible] @ 41.6 PSF, POP ON CENTER OF SOUTH MID-SPAN @ 46.8 PSF, POP ON EAST [illegible] MID-SUPPORT @ 52.0 PSF [illegible] 2 POPS ON [illegible] AT & SOUTH MID-SPAN. IW21A ALUM WALL PANEL → SCREWS STRIP-OUT [illegible] ON THE EAST SIDE OF SPECIMEN AT FAILURE.

BY

CHK'D

3/27/[illegible]

FORM 318 (Rev. [illegible])

0 9 0 3 6 · 1 9 2 4

[redacted]

TEST No. VACUUM LOAD TEST

COOPER NUCLEAR POWER STATION
L-10/IW21A ALUM WALL PANELS

PG. A-13
DATE 3/27/13

BY

CHK'D

LOAD		MID-SUPPORT DEFLECTION ③						SOUTH MID-SPAN DEFLECTION ④						SOUTH END SUPPORT DEFLECTION ⑤					
INCH OF H_2O	PRESS (PSF)	9	10	11	12	"Δ"	AVERAGE	13	14	15	16	"Δ"	AVERAGE	17	18	19	20	"Δ"	AVERAGE
0	APPLIED					-	339,50	338	338	339	343	-	339,50	342	343	343	345	-	343,25
2	10,4					1,75	337,75	336	336	337	341	2,00	337,50	342	342	343	344	0,50	342,75
3	15,6					2,25	337,25	334	335	336	340	3,25	336,25	341	342	342	344	1,00	342,25
4	20,8					3,25	336,25	[illegible]	[illegible]	[illegible]	[illegible]	4,50	335,00	341	342	342	344	1,00	342,25
5	26,0					3,75	336,15	[illegible]	[illegible]	[illegible]	337	5,50	334,00	340	341	341	343	2,00	341,25
6	31,2					3,75	336,75	[illegible]	332	[illegible]	[illegible]	5,50	334,00	341	342	342	344	1,00	342,25
7	36,4					4,25	335,25	331	331	333	336	6,75	332,75	341	341	341	344	1,50	341,75
8	41,6					4,50	335,00	330	331	331	[illegible]	7,75	331,75	340	341	342	344	1,50	341,75
9	46,8					5,25	334,25	329	329	330	334	9,00	330,50	342	341	341	343	1,50	341,75
10	52,0					6,25	333,25	328	328	[illegible]	[illegible]	10,25	329,25	340	341	340	343	2,25	341,00
11	57,2					6,75	332,75	326	327	327	330	12,00	327,50	340	340	[illegible]	343	2,50	340,75
12	62,4					8,25	331,25	[illegible]	[illegible]	[illegible]	[illegible]	13,25	326,25	[illegible]	340	[illegible]	342	3,00	340,25
13	67,6					8,25	331,25	[illegible]	[illegible]	[illegible]	[illegible]	14,00	325,50	[illegible]	340	340	342	2,75	340,50
14	72,8					9,00	330,50	[illegible]	[illegible]	[illegible]	[illegible]	15,50	324,00	340	340	[illegible]	342	3,00	340,25
15	78,0					9,25	330,25	[illegible]	[illegible]	[illegible]	[illegible]	16,50	323,0	[illegible]	340	337	[illegible]	3,00	340,25

REMARKS:

Form 318 (Rev. [illegible])

09036 1925

LOAD		NORTH MID-SPAN DEFLECTION						SOUTH MID-SPAN DEFLECTION						DEFLECTION					
NO. OF INC	PRESS (PSF)	A	B	C	D	E	F	G	H	I	J	K	L						MEAN
0	APPLIED	2 1/8	2 3/16	2 3/16	2 5/16	2 1/8	2 1/4	2 1/8	2 1/4	2 1/4	2 1/8	2 5/16	2 1/8						
2	10.4	2 1/8	2 3/16	2 1/8	2 3/8	2 3/8	2 7/16	2 1/16	2 1/4	2 1/4	2 5/16	[illegible]	2 1/2						
3	15.6	2 1/8	2 3/16	2 1/8	2 3/8	2 7/16	2 1/2	2 1/16	2 1/4	2 1/4	2 3/8	2 7/16	2 9/16						
4	20.8	2 1/8	2 3/16	2 1/8	2 3/8	2 7/16	2 9/16	2 1/16	2 5/16	2 5/16	2 7/16	2 7/16	2 9/16						
5	26.0	2 1/8	2 3/16	2 1/8	2 7/16	2 1/2	2 5/8	2 1/16	2 5/16	2 5/16	2 1/2	2 1/2	2 5/8						
6	31.2	2 1/8	2 3/16	2 3/16	2 7/16	2 1/2	2 5/8	2 1/8	2 3/8	2 5/16	2 1/2	2 1/2	2 11/16						
7	36.4	2 1/8	2 3/16	2 1/8	2 1/2	2 5/8	2 1/2	2 1/8	2 5/16	2 5/16	2 9/16	2 9/16	2 11/16						
8	41.6	2 3/16	2 3/16	2 3/16	2 1/2	2 5/16	2 13/16	2 1/8	2 5/16	2 5/16	2 9/16	2 9/16	2 3/4						
9	46.8	2 3/16	2 1/4	2 3/16	2 1/2	2 3/8	2 7/8	2 1/8	2 5/16	2 3/8	2 5/8	2 5/8	2 13/16						
10	52.0	2 3/16	2 1/4	2 1/4	2 9/16	3 1/16	2 15/16	2 1/8	2 5/16	2 3/8	2 5/8	2 5/8	2 7/8						
11	57.2	2 3/16	2 1/4	2 1/4	2 5/8	3 1/16	3	2 1/8	2 3/8	2 3/8	2 11/16	2 11/16	2 15/16						
12	62.4	2 1/4	2 1/4	2 1/4	2 5/8	3 1/4	3 1/16	2 1/8	2 3/8	2 3/8	2 3/4	2 3/4	3						
13	67.6	2 1/4	2 1/4	2 1/4	2 7/16	3 5/16	3 1/8	2 1/8	2 3/8	2 7/16	2 3/4	3 5/16	3						
14	72.8	2 1/4	2 5/16	2 5/16	2 3/4	4 1/8	3 5/16	2 1/8	2 7/16	2 7/16	2 7/8	3 1/2	3 1/8						
15	78.0	2 5/16	2 5/16	2 5/16	2 3/4	4 5/16	3 3/8	2 1/8	2 3/8	2 7/16	2 15/16	3 5/16	3 5/16						
Δmax =		.1875	.1250	.1250	.4375	.0625	.1250	0	.1250	.1875	.6250	.2500	.8125						
AVERAGE Δmax =							0.1771						.3333						

BY

CHK'D

REMARKS: THE DEFLECTIONS ABOVE ARE THE DEFLECTION BETWEEN THE 1W10 AND 1W21A ALUM. WALL PANELS AS INDICATED ON THE DRAWING NO. 4X312.

[redacted]

TEST No.
VACUUM LOAD TEST
COOPER NUCLEAR POWER STATION
L-10/1W21A ALUM. WALL PANELS

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PAGE A-14

Form 316 (Rev. 1-71)

Plastic Design in Steel

A GUIDE AND COMMENTARY

By a Joint Committee of the
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American Society of Civil Engineers

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CHAPTER 3.—ANALYSIS AND DESIGN

The purpose of this chapter is to state the important assumptions of the plastic theory and to show the essential features of plastic analysis and design. All of the available methods developed for structural engineering applications can be classified into two groups, those based on the upper-bound theorem and those based on the lower-bound theorem. It is not intended herein to present a complete discussion of all the methods; Refs. 1.1 through 1.6, 1.8, 1.9, 1.10, 3.1, 3.2, and 3.3 may be consulted for this purpose. However, brief outlines of the frequently encountered methods most illustrative of the two groups are presented in Art. 3.2 (based on upper bound) and Arts. 3.3 and 3.4 (based on lower bound).

3.1 CONCEPTS AND ASSUMPTIONS

The important concepts and assumptions with regard to the plastic behavior of structures according to the "simple plastic theory" are as follows:

1. The structure and the loads are all in the same plane, and each member has an axis of symmetry lying in that plane.
2. The material is ductile. It has the capacity of undergoing large plastic deformation without fracture.
3. Each member cross section has a maximum resisting moment (the plastic moment, M_p), a moment that is developed through plastic yielding of the entire cross section (plastification).
4. Because of the ductility of steel, rotation at relatively constant moment will occur through a considerable angle; in other words, a plastic hinge will form.
5. Connections proportioned for full continuity will transmit the calculated plastic moment. This condition is idealized as a plastic hinge at a point.
6. Plastic hinges will first form at sections where the moments under elastic condition reach M_p. With these sections rotating at constant moment, additional loading will be accompanied by a redistribution of moments in the structure, so that plastic hinges will appear at some other locations where the moments under elastic conditions were less than M_p.
7. The plastic limit load is reached when enough plastic hinges have formed to create a mechanism.

8. The deformations are small, and therefore the equilibrium equations can be formulated for the undeformed structure (as in ordinary elastic analysis). Similarly, virtual-work expressions for mechanism displacement are based on small deflections.
9. No instability will occur before the attainment of the plastic limit load.
10. The influence of normal force and shearing force on the plastic moment is not considered.
11. The loading is proportional, that is, the ratios between different loads remain constant during loading.

The experimental verification of some of these assumptions is presented in Chapters 5 and 8. Other assumptions may need implementation to assure that the appropriate requirements are met, and this is the concern of parts of Chapter 4 and of Chapters 6 and 7.

3.2 MECHANISM METHOD

The mechanism method is based on the upper-bound theorem. As is intimated in Art. 2.2, its objective is to select from all the possible modes of failure the one that corresponds to the lowest possible plastic limit load. A check of the moments should be used to verify that the plastic-moment condition is not violated ($M \leq M_p$). In essence, therefore, the method gives a solution that is the correct value of the plastic load only when it also satisfies the plastic-moment condition. An example follows.

Analysis of Rectangular Portal Frame.

It is required to find the plastic limit load for the structure shown in Fig. 3.1(a). All members are of uniform moment capacity, M_p.

The first step in the mechanism method is to find the position of possible plastic hinges. These hinges may form at points of peak moment, that is, where the shear passes through zero, for example, at such places as concentrated load points, connections, etc. Therefore, the possible plastic-hinge locations are at sections 1, 2, 3, 4, and 5.

The next step is to select for investigation the various possible failure mechanisms. Three of these are shown in Figs. 3.1(b), (c), and (d). Mechanism 1 corresponds to the action of vertical load P and is called a "beam" mechanism. Mechanism 2 corresponds to the action of the horizontal load P and is often referred to as a "sway" or "panel" mechanism. Mechanism 3, on the other hand, is a combined mechanism representing the action of both loads. It is a combination of mechanisms 1 and 2 and does not require a plastic hinge at section 2.

The correct failure mechanism will be the one which results in the lowest loading, because any greater loading would lead to the violation of the plastic-moment condition. The loading that corresponds to each mecha-

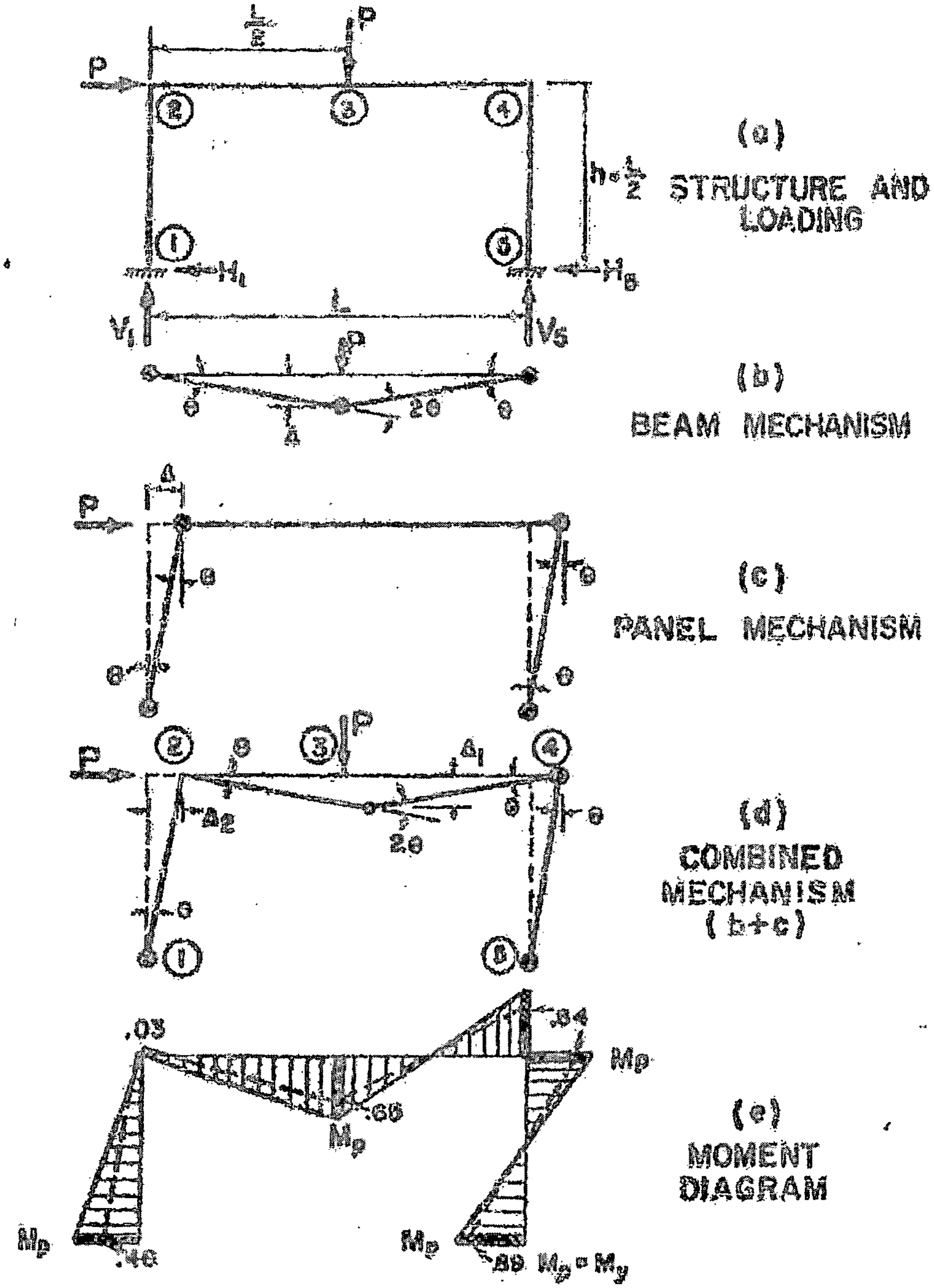


FIG. 3.1.—MECHANISM METHOD OF ANALYSIS APPLIED TO A FIXED-BASE RECTANGULAR PORTAL FRAME

nism may be computed by applying a virtual displacement (one that satisfies the constraints on that mechanism) and then using the upper-bound theorem. Referring to Fig. 3.1(b), the beam is allowed to move through a virtual displacement, Δ. The value of P, obtained by equating the *external work*, W_E, done by the load as it moves through this displacement to the *internal work*, W_I, absorbed at the plastic hinges as they rotate through corresponding angles, is greater than or equal to the plastic limit load. According to this procedure

$$W_E = W_I \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.1)$$

For mechanism 1 the external work is given by

$$W_E = P\,\Delta = P\,\frac{L}{2}\,\theta \quad \ldots\ldots\ldots\ldots\ldots \quad (3.2)$$

The internal work is given by

$$W_I = M_p\,\theta + M_p\,2\,\theta + M_p\,\theta = 4\,M_p\theta \quad \ldots\ldots \quad (3.3)$$

Equating Eqs. 3.2 and 3.3 according to Eq. 3.1

$$P\,\frac{L}{2}\,\theta = 4\,M_p\,\theta$$

and

$$P_1 = \frac{8\,M_p}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.4)$$

Similarly, for mechanism 2

$$P\,\frac{L}{2}\,\theta = M_p\,(\theta + \theta + \theta + \theta)$$

and

$$P_2 = \frac{8\,M_p}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.5)$$

For mechanism 3

$$P\,\Delta_1 + P\,\Delta_2 = M_p\,(\theta + 2\,\theta + 2\,\theta + \theta)$$

$$P\,\frac{L}{2}\,\theta + P\,\frac{L}{2}\,\theta = 6\,M_p$$

and

$$P_3 = \frac{6\,M_p}{L} \leftarrow \boxed{P_p} \quad \ldots\ldots\ldots\ldots\ldots \quad (3.6)$$

Since the lowest value is P_3, this is assumed to be the true limit load.

To make sure that some other mechanism was not overlooked and that P_3 is the true plastic limit load, the plastic-moment condition should be checked. An efficient way of doing this is to construct the moment diagram for the entire structure. The complete diagram is shown in Fig. 3.1(e), the moment at each section being determined by statics (the moments are plotted on the tension side of the member). It is found that $M \leq M_p$ throughout, and thus the answer determined above is verified. It is only a coincidence that the moment at section 2 is equal to zero.

The dotted line in Fig. 3.1(e) traces the moment diagram at the stage at which the hypothetical yield moment would first be reached in the frame—namely at section 5, where the moment is shown as M_y. The first plastic hinge would form there, followed by hinges at sections 4, 3, and finally at section 1.

If the problem were to design the frame for the loads P, the required section would be found from Eq. 3.6 as

$$M_p = \frac{PL}{6} \qquad (3.7)$$

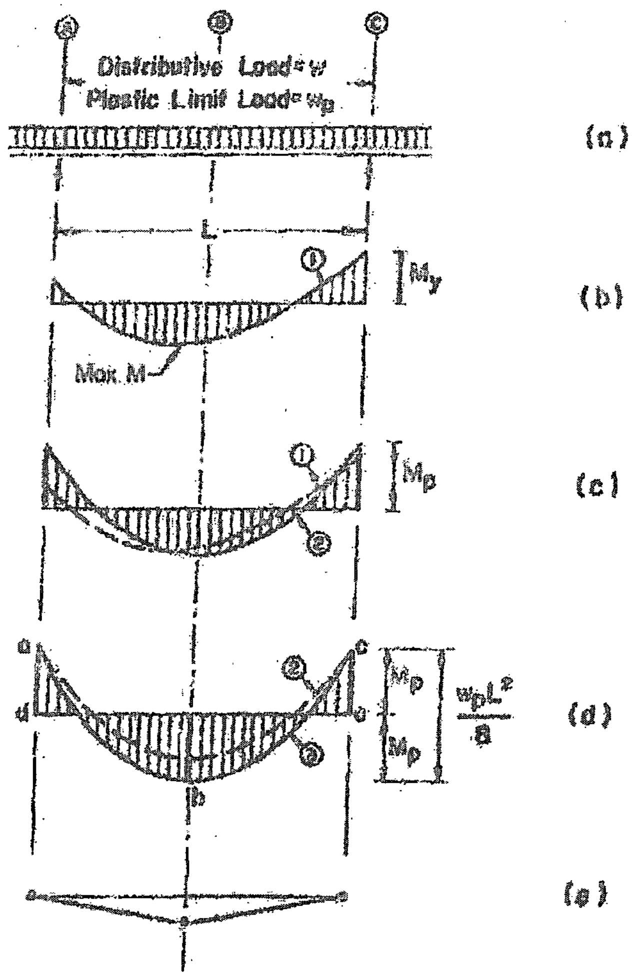
By using the principles illustrated in the example much more complex frames can also be analyzed.

3.3 STATICAL METHOD

The statical method of analysis employs the lower-bound theorem and is suitable for continuous beams and for frames in which the number of redundants is one or two. As described in Art. 2.2, the objective of this method is to find a possible moment diagram in which the plastic-moment condition is not violated ($M \leq M_p$). To make sure that this also gives the maximum possible load, a sufficient number of plastic hinges must be formed to create a mechanism. An example follows.

Suppose it is desired to determine the load, w_u, that may be carried by a beam of given constant moment capacity in a multispan structure. One of the interior spans, AC, is shown in Fig. 3.2(a).

As the loading, w, increases from zero, there are peak moments at the two supports and a peak moment within the span. Unless the structure and loading are symmetrical about span AC, the moments at the two supports will be unequal, and a typical moment diagram at the elastic limit is shown in Fig. 3.2(b). The maximum moment within the span is not at the span center line but is near it. As the load increases still further, the greater end moment (C) attains M_p and a plastic hinge forms there. As the load increases still further, the other support moment (A) attains M_p, and the maximum moment within the span increases as shown by curve 2 in Fig. 3.2(c). Since there is a hinge at each end of the span, the point of maximum moment has shifted to the center of the span and the moment curve 2 is



IG. 3.2.—STATICAL METHOD OF ANALYSIS APPLIED TO A CONTINUOUS EAM

mmetrical. Any of these load values gives a lower-bound solution. The ue plastic limit load will be reached only when a mechanism develops in te span. In the example this is attained when a plastic hinge finally forms t midspan. The moment diagram at the plastic load is shown by curve 3 Fig. 3.2(d), and the corresponding failure mechanism in Fig. 3.2(e).

The maximum ordinate of the determinate moment diagram must be $_pL^2/8$. Thus, referring to Fig. 3.2(d)

$$\frac{w_p L^2}{8} = M_p + M_p \qquad (3.8)$$

and

$$w_p = \frac{16\,M_p}{L^2} \ldots\ldots\ldots\ldots\ldots (3.9)$$

As described in Art. 4.6, the plastic moment of any beam cross section may be computed from a knowledge of the yield-stress level and the cross-section geometry. Thus the plastic load, w_p, is determined.

For design, that is, if the required plastic limit load is given by the terms of the problem, the required M_p for the beam may be calculated and the beam selected. It will be seen from Fig. 3.2(d) that the plastic design for a uniform, restrained beam may be expressed as follows:

> Draw the determinate moment diagram for load w_p on a beam of the same span but with hinged ends. Draw a new base line for zero moments (a "fixing line") at the midheight of this curve. Equate the three maximum moments thus found to M_p.

This completes the solution for an interior span of a continuous beam with restraining moments at least as great as the strength of the beam selected. The same reasoning as followed previously for a uniform load will lead to the same solution method for other types of loading, such as partial uniform or concentrated.

Of course, where a smaller beam adjoins an interior span, or in the case of an end span with simple exterior support, or in the case of nonprismatic section, the fixing line would not be drawn at midheight of the determinate moment diagram. Its position should reflect the member moment capacity at the points of peak moment. In essence, then, the failure moment diagram is drawn in such a way that a mechanism forms. Curve 3 of Fig. 3.2(d) satisfies this requirement, of course, because the three plastic hinges at a, b, and c result in the mechanism shown in Fig. 3.2(e).

In plastic design of continuous beams and frames it is unnecessary to consider the elastic distribution of moments such as that shown in Fig. 3.2(b), or the behavior under increasing load. A systematized procedure for the general case would simply involve the construction of a determinate moment diagram [corresponding to curve abc of Fig. 3.2(d)], which is then combined with a redundant moment diagram (adec) in such a way that a mechanism is formed.

Although the statical method is quite simple when applied to continuous beams, it is inefficient for frames with more than one degree of indeterminacy. For such frames recourse should be made to other methods, such as the moment balancing method described in Art. 3.4.

handling of these problems in the calculation of deflections in the plastic range.

Digital computer programs can be formulated which apply each of the methods listed above for determination of the deformed shape of members loaded into the inelastic range. Programs using numerical integration have been most helpful in computing Column Deflection Curves applicable for design aids in a large variety of beam-column problems (1.33). In the computation of deflections for more complex structures, the normal capacity of computing equipment frequently requires a compromise between refinement of the moment-curvature relationship for individual members and the ability to handle a larger number of members. Here again the "plastic hinge" method has been found to be most practical for deflection computations (3.9, 9.8, 9.9).

9.4 SAMPLE CALCULATION OF DEFLECTION AT ULTIMATE LOAD

Slope-Deflection Method.

In this section, the approximate deflection at ultimate load of the simple portal frame shown in Fig. 9.2(a) will be calculated. The assumptions used will be those given in Art. 9.3 for the "plastic hinge" method. One method of calculation is to make use of the slope-deflection equation (1.2, 1.6, 1.27, 3.1, 9.6)

$$\theta_A = \theta_A' + \frac{\Delta}{\ell} + \frac{\ell}{3EI}\left(M_{AB} - \frac{1}{2}M_{BA}\right) \quad \ldots\ldots (9.3)$$

(See Fig. 9.3 for nomenclature. Clockwise M and θ are positive. When the equation is used in this form, EI must be constant in segment AB.)

The mechanism and moment diagram at ultimate load are given in Fig. 9.2(b) and 9.2(c), respectively. In Fig. 9.2(d) are given free body diagrams of the portions of the frame. To solve for the deflections δ_V and δ_H, two boundary conditions are needed. One of these is that θ_{21} equals θ_{23} because of continuity at joint 2. The second boundary condition depends on whether the last plastic hinge forms at section 3 or section 4. A trial solution of the problem will be made for each of these assumptions, and then the correct solution will be selected.

Continuity at Section 2.—($\theta_{21} = \theta_{23}$)

$$\theta_A = \theta_A' + \frac{\Delta}{\ell} + \frac{\ell}{3EI}\left(M_{AB} - \frac{M_{BA}}{2}\right)$$

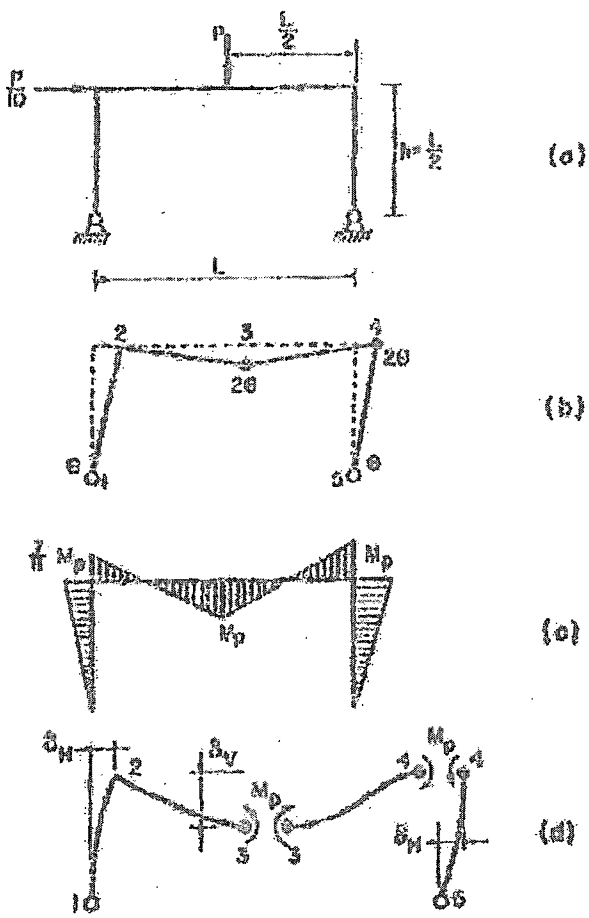


FIG. 9.2.—DEFLECTION OF PORTAL FRAME AT ULTIMATE LOAD

First, considering segment 2-1 and using the moments of Fig. 9.2(c)

$$\theta_{21} = 0 + \frac{\delta_H}{\frac{L}{2}} + \frac{\frac{L}{2}}{3EI}\left(\frac{7}{11} M_p - 0\right)$$

$$\theta_{21} = \frac{2\,\delta_H}{L} + \frac{7}{66}\,\frac{M_p L}{EI}$$

Next, considering segment 2-3

$$\theta_{23} = 0 + \frac{\delta_V}{\frac{L}{2}} + \frac{\frac{L}{2}}{3EI}\left(-\frac{7}{11}M_p + \frac{1}{2}M_p\right)$$

$$\theta_{23} = \frac{2\delta_V}{L} - \frac{1}{44}\frac{M_p L}{EI}$$

Equating θ_{21} and θ_{23}

$$\frac{2\delta_H}{L} + \frac{7}{66}\frac{M_p L}{EI} = \frac{2\delta_V}{L} - \frac{1}{44}\frac{M_p L}{EI}$$

$$\delta_V = \frac{17}{264}\frac{M_p L^2}{EI} + \delta_H \quad \text{. . . . . (9.4)}$$

This gives one desired relationship for δ_V and δ_H which applies for either of the following trial solutions.

Trial at Section 3.—($\theta_{32} = \theta_{34}$)

Considering segment 3-2

$$\theta_{32} = 0 + \frac{\delta_{V3}}{\frac{L}{2}} + \frac{\frac{L}{2}}{3EI}\left(-M_p + \frac{7}{22}M_p\right)$$

$$\theta_{32} = \frac{2\delta_{V3}}{L} - \frac{5}{44}\frac{M_p L}{EI}$$

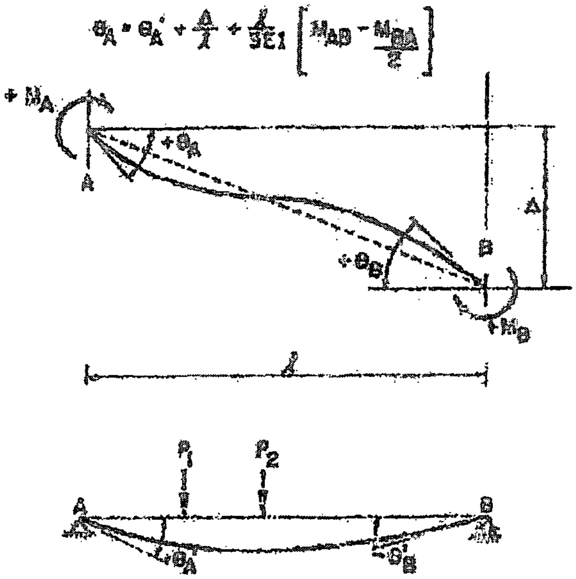


FIG. 9.3.—NOMENCLATURE FOR SLOPE-DEFLECTION EQUATION

Considering segment 3-4

$$\theta_{34} = 0 - \frac{\delta_{V3}}{\frac{L}{2}} + \frac{\frac{L}{2}}{3EI}\left(M_p - \frac{1}{2}M_p\right)$$

$$\theta_{34} = -\frac{2\delta_{V3}}{L} + \frac{1}{12}\frac{M_p L}{EI}$$

In the above expressions, δ_{V3} is the vertical deflection with continuity assumed at section 3.
Equating θ_{32} and θ_{34}

$$\frac{2\delta_{V3}}{L} - \frac{5}{44}\frac{M_p L}{EI} = -\frac{2\delta_{V3}}{L} + \frac{1}{12}\frac{M_p L}{EI} \quad \ldots\ldots (9.5)$$

Thus

$$\delta_{V3} = \frac{13}{264}\frac{M_p L^2}{EI}$$

and from Eq. 9.4

$$\delta_{H3} = -\frac{1}{66}\frac{M_p L^2}{EI} \quad \ldots\ldots\ldots\ldots (9.6)$$

These values for δ_V and δ_H would be the correct values for the deflections at ultimate load if the last plastic hinge formed at point 3.

Trial at Section 4.—($\theta_{43} = \theta_{45}$)

Considering segment 4-3

$$\theta_{43} = 0 - \frac{\delta_{V4}}{\frac{L}{2}} + \frac{\frac{L}{2}}{3EI}\left(M_p - \frac{1}{2}M_p\right)$$

$$\theta_{43} = -\frac{2\delta_{V4}}{L} + \frac{1}{12}\frac{M_p L}{EI}$$

Considering segment 4-5

$$\theta_{45} = 0 + \frac{\delta_{H4}}{\frac{L}{2}} + \frac{\frac{L}{2}}{3EI}\left(-M_p - 0\right)$$

$$\theta_{45} = \frac{2\delta_{H4}}{L} - \frac{1}{6}\frac{M_p L}{EI}$$

Equating θ_{43} and θ_{45} and solving simultaneously with Eq. 9.4 gives

$$\delta_{V4} = \frac{25}{264}\frac{M_p L^2}{EI}$$

and

$$\delta_{U4} = \frac{1}{33}\frac{M_p L^2}{EI} \quad \ldots\ldots\ldots\ldots\ldots \quad (9.7)$$

The deflections calculated assuming continuity at section 4 are larger than those obtained on the basis of continuity at section 3. Consideration of the rotations at section 4 when continuity is assumed at section 3 shows that an impossible reverse "kink" would occur at section 4. Because of this reverse "kink," the assumption resulting in such a solution is discarded. Therefore, the deflections assuming continuity at section 4 are the correct deflections at ultimate load. It has been established elsewhere (1.27) that the correct set of calculated deflections will be the largest set from any series of calculations based on different assumptions of the last plastic hinge. The last plastic hinge to form will be the hinge at which continuity is assumed in calculating the largest deflection.

Dummy Load Method.

Another example of the calculation of deflection at ultimate load will be given using the "dummy load" method, which is a variation of the "virtual work" method. This method is used in many textbooks on structures—for instance, Ref. 9.10. In plastic range applications, the moment-curvature relationship boundary conditions will be treated according to the assumptions of the plastic hinge method.

The operations of the method are as follows: (1) the moments M due to the applied loads are determined, and (2) a dummy unit load is applied to the unloaded structure at the point of the desired deflection and in the direction of the desired deflection. A set of moments m in equilibrium with the unit load is then determined.

Additional special conditions used to adapt this method for the solution of deflections at ultimate load are:

1. All members are assumed to be fully elastic between plastic hinges.
2. Continuity is assumed at the last plastic hinge.
3. The dummy load is applied to an auxiliary structure with frictionless hinges assumed at all but the last plastic hinge.
4. If the location of the last plastic hinge is unknown, a solution must be made for each assumed last plastic hinge.

After applying these conditions, the deflection at the unit load due to the original applied loads is determined from a virtual work equation of the form

$$\delta\,(1^k) = \int \phi\, m\, ds = \int \frac{Mm}{EI}\, ds \quad \ldots\ldots\ldots \quad (9.8)$$

5/7 FLEXIBLE CABLES. One important type of structural member is the flexible cable which is used in suspension bridges, transmission lines, messenger cables for supporting heavy trolley or telephone lines, and many other applications. In the design of these structures it is necessary to know the relations involving the tension, span, sag, and length of the cables. We determine these quantities by examining the cable as a body in equilibrium. In the analysis of flexible cables we assume that any resistance offered to bending is negligible. This assumption means that the force in the cable is always in the direction of the cable.

Flexible cables may support a series of distinct concentrated loads, as shown in Fig. 5/26*a*, or they may support loads that are continuously distributed over the length of the cable, as indicated by the variable-intensity loading w in Fig. 5/26*b*. In some instances the weight of the cable is negligible compared with the loads it supports, and in other cases the weight of the cable may be an appreciable load or the sole load, in which case it cannot be neglected. Regardless of which of these conditions is present, the equilibrium requirements of the cable may be formulated in the same manner.

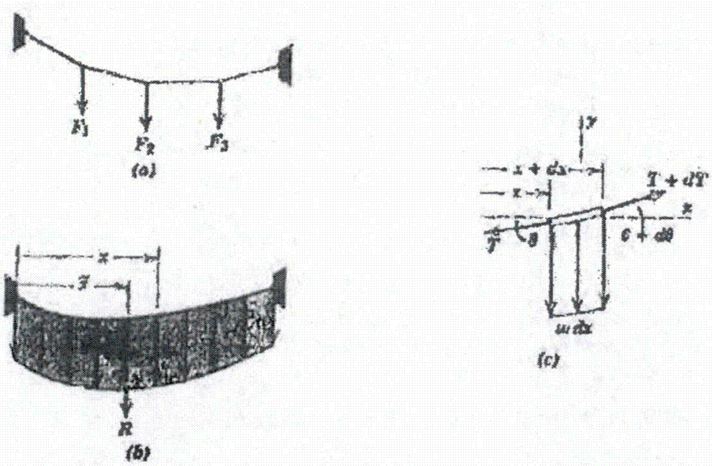


Figure 5/26

(a) General Relationships. If the intensity of the variable and continuous load applied to the cable of Fig. 5/26*b* is expressed as w units of force per unit of horizontal length x, then the resultant R of the vertical loading is

$$R = \int dR = \int w\,dx$$

where the integration is taken over the desired interval. We find the position of R from the moment principle, so that

$$R\bar{x} = \int x\,dR \qquad \bar{x} = \frac{\int x\,dR}{R}$$

The elemental load $dR = w\,dx$ is represented by an elemental strip of vertical length w and width dx of the shaded area of the loading diagram, and R is represented by the total area. It follows from the foregoing expressions that R passes through the *centroid* of the shaded area.

The equilibrium condition of the cable will be satisfied if each infinitesimal element of the cable is in equilibrium. The free-body diagram of a differential element is shown in Fig. 5/26c. At the general position x the tension in the cable is T, and the cable makes an angle θ with the horizontal x-direction. At the section $x + dx$ the tension is $T + dT$, and the angle is $\theta + d\theta$. Note that the changes in both T and θ are taken positive with positive change in x. The vertical load $w\,dx$ completes the free-body diagram. The equilibrium of vertical and horizontal forces requires, respectively, that

$$(T + dT)\sin(\theta + d\theta) = T\sin\theta + w\,dx$$

$$(T + dT)\cos(\theta + d\theta) = T\cos\theta$$

The trigonometric expansion for the sine and cosine of the sum of two angles and the substitutions $\sin d\theta = d\theta$ and $\cos d\theta = 1$, which hold in the limit as $d\theta$ approaches zero, yield

$$(T + dT)(\sin\theta + \cos\theta\,d\theta) = T\sin\theta + w\,dx$$

$$(T + dT)(\cos\theta - \sin\theta\,d\theta) = T\cos\theta$$

Dropping the second-order terms and simplifying give us

$$T\cos\theta\,d\theta + dT\sin\theta = w\,dx$$

$$-T\sin\theta\,d\theta + dT\cos\theta = 0$$

which we may write as

$$d(T\sin\theta) = w\,dx \qquad \text{and} \qquad d(T\cos\theta) = 0$$

The second relation expresses the fact that the horizontal component of T remains unchanged, which is clear from the free-body diagram. If we introduce the symbol $T_0 = T\cos\theta$ for this constant horizontal force, we may then substitute $T = T_0/\cos\theta$ into the first of the two equations just obtained and get $d(T_0\tan\theta) = w\,dx$. But $\tan\theta = dy/dx$, so that the equilibrium equation may be written in the form

$$\boxed{\frac{d^2y}{dx^2} = \frac{w}{T_0}} \qquad (5/13)$$

Equation 5/13 is the *differential equation* for the flexible cable. The solution to the equation is that functional relation $y = f(x)$

which satisfies the equation and also satisfies the conditions at the fixed ends of the cable, called *boundary conditions*. This relationship defines the shape of the cable, and we will use it to solve two important and limiting cases of cable loading.

(b) *Parabolic Cable.* When the intensity of vertical loading w is constant, the description closely approximates a suspension bridge where the uniform weight of the roadway may be expressed by the constant w. The mass of the cable is not distributed uniformly with the horizontal but is relatively small and its weight is neglected. For this limiting case we will prove that the cable hangs in a parabolic arc. Figure 5/27 shows such a suspension bridge of span L and sag h with origin of coordinates taken at the midpoint of the span. With

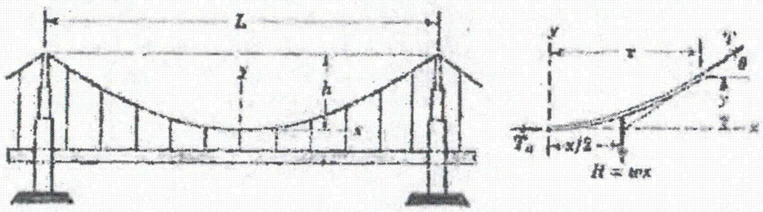


Figure 5/27

both w and T_0 constant, we may integrate Eq. 5/13 once with respect to x to obtain

$$\frac{dy}{dx} = \frac{wx}{T_0} + C$$

where C is a constant of integration. For the coordinate axes chosen, $dy/dx = 0$ when $x = 0$, so that $C = 0$. Hence

$$\frac{dy}{dx} = \frac{wx}{T_0}$$

which defines the slope of the curve as a function of x. One further integration yields

$$\int_0^y dy = \int_0^x \frac{wx}{T_0}\,dx \qquad \text{or} \qquad \boxed{y = \frac{wx^2}{2T_0}} \qquad (5/14)$$

Readers should make certain that they can obtain the identical results with the indefinite integral together with the evaluation of the constant of integration. Equation 5/14 gives the shape of the cable, which we see is a vertical parabola. The constant horizontal component of cable tension becomes the cable tension at the origin.

Inserting the corresponding values $x = L/2$ and $y = h$ in Eq. 5/14 gives

$$T_0 = \frac{wL^2}{8h} \qquad \text{and} \qquad y = \frac{4hx^2}{L^2}$$

The tension T is found from a free-body diagram of a finite portion of the cable, shown in Fig. 5/27, which requires that

$$T = \sqrt{T_0^2 + w^2x^2}.$$

Elimination of T_0 gives

$$T = w\sqrt{x^2 + \left(\frac{L^2}{8h}\right)^2} \qquad (5/15)$$

The maximum tension occurs when $x = L/2$ and is

$$T_{max} = \frac{wL}{2}\sqrt{1 + \frac{L^2}{16h^2}} \qquad (5/15a)$$

We obtain the length S of the complete cable from the differential relation $ds = \sqrt{(dx)^2 + (dy)^2}$. Thus

$$\int_0^{S/2} ds = \frac{S}{2} = \int_0^{L/2} \sqrt{1 + \left(\frac{dy}{dx}\right)^2}\, dx = \int_0^{L/2} \sqrt{1 + \left(\frac{wx}{T_0}\right)^2}\, dx$$

For convenience in computation we change this expression to a convergent series and then integrate it term by term. From the expansion for a variable x

$$(1 + x)^n = 1 + nx + \frac{n(n-1)}{2!}x^2 + \frac{n(n-1)(n-2)}{3!}x^3 + \cdots$$

the integral may be written as

$$S = 2\int_0^{L/2} \left(1 + \frac{w^2x^2}{2T_0^2} - \frac{w^4x^4}{8T_0^4} + \cdots\right) dx$$

$$= L\left(1 + \frac{w^2L^2}{24T_0^2} - \frac{w^4L^4}{640T_0^4} + \cdots\right)$$

Substitution of $w/T_0 = 8h/L^2$ yields

$$S = L\left[1 + \frac{8}{3}\left(\frac{h}{L}\right)^2 - \frac{32}{5}\left(\frac{h}{L}\right)^4 + \cdots\right] \qquad (5/16)$$

When we examine the properties of this series, we find that it converges for all values of $h/L \leq 1/4$. In most cases h is much smaller than $L/4$, so that the three terms of Eq. 5/16 give a sufficiently accurate approximation.

(c) ***Catenary Cable.*** Consider now a uniform cable, Fig. 5/28, suspended at two points in the same horizontal plane and hanging under the action of its own weight only. The free-body diagram of a finite portion of the cable of length s is shown in the right-hand part of the figure. This free-body diagram differs from that in Fig. 5/27 in that the total vertical force supported is equal to the weight of the section of cable of length s in place of the load distributed uniformly

The integration constant K is evaluated from the boundary condition $x = 0$ when $y = 0$. This substitution requires that $K = -T_0/\mu$, and hence

$$y = \frac{T_0}{\mu}\left(\cosh\frac{\mu x}{T_0} - 1\right) \qquad (5/19)$$

Equation 5/19 is the equation of the curve (catenary) assumed by the cable hanging under the action of its weight only.

From the free-body diagram in Fig. 5/28 we see that $dy/dx = \tan\theta = \mu s/T_0$. Thus, from the previous expression for the slope,

$$s = \frac{T_0}{\mu}\sinh\frac{\mu x}{T_0} \qquad (5/20)$$

We obtain the tension T in the cable from the equilibrium triangle of the forces in Fig. 5/28. Thus

$$T^2 = \mu^2 s^2 + T_0^2$$

which, upon combination with Eq. 5/20, becomes

$$T^2 = T_0^2\left(1 + \sinh^2\frac{\mu x}{T_0}\right) = T_0^2\cosh^2\frac{\mu x}{T_0}$$

or

$$T = T_0\cosh\frac{\mu x}{T_0} \qquad (5/21)$$

We may also express the tension in terms of y with the aid of Eq. 5/19, which, when substituted into Eq. 5/21, gives

$$T = T_0 + \mu y \qquad (5/22)$$

Equation 5/22 shows us that the increment in cable tension from that at the lowest position depends only on μy.

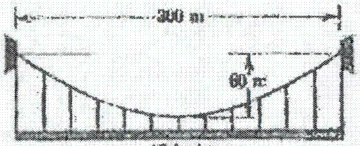
Most problems dealing with the catenary involve solutions of Eqs. 5/19 through 5/22, which may be handled by a graphical approximation or solved by computer. The graphical procedure is illustrated in the sample problem following this article.

The solution of catenary problems where the sag-to-span ratio is small may be approximated by the relations developed for the parabolic cable. A small sag-to-span ratio means a tight cable, and the uniform distribution of weight along the cable is not much different from the same load intensity distributed uniformly along the horizontal.

Many problems dealing with both the catenary and parabolic cable involve suspension points that are not on the same level. In such cases we may apply the relations just developed to the part of the cable on each side of the lowest point.

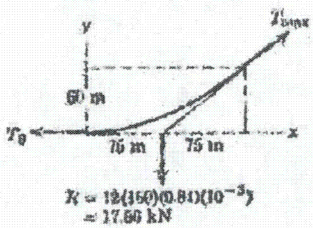
Sample Problem 5/10

The light cable supports a mass of 12 kg per meter of horizontal length and is suspended between the two points on the same level 300 m apart. If the sag is 60 m, find the tension at midlength, the maximum tension, and the total length of the cable.



Solution. With a uniform horizontal distribution of load, the solution of part (*b*) of Art. 5/7 applies, and we have a parabolic shape for the cable. For $h = 60$ m, $L = 300$ m, and $w = 12(9.81)(10^{-3})$ kN/m the relation following Eq. 5/14 gives for the midlength tension

$$\left[T_0 = \frac{wL^2}{8h}\right] \qquad T_0 = \frac{0.1177(300)^2}{8(60)} = 22.07 \text{ kN} \qquad \textit{Ans.}$$



The maximum tension occurs at the supports and is given by Eq. 5/15*a*. Thus

① $$\left[T_{max} = \frac{wL}{2}\sqrt{1 + \left(\frac{L}{4h}\right)^2}\right]$$

$$T_{max} = \frac{0.1177(300)}{2}\sqrt{1 + \left[\frac{300}{4(60)}\right]^2} = 28.27 \text{ kN} \qquad \textit{Ans.}$$

The sag-to-span ratio is $60/300 = 1/5 < 1/4$. Therefore the series expression developed in Eq. 5/16 is convergent, and we may write for the total length

$$S = 300\left[1 + \frac{8}{3}\left(\frac{1}{5}\right)^2 - \frac{32}{5}\left(\frac{1}{5}\right)^4 + \cdots\right]$$

$$= 300[1 + 0.1067 - 0.01024 + \cdots] = 329 \text{ m} \qquad \textit{Ans.}$$

① *Suggestion:* Check the value of T_{max} directly from the free-body diagram of the right-hand half of the cable from which a force polygon may be drawn.

0.050 in. (1.27 mm) thick.

E4.4.1 Pull-Out

The nominal pull-out strength [*resistance*], P_{not}, shall be calculated as follows:

$$P_{not} = 0.85\, t_c\, d\, F_{u2} \quad \text{(Eq. E4.4.1-1)}$$

E4.4.2 Pull-Over

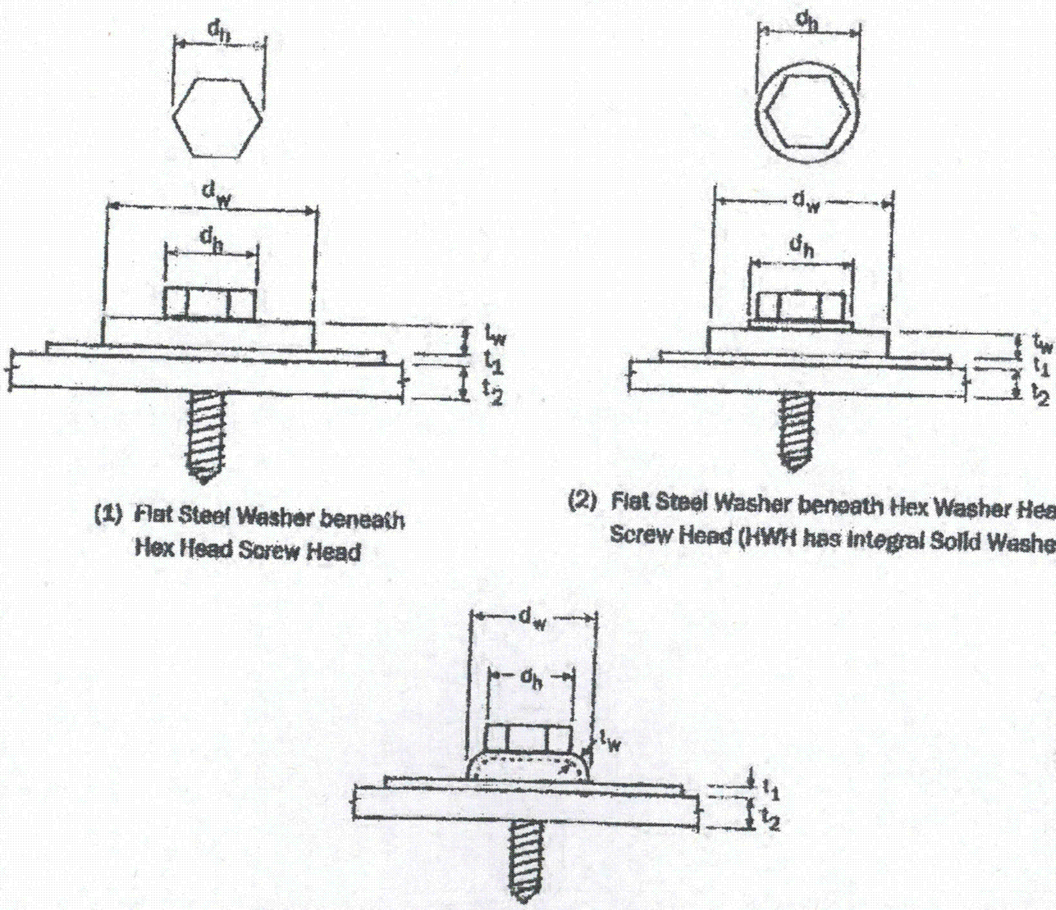
The nominal pull-over strength [*resistance*], P_{nov}, shall be calculated as follows:

$$P_{nov} = 1.5 t_1 d'_w F_{u1} \quad \text{(Eq. E4.4.2-1)}$$

where

d'_w = Effective pull-over diameter determined in accordance with (a), (b), or (c) as follows:

(a) For a round head, a hex head (Figure E4.4.2(1)), or hex washer head (Figure E4.4.2(2)) screw with an independent and solid steel washer beneath the



(1) Flat Steel Washer beneath Hex Head Screw Head

(2) Flat Steel Washer beneath Hex Washer Head Screw Head (HWH has integral Solid Washer)

(3) Domed Washer (Non-Solid) beneath Screw Head

Figure E4.4.2 Screw Pull-Over with Washer screw head

$$d'_w = d_h + 2t_w + t_1 \le d_w \quad (Eq.\ E4.4.2\text{-}2)$$

where

d_h = Screw head diameter or hex washer head integral washer diameter

t_w = Steel washer *thickness*

d_w = Steel washer diameter

(b) For a round head, a hex head, or hex washer head screw without an independent washer beneath the screw head:

$d'_w = d_h$ but not larger than 1/2 in. (12.7 mm)

(c) For a domed (non-solid and independent) washer beneath the screw head (Figure E4.4.2(3)), it is permissible to use d'_w as calculated in Eq. E4.4.2-2, with d_h, t_w, and t_1 as defined in Figure E4.4.2(3). In the equation, d'_w can not exceed 5/8 in. (16 mm). Alternatively, pull-over design values for domed washers, including the *safety factor*, Ω, and the *resistance factor*, ϕ, shall be permitted to be determined by test in accordance with Chapter F.

E4.4.3 Tension in Screws

The nominal tension strength [resistance] of the screw shall be taken as P_{ts}.

In lieu of the value provided in Section E4, the *safety factor* or the *resistance factor* shall be permitted to be determined in accordance with Section F1 and shall be taken as $1.25\Omega \le 3.0$ (*ASD*), $\phi/1.25 \ge 0.5$ (*LRFD*), or $\phi/1.25 \ge 0.4$ (*LSD*).

E4.5 Combined Shear and Pull-Over

E4.5.1 ASD Method

For screw *connections* subjected to a combination of shear and tension forces, the following requirement shall be met:

$$\frac{Q}{P_{ns}} + 0.71\frac{T}{P_{nov}} \le \frac{1.10}{\Omega} \quad (Eq.\ E4.5.1\text{-}1)$$

In addition, Q and T shall not exceed the corresponding *allowable strength* determined by Sections E4.3 and E4.4, respectively.

where

Q = Required allowable shear strength of connection

T = Required allowable tension strength of connection

P_{ns} = Nominal shear strength of connection

$= 2.7t_1dF_{u1}$ (*Eq.* E4.5.1-2)

P_{nov} = Nominal pull-over strength of connection

$= 1.5t_1d_wF_{u1}$ (*Eq.* E4.5.1-3)

where

d_w = Larger of screw head diameter or washer diameter

Ω = 2.35

Eq. E4.5.1-1 shall be valid for connections that meet the following limits:

(1) 0.0285 in. (0.724 mm) ≤ t_1 ≤ 0.0445 in. (1.130 mm),

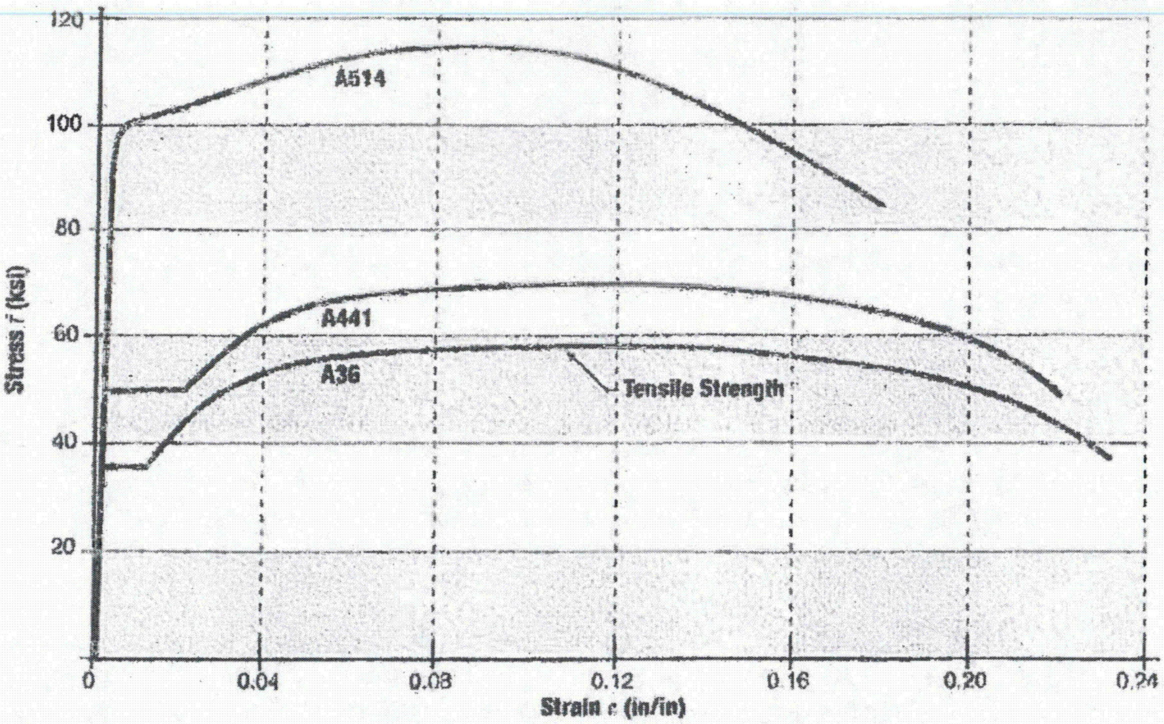


Figure B-2 Tensile stress-strain curves for three ASTM-designation steels (Brockenbrough and Johnston 1968, Tall 1974).

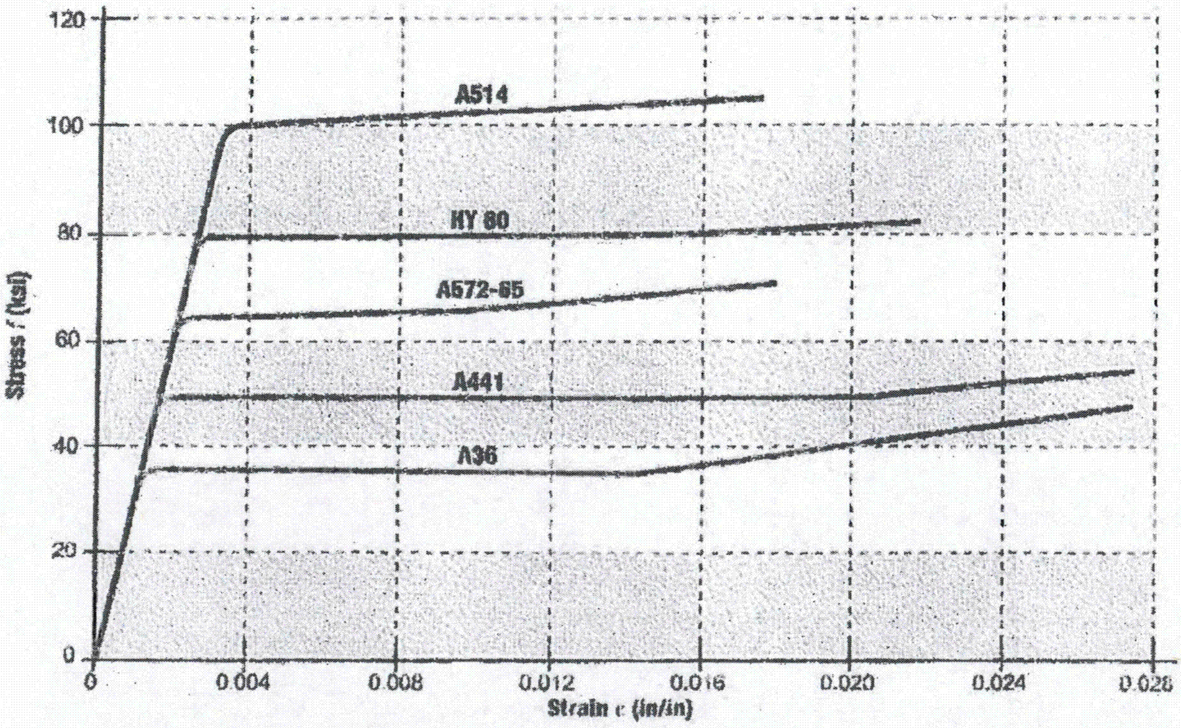


Figure B-3 Expanded yield portion of the tensile stress-strain curves (Tall 1974).

DESIGN VERIFICATION COVER PAGE

Document No. NEDC 13-028	Revision No. 1	Page 1 of 11
Title: Ultimate Internal Pressure of Turbine Building Blowout Panels and Metal Wall System		

☒ Quality Related

DV Method: ☒ Design Review ☐ Alternate Calculation ☐ Qualification Testing

VERIFICATION REQUIRED	DISCIPLINE	VERIFICATION COMPLETE AND COMMENTS RESOLVED (DV print, sign, and date)
☐	Electrical	
☐	Mechanical	
☐	Instrument and Control	
☒	Civil/Structural	Matthew Nienaber *[signature]* 7-30-15
☐	Nuclear	
☐		
☐		

Originator: (individual requesting DV)	Taylor Sutton *[signature]* 7/30/15 Print/Sign/Date After Comments Have Been Resolved

ATTACHMENT 9.6 DESIGN VERIFICATION CHECKLIST

Sheet 1 of 4

IDENTIFICATION:				DISCIPLINE:
Document Title: Ultimate Internal Pressure of Turbine Building Blowout Panels and Metal Wall System				x Civil/Structural ☐ Electrical ☐ I & C ☐ Mechanical ☐ Nuclear ☐ Other
Doc. No.:	EE 13-028	Rev. 1	QA Cat. QR	
Verifier:	Matthew Nienaber Print	Matthew Nienaber Sign	7-30-15 Date	
Manager authorization for supervisor performing Verification ☒ N/A	 Print	 Sign	 Date	
METHOD OF VERIFICATION: Design Review x	Alternate Calculations ☐		Qualification Test ☐	

The following basic questions are addressed as applicable, during the performance of any design verification. [ANSI N45.2.11 – 1974]

NOTE The reviewer can use the "Comments/Continuation sheet" at the end for entering any comment/resolution along with the appropriate question number. Additional items with new question numbers can also be entered.

1 Design Inputs – Were the inputs correctly selected and incorporated into the design?

(Design inputs include design bases, plant operational conditions, performance requirements, regulatory requirements and commitments, codes, standards, field data, etc. All information used as design inputs should have been reviewed and approved by the responsible design organization, as applicable.

All inputs need to be retrievable or excerpts of documents used should be attached. Examples may include Material Test Reports (CMTRs), verified As-Builts, approved procedures, approved drawings, publications, etc. ®[2]

See site specific design input procedures for guidance in identifying inputs.)

Yes ☒ No ☐ N/A ☐

2. Assumptions – Are assumptions necessary to perform the design activity adequately described and reasonable? Where necessary, are assumptions identified for subsequent re-verification when the detailed activities are completed?

Yes ☒ No ☐ N/A ☐

3. Quality Assurance – Are the appropriate quality and quality assurance requirements specified?

Yes ☒ No ☐ N/A ☐

4. Codes, Standards and Regulatory Requirements – Are the applicable codes, standards and regulatory requirements, including issue and addenda properly identified and are their requirements for design met?
Yes ☒ No ☐ N/A ☐

5. Construction and Operating Experience – Have applicable construction and operating experience been considered?
Yes ☐ No ☐ N/A ☒

6. Interfaces – Have the design interface requirements been satisfied and documented?
Yes ☐ No ☐ N/A ☒

7. Methods – Was an appropriate design or analytical (for calculations) method used?
Yes ☒ No ☐ N/A ☐

8. Design Outputs – Is the output reasonable compared to the inputs?
Yes ☒ No ☐ N/A ☐

9. Parts, Equipment and Processes – Are the specified parts, equipment, and processes suitable for the required application?
Yes ☐ No ☐ N/A ☒

10. Materials Compatibility – Are the specified materials compatible with each other and the design environmental conditions to which the material will be exposed?
Yes ☐ No ☐ N/A ☒

11. Maintenance requirements – Have adequate maintenance features and requirements been specified?
Yes ☐ No ☐ N/A ☒

12. Accessibility for Maintenance – Are accessibility and other design provisions adequate for performance of needed maintenance and repair?
Yes ☐ No ☐ N/A ☒

13. Accessibility for In-service Inspection – Has adequate accessibility been provided to perform the in-service inspection expected to be required during the plant life? Does this design provide adequate access to other components that require inservice inspection and testing?
Yes ☐ No ☐ N/A ☒

14. Radiation Exposure – Has the design properly considered radiation exposure to the public and plant personnel? Include review of USAR Section XII, Subsection 3.0, for affects to plant radiation zones during operation and shutdown conditions.
Yes ☐ No ☐ N/A ☒

15. Acceptance Criteria – Are the acceptance criteria incorporated in the design documents sufficient to allow verification that design requirements have been satisfactorily accomplished?
Yes ☒ No ☐ N/A ☐

16. Test Requirements – Have adequate pre-operational and subsequent periodic test requirements been appropriately specified?
    a. Are air operated components that have been added or affected by this change, tested for fail position on loss of air, or loss of electrical power?
    b. Is adequate qualification testing specified for new or unproven equipment designs?

    Yes ☐ No ☐ N/A ☒

17. Handling, Storage, Cleaning and Shipping – Are adequate handling, storage, cleaning and shipping requirements specified?

    Yes ☐ No ☐ N/A ☒

18. Identification Requirements – Are adequate identification requirements specified?

    Yes ☐ No ☐ N/A ☒

19. Records and Documentation – Are requirements for record preparation, review, approval, retention, etc., adequately specified? Are all documents prepared in a clear legible manner suitable for microfilming and/or other documentation storage method? Have all impacted documents been identified for update as necessary?

    Yes ☒ No ☐ N/A ☐

20. Software Quality Assurance- For a calculation that utilized software applications (e.g., GOTHIC, MAAP), was it properly verified and validated in accordance with 11.SQA?
    This is an 11.SQA task. However, per 3-ENS-DC-126, for exempt software, was it verified in the calculation?

    Yes ☐ No ☐ N/A ☒

21. Has adverse impact on peripheral components and systems, outside the boundary of the document being verified, been considered?

    Yes ☒ No ☐ N/A ☐

22. Are the latest applicable revisions (including pending data)of design documents utilized? ©[1,2]

    Yes ☒ No ☐ N/A ☐

23. Has this design adequately considered hazards such as missiles, jet impingement, etc.?

    Yes ☒ No ☐ N/A ☐

24. Has the design adequately considered seismic requirements, barge impact, and Mark I loadings as appropriate?

    Yes ☐ No ☐ N/A ☒

25. If this change involves a modification to a containment isolation boundary or function, have the following documents been checked for possible impact?
    a. USAR Table V-2-2
    b. USAR Section V

    Yes ☐ No ☐ N/A ☒

ATTACHMENT 9.6 DESIGN VERIFICATION CHECKLIST

Sheet 4 of 4

26. Has this design adequately addressed internal flooding vulnerability?
    a. Does the configuration change increase potential for internal flooding?
    b. Does the configuration change reduce capability to isolate or cope with flooding?
    c. Does the design locate Essential equipment where it would be susceptible to flooding?

    Yes ☐ No ☐ N/A ☒

27. Has the design adequately considered CNS Computer System interfaces (PMIS, Annunciator, Security, Simulator)?

    Yes ☐ No ☐ N/A ☒

28. Has the design adequately addressed its impact on CNS Emergency Operating Procedures and the Emergency Procedure Guidelines?

    Yes ☐ No ☐ N/A ☒

29. For electrical and IAC changes, the results of the point-to-point analysis indicate the design satisfies the specified design requirements and fully satisfies applicable topics listed on this attachment?

    Yes ☐ No ☐ N/A ☒

30. Do all drawings affected by the modification have DIR versions created and the information is properly documented?®[1,2]

    Yes ☐ No ☐ N/A ☒

31. If applicable, has the Classification Evaluation (CE) been reviewed for completeness and technical accuracy with regard to the Structure, System, or Component (SSC) design licensing requirements?
    a. Are all question responses and required information correct with regard to the design basis?
    b. Is justification to each Equipment Application Data Sheet YES/NO answer documented on the Equipment Application Analysis Sheet and is it comprehensive and sensible?
    c. Are all CE equipment applications properly classified?

    Yes ☐ No ☐ N/A ☒

32. Did this IDV include a review of all calculations affected and/or implemented by the package?

    Yes ☒ No ☐ N/A ☐

33. Did this IDV review all SAP Notifications written for document changes to ensure consistency with the proposed design?

    Yes ☐ No ☐ N/A ☒

34. Have all CNS Computer support and other considerations been adequately addressed?

    Yes ☐ No ☐ N/A ☒

35. Have all design and configuration documents which were required to be created or revised as a result of this activity been reviewed?

    Yes ☒ No ☐ N/A ☐

ATTACHMENT 9.7 DESIGN VERIFICATION COMMENT SHEET

Sheet 1 of 1

REVIEWER COMMENT/RESOLUTION RECORD

Document: NEDC 13-028 Rev 1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
1	Minor editorial comments provided in markup	MBN 7.9.15	Comments/editorial corrections have been incorporated.	TES 7-13-15	MBN 7-30-15	

REVIEWER COMMENT/RESOLUTION RECORD

Document: NEDC 13-028 Rev 1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
2	Attachments E2 and E3: Explain Fyc component and possibly delete the ultimate strength evaluation	MBN 7.9.15	Fyc is the ratio of plastic hinges. This value is used to account for the different plastic moment capacities and cross sections of the girt channel and connection angles that are evaluated in the plastic beam analysis. This shows that the connection angles will develop plastic hinges before a plastic hinge develops in the girt channel. This comment has been added to E2 and E3. Also, the ultimate strength evaluation in section 2.3 has been removed (it is not pertinent to the evaluation).	TES 7-13-15	MBN 7-30-15	
3	Attachment E4: M_p equation is not given in the 6th edition.	MBN 7.9.15	See page 2-6 in the 6th edition. No change.	TES 7-13-15	MBN 7-30-15	

REVIEWER COMMENT/RESOLUTION RECORD

Document: NEDC 13-028 Rev 1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
4	Attachment E4, pg 3: The reviewer understands that tension generated by catenery action is a function of geometry and load, not section properties. If this is incorrect, provide a reference.	MBN 7.9.15	The reviewer's comment is correct. The point of pg.3 and pg.4 is to show the tensile (and compression) forces in the angles when the plastic moment capacity is achieved and to determine the lever arm length between the tensile stress block and the compression stress block. The level arm is then used to determine the tensile load in the angle when 0.5 psi of pressure is applied to the Turbine Building siding. No change.	TES 7-13-15	MBN 7.30.15	
5	Attachment E4 pg 6: Bolt hole diameters are given on sheets 156, 157, 158 and 159 of drawing 9150. These are not the same as what was used.	MBN 7.9.15	Comment incorporated.	TES 7-13-15	MBN 7.30.15	

REVIEWER COMMENT/RESOLUTION RECORD

Document: NEDC 13-028 Rev 1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
6	Attachment E4 pg 7: The purpose of this section is unclear. It appears to be either a bolt tear out calculation or a base metal tear out calculation, both of which need to be evaluated	MBN 7.9.15	This failure mechanism is the bolt tearing through the base material. A statement has been added for clarification. A base material check for the weld has also been added in the Weld Rupture evaluation. Comment incorporated.	TES 7-13-15	MBN 7-30-15	
7	Attachment E4 pg 9: 0.62 Fu would be a better value for ultimate shear strength	MBN 7.9.15	After reviewing the document that you referenced (AISC Design Guide 17), it appears that 0.5 of Fu is used for nominal shear capacity of bolts. This is also contained in the AISC Manual (13th Ed.), Table J3.2. Page 9 has been revised to reflect this comment.	TES 7-14-15	MBN 7-30-15	

REVIEWER COMMENT/RESOLUTION RECORD

Document: NEDC 13-028 Rev 1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE
8	Attachment E4 pg 9: Drawing DIR 450015290 indicates that the bolts used in this evaluation are machine bolts.	MBN 7.9.15	Agreed; a note has been added to this evaluation page.	TES 7-13-15		
9	Attachment E4 pg 10: Because this is a transverse weld, the weld strength follows a different relationship. Also, F_{E70} needs to be reduced by 0.6	MBN 7.9.15	Per the design guide (Structural Steel Design) used at that time, a 0.6 factor for shear loading is not used. However, this section has been rewritten to follow the methodology more closely, such as using the allowable weld strength of 15.8 ksi and removing the F.S. from the equation. Also, FE70 was used for A36 steel under contract E69-15. A note was added to this section to provide some clarify.	TES 7-15-15	MBN 7-30-15	

REVIEWER COMMENT/RESOLUTION RECORD

Document: NEDC 13-028 Rev 1

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
10	The high amount of strain present may lead to a rupture failure in the connection angle that is plastically deformed. Please discuss	MBN 7.30.15	An additional paragraph was added to the calc body that discusses this failure mechanism but is conservatively ignored.	TES 7-30-15	MBN 7.30.15	

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

CALCULATION COVER PAGE	(5) CALCULATION NO: NEDC 13-028 (6) REVISION/Change Notice No: 1	(20) Effective Date: 7/30/15 (2) Page 1 of 11

(1) EC #: EE 13-041	(7) Title: **Ultimate Internal Pressure of Turbine Building Blowout Panels and Metal Wall System**	
(3) Design Basis Calc: ☒ YES ☐ NO	(9) System(s)/Structure: Turbine Building	(10) Discipline: Civil/Structural
(11) Safety Class: ☒ Quality Related ☐ Non-Quality Related	(12) Component/Equipment/Structure: Turbine Building, Panels, Girts, Angles, Bolts, Welds	
(18) Proprietary: ☐ YES ☒ NO		
(4) Superseded: ☐ YES ☒ NO		

(14) Keywords (Description/Topical Codes):
Turbine Building, Pressure, Blowout Panels, FSAR Amendment 25, Catenary

(8) Calculation Description:

This calculation is the design basis document that reconstitutes the CNS FSAR Amendment 25 Turbine Building Blowout Panel pressure of 0.5 psig. A postulated Main Steam Line break in the Turbine Generator Building, or one which breaches the Reactor Building Steam Tunnel blowout panels, is capable of creating a differential pressure between the Turbine Generator Building and the outside environment. This calculation shows how the Amendment 25 differential pressure value at which the Turbine Generator Building steel superstructure siding experiences structural failure is achieved.

(13) Conclusion/Recommendations:

A simplified, conservative method was demonstrated for determining the maximum pressure value taken from the FSAR Amendment 25.

In this analysis, the connecting bolts are shown to be the weakest component in the wall system. As the beam develops plastic deformation in the connections, its behavior will change from beam bending to catenary action. This will result in a major increase in the axial load on the angles, which will cause shear failure of the connecting bolts.

REVIEWS

(15) Name/Signature/Date	(16) Name/Signature/Date	(17) Name/Signature/Date
Taylor Sutton 7/27/15 Taylor Sutton **Responsible Engineer**	Matthew Nygaard Matthew Nygaard 7-30-15 ☒ **Design Verifier** ☒ **Technical Reviewer** ☒ Comments Attached	Alan L Able Alan L Able 7-30-15 **Supervisor/Approval** ☐ Comments Attached

CALCULATION REFERENCE SHEET	CALCULATION NO: NEDC 13-028 REVISION/Change Notice: 1

I.a. Change Notices Incorporated
List: 0C1

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. 4088	1	00/AA	☒	None	☐	N	N/A
2.	DWG. 4089	2	00/AA	☒	None	☐	N	N/A
3.	DWG. 9150	E106	00/AA	☒	None	☐	N	N/A
4.	DWG. 9150	E107	00/AA	☒	None	☐	N	N/A
5.	DWG. 9150	10	00/AA	☒	None	☐	N	N/A
6.	DWG. 9150	133	00/AA	☒	None	☐	N	N/A
7.	DWG. 9150	156	00/AA	☒	None	☐	N	N/A
8.	DWG. 9150	157	00/AA	☒	None	☐	N	N/A
9.	DWG. 9150	158	00/AA	☒	None	☐	N	N/A
10.	DWG. 9150	159	00/AA	☒	None	☐	N	N/A
11.	DWG. 49054	2N	00/AA	☒	None	☐	N	N/A
12.	DWG. 49054	2R	00/AA	☒	None	☐	N	N/A
13.	DWG. 49054	2T	00/AA	☒	None	☐	N	N/A
14.	DWG. 49054	2W	00/AA	☒	None	☐	N	N/A
15.	DWG. 49054	6T	00/AA	☒	None	☐	N	N/A
16.	DWG. 49054	7T	00/AA	☒	None	☐	N	N/A
17.	DWG. 49054	8T	00/AA	☒	None	☐	N	N/A
18.	DWG. 49054	9T	00/AA	☒	None	☐	N	N/A
19.	DWG. 49054	9W	00/AA	☒	None	☐	N	N/A
20.	DWG. 49054	10T	00/AA	☒	None	☐	N	N/A
21.	DWG. 49054	10W	00/AA	☒	None	☐	N	N/A
22.	DWG. 49054	11T	00/AA	☒	None	☐	N	N/A
23.	DWG. 49054	12T	00/AA	☒	None	☐	N	N/A
24.	FSAR Amend. 25	N/A	N/A	☒	None	☐	N	N/A
25.	EE 13-041	N/A	2	☐	None	☒	Y	N/A

III. REFERENCES:

1. "Manual of Steel Construction," Sixth Edition (1967), published by American Institute of Steel Construction, Inc.
2. "Manual of Steel Construction," Thirteenth Edition (2005), published by American Institute of Steel Construction, Inc.

3. "Plastic Design in Steel - A Guide and Commentary," Manuals and Reports on Engineering Practice No. 41 (1971). published by American Society of Civil Engineering 245 East 47th Street, New York
4. "Engineering Mechanics Statics and Dynamics," prepared by J.L. Meriam, published by John Wiley Sons, New York
5. "World Trade Center Building Performance Study," published by Federal Emergency Management Agency, Washington, D.C., Document No. FEMA 403, May 2003
6. "Federal Building and Fire Safety Investigation of the World Trade Center Disaster. Mechanical Properties of Structural Steels", NIST NCSTAR. 1-3D. Lueke et al., U.S. Department of Commerce, September 2005
7. "North American Specification for the Design of Cold-Formed Steel Structural Members," 2007 Edition, published by American Iron and Steel Institute
8. "Structural Steel Design," First Edition (1965). prepared by Jack McCormac, published by International Textbook Company
9. "Steel Design Guide Series 7 - Industrial Buildings: Roofs to Column Anchorages," First Edition (1993), prepared by James M. Fisher, published by American Institute of Steel Construction, Inc.
10. "ASTM A307: Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength," 1978 Edition, published by American Society for Testing and Materials

1

IV. SOFTWARE USED: None

Title:__________ Version/Release:______ MSI No.______

V. DISK/CDS INCLUDED: None

Description of Contents:__________

VI. OTHER CHANGES: None

Revision	Record of Revision
0	Initial issue.
0C1	This CCN corrected the south walls of the Turbine Building plastic hinge girt analysis for the different connections as identified in CR-CNS-2014-00801. The method used in the original issuance of this calculation was used again to determine the pressure at which the wall panel system will fail.
1	This revision specifically evaluates the different failure mechanism at the connection angles that could occur before the weld connection ruptures, and also incorporates change notice 0C1.

4.0 Table of Contents

5.0 Purpose

This calculation reconstitutes the design basis document that validates the Turbine Building Blowout Panel pressure of 0.5 psid as stated in FSAR Amendment 25. The pressure at which the Turbine Building walls blow out is important to vent and release steam to the atmosphere following a Main Steamline break.

Revision 1 provides additional detail pertaining to the controlling failure mechanism that will cause the blowout panels to be released from the Turbine Building north and south walls before the angle connection welds will rupture.

6.0 Conclusion

The Turbine Generator Building (TGB) metal wall panel capacity for an internal pressure is limited by the ultimate strength of the connection angles on the north and south walls between column lines D, E and F. Analyses were performed to determine the controlling material failure of the connection angles after plastic hinges are developed due to the internal pressure on the wall system. The analyses conclude that the girt channel connection bolts will shear due to the induced tensile force in the connection angles when a 0.5 psid pressure is present in the Turbine Building. The shear failure of the connecting bolts causes the release of sections of panel wall siding from the Turbine Building, which will result in 6,576 ft^2 of vent area to the atmosphere.

7.0 Input and Design Criteria

CNS Safety Analysis Report

FSAR Amendment 25: 0.5 psid Turbine Building siding blowout pressure

Design Codes

AISC Manual of Steel Construction, 6th Ed.

AISC Manual of Steel Construction, 13th Ed.

AISI North American Specification for the Design of Cold-Formed Steel Structural Members, 2007 Ed.

Standards

ASTM A307: Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength, 1978 Ed.

Physical/Measured Components

Girt Channels: C10x15.3 (A36)

Connection Angles: L5x3-1/2x3/8, L4x3x3/8 (A36)

Connecting Bolts: 3/4" machine bolts (A307, Gr.A)

Angle Welds: 1/4", 5/16" (F_{E70})

Performance Requirement

Turbine Building siding will blowout at the FSAR defined blowout pressure thereby venting released steam and moisture to the atmosphere, completely relieving the confined space.

Other Documents

All other documents used as inputs to the analyses are contained in the Calculation Reference Sheet.

8.0 Assumptions

There are no assumptions.

9.0 Method of Analysis

Although the USAR does not specify the plastic analysis method, it is, however, contained in the AISC Manual of Steel Construction, 6th edition, which is reference in the USAR.

This analysis does not consider dynamic effects. Being equal to or less than the speed of sound at ambient conditions, HELB pressure wave effects are insufficient in duration to effect the high strain rates needed to reconsider the published mechanical properties of structural members [Ref. R6].

The design methodology used is plastic design. Looking at the different types of connections, the most susceptible are the connections between columns D, E, and F on the north and south walls. The angle steel elements used in these connections are smaller than any others in the building. As documented in CR-CNS-2014-00801, the angle connections on the north and south walls are not identical; however, the analysis shows that both walls will experience failure at or before an internal pressure of 0.5 psid. The pressure is spread across a 7ft (84in) span. The 7ft spans are not the longest span, but as they occur in a group of four when the interior panels move into membrane action, the tributary area will be half of both neighboring spans.

The flexural load and section geometry differences result in plastic deformation occurring at the connection angles. At this point, no further flexural load may be carried by the angles. There may be some strain hardening or load redistribution that allows the angles to carry tension load only, but is countered by catenary action.

When the loads develop moments larger than the section's plastic moment, deflection will take place with no further load increases. The beam behavior will follow catenary action. This causes an increase in tension in the connection angles. The connecting bolts will shear prior to weld failure, as documented in Attachment E4. This releases the stress suddenly, causing a zipper like failure vertically along the column bays, as well as a release of the internal pressure. The deflection at maximum applied pressure is determined by Slope Deflection, as documented in Attachments E2 and E3.

10.0 Calculations

The load is uniformly distributed over the interior wall panels (Inryco designation L10) which are attached to the girts with #14 screws (see Design Input 14). The spans between girts vary by several feet, but most of them are 7ft. The interior panels are joined together through the interlocking flanges with a screw. The screw is anchored in a sub-girt. The sub-girt is spaced at approximately 4ft intervals. The exterior panels (Inryco designation IW21) are also secured to the sub-girt with #12 screws (see Design Input 14). 1 1

The system was tested with a mock-up by the metal wall manufacturer. A wall system 7ft by 14ft was built and tested with a vacuum pressure on the exterior panel. The test was recorded (see Attachment B); the results show that the exterior panels pop off at 0.54 psid with no damage done to the interior panels. A plastic sheet between the two panels causes most of the differential pressure to be borne by the exterior panel once small leaks develop in the interior panels. Small leaks will not establish a vent path capable of releasing the pressure. This test shows that the interior panels are strong enough to withstand a pressure of 0.54 psid.

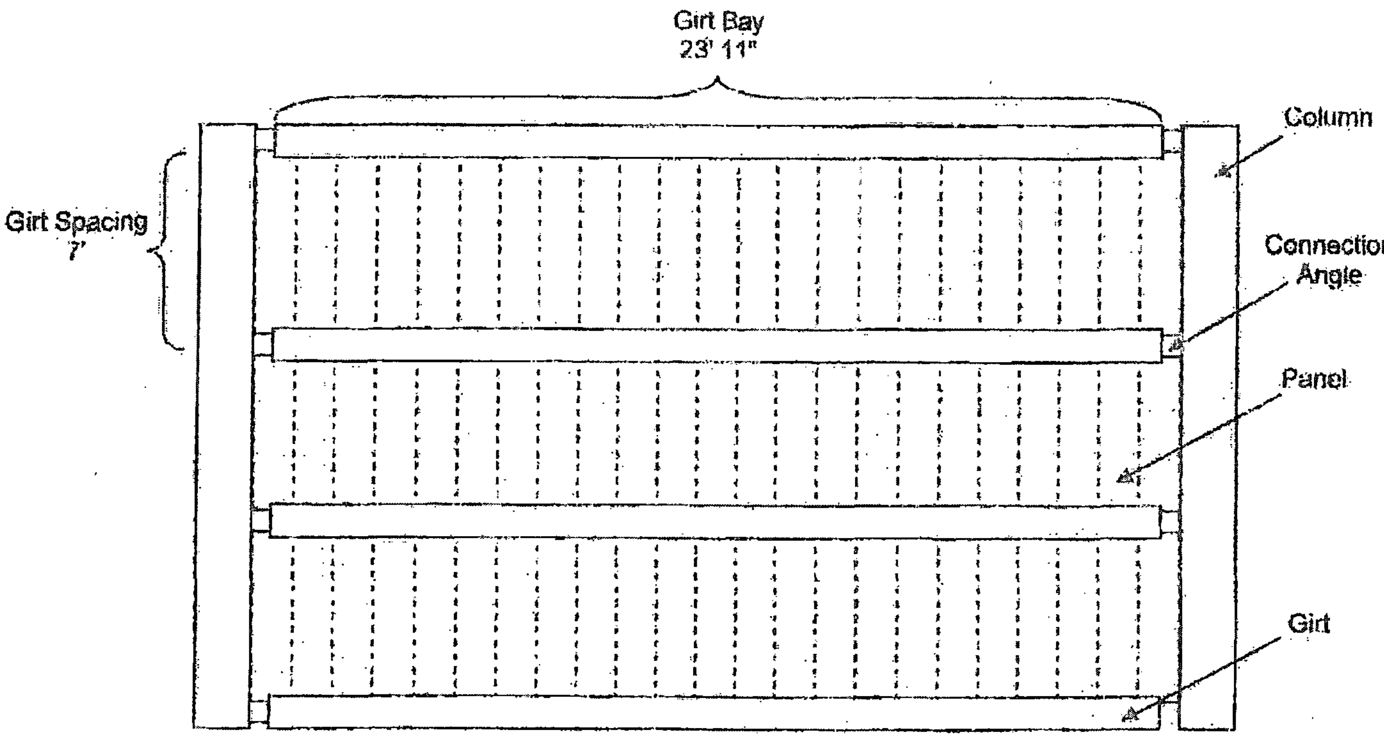


Figure 1: Typical Wall Section View

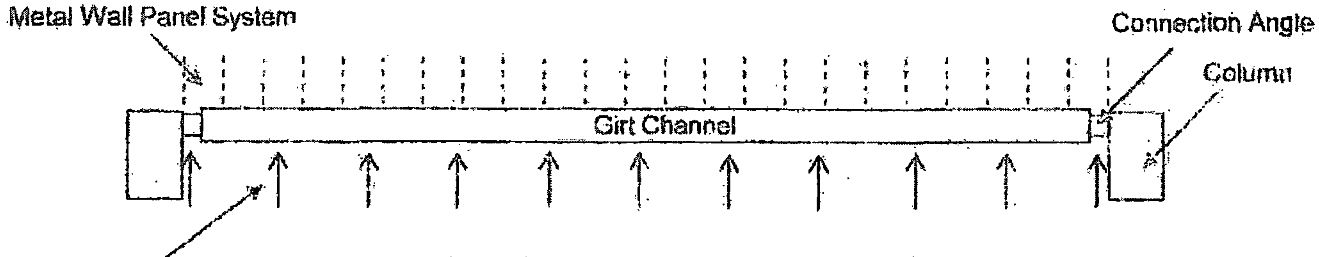


Figure 2: Typical Girt Top View

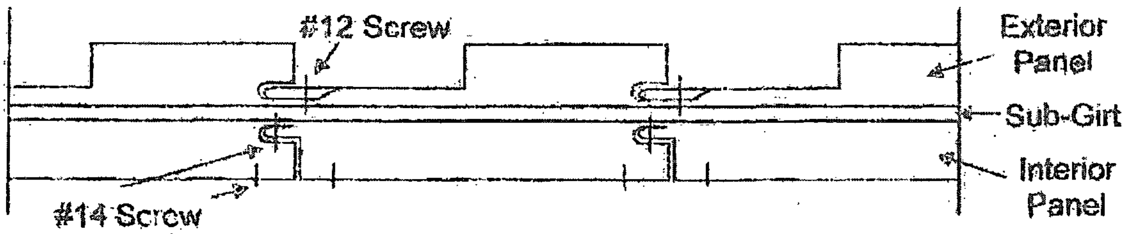


Figure 3: Typical Sub-Girt Cross Section

The interior panel design sheets were provided by the manufacturer (see Attachment A). These sheets give an allowable pressure near 0.5 psid for a triple span condition. The as-installed span condition is more than 3 spanning sections, but this difference is small. As the edges of the panel are supported continuously by the neighboring panels, the wall will distort, but no panels will open to establish a vent path for interior pressure. The behavior of the panels will then switch to that of a membrane. This greatly increases the carrying capacity of the member.

The pressure put on the panels will force the screws that tie the panels to the girts into tension. A single panel is fastened to the girt with two #14 stainless steel screws. For metal buildings and walls such as this, failure is typically initiated at these connections either by screw pullout of the base material, screw tensile rupture, or sheet pullover. These failure modes are evaluated in Attachment E1, with the lowest being a pressure of 5.6 psid. Failure of the panels will not occur, therefore, by this mode. 1

Beam material will change behavior when operating beyond the elastic range or elastic buckling range due to excessive loading. A plastic hinge will form when yield stress is passed. This causes a redistribution of the moments on the beam and a consequential change in the beam behavior. When the beam reaches its plastic moment (Mp), any further increase in load will cause continued deformation. When the deflection is large enough, the beam will no longer be able to carry the load through bending and will have to carry it in tension through catenary action. This will result, in this case, in plasticizing of the connection angles that causes shearing of the bolts. 1

The Turbine Building structural steel columns are designed for 40 psf with a 0.6 safety factor (as seen in B&R Book 58). If the safety factor is removed this will result in a yielding pressure of about 0.5 psid. These beams are very large and will not fail immediately at 0.5 psid, due to their depth, and will not allow the pressure to be released. The columns are rigid enough to resist the catenary action of the beam.

Several different beam configurations are used in the girts of the metal walls of the Turbine Building. As seen in B&R Book 58, the girts are designed for a 40 psf distributed load. Like item 6 above, with the safety factor removed the yield pressure is near 0.5 psid. The book's analysis is bounding for the longest girt.

The bolts connecting the girt channels to the connection angles are the weakest part of the loading chain. The connection angles between column lines D, E and F in the north and south walls are different. The north wall connection angles are L4x3x3/8, and the south wall connection angles are L5x3-1/2x3/8. The girt channels are C10x15.3 for both walls between column lines D, E and F. Shear loads, caused by the internal pressure acting normal to the metal walls, are not capable of causing failure.

This calculation demonstrates the methodology used to reconstitute the 0.5 psid value presented in the FSAR Amendment 25. The connecting bolts will shear before a pressure 0.5 psid is present in the Turbine Building, as seen in Attachment E4. Therefore, the value is conservative.

Catenary action is initiated at the development of a full plastic moment at the end connection angles resulting in failure. This evaluation also demonstrates a tensile force from catenary action. This calculation does not explicitly consider the possibility of strain-hardening and load distribution. The large catenary tension force demonstrates the conservatism in this methodology. The conclusion is the same for both cases: 1) catenary action is initiated below 0.5 psid and; 2) when catenary action begins, tensile loads increase rapidly and shear the bolts.

Attachment E3 contains the evaluation performed in change notice 0C1. This change notice was performed to evaluate the south wall connection angles.

Attachment E4 contains an evaluation of the material failure mechanisms. The result of the evaluation determined that the shear failure of the two 3/4in A307 bolts connecting the girt channel to the connection angles is the controlling failure of the metal panel wall system.

The analysis in Attachment E4 does not evaluate the strain levels of the connection angle at the location where the plastic hinge is developed. An advanced fracture mechanics evaluation could be performed to show that the angle experiences fracture when the strain limits of the material (A36) are exceeded before full catenary behavior is developed in the girt channel. Further analysis could prove exceeding the angle material strain limit is the controlling failure mechanism in system; however, strain limits were conservatively ignored. Stress evaluations were performed using standard structural engineering design methodologies.

The loss of a connection angle (bolts) causes a redistribution of the loads on the whole girt bay. Loads supported initially by the broken connection are distributed to the connections above and below, which then fail under increased load. The combined vent path area created on the north and south walls of the Turbine Building, due to the shear failure of the connection angle bolts, is 6,576 ft^2.

Attachments:

- E1. Panel Fastener Evaluation (pages E1-1 to E1-2)
- E2. North Wall Plastic Evaluation (pages E2-1 to E2-3)
- E3. South Wall Plastic Evaluation (E3-1 to E3-4)
- E4. Failure Mechanisms Evaluation (E4-1 to E4-12)
- A. Inryco Wall Systems Technical Data sheet for L10 Series Liner Panels (pages A1-A2)
- B. Vacuum Load Test (pages B1-B28)
- C. Plastic Design in Steel *excerpt* (pages C1-C13)
- D. Engineering Mechanics Statics and Dynamics *excerpt* (pages D1-D6)
- E. Fastener equations from reference R7
- F. Figure B-2 from reference R5

Evaluation 1

Modern AISI code (07) is used as it gives a more thorough treament of fasteners citing more recent research.

The screws which fasten the panels to the structural girt are evaluated below. These will be subjected to a tensile force which may cause:

Pull-over : The sheet metal will rip over fasteners
Pull-out : The screw will rip out of the structural girt
Tensile failure: The screw will fail in tension

1.1 Pull over

$t1 := 0.048$	in: 18 gage steel
$dw := 0.5$	in : width across screw head (AISI equation max)
$Fu1 := 90$	ksi ultimate strength of sheet stee (Attachment A)
$Pn\_ov := 1.5 \cdot t1 \cdot dw \cdot Fu1 = 3.2$	kips

The bolt heads cannot be directly observed and the true ultimate strength is unknown so conservative assumptions have been made.

1.2 Pull out

$tc := \frac{7}{16}$	in: depth of penetration (flange thickness of C9x13.4)
$d := 0.242$	in: Nominal diameter for #14 screw
$Fu2 := 58$	ksi: Tensile strength of Structural Steel
$Pn\_ot := 0.85 \cdot tc \cdot d \cdot Fu2 = 5.2$	kip

1.3 Tensile failure

$Fus := 90$	Tensile strength, assumed from 316 SS
$d1 := 0.242$	in: nominal diameter
$A := \frac{\pi (d1)^2}{4} = 0.046$	in^2: Tensile cross section
$Pts := Fus \cdot A = 4.1$	kips

1.4 Failure Pressure

Span is assumed 7' 0". Strength of a fastener group is assigned to the support reaction. "6 span condition is used. The lowest pressure evaluation for screw strength is used.

$R := 2 \cdot Pn\_ov = 6.5$ kips (2 screws installed per panel to girt)

$L := 7 \cdot 12 = 84$ in

$R := w \cdot L \cdot \frac{(65 + 55)}{104}$ Ref R2, 6 span condition

$wf := \frac{R}{1.154 \cdot L} = 0.067$ $\frac{kips}{in}$

$Pf := \frac{1000wf}{12} = 5.6$ psi

Using the minimum strength of the fasteners, the screws fail in tension when the interior pressure reaches 5.6 psi.

Evaluation 2.

2.1 Given:

E := 29000000	psi : modulus of elasticiy	Channel C10x15.3		
Fy := 36000	psi : yield stress (ref R1)	Ixx := 66.9	in4 : moment of inertia (ref R1)
Fu := 55000	psi : ultimate stress	L := 287	in : girt length
Ls := 84	in : girt spacing	εs := 0.12	in/in : ultimate strain (ref R5, Fig B-2)
La := 4.5	in : length of angle	εp := 0.01	in/in : plastic strain (ref R5, Fig B-2)
FE70 := 70000	psi : ultimate weld strength		

Section C10x15.3: Member 2	Section L4x3x3/8: Member 1/3	
Axxc := 4.47	Axxl := 2.48	in2 : Area (ref R1)
Zxc := 15.9	Zxl := 2.6	in3 : Plastic section Modulus (ref R2)
Mp2 := Fy·Zxc = 572400	Mp1 := Fy·Zxl = 93600	lbs-in: Plastic moment

2.2 Using the Mechanism method from reference R3 the max pressure is evaluated. The main beam of the girt does not yield. The load is balanced to find the bending moment in the middle of the elastic beam when the angle connections reach plastic moment (Mp).

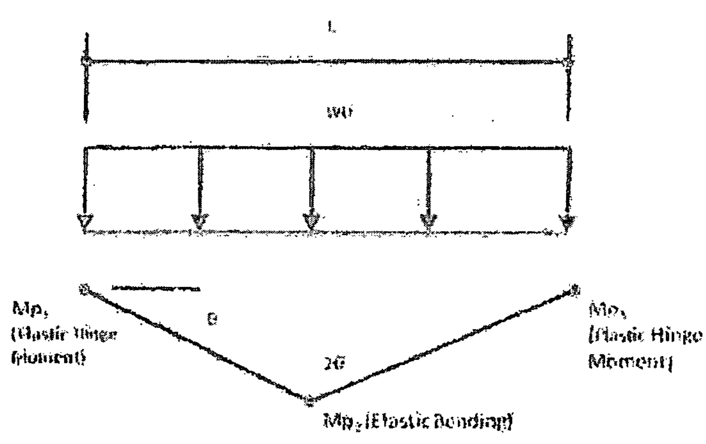


Fig. 3 Girt Free Body Diagram

$$Fyc := 2\frac{Mp1}{Mp2} = 0.327$$

Fyc = Ratio of Plastic Hinges

Note: This value is used to account for the different plastic moment capacities and cross sections of the girt channel and the connection angles. Due to the smaller capacity of the connection angles, the girt channel will not develop a plastic hinge when plastic hinges develop in the connection angles.

2.3 Determination of maximum pressure applied to the Girt based on external versus internal work for upper limit condition:

Where $We = Wi$ (ref R3, Art 3.2)

$We = wu*L*(\theta/2)*(L/2) = (wu*L2*\theta)/4$

$Wi = Mp1*\theta+Mp3*\theta+2*Fyc*Mp2*\theta$

$Mp1 = Mp3$

Fy

$Wip := 2 \cdot Mp1 + 2 \cdot Fyc \cdot Mp2 = 561600 \cdot \theta$

$$wup := \frac{(4 \cdot Wip)}{L^2} = 27.3 \quad \frac{lbs}{in}$$

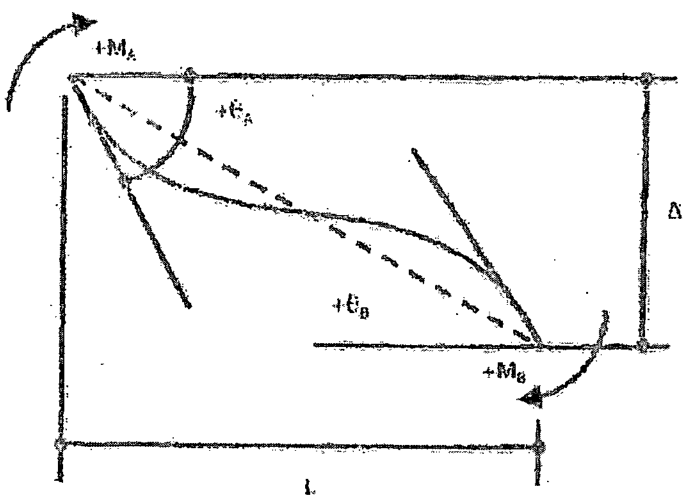
$$Pp := \frac{wup}{Ls} = 0.3 \quad \frac{lbs}{in^2}$$

This demonstrates that the USAR load of 0.5 psi can be obtained. True failure occurs at 0.3 psi. Accuracy should not be given to more than 1 significant figure due to uncertainties in yield stress.

2.4 Determination of the deflection at mid-span using slope-deflection on a beam using uniform loading. Maximum deflection occurs at the center of the girt.

Where:

$\theta A = \theta A' + \Delta/L + L/(3 * E * I) * (MAB - MBA/2)$ (ref R3, Art 9.4)



Note: θA' is a support displacement and is ignored in this evaluation.

θ21 = 0 + δV2/(L/2) + (L/6EI)(-Mp2 + Mp1/2)
θ23 = 0 - δV2/(L/2) + (L/6EI)(Mp2 - Mp3/2)

θ21 = θ23
2δV2/L + (L/6EI)(-Mp2 + Mp1/2) = -2δV2/L + (L/6EI)(Mp2 - Mp3/2)
2δV2/L + 2δV2/L = (L/6EI)(Mp2 - Mp3/2) - (L/6EI)(-Mp2 + Mp1/2)
4δV2/L = (L/6EI)(2Mp2 - Mp1/2 - Mp3/2)
δV2 = (L²/24EI)(2Fyc*Mp2 - Mp1/2 - Mp3/2)

$$\delta v2p := \frac{L^2 \cdot (2Fyc \cdot Mp2 - Mp1)}{(24 \cdot E \cdot Ixx)} = 0.5$$

2.5 To estimate the deflection beyond yield, Es is determined based on the angle permanent deformation strain value. The values for ε are taken from the stress-strain curve from ref R5.

$$\delta sp := 2\varepsilon p \cdot La = 0.09$$

$$\delta p := \delta v2p + \delta sp = 0.59$$

2.6 Determination of the applied tension at support ends as a result of the catenary action which occurs when Mp is achived; the beam deflection from Section 2.5 is used.

$$Tp := \frac{wup \cdot L^2}{8 \cdot \delta p} = 478580$$ lbs (ref R4)

Evaluation 3

Given:

E := 29000000	psi : modulus of elasticiy	Channel C10x15.3	
Fy := 36000	psi : yield stress (ref R1)	Ixx := 66.9	in4 : moment of inertia (ref R1)
Fu := 55000	psi : ultimate stress	L := 287	in : girt length
Ls := 84	in : girt spacing	εs := 0.12	in/in : ultimate strain (ref R5, Fig B-2)
La := 4.75	in : length of angle	εp := 0.01	in/in : plastic strain (ref R5, Fig B-2)
FE70 := 70000	psi : ultimate weld strength		

Section C10x15.3: Member 2	Section L5x3.5x3/8: Member 1/3	
Axxc := 4.47	Axxl := 3.05	in2 : Area (ref R1)
Zxc := 15.9	Zxl := 4.09	in3 : Plastic section Modulus (ref R2)
Mp2 := Fy·Zxc = 572400	Mp1 := Fy·Zxl = 147240	lbs-in: Plastic moment

Using the Mechanism method from reference R3 the max pressure is evaluated. The main beam of the girt does not yield. The load is balanced to find the bending moment in the middle of the elastic beam when the angle connections reach plastic moment (Mp).

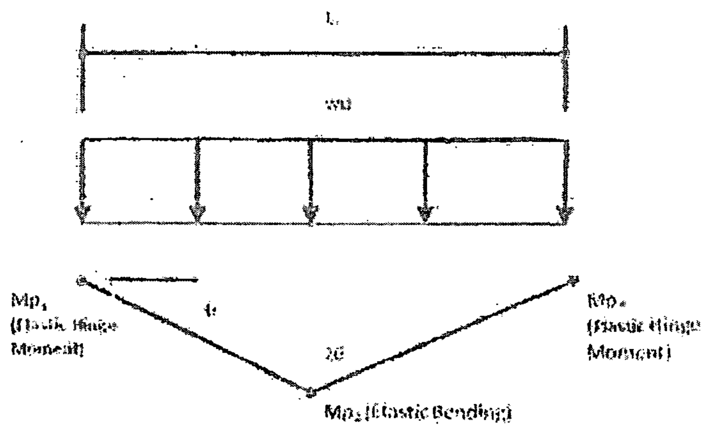


Fig. 1 Girt Free Body Diagram

$$Fyc := 2\frac{Mp1}{Mp2} = 0.514$$

Fyc = Ratio of Plastic Hinges

Note: This value is used to account for the different plastic moment capacities and cross sections of the girt channel and the connection angles. Due to the smaller capacity of the connection angles, the girt channel will not develop a plastic hinge when plastic hinges develop in the connection angles.

Determination of maximum pressure applied to the Girt based on external versus internal work for upper limit condition

Where: We = Wi (ref R3, Art 3.2)

We = wu*L*(θ/2)*(L/2) = (wu*L2*θ)/4

Wi = Mp1*θ+Mp3*θ+2*Fyc*Mp2*θ

Mp1 = Mp3

Fy

$$Wip := 2 \cdot Mp1 + 2 \cdot Fyc \cdot Mp2 = 683440 \cdot 0$$

$$wup := \frac{(4 \cdot Wip)}{L^2} = 42.9 \quad \frac{lbs}{in}$$

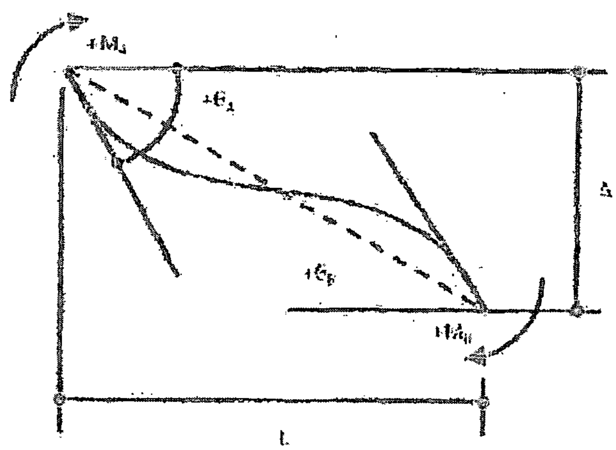
$$Pp := \frac{wup}{Ls} = 0.5 \quad \frac{lbs}{in^2}$$

This demonstrates that the USAR load of 0.5 psi can be obtained. Accuracy should not be given to more than 1 significant figure due to uncertainties in yield stress.

Determination of the deflection at mid-span using slope-deflection on a beam using uniform loading. Maximum deflection occurs at the center of the girt.

Where:

θA = θA' + Δ/L + L/(3 * E * I) * (MAB - MBA/2) (ref R3, Art 8.4)



Note: θA' is a support displacement and is ignored in this evaluation

$\theta 21 = 0 + \delta V2/(L/2) + (L/6EI)(-Mp2 + Mp1/2)$
$\theta 23 = 0 - \delta V2/(L/2) + (L/6EI)(Mp2 - Mp3/2)$

$\theta 21 = \theta 23$
$2\delta V2/L + (L/6EI)(-Mp2 + Mp1/2) = -2\delta V2/L + (L/6EI)(Mp2 - Mp3/2)$
$2\delta V2/L + 2\delta V2/L = (L/6EI)(Mp2 - Mp3/2) - (L/6EI)(-Mp2 + Mp1/2)$
$4\delta V2/L = (L/6EI)(2Mp2 - Mp1/2 - Mp3/2)$
$\delta V2 = (L^2/24EI)(2Fyc{\cdot}Mp2 - Mp1/2 - Mp3/2)$

$$\delta v2p := \frac{L^2 \cdot (2Fyc \cdot Mp2 - Mp1)}{(24 \cdot E \cdot Ixx)} = 0.78$$

To estimate the deflection beyond yield, Es is determined based on the angle permanent deformation strain value. The values for ε are taken from the stress-strain curve from ref R5.

$$\delta sp := 2\varepsilon p \cdot La = 0.095$$

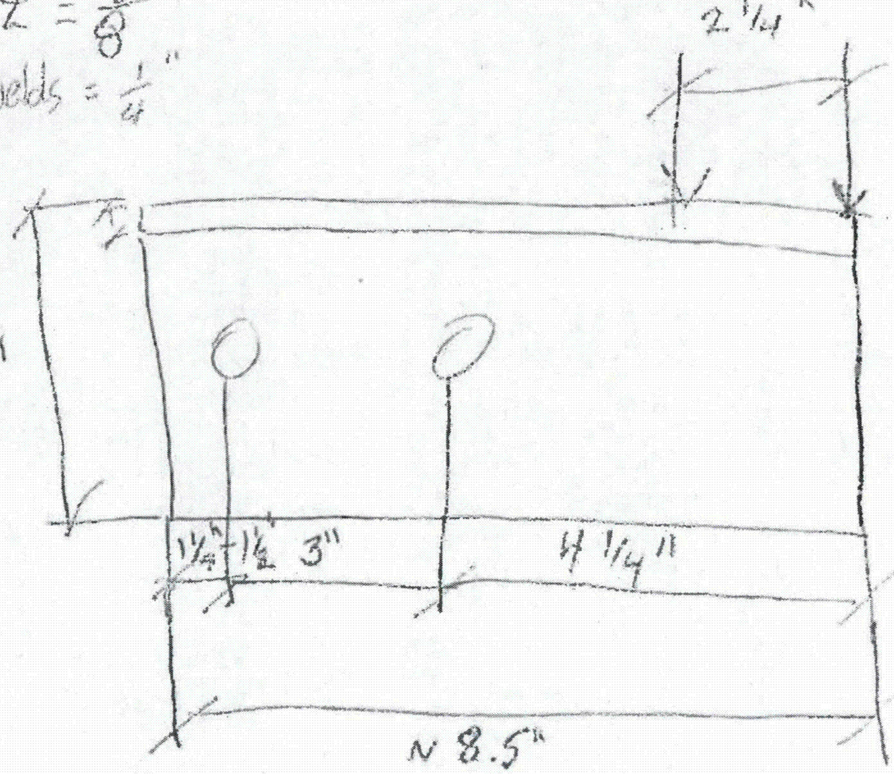
$$\delta p := \delta v2p + \delta sp = 0.88$$

Determination of the applied tension at support ends as a result of the catenary action which occurs when Mp is achived; the beam deflection from Section 2.5 is used.

$$Tp := \frac{wup \cdot L^2}{8 \cdot \delta p} = 504014 \qquad \text{lbs (ref R4)}$$

1

height = 3.5"
t = 3/8"
welds = 1/4"



Measured at 948'-8" south turbine building wall on the east side of "F" column line

Evaluation 4

Moments on Girt Channel:

Girt Length (L)

$L = 287$ in — Ref: DWG. 9150 Sht. 123

Applied Pressure (P)

$P = 0.5$ lb/in^2 — Ref: FSAR Amendment 25

Girt Spans (S)

$S = 84$ in — Ref: DWG. 9150 Sht. E106, E107

Distributed Load (w)

$w = P \times S = 42$ lb/in

Fixed-Fixed End Conditions

Load Induced Moments: — Ref: AISC Manual, 6th Ed.

Midspan Moment = $wL^2/24$ = 144146 lb-in → 144 kip-in

End Moments = $wL^2/12$ = 288292 lb-in → 288 kip-in

Pinned-Pinned End Conditions

Load Induced Moments: — Ref: AISC Manual, 6th Ed.

Midspan Moment = $wL^2/8$ = 432437 lb-in → 432 kip-in

End Moments = 0

Moment Capacities for Each of the Given Cross Sections:

F_y = 36 ksi F_u = 58 ksi (for all cross sections)

$M = Fy \times S_x$ Ref: AISC Manual, 6th Ed.

$M_p = Fy \times Z_x$ Ref: AISC Manual, 6th Ed.

Cross Section	S_x	M	Z_x	M_p
C10x15.3	13.4 in^3	482.4 kip-in	15.9 in^3	572.4 kip-in
L5x3-1/2x3/8	2.28 in^3	82.1 kip-in	4.09 in^3	147.2 kip-in
L4x3x3/8	1.44 in^3	51.8 kip-in	2.60 in^3	93.6 kip-in

Check for Reduced Strength Capacity of the Girt Channel:

$F_b = 12000000/(Ld/A_f)$ = 18959 psi Ref: AISC Manual, 6th Ed.

L = 287/4 = 72 in Ref: DWG. 9150 Shts. E101, E107

d = 10 in

A_f = 2.6 in x 0.436 in = 1.134 in^2

$M = F_b \times S_x$ = 254053 lb-in → 254.1 kip-in

or

$F_b = [1 - (L/r)^2/(2C_c^2C_b)] \times 0.6F_y$ = 21.6 ksi Ref: AISC Manual, 6th Ed.

r_y = 0.72 in

$C_c = [(2\pi^2E)/F_y]^{0.5}$ = 3988 Ref: AISC Manual, 6th Ed.

C_b = 1 (conservative)

E = 29000000 psi

$M = F_b \times S_x$ = 289.3 kip-in CONTROLS!

Summary:

Due to the unbraced length of the girt channel being 72 inches between the sag rods (which provide lateral support [Ref. AISC Design Guide 7 - Industrial Buildings]), the effects of lateral torsional buckling will control over the yield capacity of channel. Therefore, the channel will buckle (289.3 kip-in) before the plastic bending capacity (572.4 kip-in) is achieved. However, this buckling phenomenon will not be observed before the plastic moment capacities of the connection angles are exceeded (147.2 kip-in, 93.6 kip-in < 289.3 kip-in).

Midspan Check:

The largest midspan moment the girt (C10x15.3) will experience occurs when it is modeled with pinned-pinned end conditions, which is unrealistic. The elastic and plastic moment capacities of the channel cross section are 289 kip-in and 572 kip-in, respectively. Due to the weaker connection angles at the end connections, the girt channel will not yield before the angles exceed their plastic moment capacities. Therefore, the channel will not experience plastic failure at 0.5 psi, but will remain within the elastic stress zone.

End Connection Check:

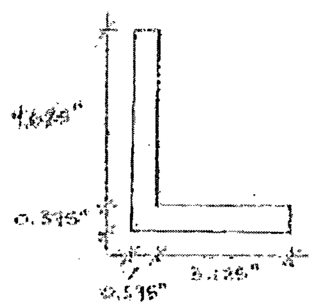
The north and south Turbine Building walls have two different sized connection angles. Considering the fixed-fixed end conditions (two 3/4" bolts), the pressure induced moments at the girt channel ends, and therefore at the connection angles, is 288 kip-in. This applied moment is greater than the plastic moment capacity of the larger connection angle on the south Turbine Building wall (288 kip-in > 147 kip-in); therefore, significant plastic deformation will occur at the plastic hinge point in the connection angles. After the plastic moment capacity is exceed, catenary action forces (axial tensile forces) develop in the connection angles to compensate for the loss of moment capacity. Rupture of the cross section occurs not due to bending but due to excessive tension in the member. The internal forces in the connection angle must be identified when the Turbine Building is pressurized at 0.5 psi to determine the exact failure mechanism. The tensile force that is developed at 0.5 psi is then compared to the pertinent material failure mechanisms to determine the controlling failure.

Determine Internal Forces on Cross Section:

When M_p is reached in the cross section, the stress block areas above and below the plastic neutral axis are equal ($A_T = A_C$). This means that the tension and the compression forces in the cross section are also equal (T = C). The distance between the resultant forces is the lever arm (d_{LA}). The lever arm multiplied by the T or C force equals the plastic moment capacity of the cross section.

L5x3-1/2x3/8 Connection Angle

M_p = 147 kip-in
PNA Location = 0.938 in (from bottom)
A_T = 0.375 in x (5 in - 0.938 in)
A_T = 1.523 in^2
Resultant Location A_T = (5 in - 0.938 in) / 2
Resultant Location A_T = 2.031 in (from top)



Resultant Location A_C is determined by locating the $\bar{Y}$ in the A_C

	A (in^2)	$\bar{y}$ (in)	$A\bar{y}$ (in^3)
Top	0.2109	0.6563	0.1384
Bottom	1.3125	0.1875	0.2461
$A_C = \Sigma A$ =	1.523	$\Sigma A\bar{y}$ =	0.3845

$\bar{Y} = \Sigma A\bar{y} / \Sigma A$ = 0.252 in (from bottom)
Resultant Location A_C = 0.252 in (from bottom)

d_{LA} = 5 in - 2.031 in - 0.252 in
d_{LA} = 2.716 in

T = A_T x Fy = 54.8 kips
or T = M_p / d_{LA} = 54.2 kips

P = 0.5 psi T = M / d_{LA} = 288 kip-in / 2.716 in = 106 kips

The tension value is 106 kips. After the plastic moment capacity of the cross section is exceeded the compression area starts to decrease, causing the tension area to increase. Eventually, the compression area will be eliminated entirely from the cross section, leaving only the tension properties of the cross section to carry the entire load. At this point pure catenary behavior is achieved, which would result in higher tension forces and therefore shows that using a tension value at 106 kips is conservative.

Determine Internal Forces on Cross Section: (continued)

L4x3x3/8 Connection Angle

$M_p =$ 93.6 kip-in

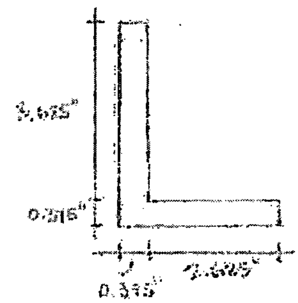
PNA Location = 0.688 in (from bottom)

$A_t = 0.375 \text{ in} \times (4 \text{ in} - 0.9375 \text{ in})$

$A_t =$ 1.242 in^2

Resultant Location $A_t = (4 \text{ in} - 0.9375 \text{ in}) / 2$

Resultant Location $A_t =$ 1.656 in (from top)



Resultant Location A_c is determined by locating the $\bar{Y}$ in the A_c

	A (in²)	ȳ (in)	Aȳ (in³)
Top	0.1174	0.5315	0.0624
Bottom	1.1250	0.1875	0.2109
$A_c = \Sigma A =$	1.242	$\Sigma A\bar{y} =$	0.2733

$\bar{Y} = \Sigma A\bar{y} / \Sigma A =$ 0.220 in (from bottom)

Resultant Location $A_c =$ 0.220 in (from bottom)

$d_{tA} = 4 \text{ in} - 1.656 \text{ in} - 0.220 \text{ in}$

$d_{tA} =$ 2.124 in

$T = A_t \times Fy =$ 44.7 kip

or $T = M_p / d_{tA} =$ 44.1 kip

P = 0.5 psi $T = M / d_{tA} = 288 \text{ kip-in} / 2.124 \text{ in} = 136$ kip

The tension value is 136 kips. After the plastic moment capacity of the cross section is exceeded the compression area starts to decrease, causing the tension area to increase. Eventually, the compression area will be eliminated entirely from the cross section, leaving only the tension properties of the cross section to carry the entire load. At this point pure catenary behavior is achieved, which would result in higher tension forces and therefore show that using a tension value at 136 kips is conservative.

Check Failure Mechanisms:

1. Base Material Rupture
2. Base Material Tearing
3. Bolt Bearing
4. Bolt Shear
5. Weld Rupture

The plastic moment capacities of the connection angles are exceeded by at least a factor of 2 (288 kip-in / 147 kip-in = 1.96 or 288 kip-in / 93.6 kip-in = 3.1), approximately, when an internal pressure value of 0.5 psi is present in the Turbine Building. The controlling mechanism will be a localized tension or shear failure due to the induced catenary behavior in the connection angles. The failure will occur as either tensile rupture or shear rupture of the base material or the fastening materials (bolts or welds). The failure mechanisms that are listed above will be evaluated to determined the initial failure that will control the wall panels release from the Turbine Building.

1. **Base Material Rupture**

The failure mechanism is tensile rupture.

$T_u = F_u \times A_e$

$F_u =$ 58 ksi

$A_e =$ Net Cross Sectional Area

1.1 *L5x3-1/2x3/8 Connection Angle*

$A_g =$ 3.05 in^2			Ref: AISC Manual, 6th Ed.
Bolt Diameter (b_d) = 0.75 in			Ref: DWG. 9150 Sht. E101
Bolt Hole Diameter (h_d) = 0.81 in			Ref: DWG. 9150 Sht. 156
t = 0.38 in	(leg thickness)		Ref. DWG. 9150 Sht. 156
$A_e = A_g - h_d \times t =$ 2.75 in^2			

$T_u =$ 159 kips

1.2 *L4x3x3/8 Connection Angle*

$A_g =$ 2.48 in^2			Ref: AISC Manual, 6th Ed.
Bolt Diameter (b_d) = 0.75 in			Ref: DWG. 9150 Sht. E101
Bolt Hole Diameter (h_d) = 0.81 in			Ref: DWG. 9150 Sht. 157, 158, 159
t = 0.38 in	(leg thickness)		Ref: DWG. 9150 Sht. 157, 158, 159
$A_e = A_g - h_d \times t =$ 2.18 in^2			

$T_u =$ 126 kips CONTROLS!

1.3 *C10x15.3 Girt Channel*

$A_g =$ 4.47 in^2			Ref: AISC Manual, 6th Ed.
Bolt Diameter (b_d) = 0.75 in			Ref: DWG. 9150 Sht. E101
Bolt Hole Diameter (h_d) = 0.81 in			Ref: DWG. 9150 Sht. 133
t = 0.24 in	(web thickness)		Ref: AISC Manual, 6th Ed
$A_e = A_g - h_d \times t =$ 4.28 in^2			

$T_u =$ 248 kips

The L4x3x3/8 Connection Angle is the controlling cross section for this failure mechanism.

2. Base Material Tearing

The failure mechanism is shear rupture of the base material. This is caused by the bolts tearing through the base material.

$T_u = F_u \times 2L_s \times t$ ($2L_s$ = Two Shear Planes)

F_u = 58 ksi

L_s = Shear Plane Length

t = Base Material Thickness

2.1 L5x3-1/2x3/8 Connection Angle

L = 8.25 in		Ref. DWG. 9150 Sht. 156
L_s = 4.25 in		Ref. DWG. 9150 Sht. 156
t = 0.38 in	(leg thickness)	Ref. DWG. 9150 Sht. 156

T_u = 185 kips

2.2 L4x3x3/8 Connection Angle

L = 8.75 in		Ref: DWG. 9150 Sht. 157, 158, 159
L_s = 4.25 in		Ref: DWG. 9150 Sht. 157, 158, 159
t = 0.38 in	(leg thickness)	Ref: DWG. 9150 Sht. 157, 158, 159

T_u = 185 kips

2.3 C10x15.3 Girt Channel

L_s = 4.5 in		Ref: DWG. 9150 Sht. 133
t = 0.24 in	(web thickness)	Ref: AISC Manual, 6th Ed.

T_u = 125 kips CONTROLS!

The C10x15.3 Girt Channel is the controlling cross section for this failure mechanism. This is occuring from the edge of the end of channel to the inside edge of the bolt line.

3. **Bolt Bearing**

The failure mechanism is yielding of the base material around the bolts.

$F_B = b_N \times t \times b_d \times 1.35F_y$ Ref: Struct. Steel Design (1965)

b_N = Number of Bolts

t = Base Material Thickness

b_d = Bolt Diameter

F_y = 36 ksi

3.1 *L5x3-1/2x3/8 Connection Angle*

b_N =	2	bolts	Ref. DWG. 9150 Sht. 156
t =	0.375	in	Ref. DWG. 9150 Sht. 156
b_d =	0.75	in	Ref: DWG. 9150 Sht. E101
F_B =	27.3	kips	

3.2 *L4x3x3/8 Connection Angle*

b_N =	2	bolts	Ref: DWG. 9150 Sht. 157, 158, 159
t =	0.375	in	Ref: DWG. 9150 Sht. 157, 158, 159
b_d =	0.75	in	Ref: DWG. 9150 Sht. E101
F_B =	27.3	kips	

3.3 *C10x15.3 Girt Channel*

b_N =	2	bolts	
t =	0.24	in	
b_d =	0.75	in	
F_B =	17.5	kips	CONTROLS!

The C10x15.3 Girt Channel is the controlling cross section for this failure mechanism.

4. **Bolt Shear**

The failure mechanism is shear rupture of the bolt material.

$R_v = b_N \times F_n \times A_b$			Ref: AISC Manual, 13th Ed.
b_N = Number of Bolts			
F_u = 60 ksi		(A307)	Ref. ASTM A307, DWG. 9150 Shts. 10 and E101
F_n = 37.2 ksi		(A307X)	Ref: AISC Manual, 13th Ed.
A_b = Bolt Cross Sectional Area			
b_N = 2 bolts		(both angle sizes)	Ref: DWG. 9150 Sht. 133
t_d = 0.75 in			
A_b = 0.442 in^2			
R_v = 32.9 kips			

Shear rupture of both bolts will occur at 32.9 kips (tension between girts and connection angles).

Note: DWG. 9150 Shts. 10 and E101 indicate that the bolts used in these connections are 3/4 in machine bolts. Per contract E69-15, these machine bolts are of A307 (assuming grade A). Also, it is assumed that the threads are excluded from the shear plane. The 13th edition uses 62% tensile strength for the nominal shear capacity of bolts.

5. **Weld Rupture**

The failure mechanism is tensile failure of the welds that fasten the connecting angles to the structural columns. (F_{E70} welds are assumed.)

$R_w = t_w \times L_w \times 0.707 \times S_s \times F.S.$		Ref: Struct. Steel Design (1965)
t_w = Weld Thickness		
L_w = Weld Length	(total combined length)	
F_{E70} = 70 ksi		
S_s = 15.8 ksi	(Allowable Shear Stress)	Ref: AISC Manual, 6th Ed.
F.S. = 3	(Commentary)	Ref: AISC Manual, 6th Ed.

5.1 *L5x3-1/2x3/8 Connection Angle*

t_w = 0.25 in		Ref. DWG. 9150 Sht. 156
L_w = 7.0 in		Ref. DWG. 9150 Sht. 156
R_w = 58.6 kips	CONTROLS!	

5.2 *L4x3x3/8 Connection Angle*

t_w = 0.31 in	Ref: DWG. 9150 Sht. 157, 158, 159
L_w = 6.0 in	Ref: DWG. 9150 Sht. 157, 158, 159
R_w = 62.8 kips	

The controlling weld connection is on the L5x3-1/2x3/8 connection angle.

Note: The 6th edition provides an allowable weld strength in the specifications; the commentary section indicates that a factor of safety of 3 was used on the ultimate strength test results to produce the allowables. The F.S. is multiplied back into the strength equal to generate the ultimate strength of the welds. Also, E70 electrodes were identified via weld procedures associated with contract E69-15.

Validate weld strength controls over base material strength: Ref: AISC Manual, 13th Ed.

Weld Metal:	$Rn = F_w \times A_w$	$= 0.60 \times t_w \times F_{E70}$	=	13.1	kip/in	(5/16 in. weld thickness)
Base Metal:	$Rn = F_{BM} \times A_{BM}$	$= 0.60 \times t \times Fu$	=	13.1	kip/in	(3/8 in. leg thickness)

The base material and the weld material have the same strength; therefore, the calculated weld strength is acceptable for use.

Controlling Results Summary:

Failure Mechanism	*Failure Force*	*Failed Material*	
Base Material Rupture	126 kips	L4x3x3/8	
Base Material Tearing	125 kips	C10x15.3	
Bolt Bearing	17.5 kips	C10x15.3	(Note: this is yielding)
Bolt Shear	32.9 kips	3/4 in (A307) bolts	CONTROLS!
Weld Rupture	58.6 kips	L5x3-1/2x3/8 welds	

Minimum Girt Span Length:

The span length between girt channels effects the line force on the girt channels. An evaluation is performed below to determine the minimum girt channel span that will permit panel blowout when 0.5 psi is present in the Turbine Building.

Distributed Load: $w = P \times S$

Fixed End Moment: $M = wL^2/12$

Tension in Cross Section: $T = M_n / d_{LA}$

Girt Span: $S = (d_{LA} \times T \times 12 \times 1000\ \text{lb/kip})/(P \times L^2)$

(L5x3-1/2x3/8) d_{LA} = 2.716 in
(L4x3x3/8) d_{LA} = 2.124 in
T = 32.9 kips
P = 0.5 lb/in^2
L = 287 in

(L5x3-1/2x3/8) S = 26.015 in CONTROLS!
(L4x3x3/8) S = 20.342 in

The minimum girt channel span for bolt shearing to occur with 0.5 psi pressure in the Turbine Building is 26.01 in. All the girt channel spans between column lines D, E, and F on the north and south ends of the Turbine Building are greater than 2.5 ft (26.01 in). The only except is the top girt channel, which is a C12x25 channel.

Conclusions:

The internal tension forces acting on the two different connection angle sizes are 106 kips (L5x3-1/2x3/8) and 136 kips (L4x3x3/8) when the Turbine Building is pressurized at 0.5 psi. Ignoring the bolt bearing failure mechanism (not ultimate strength of the material), the controlling failure mechanism in the girt channel/connection angle is the shearing of the two 3/4 in A307 (machine) bolts at 32.9 kips. This failure mechanism occurs before a pressure of 0.5 psi is achieved (32.9 kips < 106 kips, 136 kips). Per the previous sections, a pressure of 0.5 psi will cause failure of the welds; however, the connection bolts will shear before the strongest connection element (welds) will rupture (32.9 kips < 58.6 kips). Therefore, the connection angles will not remain attached to the girt channels when the panels are relieved from the Turbine Building north and south walls. Panel release will occur unimpeded. At a 0.5 psi pressure, the minimum girt channel span length for panel release is 2.5 ft. All of the girt channels, except for the top girt channels, between column lines D, E, and F on the north and south ends of the Turbine Building have spans greater than 3 ft, and therefore will blowout at 0.5 psi. A vent area of 6,576 ft^2 will be created that will relieve pressure from the Turbine Building.

Note on the vent area:
The total bay width is 48 ft (2 bays x 24 ft) and the bay height (sum of individual girt spans) is 68.5 ft per drawings 4088 and 4089. The 6,576 ft^2 (2 x 68.5 ft x 48 ft) is the total vent area created on the north and south Turbine Building walls.

Inryco Wall Systems Technical Data

Field-insulated Walls

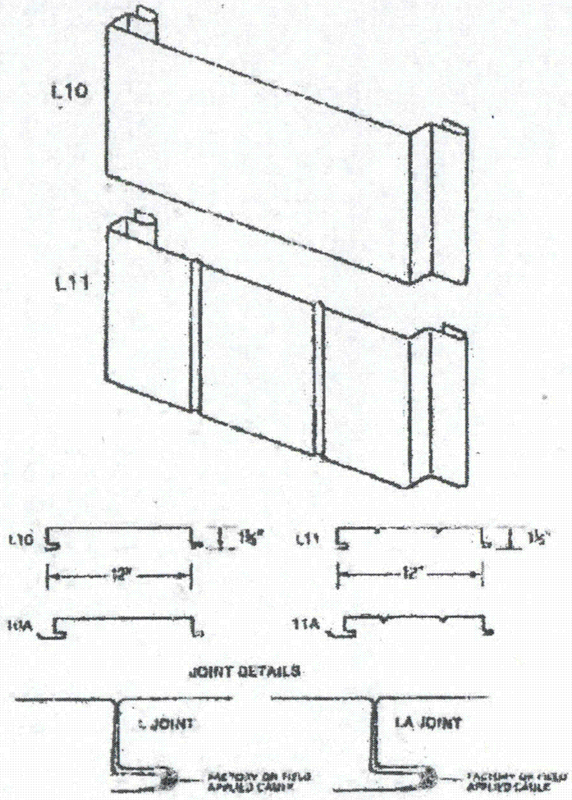
Inryco
an Inland Steel company

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 S	24 Gage D	24 Gage T	22 Gage S	22 Gage D	22 Gage T
Load		MAXIMUM SPAN LENGTHS IN FEET					
6.4 PSF*	f	13.23	13.23	14.79	15.56	15.56	17.38
15 PSF	f	8.64	8.64	9.66	10.16	10.16	11.36
	Δ	6.88	9.23	8.61	7.82	10.22	9.42
20 PSF	f	7.48	7.48	8.37	8.79	8.79	9.83
	Δ	6.26	8.39	7.73	6.92	9.28	8.56
25 PSF	f	6.69	6.69	7.48	7.87	7.87	8.79
	Δ	5.81	7.76	7.18	6.45	8.62	7.94
30 PSF	f	6.11	6.11	6.83	7.18	7.18	8.03
	Δ	5.48	7.32	6.75	6.06	8.11	7.48
35 PSF	f	5.66	5.66	6.32	6.65	6.65	7.43
	Δ	5.19	6.96	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	4.98	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

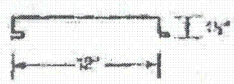
Span Type		20 Gage S	20 Gage D	20 Gage T	18 Gage S	18 Gage D	18 Gage T
Load		MAXIMUM SPAN LENGTHS IN FEET					
6.4 PSF*	f	17.80	17.83	19.90	20.88	20.88	23.35
15 PSF	f	11.62	11.62	13.00	13.64	13.64	15.25
	Δ	8.44	11.31	10.42	9.58	12.85	11.84
20 PSF	f	10.07	10.07	11.26	11.81	11.81	13.21
	Δ	7.66	10.27	9.47	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	8.53	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 (60 mph) wind load, no deflection limit.

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	.188	.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

DOCUMENT (SET)
START

VACUUM LOAD TEST
L10 STEEL/IW21A ALUMINUM
1820 GAGE WALL PANEL
COOPER NUCLEAR POWER STATION
NEBRASKA

E69-28 file

Job No. 49054

March, 1973

TEST REPORT PREPARED BY: ________________
Demetrio A. Lucap

TESTED AND WITNESSED BY:

Inland-Ryerson Personnel

Y. R. Shea
J. H. Fisk
T. J. McCabe
D. R. Heintz
K. Meyer
G. Vasbinder
R. Vasey

Burnes & Roe, Inc.

M. Goodman

TEST REPORT REVIEWED BY: ________________
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TEST REPORT APPROVED BY: ________________
Y. R. Shea, P.E.

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APPENDIX

INTRODUCTION

This report contents the results of a uniform load test continuous over two equal spans of 7'-0". The assembly was composed of L10 (steel) 18 gage and IW21A (aluminum) 20 gage uninsulated wall panels. The assembly was loaded with negative pressure. The test was conducted at Inland Ryerson Construction Products Company, Burnham Street Research Laboratory, Milwaukee, Wisconsin on March 27, 1973. The test and calculations were performed by Inryco personnel.

The test was witnessed by John H. Fisk, Inryco District Sales Manager and Milton Goodman, Burns & Roe, Inc. The assembly simulated the construction of wall panels used on the Cooper Nuclear Power Station, Nebraska.

PURPOSE

The objective of the negative pressure load test were the following:

1. Demonstrate the load carrying capacity of the IW21A Aluminum wall panel.
2. Determine the load deflection curves of the L10 Steel/IW21A Aluminum insulated wall assembly.

CONCLUSIONS

The L10 Steel/IW21A Aluminum Wall Panel assembly carried a uniform load of 78.0 psf under negative pressure applied directly to IW21A aluminum panel before the panel pulled over the screws. This load produced ultimate load of 341.25 lbs./ft. per screw. This gives a safety factor of 2.03 in term of the design load per screw of 167.8 lbs./ft. There was no damage to the L10 steel panel.

The maximum deflection of the specimen was 0.1075 inch in the north span which is 88.5% of the calculated value of 0.1212 inch. The south span deflected 0.110 inch or 106.4% of the calculated value of 0.1034 inch.

The IW21a 20 gage aluminum panel requires a screw at each subgirt adjacent to the web of the panel to maintain its profile under load. It is recommended that Inland screw, part No. 4290-122, a #12-14 x 1" nylon head, self drilling, tapping and sealing sheet metal screw be used.

TABLE I

DEFLECTION SUMMARY

Test No	Test Panel	Load PSF	DEFLECTION ( ), INCH North Span Test	North Span Calc.	South Span Test	South Span Calc.
2	L10 20 Ga. Steel	40	.1075	.1215		
	IW21A 18 Ga. Alum.	34			.110	.1034

TABLE II

SCREW LOAD CARRYING CAPACITY

Test No.	Panel Type	Design Load/Screw (Lbs/Ft)	Ultimate Load/Screw (Lbs/Ft)	Safety Factor
2	L10/IW21A Alum 18/20 gage	167.8	341.2	2.03

DESCRIPTION OF TEST RESULTS

The IW21A aluminum panel lost its carrying capacity when the profile lost its original shape. This occurred in the three east panels where the screw was fastened to the subgirts at the stretch rib. The three panels on the west side were fastened to the subgirt adjacent to the web and were not damaged in the test.

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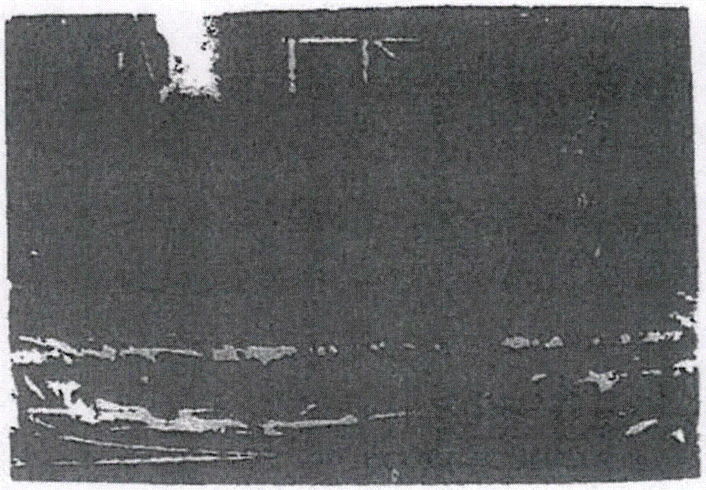

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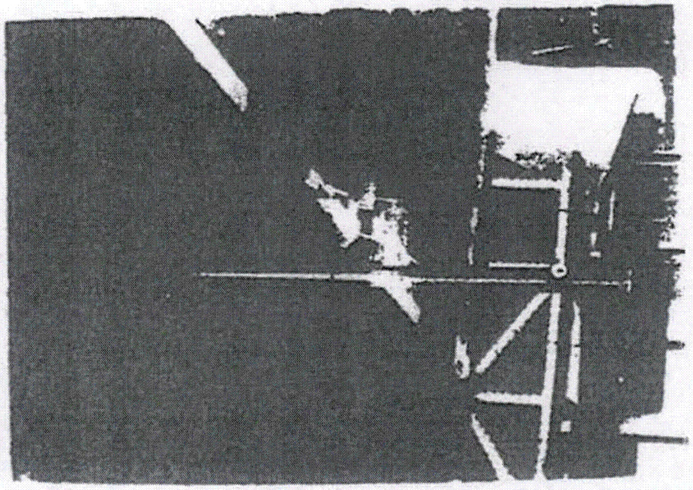

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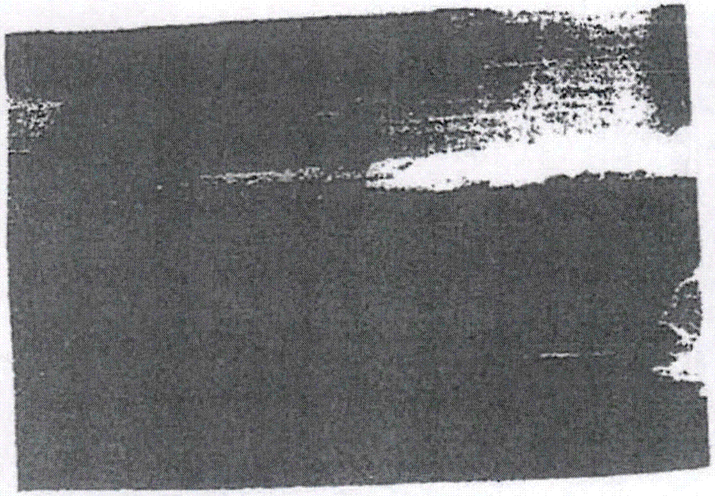

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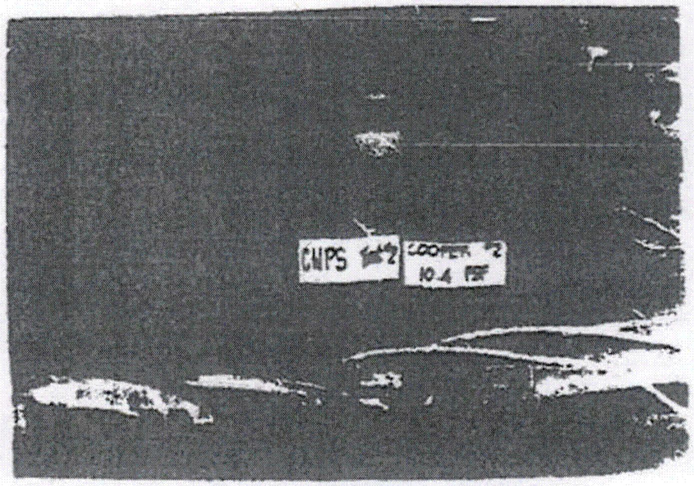
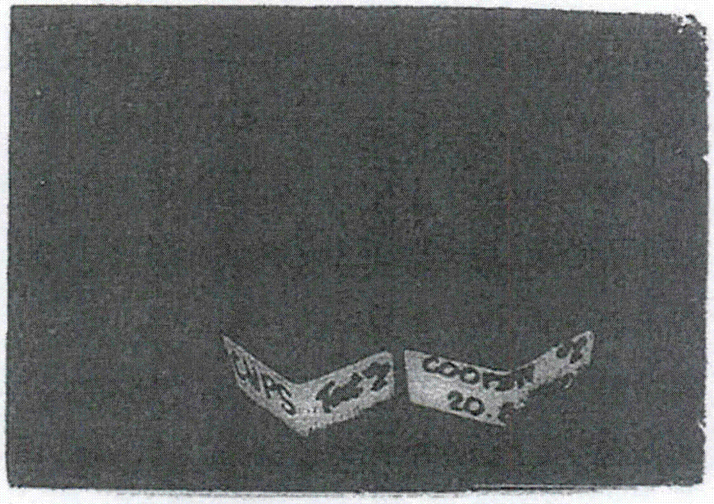
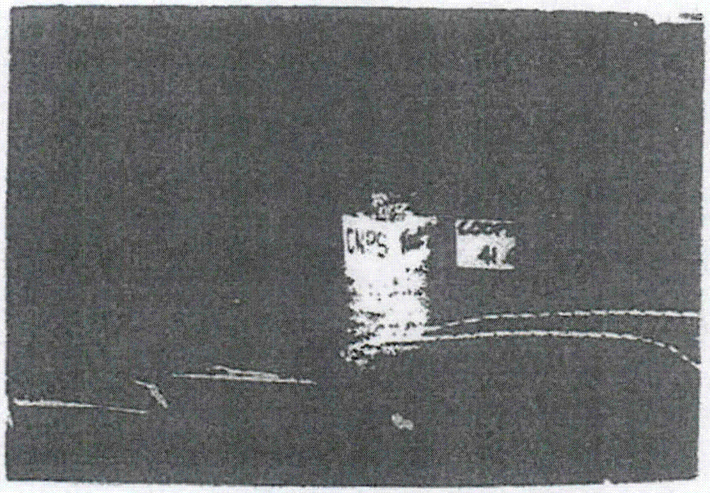


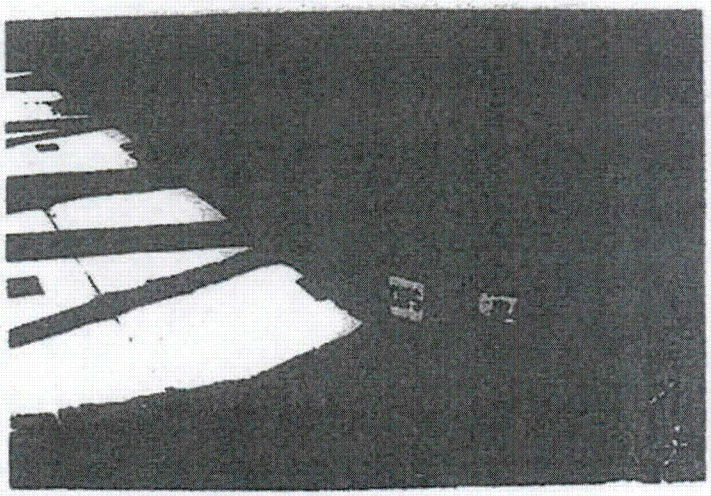
First Load at 10.4 PSF



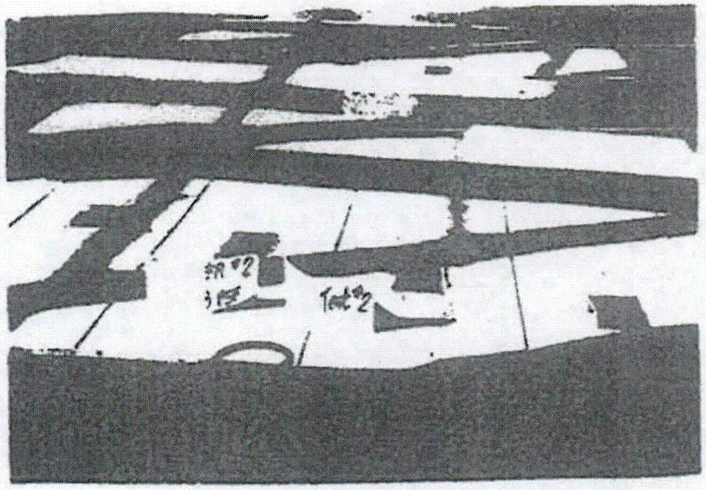
Load at 20.8 PSF



Load at 41.6 PSF



Load at 62.4 PSF



Load at 72.8 PSF

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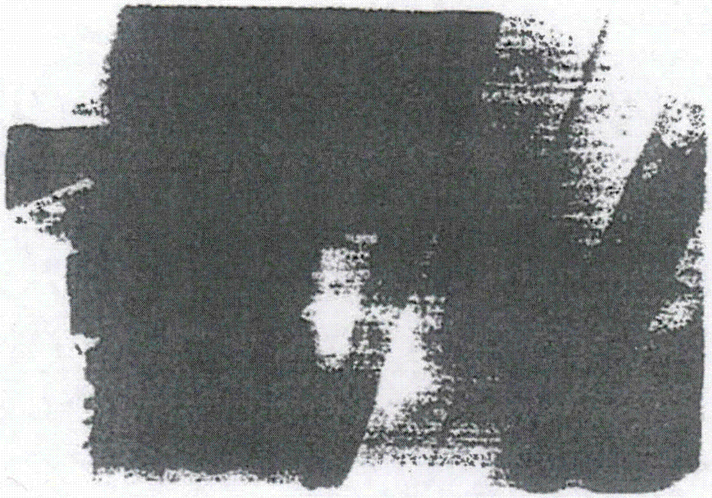
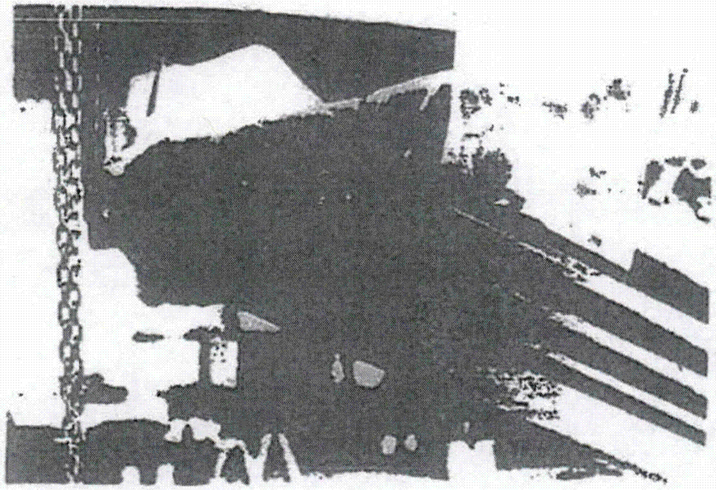
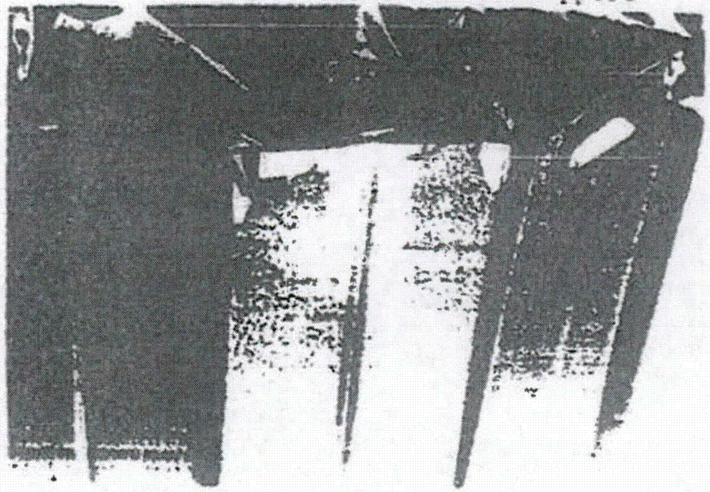
Ultimate Load (78.0 PSF) At Failure



East Panel Pull Over of Screw On Stiffened Rib at Mid-Support



North-East End Failure of Panel



End Panel Pull Over Of Screw On Stiffened Rib

DESCRIPTION OF TEST

A. Test Assembly

The liner consisted of six L10 steel 18 gage panels 12 inches wide, 14'-3" long screwed to 3-1/2" x 3-1/2" tubing supports. Two #14 x3/4" Type B screws with neoprene washer, Inryco Part No. 4290-101, attached each panel to support at 18" on centers. A drill size No. 1 was used into the tubing supports. Five Inryco hat shpaed subgirts 6'-4" long, Part No. 4290-810, were used at 3'-6" on centers. They were fastened with #12 - 14 x 1" Hex washer head, self drilling and tapping, Part No. 4209-951, screws to the liner 12" on centers. The face panel consisted of six IW21A aluminum 20 gage panels, 12 inches wide and 14'-3" long. The panels were fastened with the subgirts by two kind of screws, Part No. 4209-951 and 4290-121. The location and spacing of screws were indicated on the Drawing No. 4X312 of the Appendix.

A 6'-11" x 14'-3-1/2" (I.D.) wooden frame was constructed around the specimen. The assembly was sealed with 6 mil polyethylene plastic sheet placed between the L10 and IW21A panels that caused the applied load carried directly by IW21A panel to the floor with caulk tape.

B. Instrumentation

Two types of instrumentation were used in the tests:

1. Two - two speed Clements Blowers, Model HP3A rated at 196 CFM were bolted to the holes in the wooden frame around the assembly. Pressure was read using a manometer with a range of 50 inches (1 inch equal to 5.2 PSF). The fluid was meriam 100 red unity oil with a specific gravity of one at 73° fahrenheit.

2. A transit level and 40th scale ruler were used to measure deflections of the assemblies as indicated on the data reduction pages in the Appendix.

C. Procedure

Step 1. The assembly was pre-loaded to 10.4 PSF and held for five minutes to verify the seal.

Step 2. Load was applied to 10.4 PSF. Then loads were increased in 5.2 PSF increments until the failure of the specimen. Each load was again held for ten minutes. See Data Sheets in the Appendix for the specific loadings.

Braisted, Jonathan

From:	Sutton, Taylor E. <tesutto@nppd.com>
Sent:	Tuesday, August 04, 2015 2:19 PM
To:	Braisted, Jonathan; Reinert, Dustin
Cc:	Flaherty, James R.
Subject:	[External_Sender] Turbine Building Blow-out Calculation
Attachments:	NEDC 13-028 Rev.1 Approved (email size p1).pdf

All,

Attached is the first part of NEDC 13-028 Revision 1. The second half is to come shortly.

TAYLOR SUTTON
DESIGN CIVIL ENGINEERING DEPARTMENT
COOPER NUCLEAR STATION – NPPD
TESUTTO@NPPD.COM
O. (402) 825-2706

Braisted, Jonathan

From:	Sutton, Taylor E. <tesutto@nppd.com>
Sent:	Tuesday, August 04, 2015 2:15 PM
To:	Braisted, Jonathan; Reinert, Dustin
Cc:	Flaherty, James R.
Subject:	[External_Sender] FW: Blow-Out Panel 3D Animation Update
Attachments:	EE 13-041 Rev.2 Approved (email size).pdf

All,

Attached is EE 13-041 Revision 2. The calculation will come in two separate emails.

TAYLOR SUTTON
DESIGN CIVIL ENGINEERING DEPARTMENT
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TESUTTO@NPPD.COM
O. (402) 825-2706

From: Sutton, Taylor E.
Sent: Tuesday, August 04, 2015 12:26 PM
To: Braisted,, Jonathan (Jon) D.- U.S. Nuclear Regulatory Commission; Reinert,, Dustin- NRC
Cc: Flaherty, James R.
Subject: FW: Blow-Out Panel 3D Animation Update

From: Woods Kirby [mailto:kwoods@inntecheng.com]
Sent: Monday, August 03, 2015 6:49 AM
To: Able, Alan L.; Sutton, Taylor E.
Cc: Estrada, Roman M.; Nienaber, Matthew B.
Subject: Blow-Out Panel 3D Animation Update

I have created two 3D animations of the angle and bolt(s) fracture (see attachment).

Enjoy,

Kirby Woods
President

InnoTech Engineering Solutions, LLC
6721 North 105th Avenue
Omaha, NE 68122-3005

Phone No. (402) 740-0281
kwoods@inntecheng.com
website: http://www.inntecheng.com

EC#	(EE) 13-041	Rev. No. 2	Page 1 of 16

Created from:	ECR	11156311	EC Type:	EVAL	SubType:	Engineering Evaluation	
Proj. Pri:	None	Facility: CNS	EWO #:	None	System:	Turbine Building/Structure	
Outage: (Y/N)	NO	W/O Required: NO	Alt Ref.:	None	Temp?: N/A	Work at Risk?:	N/A

Keywords

Setpoint? (Y/N)	N	O&M Mod? (Y/N)	N	Software? (Y/N)	N	Cyber? (Y/N)	N	Quality Related? (Y/N)	Y	Proprietary Info? (Y/N)	N
Safeguards? (Y/N)	N	EQ? (Y/N)	N								

Title: Turbine Building Blowout Panels / Metal Wall System

Summary

The north and south Turbine Building panel walls have been evaluated to blowout at 0.5 psid, as stated in FSAR Amendment 25. The resulting blowout panel wall debris will not affect the surrounding structures and equipment.

☐ Vendor EC	N/A Vendor company name/Contract #/Date

Resp. Engr / Preparer	Taylor Sutton	Date: 7/27/15	Taylor Sutton

REVIEW

☐ Technical Reviewer	Date:
☒ Design Verifier Matthew Mendez Matthew Mendez 7.30.15	Date: 7.30.15
☒ EQRT Review Alan Able For Ramon Estrada per telecon	Date: 7-30-15
☐ Reviewers (See Attachment 9.21) ®[13]	

APPROVAL

☒ Supervisor	Alan Able Alan Able	Date:	7-30-15
☐ Manager		Date:	
☐ GMPO		Date:	
Expiration Date = (Four (4) years after approval date		Date:	

Copy Distribution:

[X] Master Data Group
[X] Originator
[X] Maintenance Rule Group
[X] Licensing
[X] Work Planning
[X] Simulator Services
[X] System Engineer
[X] Training DERC
[X] WCC (If Field Work Required)
[ ] SRAB (Required if TS change or NRC approval is needed)

1. Description

1.1. Structure, System, or Component Description

Turbine Building

The Turbine Building is designated as a Principle Class II Structure [Ref. 2.3.31].

The Turbine Building houses the turbine-generator and associated auxiliaries including the Condensing, Feedwater, and Water Treatment Systems. Space is provided in this building for other auxiliary power plant equipment. The Water Treatment Area, Machine Shop, Exhaust Fan Room and Heating Boiler Room are located adjacent to the Turbine Building and are referred to as the Turbine Building appendages [Ref. 2.3.32].

The Turbine Building is a reinforced concrete structure up to the operating floor. Concrete shield walls surround the turbine-generator and structural steel framing rises above the operating floor. The building is enclosed with insulated metal siding and roofing. The interior walls are reinforced concrete with concrete block enclosing smaller areas. The Turbine Pedestal is a massive reinforced concrete structure supported by the same foundation mat as the building [Ref. 2.3.32].

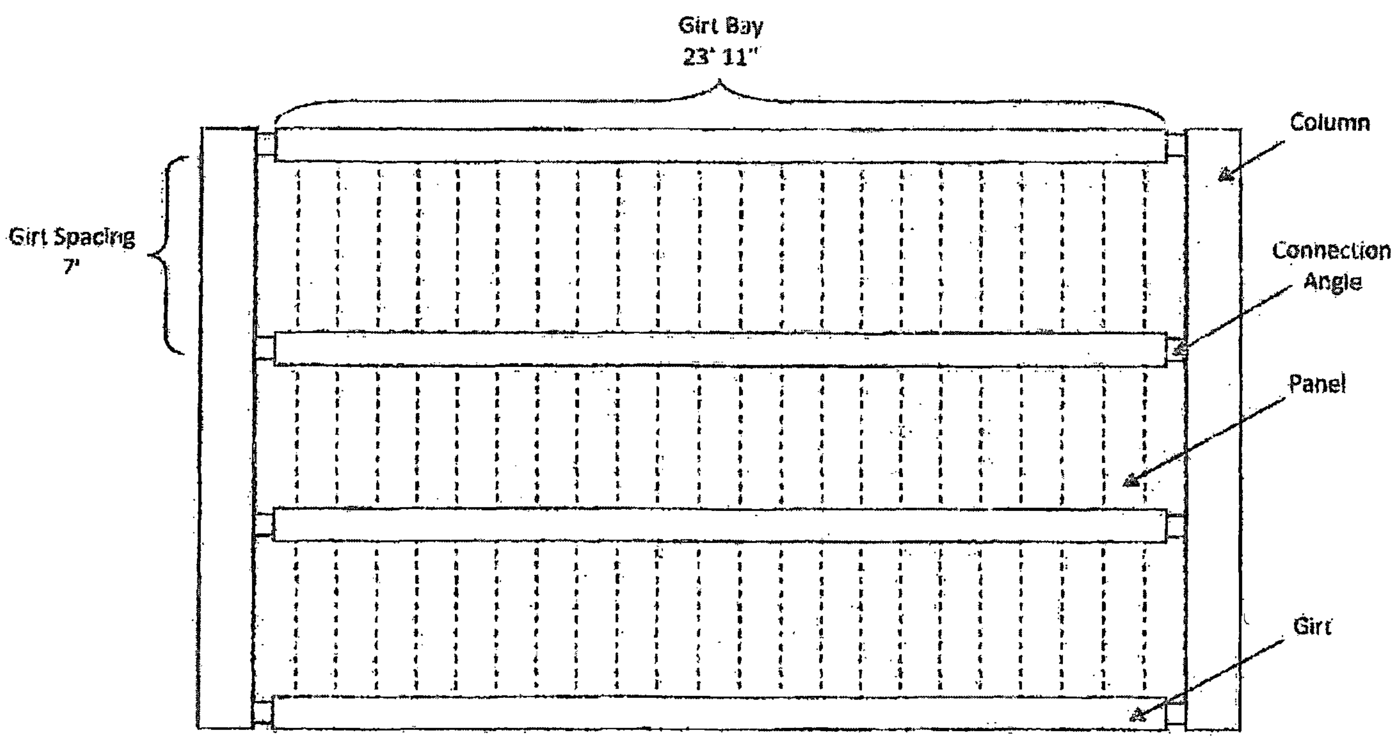

Figure 1: Typical Wall Section View

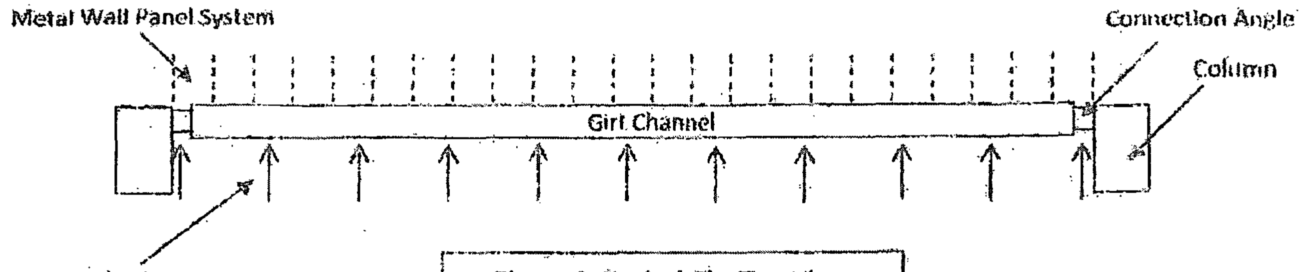

Figure 2: Typical Girt Top View

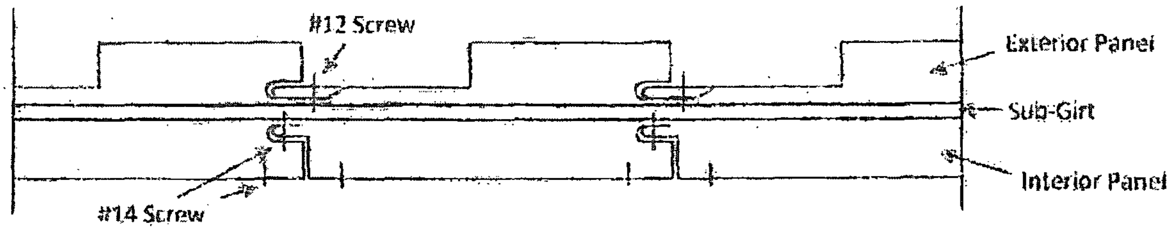

Figure 3: Typical Sub-Girt Cross Section

1.2 Reason for Change/Evaluation

EE 13-041 Revision 2

During a High Energy Line Break (HELB), such as a Main Steam Line Break, the Turbine Building will pressurize. This event will lead to over pressurization of the Control Room if the pressure is not relieved. The peak pressure that the panel wall system (metal siding) can achieve is stated in FSAR Amendment 25. This Engineering Evaluation looks at the north and south walls of the Turbine Building to determine the failure mechanism that will control the release of the panel wall system when 0.5 psid pressure is present in the Turbine Building.

1.3 Design Objective to Resolve Problem

This Engineering Evaluation is an assessment of the weak link and subsequent internal pressure that can be maintained as well as the impact of the effects of blowout on the surrounding structures and equipment.

FSAR Amendment 25 states a value of 0.5 psid for the pressure at which the building siding fails. This evaluation conservatively confirms that panel wall systems on the north and south Turbine Building walls will fail at 0.5 psid.

2. Documents

2.1 Affected Documents List

There are no documents affected by this Engineering Change.

2.2 Affected Equipment List

There are no SSCs or equipment affected by this Engineering Change.

2.3 Reference Documents

2.3.1 NEDC 13-028, Rev.1; "Ultimate Internal Pressure of Turbine Building Blowout Panels and Metal Wall System"
2.3.2 FSAR Amendment 25
2.3.3 DWG. 4088, Sht.1, Rev.00, Ver.AA
2.3.4 DWG. 4089, Sht.2, Rev.00, Ver.AA
2.3.5 DWG. 9150, Sht.E106, Rev.00, Ver.AA
2.3.6 DWG. 9150, Sht.E107, Rev.00, Ver.AA
2.3.7 DWG. 9150, Sht.10, Rev.00, Ver.AA
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2.3.24 DWG. 49054, Sht.11T, Rev.00, Ver.AA
2.3.25 DWG. 49054, Sht.12T, Rev.00, Ver.AA
2.3.26 USAR VI-4.3, Rev.loep.xxvii2
2.3.27 USAR VI-4.4, Rev.loep.xxvii2
2.3.28 USAR XI-2.4, Rev.loep.xxvii2
2.3.29 USAR XI-9.3, Rev.loep.xxvii2

2.3.30 USAR XII-2.1.2.3, Rev.loep.xxvii2
2.3.31 USAR XII-2.1.3.1, Rev.loep.xxvii2
2.3.32 USAR XII-2.2.2, Rev.loep.xxvii2
2.3.33 USAR XII-2.2.3, Rev.loep.xxvii2
2.3.34 USAR XII-2.2.6, Rev.loep.xxvii2
2.3.35 USAR XII-2.3.3.1, Rev.loep.xxvii2
2.3.36 USAR XII-2.3.3.2, Rev.loep.xxvii2
2.3.37 USAR XII-2.3.5.1.3, Rev.loep.xxvii2
2.3.38 USAR XIV-6.5.3.1.i, Rev.loep.xxvii2
2.3.39 Technical Specification 3.5.2
2.3.40 Burns and Roe Calculation Book 58

Other applicable calculation references are contained in NEDC 13-028, Revision 1.

3. Evaluation/Design Summary

3.1 Evaluation Resolution

If the panels blow off, it is hypothesized that they could damage plant structures or equipment. This discussion looks at the various equipment surrounding the Turbine Building and what effect the discharged building siding could have.

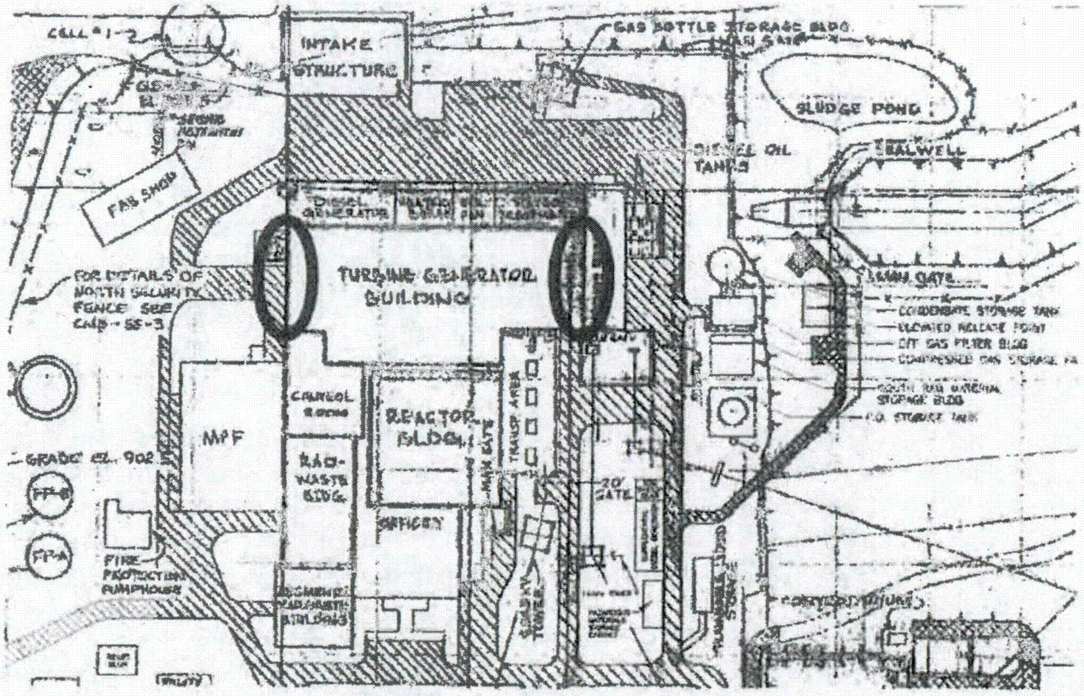

Figure 4: Location of Blowout Panels

Calculation NEDC 13-028 concludes that the wall system will fail in the middle of the north and the south walls, as noted in Figure 4. (The east and west walls of the Turbine Building have more robust connections to the girts and will require a higher internal pressure for the connecting bolts to shear.) The calculation also validates that the walls will fail at the FSAR Amendment 25 pressure value. Bolt failure is to occur in the shear plane between the girt channels and the connection angles. This failure mechanism controls the release on the blowout panels.

Localized dynamic loading was not considered in NEDC 13-028. Localized pressures from a HELB event could break the panel fasteners, which would result in releasing the panels. If this were to occur, it would establish a vent path sooner than what is calculated. From the point of view of the HELB event, ignoring dynamic effects is conservative.

The internal pressure acting on the panels puts the girts in bending. The connection angles have smaller plastic moment capacities compared to the girt channel. After the connection angles deform plastically, plastic hinges form that induce catenary behavior in the connection angles. A catenary is formed by lack of flexural strength such as is present in a string or suspension bridge. This is reasonable as deflection will continue to increase with no increase in load. The catenary action leads to a much larger tensile force that must be resisted by the bolted connections between the girt channel and the connection angles. (In the end, the bolts fail due to excessive loading.)

After the first connection fails, there will be a redistribution of load, which in turn, causes more bolts to shear. Sag rods tie the girts to each other from the lowest girt to the ceiling. The sag rods provide dead load and lateral support of the panels and girts, and are supported from above in the eave strut.

The Turbine Building's concrete walls and metal roof were not considered. They are not the weakest components of the building envelope. An alternate blowout location, such as the Office Building or Turbine Building ground level exit, does not offer a large enough vent path and so are conservatively ignored.

The interior and exterior panels do not fail within this evaluation. The panels were subjected to a vacuum test (retrieved from microfilm and is attached to calculation NEDC 13-028). The failure mechanism of the panels is sheet pullover near the pressure that is considered in that test/analysis. The panels, as tested in the setup, do not fail and remain undamaged.

However, it is foreseeable that unusual conditions may cause panels to separate from the main mass of the wall. The resulting pieces would be small and would fly anywhere within a wide arc of the breach. It is also foreseeable that larger sections of the wall could break off. These larger pieces would not travel far and may land close to the north or south walls. In addition to wall debris, small, lightweight, loose items within the Turbine Building may also be swept up in the resulting airstream and become airborne. The total debris is equivalent to the vent area, which is 6,576 ft^2.

As stated in USAR XII-2.3.3.2.1, all Class I structures are designed for tornado loading. Also, USAR XII-2.3.3.2.2 states Class I structures are designed for tornado generated missiles. Any missile generated by release of the blowout panels will have less kinetic energy than a 35' utility pole with a velocity of 200 miles per hour due to the significantly smaller mass and slower velocity

Class I structures include the Service Water Pump Room, the Control Building, the Diesel Generator Building, the Controlled Corridor, and the Drywell and Suppression Chamber. Not included is the refueling floor which is enclosed in a metal wall similar to the Turbine Building. This part of the structure is not located near the blowout, so it is protected from damage by separation.

Diesel fuel oil storage tanks are classified as Class I equipment as stated in USAR XII-2.1.2.3. The diesel fuel oil storage tanks are protected from tornado missiles by a double manhole and a soil cover. They are not affected by airborne debris.

USAR Section XII-2.2.6 states that the ERP was not designed for tornado loading. It is not required for safe shutdown. It's failure does not cause an accident.

While the Emergency Station Service Transformer (ESST) and Start-up Station Service Transformer (SSST) are not located directly adjacent to the walls that blow out, it is possible that panel fragments or other airborne missiles could damage the transformers or short the buses. Per USAR XIV-6.5.3.1.i, during a main steam line break accident, the plant assumes a Loss Of Offsite Power (LOOP). The Emergency Station Service Transformer (ESST) and Start-up Station Service Transformer (SSST) are both assumed down at this point. Therefore, they are not required.

The Condensate Storage Tanks provide station system makeup, receive system reject flow, and provide condensate for any continuous service needs and intermittent batch type services. While CST-A is used in Tech Spec 3.5.2 and is referenced as ECCS suction in USAR sections XI-9.3, VI-4.3, and VI-4.4, both Condensate Storage Tanks A and B are not designed to withstand tornado missiles. The Tech Spec and USAR sections are under revision to remove this requirement due to the lack of seismic qualification of the tanks. CST's are not required after a Main Steam Line Break.

Summary/Conclusions:

The 0.5 psid value stated in FSAR Amendment 25 was shown to be calculated by a simplified analysis (NEDC 13-028). This value is shown to be conservative when compared with the shearing force that is required for the controlling failure mechanism of the system.

The north and south Turbine Building walls between columns D, E, and F will experience blowout before 0.5 psid is achieved. When the bolt connections fail, pressure inside the structure is relieved to the atmosphere.

When the wall panels detach form the Turbine Building, they do not impact structures or equipment important to nuclear safety.

3.2 <u>Licensing and Design Bases Discussion</u>

FSAR Amendment 25 states that the Turbine Building siding will blow out at 0.5 psid thereby venting the released steam and steam/water mixture to atmosphere pressure causing complete pressure relief for the confined space (Turbine Building).

An evaluation of the Turbine Building concrete structure was performed to confirm that it is capable of remaining structurally intact without gross structural failure following a postulated SSE. This allows crediting the dose consequence mitigation assumptions related to leakage holdup and the resulting iodine plateout within the Main Condenser following a postulated Loss-of-Coolant Accident [Ref. 2.3.31].

The Turbine Building is a Class II structure [Ref. 2.3.31] that is designed to withstand a minimum wind velocity of 100 mph [Ref. 2.3.35]. Class I structures are designed to withstand tornado loads, such as tornado missiles [Ref. 2.3.36]; the Turbine Building is not designed to withstand tornado loads.

The Turbine structure is designed to withstand missile effects for normal operation, i.e., the inner and outer casings are designed to resist disk missile impact at more than 120% of rated speed [Ref. 2.3.28].

Burns and Roe Calculation Book 58 contains the structural design of columns, roofing, girts and other wall panel components in Turbine Building. This book was reviewed in the preparation of NEDC 13-028.

Samples of key Turbine Building substructures (e.g., walls, floor slabs, and columns) were evaluated for increased seismic loading resulting from a postulated SSE. The horizontal seismic acceleration input to the operating floor of the Turbine Building at elevation 932 ft-6 in. due to the Turbine Building response was assumed to be 0.3g based on a comparison with Class I structures (Reactor Building and Control Building). The evaluations show that the increase in design loadings from the original seismic Class II criteria to the postulated SSE condition do not result in stresses that exceed the allowable limits applicable to the SSE load case. Therefore, it is concluded that there is sufficient margin in the original design to ensure that the concrete portion of the Turbine Building structure will remain intact during and following an SSE. These results are based primarily on the fact that allowable stresses are increased for the SSE load case and, consequently, the increase in seismic loading is offset by the increase in allowable stresses [Ref. 2.3.32].

The Turbine Building was specifically evaluated for the potential to impact adjacent Class I structures. It was determined that the only zone of the Turbine Building that can adversely affect the integrity of an adjoining Class I structure is the area in close proximity to the Diesel Generator Building. This particular area has been designed to Class I standards [Ref. 2.3.37].

The Control Building houses instrumentation and switches required for station operation. Also located in this building is a computer room, station batteries, and components of the Reactor Protection System [Ref. 2.3.33]. This evaluation is in support of protecting the Control Building. These other areas are evaluated separately.

The applicable drawings of the Turbine Building panel walls were utilized in NEDC 13-028 and are listed as references to this document. When possible, the drawings were field verified to validate materials and dimensions of the metal wall system. In such cases, the drawing details did match the field conditions. The drawings that could not be easily verified were assumed to be accurate.

The use of several structural design codes, such as the AISC Manual of Steel Construction 6th Edition, were used, when applicable, to determine the ultimate strength of the metal wall system girt channels, connection angles, bolts and welds. Design codes are typically used to evaluate the acceptability of structural members, and not the point of ultimate failure. This evaluation uses engineering principles when the standard engineering practices (design codes) lack specific design criteria for evaluating material failure.

3.3 Design Input Considerations

CNS Safety Analysis Report

FSAR Amendment 25: 0.5 psid Turbine Building siding blowout pressure

Design Codes

AISC Manual of Steel Construction, 6th Ed.
AISC Manual of Steel Construction, 13th Ed.
AISI North American Specification for the Design of Cold-Formed Steel Structural Members, 2007 Ed.

Standards

ASTM A307: Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength, 1978 Ed.

Physical/Measured Components

Girt Channels: C10x15.3 (A36)
Connection Angles: L5x3-1/2x3/8, L4x3x3/8 (A36)
Connecting Bolts: 3/4" machine bolts (A307, Gr.A)
Angle Welds: 1/4", 5/16" (F_{E70})

Performance Requirement

Turbine Building siding will blowout at the FSAR defined blowout pressure thereby venting released steam and moisture to the atmosphere, completely relieving the confined space.

Other Documents

All other documents are referenced in Section 2.3 or referenced as inputs to the analyses in NEDC 13-028.

3.4 Discussion of Impact Screen Results

See Attached 9.1 for the Impact Screening Summary.

3.5 Relationship with Other Engineering Changes

This EC is not associated with any other Engineering Change document. However, the following discussion has been added for clarification on the different calculations associated with Turbine Building blowout panels and HELB.

NEDC 12-012, "Turbine Generator Building Siding Blowout Pressure, other than EQ Purposes," is superseded by NEDC 13-028, "Ultimate Internal Pressure of Turbine Building Blowout Panels and Metal Wall System." However, NEDC 12-012 supports NEDC 11-075, "Turbine Building High Energy Line Break," which supports a closed CAP item. For this purpose, these two calculations have not been deleted. Calculation NEDC 03-005, "Turbine Generator Building Siding Blowout Pressure," which also looks at the blowout of the Turbine Building walls, has been left in place. It provides conservative results to NEDC 02-006, "HELB EQ - RB Gothic Model." Calculation NEDC 11-122, "Design Check for Door Mark H-307 for an Overpressure of 0.75 psi," evaluates door H-307 for an overpressure of 0.75 psid. Because the blowout pressure has been determined to be lower than 0.75 psid, there are no pending changes.

3.6 Operating Experience Search Results

3.6.1 The search criterion of "Turbine Building Blowout" was used in the INPO ICES database; there were 21 results using that criterion. Only two reports appeared to be applicable to this evaluation.

3.6.1.1 **Report #164109** - Safety Analysis Overpressure Protection Not Provided for the Turbine Building

Event Summary: On March 3, 1997, with Vermont Yankee operating at 100 percent power, it was discovered that the design for Turbine Building overpressure protection, as described in the final safety analysis report (FSAR), was never installed. The Vermont Yankee FSAR description of Turbine Building overpressure protection includes a 1,000 square foot blowout panel that would actuate at an internal pressure of 0.25 psi. No venting areas of this size have been found in the Turbine Building, and investigation to date has been unable to identify any other venting methods that may have been assumed, in lieu of the blowout panels, when the plant was 2 constructed. The analysis for high energy line breaks of the main steam and feedwater piping in the Turbine Building credits the blowout panel. Once the blowout panels actuate, a steam cloud would be released in a matter seconds. Absent the blowout panel, the potential higher pressure raised concern for the structural capability of certain block walls and ductwork in operation.

The preliminary FSAR states that steam would travel up through the Turbine Building, partially condensing on the walls and equipment. The increase in pressure in the Turbine Building would result in portions of the siding and roof of the building being torn from the building structural members releasing steam to the environment. It is not clear from the current FSAR description and that of the preliminary FSAR as to whether the anticipated change to blowout panels was to be made as a result of a decision that an engineered feature needed to be installed or simply a case of interpretation.

Due to the age of this event, it is not possible to determine the root cause for the failure to install the device described in the FSAR. The difference in the terminology between the plant's preliminary safety analysis report (PSAR) and the original final safety analysis report alludes to an apparent cause. The PSAR identified that building overpressure was to be prevented by failure of Turbine Building siding. In contrast, the original FSAR described a blowout panel, indicating that a formally engineered relief path was intended. It appears that a design change was planned which would have installed a specific relief device in the Turbine Building. However, because the investigation was hampered by the lack of documentation and unavailability of individuals involved in the early licensing process, it could not be positively determined if the change was simply a change in wording or the result of actual change of intent.

Corrective actions include modification of the vital switchgear room heating, ventilation and air conditioning system to prevent an HELB in the Turbine Building from introduction steam into the switchgear rooms and developing operability determinations to support bases for continued safe operation of the plant.

Applicability: The original Vermont Yankee PSAR indicated that the Turbine Building siding would fail and thereby create a relief path for internal pressure to escape to the atmosphere.

3.6.1.2 **Report #153158** - Turbine and Reactor Building Blowout Panels Determined to be Outside Plant Design Basis

Event Summary: At 1000 hours on October 25, 1993, with the reactor at 100 percent power, it was determined that the relief (blowout) panels in the Nine Mile Point Unit 1 (NMP1) Turbine and Reactor Buildings would not blow out at the design pressure of 45 psf because the bolt fasteners for the panels were larger than designed and had a higher ultimate strength than designed. The initial engineering evaluation of this condition erroneously determined that the Turbine and Reactor Building panels would blow out at 60 and 53 psf, respectively, to relieve internal building pressure prior to structural failure of the buildings.

On March 27, 1995, during refueling outage 13 (RFO13), it was determined that there was an error in the design assumptions for load distribution. Revised calculations confirmed that the relief panels would not blow out until the internal building pressure exceeded the minimum documented building structural design of 80 psf. The cause of this event was inadequate quality control measures during initial construction which resulted in oversize bolts being installed in the relief panels.

Prior to restart from RFO13, the panel attachment design was revised, the size of the bolts was verified, and every other bolt was removed from the relief panels to reduce their blowout point to a value below the documented building structural capability. Additionally, appropriate Engineering personnel were counseled regarding verification of design assumptions.

The blowout panel design uses panels of corrugated siding approximately 2 feet high by 20 feet long. The horizontal ends of these panels are firmly fastened to steel angle bars which are then connected to the building structural steel by shear bolts. The top and bottom of the individual panels are not structurally fastened to adjacent panels, and, because of the corrugation, the panels are relatively flexible in the vertical direction. For these reasons, the load resulting from an internal pressure is transferred horizontally only, however, the engineer evaluated both a horizontal and vertical transfer of load in the design calculation. As a result, the incorrect calculation conclusions were used to determine the blowout panels would separate from the building structures at internal pressures of 60 psf (Turbine Building) and 53 psf (Reactor Building) by tearing of the panel metal at the top and bottom of the Blowout Panels.

Applicability: The Nine Mile Point Turbine and Reactor Building Blowout Panel analysis determined the limiting failure mechanism of the system to be the bolt fasteners.

3.7 Margin Analysis

The analysis preformed in NEDC 13-028 determined the metal wall systems on the north and south Turbine Building walls will fail (create a vent path to the atmosphere) before 0.5 psid pressure is present in the Turbine Building. However, the purpose of the calculation, and this EC, is to reconstitute the FSAR Amendment 25 pressure value of 0.5 psid; therefore, margin is available in the calculation to confidently claim that the applicable panels will fail at 0.5 psid.

4. Impact on Current Operational Basis

This Engineering Change does not adversely impact the current operational basis of the plant, but on the contrary, by validating the FSAR Amendment 25 blowout pressure the current operational basis is reinforced.

5. Engineering Requirements Necessary to Implement the Change

There are no engineering requirements associated with this Engineering Change.

6. Engineered Materials List and Vendor Technical Information

6.1 Engineered Materials

None

6.2 Vendor Technical Information

The applicable vendor as-built drawings are listed in Section 2.3.

7. Special Process Requirements

No special process requirements are associated with this Engineering Change.

8. Test & Inspections

No new tests or inspections are associated with this Engineering Change.

9. Attachments

9.1 Impact Screening Summary

9.2 PAD (Revision 1)

ATTACHMENT 9.3 — **IMPACT SCREENING SUMMARY**

SHEET 1 OF 7

INSTRUCTIONS:

Identify the potentially impacted and the unaffected areas by checking the appropriate Yes or No block on the Impact Screening Criteria.

If a direct determination cannot be made based on the impact screening questions, the detailed questions in Attachment 9.4 may be used to identify potential impacts.

The completion of detailed questions in Attachment 9.4 is recommended but not required.

Contact potentially impacted groups if assistance is required.

Engineering Change No.: (EC) EE 13-041 Rev. No.: 2

Prepared by: Taylor Sutton Date: 7/27/15

DESIGN ENGINEERING DISCIPLINES	Potential Impact	
Civil / Structural Design Engineering • Does the proposed activity involve any civil / structural (including seismic) design changes or activities? OR Does the proposed activity involve any piping engineering design changes or activities?	☒ YES	☐ NO
Paintings and Coatings Engineering • Does the proposed activity involve the use of a coating or a component coated with a coating not previously evaluated under the Paintings and Coatings program?	☐ YES	☒ NO
Electrical Design Engineering • Does the proposed activity involve any station or switchyard electrical design, large power transformers, or settings changes or activities? (SOER 10-1)	☐ YES	☒ NO
Instrumentation and Controls Design Engineering • Does the proposed activity involve any I&C design or settings changes or activities?	☐ YES	☒ NO
Mechanical Design Engineering • Does the proposed activity add/remove/replace insulation, aluminum or other metallic/non-metallic sources of debris in the reactor/containment building or involve any mechanical design changes or activities?	☐ YES	☒ NO
PSA Engineering • Does the proposed activity impact or involve changes to plant evaluations or probabilistic safety assessments? • Does the proposed activity impact or involve changes to the Emergency Operating Procedures, Abnormal Operating Procedures, or Severe Accident Procedures or does it add, remove or modify SSCs included in a Maintenance Rule function?	☐ YES	☒ NO
Nuclear Analysis • Does the proposed activity impact or involve changes to plant evaluations, Technical Specifications, Technical Requirements Manual, or require a full 50.59 Evaluation?	☐ YES	☒ NO
Cyber Security Analysis • Does the proposed activity impact or involve changes to Cyber Security Controls, or require cyber security assessment, evaluation as per 10CFR73.54 and Procedure 11.CYBER-SECURITY?	☐ YES	☒ NO

DESIGN ENGINEERING DISCIPLINES (CONTINUED)	Potential Impact	
FERC / NERC Impact • Does the proposed activity impact or involve change to FERC / NERC or Regional Entity compliance documents?	☐ YES	☒ NO
B.5.b and other "Beyond Design Basis" considerations • Does the proposed activity impact or involve changes to strategies related to B.5.b or other issues considered "beyond design basis" including Severe Accident Procedures and Guidelines?	☐ YES	☒ NO

DESIGN ENGINEERING PROGRAMS	Potential Impact	
ASME Section III Specifications • Does the proposed activity add, delete, or modify information required by ASME Section III to be contained in a design specification?	☐ YES	☒ NO
Cable and Raceway Program • Does the proposed activity involve any changes to cable trays, raceways or the associated documentation?	☐ YES	☒ NO
Electronic Databases (EDB) • Does the proposed activity involve any changes to electronic databases?	☐ YES	☒ NO
EQ Program (10CFR50.49, NUREG 0588, Reg Guide 1.89) • Does the proposed activity involve any changes to the EQ Program?	☐ YES	☒ NO
Hydrogen Control Program (10 CFR 50.44) *(if applicable)* • Does the proposed activity impact equipment or materials related to hydrogen control?	☐ YES	☒ NO
Human Factors Program • Does the proposed activity involve control panel design including layout and labeling, visual displays, operator aids, auditory signals or environment in the control room, cable spreading room, and other control panel locations?	☐ YES	☒ NO
Margin Management • Does the proposed activity impact or involve any change to design, licensing or operations margins?	☐ YES	☒ NO
Reg. Guide 1.97 / PAM (Post Accident Monitoring) • Does the proposed activity involve Reg. Guide 1.97 (Post Accident Monitoring) Indications?	☐ YES	☒ NO

MAINTENANCE	Potential Impact	
Electrical Maintenance • Does the proposed activity require an Electrical Maintenance review to identify affected procedures, required actions and required training?	☐ YES	☒ NO
I&C Maintenance • Does the proposed activity require an I&C Maintenance review to identify affected procedures, required actions and required training?	☐ YES	☒ NO

MAINTENANCE (CONTINUED)	Potential Impact	
Mechanical Maintenance • Does the proposed activity require a Mechanical Maintenance review to identify affected procedures, required actions and required training?	☐ YES	☒ NO

NUCLEAR ENGINEERING	Potential Impact	
Nuclear Fuel Design • Does the proposed activity impact or involve the design, performance or storage of nuclear fuel?	☐ YES	☒ NO
Reactivity Management Program • Does the proposed activity impact or involve the reactor system, reactor controls, reactor chemistry, related systems, potential core and spent fuel damage, spent fuel, reactor coolant pressure boundary or reactor system procedures?	☐ YES	☒ NO

PROCESS OR PROGRAM IMPACT SCREENING CHECKLIST	Potential Impact	
Computer Support and Software • Does the proposed activity impact or involve changes to plant computer software or firmware or impact Software Quality Assurance (SQA)?	☐ YES	☒ NO
Chemistry and Environmental Impact • Does the proposed activity impact or involve any changes to plant chemistry requirements, operations or procedures, or any changes to the environment? • Does the proposed activity impact or involves the site radiological ground water monitoring program?	☐ YES	☒ NO
Radiation Protection (RP) Program Impact • Does the proposed activity impact or involve any changes to the RP program?	☐ YES	☒ NO
Operations • Does the proposed activity impact or involve changes to Operations procedures, training or operator actions?	☐ YES	☒ NO
Planning, Scheduling and Outage (PS&O) • Does the proposed activity require a PS&O review to identify required design and installation information?	☐ YES	☒ NO
MP&C (Inventory) • Does the proposed activity impact or involve any addition or removal of equipment from the inventory? • Does the proposed activity impact or involve any Procurement of Quality material from non-qualified suppliers?	☐ YES	☒ NO
Procurement Engineering • Does the activity impact or involve any procurement activities?	☐ YES	☒ NO
Vendor Manual Control • Does the change add, remove, or modify a structure, system, or component?	☐ YES	☒ NO

PROGRAMS AND COMPONENTS	Potential Impact	
ASME Appendix J (Primary Containment Leak Rate Testing) Program • Does the proposed activity impact or involve any changes to primary containment leak rate testing?	☐ YES	☒ NO
ASME Containment In-service Inspection (IWE / IWL) Program • Does the proposed activity impact or involve the containment pressure boundary or associated moisture barriers or a support for the containment pressure boundary? This includes "software only" changes that do not physically change the hardware such as re-rating of pressures or temperatures to include changes to documented information on these items or additional documented information for these items. • Does the proposed activity limit access to containment surfaces for inspection? • Involve disassembly of a bolted connection which forms a portion of the containment boundary?	☐ YES	☒ NO
ASME In-service Inspection (ISI) Program • Does the proposed activity add, delete, or modify an ASME Section XI pressure boundary item or a support for an ASME Section XI pressure boundary item? This includes "software only" changes that do not physically change the hardware like such as re-rating of pressures or temperatures. (ASME Section XI items include ASME Class 1, 2, 3, or B31.1 treated as ISI Class 2 or 3 (T2 and T3) components, parts, or appurtenances such as pipe or pressure vessel walls, valve bodies and pump casings.	☐ YES	☒ NO
ASME Section XI Repair / Replacement Program • Does the proposed activity involve any mechanical component within the ASME XI program / boundaries? Does the proposed activity add, delete, or modify an ASME Section XI pressure boundary item or a support for an ASME Section XI pressure boundary item? (ASME Section XI items include ASME Class 1, 2, 3, MC, and CC, as well as B31.1 treated as ISI Class 2 or 3 (T2 and T3) components, parts, or appurtenances such as pipe or pressure vessel walls, valve bodies and pump casings).	☐ YES	☒ NO
ASME In-service Testing Program • Does the proposed activity impact or involve any item (safety related or non safety related) that may affect the performance or testing of a safety related pump or valve? • Does the proposed activity impact the function or functional classification of any pump or valve as stated in the IST program documents?	☐ YES	☒ NO
Air Operated Valve (AOV) Program • Does the proposed activity impact or involve the design, operation or testing of AOVs?	☐ YES	☒ NO
Buried Piping and Tanks (BP&T) Program • Does the proposed activity impact or involve any changes to piping or cathodic protection of Buried Piping & Components?	☐ YES	☒ NO
Boric Acid Corrosion (BAC) Program • Does the proposed activity impact or involve any increase in the likelihood of boric acid formation or corrosion?	☐ YES	☒ NO
Check Valve Program • Does the proposed activity impact or involve the design, operation or testing of check valves?	☐ YES	☒ NO

PROGRAMS AND COMPONENTS (CONTINUED)	Potential Impact	
Cobalt Reduction • Does the proposed activity involve the potential to create conditions that could impact personnel dose such as the introduction (or reintroduction) of any cobalt containing materials (such as Stellite or high cobalt stainless steel) into the primary system or processes such as electro-polishing or advanced surface treatments?	☐ YES	☒ NO
Control Room Habitability • Does the proposed activity impact or involve changes that affect the temperature or radiological environmental conditions in the Main Control Room?	☐ YES	☒ NO
Electrical Circuit Breaker, Relay and Electrical Equipment Testing • Does the proposed activity impact or involve changes to the functional testing of circuit breaker, relay or electrical equipment?	☐ YES	☒ NO
Flow Accelerated Corrosion (FAC) Program • Does the proposed activity involve any changes (e.g.: configuration, velocity, flow rate, pressure, temperature, material, weld location, etc.) to piping systems included in the FAC Program ?	☐ YES	☒ NO
Heat Exchanger Program • Does the proposed activity impact or involve the design, operation or testing of a component in the heat exchanger program?	☐ YES	☒ NO
Predictive Maintenance Program • Does the proposed activity impact or involve any aspect of the storage, use and testing of lubricants? • Does the proposed activity involve thermography? • Does the proposed activity involve vibration monitoring?	☐ YES	☒ NO
Microbiological Induced Corrosion (MIC) Program Impact • Does the proposed activity involve piping containing untreated or stagnant water or open to the atmosphere?	☐ YES	☒ NO
Motor Operated Valve (MOV) Program • Does the proposed activity impact or involve the design, operation or testing of MOVs?	☐ YES	☒ NO
Plant Thermal Performance Program • Does the proposed activity impact or involve plant thermal performance?	☐ YES	☒ NO
Preventive Maintenance Program • Does the proposed activity impact or involve periodic testing or performance of SSCs? or • Does the proposed activity add, modify or delete any Environmental Qualification (EQ) maintenance requirement or replacement frequency to ensure the component(s) maintains its qualification status?	☐ YES	☒ NO
PT Curves • Does the proposed activity add, delete, or modify the basis (as contained in each sites reactor vessel surveillance material testing program) for the Pressure / Temperature Limit Curves (fluence, pressure, temperature, reactor materials)?	☐ YES	☒ NO

PROGRAMS AND COMPONENTS (CONTINUED)	Potential Impact	
Relief Valve Program • Does the proposed activity impact or involve the design, operation and testing of relief valves, safety valves, vacuum breaker valves or rupture disc?	☐ YES	☒ NO
RPV Internals Program (BWR VIP) • Does the proposed activity impact or involve any change to the reactor internals? This includes "software only" changes that do not physically change the hardware such as re-rating of pressures or temperatures. • Does the proposed activity impact or involve or related documentation to include neutron fluence or neutron fluence calculations? • Does the proposed activity impact or involve or changes to core flow characteristics or core flow characteristics?	☐ YES	☒ NO
Welding Program • Does the proposed activity impact or involve a special process, such as welding, brazing or soldering?	☐ YES	☒ NO
Safety Program • Does the proposed impact or activity involve personnel or industrial safety?	☐ YES	☒ NO
System Engineering • Does the proposed activity impact or involve any changes to system configuration, function or performance, etc., for Maintenance Rule or other system? • Does the proposed activity impact or involve any changes to system procedures, maintenance or operation, etc.?	☐ YES	☒ NO
Training Program • Does the proposed activity involve existing training requirements or create the need for new training?	☐ YES	☒ NO
Simulator Impact • Does the proposed activity impact or involve changes to the Simulator?	☐ YES	☒ NO
Fire Protection Program • Will the proposed activity be performed as "Work at Risk" thus requiring notification of NEIL representative at conceptual design phase? • Does the proposed activity result in a change to any existing or install a new fire protection water supply system (tanks, pumps, underground fire main, hydrants), fire suppression system (water, gaseous), fire alarm or detection system? • Does the proposed activity result in a change to any existing fire barrier, fire barrier penetration seal, or install new fire barriers or fire barrier penetration seals? • Does the proposed activity result in a change to any in situ combustible loading or add new in-situ combustible materials (cable insulation, thermal insulation, oil, lubricants, plastics, etc.)? • Does the proposed activity result in a change to any administrative control of the Fire Protection Program (FPP) (e.g., hot work, transient combustible, impairments, fire brigade composition or training)? • Does the proposed activity result in a change to any system or component credited in the Nuclear Safety Capability Assessment (NSCA)? Reference NEDC 11-019. • Does the proposed activity result in a change to the Fire Protection Program (FPP) as described in Procedure 0.23? Also, refer to the Fire Safety Analysis calculation for the specific fire areas affected by the change.	☐ YES	☒ NO

PROGRAMS AND COMPONENTS (CONTINUED)	Potential Impact	
Fire PRA • Does the proposed activity result in a change that would affect the PRA fire model? Change to a pump, motor, fan, MOV. Change to any cable(s) or cable routing. Change to equipment location. Change to in-situ combustibles (cable insulation, lubricants, plastics). Change to or addition of an ignition source (transformer, motor, electrical cabinet, junction box or switchgear).	☐ YES	☒ NO
NEIL Design Review • Is this design change a new structure? • Does addition, renovation or alteration change the occupancy classification of any part of a NEIL insured structure? • Does the design change involve the addition of a new NEIL required fire protection system? • Does the change significantly add to, remove or alter an existing NEIL required fire detection or fire protection system? For example, this does not include relocation of less than 10% of the fire detectors or suppression heads/nozzles for a single system while still maintaining code compliance? • Does the change create an addition to an existing NEIL insured structure? • Does the change involve replacement of the roof decking and/or covering? • Does the design change affect an interior finish such that it would not meet the requirements of the NEIL Property Loss Control standards (PLCS)? • Does the design change add to, renovate or alter the cooling tower fill or supports? • Does the design change add oil filled components over 50 gallons oil capacity? • Does the change add to, renovate or alter oil collection systems, fire barrier or fire protection systems for oil filled components?	☐ YES	☒ NO

I. OVERVIEW

PAD Tracking #: 1120

PAD Rev. #: 1

Proposed Activity / Document: EE13-041 and/or NEDC13-028 Change/Rev. #: 2

Description of Proposed Activity: Turbine Building Blowout Panels / Metal Wall System

II. DOCUMENT REVIEW

Provide the requested information for each item below.

1. For documents available electronically:

   a. List search engine or documents searched, and keywords used: ESAR, Tech Specs. "Siding" "HELB" "BLDG" "Panel" "Girt" "Turbine Building"

   b. List relevant sections of controlled electronic documents reviewed:

   CNS USAR Section II-3.2.2, *"Wind"*.
   CNS USAR Section XII-2.1.3.1, *"Principal Class II Structures"*.
   CNS USAR Section XII-2.2.2, *"Turbine Building"*.
   CNS USAR Section XII-2.3.3.1, *"Wind Loads"*.
   CNS USAR Section XII-2.3.3.2.2, *"Tornado Generated Missiles"*
   CNS USAR Section IV-12, *"High Energy Line Break (HELB) Study"*

2. Documents reviewed manually (hardcopy):

   NEDC 03-005, Rev. 0 thru 4
   NEDC 12-012, Rev. 1
   NEDC 11-075, Rev. 1

3. For those documents that are not reviewed either electronically or manually, use the specific questions provided in Sections III and IV of Attachment 9.2 of 0-EN-LI-100 as needed. Document below the extent to which the Attachment 9.2 questions were used.

   N/A

III. PROCESS REVIEW

Does the proposed activity affect, invalidate, or render incorrect, OR have the potential to affect, invalidate, or render incorrect, information contained in any of the following processes? Associated regulations and procedures are identified with each process below.

PROCESS (Regulations / Procedures)	YES	NO	REVIEW RESULTS
Radwaste / Process Control Program (PCP) (0.PCP.1 or contact the Radiation Protection Dept.)	☐	☒	——
Radiation Protection / ALARA (10 CFR 20 / 9.EN-RP-110 or contact the Radiation Protection Dept.)	☐	☒	——

ATTACHMENT 9.1 PROCESS APPLICABILITY DETERMINATION FORM

PROCESS (Regulations / Procedures)	YES	NO	REVIEW RESULTS
Inservice Inspection Program (10 CFR 50.55a / EN-DC-120, -351)	☐	☒	———
Inservice Testing Program (10 CFR 50.55a / 3-EN-DC-332)	☐	☒	———
Maintenance Rule Program (10 CFR 50.65 / EN-DC-203, -204, -205, -206, -207)	☐	☒	———
Containment Leakage Rate Testing (Appendix J) Program (10 CFR 50 Appendix J / 3.40)	☐	☒	———

IF any box is checked "Yes," THEN contact the appropriate department to ensure that the proposed change is acceptable and document the results in the REVIEW RESULTS column.

IV. LICENSING BASIS DOCUMENT REVIEW

Does the proposed activity affect, invalidate, or render incorrect, OR have the potential to affect, invalidate, or render incorrect, information contained in any of the following Licensing Basis Document(s)? Associated regulations and procedures are identified with each Licensing Basis Document below.

LICENSING BASIS DOCUMENTS (Regulations / Procedures)	YES	NO	REVIEW RESULTS OR SECTIONS AFFECTED OR LBDCR #
Quality Assurance Program for Operation Policy Document (QAPD) (10 CFR 50.54(a) / 0.29.4)	☐	☒	———
Fire Protection Program (FPP) [includes the Fire Hazards Analysis (FHA), Appendix R SSAR, 0.23] OL Condition, 10 CFR 50.48 / 0.29.4)	☐	☒	———
Emergency Plan (10 CFR 50.54(q) / 0.29.4)	☐	☒	———
Safeguards Plan and Cyber Security Plan (10 CFR 50.54(p) / 0.29.4 or contact the site Security / IT Dept.)	☐	☒	———
Operating License (OL) / Technical Specifications (TS) (10 CFR 50.90 / 0.29.1, 0-EN-LI-103)	☐*	☒	———
TS Bases (10 CFR 50.59 / 0-EN-LI-100 /0-EN-LI-101, 0.29.1)	☐	☒	———
Technical Requirements Manual (TRM) (including TRM Bases) (10 CFR 50.59 / 0-EN-LI-100 / 0-EN-LI-101, 0.29.1)	☐	☒	———
Core Operating Limits Report (COLR), (TS Administrative Controls, 10.32, 0-EN-LI-100, 0-EN-LI-101)	☐	☒	———
Off-site Dose Assessment Manual (ODAM) (TS Administrative Controls or 10 CFR 50.59 / 0.29.1 or 0-EN-LI-100 / 0-EN-LI-101)	☐	☒	———
Updated Safety Analysis Report (USAR) (10 CFR 50.71(e) / 0.29.2, 0-EN-LI-100, 0-EN-LI-101)	☐	☒	———
Storage Cask Certificate of Compliance (10 CFR 72.244 / 0.29.9)	☐*	☒	———
Cask Safety Analysis Report (CSAR) (10 CFR 72.70 or 72.248 / 0.29.9, 0-EN-LI-100, 0-EN-LI-112)	☐*	☒	———
10 CFR 72.212 Evaluation Report (212 Report) (10 CFR 72.48 / 0-EN-LI-100, 0-EN-LI-112, 0.29.1)	☐	☒	———

LICENSING BASIS DOCUMENTS (Regulations / Procedures)	YES	NO	REVIEW RESULTS OR SECTIONS AFFECTED OR LBDCR #
NRC Orders (10 CFR 50.90 / 0-EN-LI-103 or as directed by the Order)	☐*	☒	———
NRC Commitments and Obligations (0.42.1)	☐*	☒	———
Site Specific CFR Exemption (10 CFR 50.12, 10 CFR 55.11, 10 CFR 55.13, 10 CFR 72.7)	☐*	☒	———

*STOP. Contact the site Licensing Department for further guidance.

IF any box is checked "Yes," THEN ensure that any required regulatory reviews are performed in accordance with the referenced procedures. Prepare an LBDCR per procedure 0.29.1 if a LBD is to be changed, and document any affected sections or the LBDCR #. Briefly discuss how the LBD is affected in Section VII.A.

V. 10 CFR 50.59 / 10 CFR 72.48 APPLICABILITY

Can the proposed activity be dispositioned by one of the following criteria? Check the appropriate box (if any).

☐	An approved, valid 50.59/72.48 Evaluation covering associated aspects of the proposed activity already exists. Reference 50.59/72.48 Evaluation # N/A (if applicable) or attach documentation. Verify the previous 50.59/72.48 Evaluation remains valid.
☐	The NRC has approved the proposed activity or portions thereof or an approved license amendment addresses the proposed activity. Reference the approval document: N/A
☐	The proposed activity is controlled by one or more specific regulations. Examples of specific regulations are: • Quality Assurance Program (10 CFR 50 Appendix B) • Safeguards Plan (50.54(p)) • Emergency Plan (50.54(q)) • Fire Protection (operating license condition) See NEI 96-07 Section 4.1 for additional guidance on specific regulations. Reference the controlling specific regulation(s): N/A

IF the entire proposed activity can be dispositioned by the criteria in Section V, THEN proceed to Section VII and provide basis for conclusion in Section VII.A.

Otherwise, continue to Section VI to perform a 50.59 and/or 72.48 Screening, or perform a 50.59 and/or 72.48 Evaluation in accordance with 0-EN-LI-101 and/or 0-EN-LI-112.

VI. 50.59 / 72.48 SCREENING REVIEW

VI.A 50.59/72.48 SCREENING (Check the appropriate boxes.)

☒ 10 CFR 50.59 Screening criteria are met. [10 CFR 50.59(c)(1)]

The proposed activity meets all of the following criteria regarding design function:

- Does not adversely affect the design function of an SSC as described in the USAR; AND
- Does not adversely affect a method of performing or controlling a design function of an SSC as described in the USAR; AND
- Does not adversely affect a method of evaluation that demonstrates intended design function(s) of an SSC will be accomplished as described in the USAR; AND
- Does not involve a test or experiment not described in the USAR.

☐ The proposed activity does not involve structures, systems, or components controlled by 10 CFR 50.59.

☐ 10 CFR 72.48 Screening criteria are met. [10 CFR 72.48(c)(1)] (Applicable to sites with an ISFSI)

The proposed activity meets all of the following criteria regarding design function:

- Does not adversely affect the design function of an SSC as described in the CSAR; AND
- Does not adversely affect a method of performing or controlling a design function of an SSC as described in the CSAR; AND
- Does not adversely affect a method of evaluation that demonstrates intended design function(s) of an SSC will be accomplished as described in the CSAR; AND
- Does not involve a test or experiment not described in the CSAR.

☒ The proposed activity does not involve structures, systems, or components controlled by 10 CFR 72.48.

IF either of the 50.59 or 72.48 Screening criteria are met, THEN complete VI.B below as appropriate and proceed to Section VII.

IF the proposed activity does not meet the applicable criteria, THEN perform a 50.59 or 72.48 Evaluation in accordance with 0-EN-LI-101 or 0-EN-LI-112, as appropriate, attach a copy of the Evaluation to this form, and proceed to Section VII.

IF the activity does not involve systems, structures, or components controlled by 10 CFR 50.59 or by 10 CFR 72.48, THEN a 50.59 or 72.48 Screening is not required, as appropriate, and proceed to Section VII.

VI.B BASIS

Provide a clear, concise basis for determining the proposed activity may be screened out such that a third-party reviewer can reach the same conclusions. Refer to NEI 96-07 Section 4.2 for guidance. Provide supporting documentation or references as appropriate.

- Does not adversely affect the design function of an SSC as described in the USAR; AND

  The siding (such as: panels, girts, sub-girts, sag rods and connections) is considered as an architectural component and is not credited as a structural SSC for the design of the Turbine Generator Building (TGB) superstructure (Note: the siding only transfers load to the TGB Superstructure SCC) . The HELB Event has the potential to generate debris and/or missiles', but will be minimized by the sag rod and attachment to the roof truss and girds. In addition, the Reactor Building Tornado Event for missiles impact (Ref. CNS USAR Section XII-2.3.3.2.2) envelops this siding system missile type.

- Does not adversely affect a method of performing or controlling a design function of an SSC as described in the USAR; AND

  There are no changes to any procedure, or any other document, as a result of this assessment. EE13-041 ~~Rev. 0~~ *Rev. 2* and/or Calculation NEDC13-028~~, Rev. 0~~, determines the maximum internal pressure at which the TGB siding can be maintained. Therefore, there are no adverse effects on USAR described SSC design function performance or control.

- Does not adversely affect a method of evaluation that demonstrates intended design function(s) of an SSC will be accomplished as described in the USAR; AND

  This assessment of the TGB siding(s) capability to resist an internal pressure does not impact either the design bases for the structure or safety analyses for the HELB. The HELB analyses' bounding peak and temperature curves will not be altered and/or changed by this assessment. However, this evaluation does demonstrate the code approach and/or methodology and values used to arrive at the FSAR Amendment 25 (Ref. CNS USAR Section IV-12, *"High Energy Line Break (HELB) Study"*, 110) bounding value for the TGB limiting internal pressure capacity as being conservative.

- Does not involve a test or experiment not described in the USAR.

  No USAR-described tests or experiments are involved in the evaluation of the TGB siding system internal pressure capacity.

Summary:

This EE13-041 and subsequent supporting analysis NEDC13-028 is an assessment of the architectural siding's structure, system and/or components (SSC) to determine the weak link and subsequently the maximum internal pressure that can be maintained as a result of a High Energy Line Break Event. This assessment does not change the Design Basis of the TGB and therefore has no impact on the design requirements for this Class II structure as defined in the USAR. However, this evaluation of progressive collapse of the TGB architectural siding system, consisting of the girt and connections, documents the code methodology and input values to derive an internal pressure value of 0.5 psid as reported in the FSAR Amendment 25. In addition, the analysis also ~~demonstrates~~ *identifies* this system's ~~internal pressure limiting condition as 0.3 psid occurring at full yield of angle connections relative to this event scenario~~ *weakest component that will fail and subsequently initiate the release of panels from the north and south TGB walls when the FSAR defined pressure is present in the TGB.*

VII. REGULATORY REVIEW SUMMARY

VII.A GENERAL REVIEW COMMENTS (Provide pertinent review details and basis for conclusions if not addressed elsewhere in form.)

None

VII.B CONCLUSIONS

1. Is a change to an LBD being initiated? ☐ Yes ☒ No

   IF "Yes," THEN enter the appropriate change control process and include this form with the change package.

2. Is a 10 CFR 50.59 Evaluation required? ☐ Yes ☒ No

   IF "Yes," THEN complete a 50.59 Evaluation in accordance with 0-EN-LI-101 and attach a copy to the change activity.

3. Is a 10 CFR 72.48 Evaluation required? ☐ Yes ☒ No

   IF "Yes," THEN complete a 72.48 Evaluation in accordance with 0-EN-LI-112 and attach a copy to the change activity.

4. Are any other regulatory reviews required? ☐ Yes ☒ No

   IF "Yes," THEN specify the review(s) required, below, complete the review, and attach a copy of the review result(s) to this PAD.

   Review(s) required prior to PAD completion: N/A

VIII. SIGNATURES [1]

Preparer: TQD-901P, R, or 926 — Jerry Horn / [signature] / DED / 7/28/15
Name (print) / Signature / Department / Date

Reviewer: TQD-901R or 926 — Jeff Daniel / [signature] / DED / 7/28/15
Name (print) / Signature / Department / Date

Process Applicability Exclusion

Procedure Owner: ______________________
Name (print) / Signature / Department / Date

Upon completion, forward this PAD form to the appropriate organization for record storage. If the PAD form is part of a process that requires transmittal of documentation, including PAD forms, for record storage, then the PAD form need not be forwarded separately.

[1] Signatures may be obtained via manual methods (e.g., ink signature) or telecommunication.

DESIGN VERIFICATION COVER PAGE

Document No. EE 13-041	Revision No. 2	Page 1 of 6
Title: Turbine Building Blowout Panels / Metal Wall System		
☒ Quality Related		
DV Method: ☒ Design Review ☐ Alternate Calculation ☐ Qualification Testing		

VERIFICATION REQUIRED	DISCIPLINE	VERIFICATION COMPLETE AND COMMENTS RESOLVED (DV print, sign, and date)
☐	Electrical	
☐	Mechanical	
☐	Instrument and Control	
☒	Civil/Structural	Matthew Nienaber [signature] 7-30-15
☐	Nuclear	
☐		
☐		

Originator: (individual requesting DV)	Taylor Sutton [signature] 7/30/15 Print/Sign/Date After Comments Have Been Resolved

ATTACHMENT 9.6 — DESIGN VERIFICATION CHECKLIST

Sheet 1 of 4

IDENTIFICATION:				DISCIPLINE:
Document Title: Turbine Building Blowout Panels / Metal Wall System				x Civil/Structural
				☐ Electrical
Doc. No.:	EE 13-041	Rev. 2	QA Cat. QR	☐ I & C
Verifier:	Matthew Nienaber (Print)	[signature] (Sign)	7.30.15 (Date)	☐ Mechanical
				☐ Nuclear
Manager authorization for supervisor performing Verification. ☒ N/A	(Print)	(Sign)	(Date)	☐ Other

METHOD OF VERIFICATION:

Design Review x — Alternate Calculations ☐ — Qualification Test ☐

The following basic questions are addressed as applicable, during the performance of any design verification. [ANSI N45.2.11 – 1974]

NOTE The reviewer can use the "Comments/Continuation sheet" at the end for entering any comment/resolution along with the appropriate question number. Additional items with new question numbers can also be entered.

1. Design Inputs – Were the inputs correctly selected and incorporated into the design?

   (Design inputs include design bases, plant operational conditions, performance requirements, regulatory requirements and commitments, codes, standards, field data, etc. All information used as design inputs should have been reviewed and approved by the responsible design organization, as applicable.

   All inputs need to be retrievable or excerpts of documents used should be attached. Examples may include Material Test Reports (CMTRs), verified As-Builts, approved procedures, approved drawings, publications, etc. Ⓟ[2]

   See site specific design input procedures for guidance in identifying inputs.)

   Yes ☒ No ☐ N/A ☐

2. Assumptions – Are assumptions necessary to perform the design activity adequately described and reasonable? Where necessary, are assumptions identified for subsequent re-verification when the detailed activities are completed?

   Yes ☒ No ☐ N/A ☐

3. Quality Assurance – Are the appropriate quality and quality assurance requirements specified?

   Yes ☒ No ☐ N/A ☐

4. Codes, Standards and Regulatory Requirements – Are the applicable codes, standards and regulatory requirements, including issue and addenda properly identified and are their requirements for design met?
Yes ☒ No ☐ N/A ☐

5. Construction and Operating Experience – Have applicable construction and operating experience been considered?
Yes ☐ No ☐ N/A ☒

6. Interfaces – Have the design interface requirements been satisfied and documented?
Yes ☐ No ☐ N/A ☒

7. Methods – Was an appropriate design or analytical (for calculations) method used?
Yes ☒ No ☐ N/A ☐

8. Design Outputs – Is the output reasonable compared to the inputs?
Yes ☒ No ☐ N/A ☐

9. Parts, Equipment and Processes – Are the specified parts, equipment, and processes suitable for the required application?
Yes ☐ No ☐ N/A ☒

10. Materials Compatibility – Are the specified materials compatible with each other and the design environmental conditions to which the material will be exposed?
Yes ☐ No ☐ N/A ☒

11. Maintenance requirements – Have adequate maintenance features and requirements been specified?
Yes ☐ No ☐ N/A ☒

12. Accessibility for Maintenance – Are accessibility and other design provisions adequate for performance of needed maintenance and repair?
Yes ☐ No ☐ N/A ☒

13. Accessibility for In-service Inspection – Has adequate accessibility been provided to perform the in-service inspection expected to be required during the plant life? Does this design provide adequate access to other components that require inservice inspection and testing?
Yes ☐ No ☐ N/A ☒

14. Radiation Exposure – Has the design properly considered radiation exposure to the public and plant personnel? Include review of USAR Section XII, Subsection 3.0, for affects to plant radiation zones during operation and shutdown conditions.
Yes ☐ No ☐ N/A ☒

15. Acceptance Criteria – Are the acceptance criteria incorporated in the design documents sufficient to allow verification that design requirements have been satisfactorily accomplished?
Yes ☒ No ☐ N/A ☐

16. Test Requirements – Have adequate pre-operational and subsequent periodic test requirements been appropriately specified?
    a. Are air operated components that have been added or affected by this change, tested for fail position on loss of air, or loss of electrical power?
    b. Is adequate qualification testing specified for new or unproven equipment designs?

    Yes ☐ No ☐ N/A ☒

17. Handling, Storage, Cleaning and Shipping – Are adequate handling, storage, cleaning and shipping requirements specified?

    Yes ☐ No ☐ N/A ☒

18. Identification Requirements – Are adequate identification requirements specified?

    Yes ☐ No ☐ N/A ☒

19. Records and Documentation – Are requirements for record preparation, review, approval, retention, etc., adequately specified? Are all documents prepared in a clear legible manner suitable for microfilming and/or other documentation storage method? Have all impacted documents been identified for update as necessary?

    Yes ☒ No ☐ N/A ☐

20. Software Quality Assurance- For a calculation that utilized software applications (e.g., GOTHIC, MAAP), was it properly verified and validated in accordance with 11.SQA?
    This is an 11.SQA task. However, per 3-ENS-DC-126, for exempt software, was it verified in the calculation?

    Yes ☐ No ☐ N/A ☒

21. Has adverse impact on peripheral components and systems, outside the boundary of the document being verified, been considered?

    Yes ☒ No ☐ N/A ☐

22. Are the latest applicable revisions (including pending data)of design documents utilized? ©[1,2]

    Yes ☒ No ☐ N/A ☐

23. Has this design adequately considered hazards such as missiles, jet impingement, etc.?

    Yes ☒ No ☐ N/A ☐

24. Has the design adequately considered seismic requirements, barge impact, and Mark I loadings as appropriate?

    Yes ☐ No ☐ N/A ☒

25. If this change involves a modification to a containment isolation boundary or function, have the following documents been checked for possible impact?
    a. USAR Table V-2-2
    b. USAR Section V

    Yes ☐ No ☐ N/A ☒

26. Has this design adequately addressed internal flooding vulnerability?
    a. Does the configuration change increase potential for internal flooding?
    b. Does the configuration change reduce capability to isolate or cope with flooding?
    c. Does the design locate Essential equipment where it would be susceptible to flooding?

    Yes ☐ No ☐ N/A ☒

27. Has the design adequately considered CNS Computer System interfaces (PMIS, Annunciator, Security, Simulator)?

    Yes ☐ No ☐ N/A ☒

28. Has the design adequately addressed its impact on CNS Emergency Operating Procedures and the Emergency Procedure Guidelines?

    Yes ☐ No ☐ N/A ☒

29. For electrical and IAC changes, the results of the point-to-point analysis indicate the design satisfies the specified design requirements and fully satisfies applicable topics listed on this attachment?

    Yes ☐ No ☐ N/A ☒

30. Do all drawings affected by the modification have DIR versions created and the information is properly documented?©[1,2]

    Yes ☐ No ☐ N/A ☒

31. If applicable, has the Classification Evaluation (CE) been reviewed for completeness and technical accuracy with regard to the Structure, System, or Component (SSC) design licensing requirements?
    a. Are all question responses and required information correct with regard to the design basis?
    b. Is justification to each Equipment Application Data Sheet YES/NO answer documented on the Equipment Application Analysis Sheet and is it comprehensive and sensible?
    c. Are all CE equipment applications properly classified?

    Yes ☐ No ☐ N/A ☒

32. Did this IDV include a review of all calculations affected and/or implemented by the package?

    Yes ☒ No ☐ N/A ☐

33. Did this IDV review all SAP Notifications written for document changes to ensure consistency with the proposed design?

    Yes ☐ No ☐ N/A ☒

34. Have all CNS Computer support and other considerations been adequately addressed?

    Yes ☐ No ☐ N/A ☒

35. Have all design and configuration documents which were required to be created or revised as a result of this activity been reviewed?

    Yes ☒ No ☐ N/A ☐

ATTACHMENT 9.7 DESIGN VERIFICATION COMMENT SHEET

Sheet 1 of 1

REVIEWER COMMENT/RESOLUTION RECORD

Document: EE 13-041 Rev 2

NUMBER	REVIEWER COMMENTS	REVIEWER INIT/DATE	PREPARER RESPONSE	PREPARER INIT/DATE	REVIEWER ACCEPT INIT/DATE	CODE[1]
1	Minor editorial comments provided in markup	MBN 7.16.15	Incorporated comment.	TES 7-20-15	MBN 7-30-15	
2	Applicable Design Basis: Document the Turbine Building design wind speeds, applicable tornado missiles and applicable turbine missiles. Discuss the use of not use of design codes. Document drawings used and verification methods.	MBN 7.16.15	Incorporated comment	TES 7-20-15	MBN 7-30-15	

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