Systems	7127
Calc. Sub-Type	-
Priority Code	3
Quality Class	S

NUCLEAR GENERATION GROUP

ANALYSIS / CALCULATION

S10-0063
(Calculation #)

Auxiliary Building Overhead Crane (FHCR-5) Supporting Steel Structure – Connection Evaluation
(Title including structures, systems, components)

☐ BNP UNIT ______

☒ CR3 ☐ HNP ☐ RNP ☐ NES ☐ ALL

APPROVAL ☒ Electronically Approved

Rev	Prepared By	Reviewed By	Supervisor
0	Signature Glen Carter	Signature Michael P. Black	Signature
	Name Glen Carter (ENERCON)	Name Michael P. Black (ENERCON)	Name Kyong S. Pak (ENERCON)
	Date 8-5-2011	Date 8/5/2011	Date

(For Vendor Calculations)

Vendor Enercon Services Inc. Vendor Document No. N/A

Owner's Review By ______________________ Date ________

List Of Effective Pages

Page	Rev	Page	Rev	Page	Rev	Page	Rev
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Attachments

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1	0	406	2	0	611	3	0	77
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Amendments

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Revision Summary

Revision #	Revision Summary (Include brief description of revision and a list of EC's and other modifications incorporated into revision)
0	Original Issue per EC 70139

Document Indexing Tables

Document Management System Data (For update of PassPort Controlled Document information — Document Service is to delete roll over data only if shown for DELETE in the following tables)

Notes - General

Doc Services Action (Enter ADD, DELETE, or —)	**Text of General Notes**
ADD	This calculation is issued to support the ISFSI project (EC 70139).

Reference Numbers – Reference Systems

Doc Services Action (Enter ADD, DELETE, or —)	**System** (Two letter code for systems affected by results)
ADD	7127

Reference Numbers – Other References (references to PassPort products)

Doc Services Action (Enter ADD, DELETE, or —)	**Type** (e.g. AR, EC, WO, etc)	**Reference** (e.g. AR No, EC No, WO No, etc)	**Sub** (AR Assign No, WO Task No, etc.)	**Title**
ADD	EC	70139		ISFSI Auxiliary Building Crane Upgrade (FHCR-5)

Input Document References – Controlled Documents with Cross References

Doc Services Action (Enter ADD, REV, DELETE, or —)	**Doc. Type** (e.g. CALC, DWG, NPAS, POM, etc)	Document **Sub-Type**	**Document** ID (e.g., Calc No., Dwg. No., Procedure No)	**Sheet** (Dwg. sheet number if Applicable)	Doc **Rev**	**Minor Rev** (for Calc Amendments)	**Ref Type** (for NPAS Docs)
ADD	CALC		S09-0036		0		
ADD	DWG		422-057	1	0		
ADD	DWG		521-142	1	0		
ADD	DWG		521-142	2	0		
ADD	DWG		521-142	3	0		
ADD	DWG		521-142	4	0		
ADD	DWG		521-142	5	0		
ADD	DWG		521-142	6	0		
ADD	DWG		522-041	1	0		
ADD	DWG		522-041	2	0		
ADD	DWG		522-041	3	0		
ADD	DWG		522-041	4	0		
ADD	DWG		522-041	5	0		
ADD	DWG		522-041	6	0		
ADD	DWG		522-041	7	0		
ADD	DWG		522-041	8	0		
ADD	DWG		522-041	9	0		
ADD	DWG		522-041	10	0		
ADD	DWG		522-041	11	0		
ADD	DWG		522-041	12	0		
ADD	DWG		522-041	13	0		
ADD	DWG		522-041	14	0		
ADD	DWG		522-041	15	0		
ADD	DWG		522-041	16	0		
ADD	DWG		522-041	17	0		
ADD	DWG		522-041	18	0		
ADD	DWG		522-041	19	0		
ADD	DWG		522-041	20	0		
ADD	DWG		522-041	21	0		
ADD	DWG		522-041	22	0		
ADD	DWG		522-041	23	0		
ADD	DWG		522-041	24	0		
ADD	CALC		2:01.10		-		

Doc Services Action (Enter ADD, REV, DELETE, or —)	Doc. Type (e.g. CALC, DWG, NPAS, POM, etc)	Document Sub-Type	Document ID (e.g., Calc No., Dwg. No., Procedure No)	Sheet (Dwg. sheet number if Applicable)	Doc Rev	Minor Rev (for Calc Amendments)	Ref Type (for NPAS Docs)
ADD	CALC		2:01.12		-		
ADD	CALC		2:01.13		-		
ADD	CALC		2:01.14		-		
ADD	CALC		2:01.15		-		

Description Codes (Key Words)

Doc Services Action (Enter ADD, DELETE, or —)	Code (Codes for Key Words) (To be recorded as document description codes in PassPort)
ADD	ISFSI
ADD	AUXILIARY BUILDING
ADD	FHCR-5
ADD	OVERHEAD CRANE

Output Document References (Doc Service is to open listed documents and add or delete this Calc as a reference)

Doc Services Action (Enter ADD, DELETE, or —)	Document Type (e.g. CALC, DWG, TAG, PROCEDURE, SOFTWARE)	Document Sub-Type	Document ID (e.g., Calc No., Dwg. No., Procedure No., Software name and version)	Revision	Action Tracking (AR number or EC number that will track revision of affected document for the results of this calculation)

Equipment Database Data (For update of PassPort Equipment Database information)

Equipment Document References

Config Mgt Action (Enter ADD, DELETE, or —)	Equipment Tag	Equipment Type (includes SFTAPL for analysis software)	Relationship to Calc. (e.g. equipment operation affected by results, equipment design affected by results, analysis software)
ADD	FHCR-5, CRN	CRN	Evaluation of supporting structure for new/replacement crane

Record of Lead Review

Document S1 0-0063 Revision A, B, 0

The signature below of the Lead Reviewer records that:
- the review indicated below has been performed by the Lead Reviewer;
- appropriate reviews were performed and errors/deficiencies (for all reviews performed) have been resolved and these records are included in the design package;
- the review was performed in accordance with EGR-NGGC-0003.

☒ **Design Verification Review** ☐ **Engineering Review** ☐ **Owner's Review**
- ☒ Design Review
- ☐ Alternate Calculation
- ☐ Qualification Testing

☐ **Special Engineering Review**____________________

☐ YES ☐ N/A **Other Records are attached.**

Lead Reviewer (print/sign)	**Discipline**	**Date**
Michael Black	**Civil/Structure**	**12/12/2010** **2/9/2011**

Item No.	Deficiency	Resolution
1	NONE	

FORM EGR-NGGC-0003-2-10
This form is a QA Record when completed and included with a completed design package. Owner's Reviews may be processed as stand alone QA records when Owner's Review is completed.

Document **S1 0-0063** **Revision** **A**

The signature below of the Lead Reviewer records that:

- the review indicated below has been performed by the Lead Reviewer;
- appropriate reviews were performed and errors/deficiencies (for all reviews performed) have been resolved and these records are included in the design package;
- the review was performed in accordance with EGR-NGGC-0003.

☐ **Design Verification Review** ☐ **Engineering Review** ☒ **Owner's Review**
- ☐ Design Review
- ☐ Alternate Calculation
- ☐ Qualification Testing

☐ **Special Engineering Review**____________________

☐ YES ☐ N/A **Other Records are attached.**

Casaba Ranganath **Civil/Structure**
Lead Reviewer **(print/sign)** **Discipline** **Date**

Item No.	Deficiency	Resolution
1	Attachment 1: Page 121, allowable for capacity of the bracing connection can be increased to elastic limit (1.0 Sy).	The required number and size of bolts remains the same, even if 1.0 Sy is used.
2	Attachment 1: Page 132: Why is the bolt shear capacity reduced by 75% and what is the basis.	The bolt shear capacity is set to a certain percentage of the full shear capacity in order to maximize the bolt shear and bolt tension capacities needed for the given shear and tension forces applied to the bolt group. See AISC 6th edition, Section 1.6.3 "Shear and Tension".
3	Attachment 1: Page 134. Why is factor 1.16 used for E70XX electrode.	The coefficients in Table XV (pg 4-60) of AISC 6th edition are based on E60XX electrodes. When E70XX electrodes are used, multiply coefficients by 1.16 per Table XV.
4	Attachment 1, Page 135, the new brace load should be 72.48 kips per Page 12 of calculation.	Revised page 135 (now 143) to show design force of 72.48 kips instead of 71.05 kips. Page 135 (now 143) is the "Existing Capacity" calculation. Page 160 (now 168) is the "Modification" calculation where the correct design force of 72.48 kips was used. This will not impact modification requirements.

5	Attachment 1, Page 138, the new brace load should be 134.44 kips per Page 12 of calculation.	Revised page 138 (now 146) to show design force of 134.44 kips instead of 135.16 kips. Page 138 (now 146) is the "Existing Capacity" calculation. Page 166 (now 174) is the "Modification" calculation where the correct design force of 134.44 kips was used. This will not impact modification requirements.
6	Attachment 1, Page 141, New brace load from GTSTRUDL is 163.09 Kips per Page 12 of calculation not 152.63 Kips.	Revised page 141 (now 149) to show 163.09 kips instead of 152.63 kips. Page 141 (now 149) is the "Existing Capacity" calculation. Page 170 (now 178) is the "Modification" calculation where the correct design force of 163.09 kips was used. This will not impact modification requirements.
7	Attachment 1, Page 144, the new brace load is 159.12 not 148.98 kips per Page 12 of calculation.	Revised page 144 (now 152) to show design force of 159.12 kips instead of 148.98 kips. Page 144 (now 152) is the "Existing Capacity" calculation. Page 174 (now 182) is the "Modification" calculation where the correct design force of 159.12 kips was used. This will not impact modification requirements.
8	Attachment 1, Page 147, the new brace load is 101.74 and not 94.83 kips shown.	Revised page 147 (now 155) to show design force of 101.74 kips instead of 94.83 kips. Page 147 (now 155) is the "Existing Capacity" calculation. Page 177 (now 182) is the "Modification" calculation where the correct design force of 101.74 kips was used. This will not impact modification requirements.
9	Attachment 1, Page 150, the new brace load is 50.49 kips and not 34.65 Kips shown. This may require mod, verify. In Page 149 add x2 for bolt shear capacity (D.S).	Revised page 150 (now 158) to show 50.49 kips instead of 34.65 kips. A new modification is required. Calculation and drawings have been updated. Included "double shear" for bolts.

10	Attachment 1, page 153, the new brace load is 80.74 kips and not 68.87 kips shown.	Revisde page 153 (now 161) to show 80.74 kips instead of 68.87 kips. A new modification is required. Calculation and drawings have been updated.
11	Attachment 1, Page 156, the new brace load is 87.13 kips and not 96.65 kips as shown.	Revised page 156 (now 164) to show 87.13 kips instead of 96.65 kips. Page 156 (now 164) is the "Existing Capacity" calculation. Page 181 (now 196) is the "Modification" calculation where the correct design force of 87.13 kips was used. This will not impact modification requirements.
12	Attachment 1, Page 158, Why change bolts between brace and gusset plate, since the existing bolts have 72.12 x 1.33 = 96.16 kips capacity greater than the new load 72.48 kips. Also in Page 160 why is the bolt shear capacity set equal to 50% of full shear capacity.	Yes, the existing bolts will work for the new force. However, the existing bolts must be removed to install new gusset plate. Therefore, bolts must be replaced (See Detail 2 on 522-041-008). The bolts are **not** being replaced at other end of brace (See Detail 3 on 522-041-008). For 50% shear capacity, see combined shear and tension explanation given in Item #2 above.
13	Attachment 1, Page 166: the bolt shear capacity for gusset to angle brace is 105.78 (Page 137) x 1.33 = 141.04 Kips greater than the load on brace 134.44 kips, why change the bolts for this connection.	Yes, the existing bolts will work for the new force. However, the existing bolts must be removed to reinforce the existing gusset plate. Therefore, bolts must be replaced (See Detail 3 on 522-041-008)
14	Attachment 1, Page 172: Gusset to column connection, what is the reference for bolt allowable for Type I 325 bolts of 28 ksi.	28 ksi is incorrect. This should be 22 ksi. The calculation has been revised. (now 180) This will not impact modification requirements.

15	Attachment 1, Page 188, the design force should be 41.92 kips not 30 kips, in addition the bolt allowable shown 11.99 kips should be 18.06 per AISC 4-44, with this increase in allowable no change to bolt is required between gusset plate and brace, verify.	Revise page 188 (now 203) to show design force of 41.92 kips instead of 30 kips. Yes, bolt allowable can be increased. The calculation has been revised to show this increase. Page 188 (now 203) is the “Existing Capacity” calculation. Page 200 (now 215) is the “Modification” calculation where the correct design force of 41.92 kips was used. The “Modification” calculation does not need to be revised. The bolts must be removed and replaced for new gusset plate installation. However, 7/8” diameter bolts should be used instead of the 1” diameter bolts called out on Detail 1 of drawing 522-041-006. Drawing has been revised.
16	Attachment 1, Page 197: Delete inches in no. of bolts 2, same comment in Page 200. ~~Also in Page 188, the design force should be 41.92 kips not 30 kips, in addition the bolt allowable shown 11.00 kips should be 18.06 per AISC 4-44, with this increase in allowable no change to bolt is required between gusset plate and brace, verify.~~	“inches” has been deleted from pages 197 (now 212) and 200 (now 215).
17	Attachment 1, page 202: Design force should be 163.09 kips per Page 12 of calculation.	Revised page 202 (now 217) to show design force of 163.09 kips instead of 152.63 kips. Page 202 (now 217) is the “Existing Capacity” calculation. Page 209 (now 224) is the “Modification” calculation where the correct design force of 163.09 kips was used. This will not impact modification requirements.
18	Attachment 1, page 204: Design force should be 159.12 kips per Page 12 of calculation instead of 150 kips.	Revised page 204 (now 219) to show design force of 159.12 kips instead of 150 kips. Page 204 (now 219) is the “Existing Capacity” calculation. Page 211 (now 226) is the “Modification” calculation where the correct design force of 159.12 kips was used. This will not impact modification requirements.

19	Attachment 1, page 206: Design force should be 87.13 kips per Page 12 of calculation instead of 96.65 kips.	Revised page 206 (now 221) to show design force of 87.13 kips instead of 96.65 kips. Page 206 (now 221) is the "Existing Capacity" calculation. Page 214 (now 229) is the "Modification" calculation where the correct design force of 87.13 kips was used. This will not impact modification requirements.
20	Attachment 1, Page 217: Shows this as a Type 5 with L5X5X1/2 brace, however, Page 168 shows this as Type 2 with L6x6X3/4 brace, L6x6x3/4 is correct, verify. Also in Page 218 why the bolt shear capacity is reduced to 75% and the new brace force per Page 12 of calculation is 163.09 kips, this should be revised in Page 219.	Page 217 (now 232) checks the "Existing" capacity of connection at Grid 301-L, TOS EL 161'-4". The "existing" brace is L5x5x1/2 (See VB-15 sketch on page 239 (now 254)). Page 168 (now 176) is a "Modification" calculation at Grid 301-M1, TOS EL 161'-4". The "modified" angle size is L6x6x3/4 (See VB-5 sketch on page 167 (now 175)). Page 219 (now 234) has been revised to show 163.09 kips instead of 152.63 kips. Page 219 (now 234) is the "Existing Capacity" calculation. Page 242 (now 257) is the "Modification" calculation where the correct design force of 163.09 kips was used. This will not impact modification requirements. See Item 2 above for explanation of 75% shear capacity at combined tension and shear condition.
21	Attachment 1, page 225: Design force should be 159.12 kips per Page 12 of calculation instead of 148.98 kips.	Revised page 225 (now 240) to show design force of 159.12 kips instead of 148.98 kips. Page 225 (now 240) is the "Existing Capacity" calculation. Page 250 (now 265) is the "Modification" calculation where the correct design force of 159.12 kips was used. This will not impact modification requirements.

22	Attachment 1, page 228: Design force should be 101.74 kips per Page 12 of calculation instead of 94.83 kips.	Revised page 228 (now 243) to show design force of 101.74 kips instead of 94.83 kips. Page 228 (now 243) is the “Existing Capacity” calculation. Page 253 (now 268) is the “Modification” calculation where the correct design force of 101.74 kips was used. This will not impact modification requirements.
23	Attachment 1, page 231: Design force should be 51 kips per Page 12 of calculation instead of 35.86 kips. The capacity of the bracing connection as per Page 231 is 47.96 kips. This may require a mod verify.	Revised page 231 (now 246) to show design force of 51 kips instead of 35.86 kips. A new modification is required. Calculation and drawings have been updated.
24	Attachment 1, page 234: Design force should be 80.74 kips per Page 12 of calculation and the capacity of the bracing connection is 71.94 kips as per Page 234. This may required a mod, verify.	Revised page 234 (now 249) to show design force of 80.74 kips instead of 68.87 kips. A new modification is required. Calculation and drawings have been updated.
25	Attachment 1, page 240: Member 7315 shows Type 5, Page 168 shows Type 2, verify, also this appears to be a duplicate calc for this member.	See explanation in Item 20 above. Note that page 240 (now 255) is the “modification” at Grid 301-L, and page 217 (now 232) is the “existing” capacity at Grid 301-L.
26	Attachment 1, Page 249: Why reduced capacity for shear to 75% for gusset to beam connection.	See explanation concerning combined shear and tension in Item 2 above.
27	Attachment 1, Page 252: Why reduced capacity for shear to 65% for gusset to beam connection.	See explanation concerning combined shear and tension in Item 2 above.
28	Attachment 1, Page 256: Why reduced capacity for shear to 70% for gusset to beam connection.	See explanation concerning combined shear and tension in Item 2 above.
29	Attachment 1, page 258: Design force should be 51 kips per Page 12 of calculation instead of 35.86 kips.	Revised page 258 (now 280) to show force of 51 kips instead of 35.86 kips. Modification is not required.

30	Attachment 1, page 261: Design force should be 41.92 kips per Page 12 of calculation instead of 32.92 kips. Why compressive force in this brace.	Revised page 261 (now 283) to show force of 41.92 kips instead of 32.92 kips. Page 261 (now 283) is the "Existing Capacity" calculation. Page 265 (now 287) is the "Modification" calculation where the correct design force of 41.92 kips was used. This will not impact modification requirements. This is a Tension/Compression brace. See Gilbert drawing S-522-006 (+/- 30 kips)
31	Attachment 1, page 268: Design force should be 93.97 kips per Page 12 of calculation instead of 90.06 kips.	Revised page 268 (now 290) to show force of 93.97 kips instead of 90.06 kips. Page 268 (now 290) is the "Existing Capacity" calculation. Page 279 (now 301) is the "Modification" calculation where the correct design force of 93.97 kips was used. This will not impact modification requirements.
32	Attachment 1, page 271: Design force should be 101.74 kips per Page 12 of calculation instead of 94.83 kips.	Revised page 271 (now 293) to show force of 101.74 kips instead of 94.83 kips. Page 271 (now 293) is the "Existing Capacity" calculation. Page 284 (now 306) is the "Modification" calculation where the correct design force of 101.74 kips was used. This will not impact modification requirements.
33	Attachment 1, page 274: Design force should be 80.74 kips per Page 12 of calculation instead of 68.87 kips. Bolts between gusset and brace not OK with the increased load, see Page 275.	Revised page 274 (now 296) to show force of 80.74 kips instead of 68.87 kips. Page 274 (now 296) is the "Existing Capacity" calculation. Page 289 (now 311) is the "Modification" calculation where the correct design force of 80.74 kips was used. This will not impact modification requirements.
34	Attachment 1, Pages 297, 298, and 299 are not legible.	The portions of the calculation that are not legible are not applicable, and have been removed. (now 319, 320, and 321)

35	Attachment 1, Page ~~301~~361: There is a mod sketch for Members 7246 to 7253. The sketch shows modification at the end of the member, however, could not locate any calculation for this mod. and also this mod is not identified in Page 2 of calculation that provides a table of all the modifications and it is not in the modification drawings, verify.	The modification in question is a "Horizontal Bracing" connection. See modification sketches HB-1 & HB-1a (pages 360 (now 382) & 361 (now 383)). See calculation pages 365 (now 387) and 367 (now 389) ("Information at Gusset to 30WF116" and "Gusset to 30WF116 Beam Connection). See page 325 (now 347) for modification location on Gilbert drawing S-521-102. See modification Detail 2 on drawing 521-142-001.
36	Attachment 2, Page 217: Change Mark# W36 to B59E, also total applied axial force and total shear capacity should this be Connection capacity instead of applied force, clarify. This is general comment that applies at other locations.	This is a CONNECTION CAPACITY for all W36 members with ¼" **welds** in this sub-section (2.3.1) of the calculations (not only Mark #B59E.) References have been revised at the ***INPUT*** section of the spread sheet to better explain.
37	Attachment 2, Page 222: Change Mark# W36 to B51C.	This is a CONNECTION CAPACITY for all W36 members with **5/16**" **welds** in this sub-section (2.3.1) of the calculations (not only Mark #B51C.) References have been revised at the ***INPUT*** section of the spread sheet to better explain.
38	Attachment 2, Page 227: Change Mark# W14W to B56D.	This is a CONNECTION CAPACITY for W14 at the West end of B56D. References have been revised at the ***INPUT*** section of the spread sheet to better explain.
39	Attachment 2, Page 232: Change Mark# W14E to B50D.	This is a CONNECTION CAPACITY for W14 at the East end of B56D (not B5**0**D.) References have been revised at the ***INPUT*** section of the spread sheet to better explain.
40	Attachment 2, Page 237: Change Mark# C8 to B56F.	References have been revised at the ***INPUT*** section of the spread sheet to better explain.
41	Attachment 2, Page 243: Total Applied Shear 33.82 is due to EnvA not EnvC, verify.	Revised shear to match EnvC – NO CHANGE TO RESULTS OR MODIFICATIONS. [$t_{REQ'D}$ = 0.673" > t = 0.375"]
42	Attachment 2, Page 248: Total Applied Shear 33.82 is due to EnvA not EnvC, verify.	Revised shear to match EnvC – NO CHANGE TO RESULTS OR MODIFICATIONS. [$t_{REQ'D}$ = 0.704" > t = 0.375"]
43	Attachment 2, Page 258: Total Applied Shear 30.24 is due to EnvA not EnvC, verify.	Revised shear to match Env**B** (Case Env**B** noted on Revision A calculation – **not** Env**A)** – NO CHANGE TO RESULTS OR MODIFICATIONS. [$t_{REQ'D}$ = 0.781" > t = 0.375"]

44	Attachment 2, Page 263: Total Applied Shear 30.24 is due to EnvA not EnvC, verify.	Revised shear to match Env**B** (Case Env**B** noted on Revision A calculation **– not** Env**A) –** NO CHANGE TO RESULTS OR MODIFICATIONS. [$t_{REQ'D}$ = 0.781" > t = 0.375"]
45	Attachment 2, Page 281: Revise W36 shown under CASE. The total applied force and shear are not as per load table in page 274, clarify.	This is a CONNECTION CAPACITY for all W36 members BUT WAS NOT REQUIRED BECAUSE THE EXISTING FORCE WAS > THE REVISED (CURRENT) FORCE. MARKED "**VOID** (NOT REQUIRED)" UNDER ***COMMENT***. *(Pages 274 thru 276 were shifted to 275 thru 277.)*
46	Attachment 2, Page 286: Revise W30 shown under CASE.	This is a CONNECTION CAPACITY for all W30 members this sub-section (2.3.2) of the calculations (Table pgs 278-280.) References have been revised at the ***INPUT*** section of the spread sheet.
47	Attachment 2, Page 291: Revise W24 shown under CASE.	This is a CONNECTION CAPACITY for like or similar W24 members this sub-section (2.3.2) of the calculations (Table pgs 278-280.) References have been revised at the ***INPUT*** section of the spread sheet.
48	Attachment 2, Page 296: Revise W24 shown under CASE. Add "D" after EN under COMMENT.	Same as ITEM 47. Added "D" (at *NORTH EN**D***) under ***COMMENT***.
49	Attachment 2, Page 306: Revise W24 shown under CASE.	Same as ITEM 47.
50	Attachment 2, Page 311: Revise W24 shown under CASE. The total applied force and shear are not as per load table in page 275, clarify.	This is a CONNECTION CAPACITY for W24 @Mark #B57B (North End) BUT WAS NOT REQUIRED BECAUSE THE EXISTING FORCE WAS > THE REVISED (CURRENT) FORCE. MARKED "**VOID** (NOT REQUIRED)" UNDER ***COMMENT***.
51	Attachment 2, Page 316: Revise W24 shown under CASE.	Same as ITEM 47.
52	Attachment 2, Page 321: Revise W24 shown under CASE.	Same as ITEM 47.
53	Attachment 2, Page 326: Revise W18 shown under CASE. This is a general comment at various other locations in this attachment.	This is a CONNECTION CAPACITY for like or similar W18 members this sub-section (2.3.2) of the calculations (Table pgs 278-280.) References have been revised at the ***INPUT*** section of the spread sheet. *The same revision was made to all subsequent applications of the spread sheets where it (the spread sheet) was used to determine a CONNECTION CAPACITY (based upon nominal beam depth) rather than to analyze a connection for a specific load condition. This was addressed at all subsections where connection capacities where determined.*

54	Attachment 3, Page 7, Section 4.2.0: In the second paragraph should also add Member 71090 and Mz as per Attachment 3.2 is 237 kip-ft.	71090 has been added. Mz has been changed to 237 kip-ft.
55	Attachment 3, Page 7, Section 4.2.8: Why fv and ft are not taken to act at the same time.	"fv" is a crane longitudinal force and does not act at the same time as "ft" which is a crane transverse force based on the "GTStrudl" load combinations.
56	Attachment 3: Why the strong axis allowable are not increased by 1/3.	Strong axis forces are gravity forces and are viewed as constant loads. Transverse and longitudinal forces are viewed as impact loads and are present for a very short time. The design check and modification for the bracket have be based on the continuous presents of the gravity forces, therefore, the increase allowable was not taken for the gravity forces.
57	Attachment 3, Section 4.2.20: Why the axial stress not included with the bending and shear stress in this section.	Transverse and longitudinal forces are impact loads and are not present at the same time. The axial stress is less the weak axis forces, therefore, the axial stress combined with the strong axis bending will not control the design.
58	Attachment 3, Section 4.3.0: Fy is 249.9 kips as per Page 1 of Attachment 3.1. This requires to be corrected at all other locations in the calculation.	Have revised Fy to 249.9 kips and updated the calculation the reflect change
59	Attachment 3, section 4.3.12: Why is axial stress not combined with bending and shear stress in this section.	Same as 56
60	Attachment 3, Section 4.4.0: Verify if 7120 should be 71200. My should be 74 kip-ft per Attachment 3.2. Also under the lower column shaft connection, the Fy load at Column "Q1-301" is shown as 272.2 kips + 165.8 kips, as per Attachment 3.1, it is 33.8 kips + 165.8 kips, verify.	7120 should be 71200 is correct. Colum "Q1-301" does not control design, Column "Q1-302A" controls the design. The section has been corrected to show this. Vertical force in column are 236.9 kips + 143.8 kips.
61	Attachment 4: Calculation number should be S10-0063 and not S09-0063.	The calculation number has been changed to S10-0063
62	Attachment 4, Section 4.7: Why the existing wing plates not considered in this calculation. Why 22 ksi chosen in calculating M_{ALL}.	In this case, the base plate is bending about the strong axis of the column. The existing wing plates do not affect the plate bending in this direction.
63	Attachment 4, Section 4.7, Page 23: Under Connection welds, the bending stress diagram it is not clear what the 1.21" and 7.79" correspond to. The numbers shown do not correspond to the values calculated in Pages 22 and 23.	The dimensions shown in the figure are incorrect and have been corrected.

64	Attachment 4, Page 22: In the table that shows the length of shear reinforcement required, it appears that the loads at Q-301 and Q-302A are very much similar, however Q-301 has 18 and 53 length shear lugs and Q-302A do not require any shear reinforcement. Why such a big difference. Also is it acceptable to distribute the shear reinforcement on opposite sides as long as the total length of the shear reinforcement L1and L2 are maintained and they are placed symmetrically from the center line of column.	The amount of required shear reinforcement has been significantly reduced from the original calculation. The reinforcement at Q-301 has been reduced to 12" and 35" in the two orthogonal directions. The shear reinforcement design is based on supplied reactions from the GTSTRUDL analysis model. Examination of the reactions indicates that the shear reactions at Q1-301 are much larger than the shear reactions at Q1-302A. Dwg. 522-006 indicates that a vertical brace frames into the base of Q1-301 while Q1-302A has no such bracing. The basis of the owners comment is uncertain. The latest drawings have distributed the reinforcement around the edges of the base plates to clear interferences. Symmetric redistribution of the reinforcement would be permissible, but is probably no longer necessary.

Document **S1 0-0063** **Revision** **B, 0**

The signature below of the Lead Reviewer records that:

- the review indicated below has been performed by the Lead Reviewer;
- appropriate reviews were performed and errors/deficiencies (for all reviews performed) have been resolved and these records are included in the design package;
- the review was performed in accordance with EGR-NGGC-0003.

☐ **Design Verification Review** ☐ **Engineering Review** ☒ **Owner's Review**

- ☐ Design Review
- ☐ Alternate Calculation
- ☐ Qualification Testing

☐ **Special Engineering Review**____________________

☐ YES ☐ N/A **Other Records are attached.**

Casaba Ranganath	**Civil/Structure**	**3-12-11**
Lead Reviewer (print/sign)	**Discipline**	**Date**

Item No.	Deficiency	Resolution
1	Attachment 1, Pages 41& 48: 301-N1 and 302-N1 moment connection: Since the flange connection bolts at the top and bottom flanges are not symmetrical, in that one half of the flange has more bolts and the bolts are in double shear, this would induce eccentric load on the connection. This eccentric loading should be considered.	Attachment 1, Page 48 revised to account for Unbalanced Force Distribution. This modification occurs at 302A-N1 only (at elevator wall). Page 41 (301-N1) does not require a Revision.
2	Attachment 3, Page 1: Change title from Crane connection to Crane Girder Connection.	Title revised per request.
3	Attachment 3, Page 27: Strong axis Mz load (291.3 K-ft) is the envelope load due to gravity as well as seismic as per STRUDL output Attachment 3.2, Page 67. Hence the allowable for the strong axis can be increased by 1/3, since it is not just gravity load (as indicated in the response to Item 53 in Revision A of owners review comment) but also includes seismic load. The same comment applies to crane girder bracket design in Revision A of this calculation.	If we use the 1/3 increase on the gravity forces, the Unity Check (Section 4.4.12) would change from 2.46 to 2.14. This would still require the modifications to be made to the connections in question. Therefore, calculation will remain "as is" knowing that by not allowing the 1/3 increase for the gravity forces would be a conservative. Following statement added to page 27: "Conservatively, normal allowable are used for the strong axis even though the load includes seismic load."

4	Attachment 4, Page 112, Section 4.9, Last sentence: after “will” add “be” Attachment 4, Page 618: Foundation capacity at S1-301 and S1-302A: By connecting S1-301 and S1-302A with a concrete beam about 3’-6” below grade, will this change response of the steel super structure that are presently modeled in STRUDL under Calculation S09-0036. In other words does the STRUDL model in S09-0036 require to be modified to include this concrete beam. Also in the STRUDL input in Page 1492, Joint 1001 and 1012 Y coordinate should be 1.1425 E +02, verify.	The word “be” added to last sentence on page 112. No, the concrete beam about 3’-6” below the grade will not cause any change in response of the steel super structure presently evaluated in calculation S09-0036. Forces in super structure model (S09-0036) are governed by the boundary conditions defined at the column line S1, which are still true (column to foundation connection is represented in terms of boundary conditions at column end and no foundations are represented in the model, which is consistent with the original calculation). When a column is considered fixed in a particular direction, it is an inherent assumption that grade below/foundation is able to transfer the forces and moments effectively. In the case of column at S1, the foundations were not stiff enough to transfer the moments effectively and hence the beam was added between foundations S1-301 & S1-302A to reflect the assumptions made in the GTSTRUDL model in S09-0036. No further modifications are required. Yes, the dimension should been 1.1425E+02 instead of 1.145E+02 which should have insignificant impact on results. Following note added to the page 618 of this attachment: “Minor differences in the actual dimensions and GTSTRUDL dimensions will have insignificant impact on the analysis result.”
5	Attachment 4: It is mentioned in Page 113 that all Case 2 excessive bearing stress and inadequate shear capacity conditions will be reexamined. However, several of the load cases shown in Attachment 4.1 where the bearing stress has been exceeded for Case 2 the evaluation could not be located. For example Page 648, Load Case 3000, Run 79, which has Fy=406.2 kips and Mx = -230K’, so also in Page 649, Run 82, Load case 3000, which has Fy=389.5 kips and Mx=-233.2 kft. There are a few other cases where the evaluation could not be located even though bearing stress is exceeded as shown in Attachment 4.1. Also in the fourth line in Page 113 add “e” before “< l/6”.	In several cases, the “Case 2” analysis as indicated in the spreadsheet is exactly correct and is not affected by a strain compatibility analysis. This occurs when the length of bearing at the bottom of the base plate is greater than the distance from the edge of the base plate to the far anchor bolts. In these cases the anchor bolts are not in tension and excessive bearing stress is not reduced by the strain compatibility analysis. For S1-301, this occurs in four cases, P=406.2 kips and Mx = 230 kip feet, P=389.5 kips and Mx=233.2 kip feet, P=398 kips and Mx=229 kip feet, and P=411.2 kips and Mx=236.4 kip feet. The excessive bearing stress for these load cases is addressed in the write up on page 610 of the calculation. “e” added in the fourth line on page 113.

6	Attachment 6: In this Attachment 6, the column K-301 at base is fixed along the Global X-axis. This boundary condition is different from the boundary condition used in all the earlier STRUDL runs in Calculation S09-0036. Even though it is mentioned in Page 3 that fixed condition is required to eliminate the excessive loading clarify this change in the boundary condition and the effect of this change for the coupled model provided to MMH for their crane analysis.	Connection is considered as semi-rigid in this attachment. Also an evaluation is carried out to ensure that anchor bolts are the weak link in the connection configuration, as a result, under the extreme loading anchor bolt will yield and connection will act as a pin connection (consistent with GTStrudl runs in calculation S09-0036). Whereas under initial normal condition the connection will act as fixed connection and continue to serve its purpose without any adverse impact on structure as discussed in second paragraph on section 1 (pg. 3). The column K-301 at base is fixed along the Global X-axis, to determine the maximum moment acting on the connection (if connection is considered fixed) and to determine the forces at which the anchor bolt will yield to behave as pin connection as discussed in the attachment 6. *"A modification to the boundary condition of connection (fixed to pinned) is required to eliminate the excessive loading (as discussed in Calculation S09-0036). The computer model in calculation S09-0036 at joint 6101 (Ref. 7) has a pin boundary condition to reflect this issue. The pin boundary condition representation of the support is the worst case depiction of the connection behavior (more conservative for whole structure) considering that the existing anchor bolts are not present and joint is free to rotate. Hence, the results obtained for design purposes tend to be more conservative compared to when a fixed connection or semi-rigid connection is considered in the analysis."*
7 (03/25/2011)	Attachment 4: Sub Section 4.4 and Section show the original load on the vertical load on the column as P max =744 K and P min = -155K, where is it taken from the original calculation, was not able to locate them in the original calculation. Also Section 4.4 shows the moment at joint 1 from original calculation as 1640 K'. The moment at Joint 1 should be zero as per original calculation, explain.	See page 618 of Attachment 4. Also, see Gilbert calculation page 2:01.3D-7.

8 (03/25 /2011)	In response to my Comment 46 under 70% EC, it is mentioned that the concrete cut out for moment connection at Elevation 162'-0" evaluation have been added. However, could not locate this evaluation in Revision B of the calculation. Please identify where this evaluation is included.	The justification of concrete slab cut-out/repair has been added to the section 4.1.1 of Attachment 1 of this calculation.
9 (03/25 /2011)	General: Evaluation of concrete structure due to revised load on concrete from steel columns as a result of crane upgrade.	As per Attachment 4 of this calculation, the new tension force in anchor bolts is less than the original design tension force shown in Gilbert Calculation 2.01.10. Also, the concrete is checked for the bearing and shear requirements in the same attachment and necessary modifications are provided. Attachment 7 is added to compare old and new load for concrete bearing and shear load. As there is no increase in the force, further evaluation of concrete structure below steel structure is not required.
10 (03/22 /2011)	With regard to the response to comment 6 in the Owner Review, is there a comparison or a STRUDL run to show that using column K-301 base as pinned is more conservative and provides the conservative response at all locations in the building structure compared to the base being fixed in the global X direction. Calculation S10-0063 Rev. B Attachment 6, Section 1, Page 3 just makes a statement, can this be confirmed through a comparison showing numbers at critical locations.	The statement is made based on the engineering judgment. Lateral forces redistribution takes place along the column k-301 based on its boundary condition. When boundary condition is considered to be pinned: all the lateral wind force is transferred to the adjutant beams and roof beams, whereas under fixed condition column will provided direct load path to the lateral wind force and adjutant beams and roof beams will see less forces. Moments generated due to seismic excitation at this column base are approximately 50% of that of the designed wind moments. Fix connection will make the structure more rigid and the natural frequency should rise slightly in the X direction. Natural frequency of the coupled crane and structure is mainly governed by the crane natural frequency which are in the approx. range of 1.2 Hz to 8 Hz and as per the review of response spectra curve (DCD), slight shift to right in this frequency range on response spectra curve will reduce the acceleration values and will results in lower forces due to EQ.

This form is a QA Record when completed and included with a completed design package. Owner's Reviews may be processed as stand alone QA records when Owner's Review is completed.

Record of Interdisciplinary Reviews

PART I — DESIGN ASSUMPTION / INPUT REVIEW: APPLICABLE ☐ Yes ☒ No

The following organizations have reviewed and concur with the design assumptions and inputs used in this calculation:

Systems Engineering

Name	Signature	Date

Operations

Name	Signature	Date

Other

Name	Signature	Date

PART II — RESULTS REVIEW:

The following organizations are aware of the impact of the results of this calculation (on designs, programs and procedures):

Systems Engineering
☐ Yes ☒ NO

Name	Signature	Date

Comments:

Operations
☐ Yes ☒ NO

Name	Signature	Date

Comments:

Other

Name	Signature	Date

Comments:

Other

Name	Signature	Date

Comments:

Other

Name	Signature	Date

Comments:

1.0 PURPOSE AND SCOPE

The Progress Energy Crystal River Unit 3 (CR3) Auxiliary Building (AB) Overhead Crane support steel structure is being evaluated for overhead crane (FHCR-5) replacement (EC 70139, Ref. 4.1.3). The purpose of this calculation is to verify/evaluate the existing connections for Auxiliary Building (AB) Overhead Crane support steel structure, crane brackets, crane stops, crane rails and column base connections. This calculation also addresses the qualification of modified structural connections.

2.0 CONCLUSION

All of the Auxiliary Building steel structure connections are structurally acceptable and meet the necessary code requirements listed in Design Criteria Document (Ref. 4.5.3).

Table below shows the summary of modifications required to the Auxiliary Building steel structure as a result of Overhead Crane (FHCR-5) replacement.

See drawings 422-057 Sht. 1 (Ref. 4.3.15), 521-142 Sht. 1 through 6 (Ref. 4.3.13) and 522-041 Sht. 1 through 24 (Ref. 4.3.14) for modification details. See Attachment 5 for location of modifications.

Type of Modification	Total No.	Att.	Drawings
Vertical Bracing Connections	51	1.3	522-041 SH. 1 thru 9, 24 522-006
Horizontal Bracing Connections at Roof EL. 209’-1” (between column line I1 & J1)	2	1.4	521-142 SH. 1 521-102
E-W Member Connections at Roof EL. 209’-1” (at column line S1)	2	2.1	522-041 SH. 14 522-004
N-S Member Connections at Roof EL. 209’-1” (between column line P1 & Q1)	2	2.1	522-041 SH. 1 & 4 522-004
N-S Member Connections (Purlin) at Roof EL. 209’-1”	98	2.1	521-142 SH. 1 522-041 SH. 15 521-102 522-004
E-W Member Connections at EL. 189’-9” (at column line S1)	3	2.2	522-041 SH. 1 & 17 522-004
Horizontal Bracing Connections at Floor EL. 162’-0” (between column line N1 & O1)	1	1.4	522-041 SH. 10 & 15 522-003
E-W Member Moment Connections at Floor EL. 162’-0”	8	1.1	522-041 SH. 10,12,13,20 522-003
E-W Member Connections at EL. 162’-0” (at column line L, at column line S1, and between column line M1 & N1)	5	2.3	522-041 SH. 10 & 16 522-003
N-S Member Connections at EL. 162’-0”	32	2.3	522-041 SH. 10,14,15,16 522-003
Miscellaneous N-S Member Connections	8	2.4	522-041 SH. 1,10,17 522-003
Crane Girder Bracket Connections	15	3	521-142 SH. 2 522-041 SH. 11 521-102 522-004
Column Connections to Concrete Floor	13	4	521-142 SH. 3 & 4 522-041 SH. 18 & 19
Concrete Tie-Beam between S1-301 & S1-302A Concrete Column Pier at EL. 119’-0”	1	4	422-057 SH. 1

3.0 INTRODUCTION

Progress Energy Crystal River Unit 3 (CR3) is implementing the Independent Spent Fuel Storage Installation (ISFSI) for Dry Fuel Storage campaign. The Transfer Casks (TC) containing the Dry Shield Canisters (DSCs) are placed into and removed from the Spent Fuel Pool (SFP) using the AB Overhead Crane (FHCR-5). The existing overhead crane capacity (120 tons but has subsequently been derated by 40% to 72 tons, and recently derated further to 25 tons per reference 4.1.4) is inadequate to handle the proposed TC to be used at CR3. In addition, the existing overhead crane does not meet the single-failure-proof criteria of NUREG 0554 (Ref. 4.2.5) and NUREG 0612 (Ref. 4.2.6). Therefore, the overhead crane must be upgraded to increase load capacity to 130 tons/15 tons, main and aux hook capacities. The existing crane is not modified. Instead, complete new crane, including the crane bridge structure as well as the trolley, is provided by the crane vendor. Therefore, the Auxiliary Building is evaluated with the new crane loads along with other loads (e.g., dead loads, live loads, earthquake loads and wind loads).

The existing Auxiliary Building is designed to resist Operating Basis Earthquake (OBE) seismic loads and a design wind speed of 110 mph (Refs. 4.1.2, 4.4.1 and 4.4.6). This calculation and structural member evaluation calculation S09-0036 (Ref. 4.4.5) together demonstrate that the modified crane support structure can accommodate an upgraded single-failure-proof crane under heavy load cask handling to 130 tons capacity in conjunction with the loads defined by the original plant licensing basis and ASME NOG-1 (Ref. 4.2.1). This calculation and the Design Criteria Document (Ref. 4.5.3) describe the load combinations, analysis methodology, and acceptance criteria. The intent of this calculation is to use the identified critical loads from calculation S09-0036 (Ref. 4.4.5) for the design/evaluation of the member connections, column base connections, crane brackets, crane stops, crane rails and rail splice joints. The interface point between ENERCON and crane vendor is at the top of the runway rail where crane and supporting structure meet. ENERCON is responsible for the structure below the interface, i.e., the supporting structure and crane vendor is responsible for above the interface, i.e., the crane bridge.

4.0 REFERENCES

4.1 Site Specifications and Procedures

4.1.1 Design Basis Document 1/3, Major Class I Structures, Rev. 6
4.1.2 Crystal River Nuclear Unit 3 Final Safety Analysis Report, Rev. 32
4.1.3 EC 70139, ISFSI Auxiliary Building Crane Upgrades (FHCR5), Rev. 0 (Draft)
4.1.4 OP0421C, Operation of the Auxiliary Building Overhead Crane FHCR-5, Rev. 33

4.2 Industrial Codes, Standards, and Manuals

4.2.1 Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder), ASME NOG-1, 2004
4.2.2 AISC Manual of Steel Construction, Allowable Stress Design 6th Edition, 1963
4.2.3 AISC Manual of Steel Construction, Volume II Connections 1st Edition, 1992
4.2.4 AISC Manual of Steel Construction, Allowable Stress Design 9th Edition, 1989
4.2.5 NUREG-0554, “Single-Failure-Proof Cranes for Nuclear Power Plants”, May 1979
4.2.6 NUREG-0612, “Control of Heavy Loads at Nuclear Power Plants”, July, 1980

4.3 Drawings and Sketches

4.3.1 522-001, Auxiliary Building – Steel Framing Column Schedule, Rev. 1
4.3.2 522-003, Auxiliary Building South Steel Framing Roof at Elev. 167’-6” and 162’-0”, Rev. 6
4.3.3 522-004, Auxiliary Building South Steel Framing Roof at Elev. 209’-1” Crane Runway at Elev. 193’-7”, Rev. 4
4.3.4 522-006, Auxiliary Building South Steel Framing Column Bracing, Rev. 3
4.3.5 521-102, Auxiliary Building North Steel Framing Roof Steel Plan-Crane Runway. Roof Elev. 200’-4” and 209’-1”, Rev. 6
4.3.6 0129-E1, Auxiliary Building – Steel Framing Column Schedule, Dated 3-5-71
4.3.7 0129-E3, Auxiliary Building South Steel Framing Roof Elev. 167’-6” and 162’-0”, Dated 2-24-71
4.3.8 0129-E4, Auxiliary Building South Steel Framing Roof at Elev. 209’-1” Crane Runway at Elev. 193’-7”, Dated 3-5-71
4.3.9 0129-E5, Auxiliary Building South Steel Framing Column Bracing, Date not legible
4.3.10 0129-E9, Auxiliary Building North Steel Framing Roof Steel Plan-Crane Runway. Roof Elev. 200’-4” and 209’-1”, Date not legible
4.3.11 0129-D1 to 0129-D105 Shop Details
4.3.12 0129-BD2 to 0129-BD17 Vertical Bracing Layout Details
4.3.13 521-142 Sht. 1 through 6, Auxiliary Building – North Steel Framing Modifications Details, Rev. 0
4.3.14 522-041 Sht. 1 through 24, Auxiliary Building – South Steel Framing Modifications Details, Rev. 0
4.3.15 422-057 Sht. 1, Auxiliary Building Modification – South Column Modifications Details, Rev. 0

4.4 Calculations

4.4.1 Calculation 2:01.10, Steel Frames
4.4.2 Calculation 2:01.12, Vertical Bracing
4.4.3 Calculation 2:01.14, Steel Floor Framing @ 162'-0"
4.4.4 Calculation 2:01.15, Roof Framing, Girts, and Miscellaneous Steel
4.4.5 S09-0036, Auxiliary Building Overhead Crane (FHCR-5) Supporting Steel Structure - Analysis, Rev. 0
4.4.6 Calculation 2:01.50, Structural Steel – Aux. Building

4.5 Other References

4.5.1 Formulas for Stress and Strain, by Roark and Young, 5th edition
4.5.2 Experimental Investigation of Stresses in Gusset Plates, by R.E. Whitmore, Bulletin No. 16, Engineering Experiment Station, University of Tennessee, 1952
4.5.3 FPC118-PR-001, Design Criteria Document for Crystal River Unit 3 Auxiliary Building Evaluation for Crane Upgrades, Revision 1 (Attachment 1 of Ref. 4.4.5 and Attachment Z23 of Ref. 4.1.3)

NOTE: See Attachments 1, 2, 3, 4, 6 and 7 for additional reference documents used for evaluation.

5.0 ASSUMPTIONS

It is assumed that the Auxiliary Building steel structure has not experienced degradation and that the as-built condition of the structural members and their connections are consistent with the original structural design drawings and the original fabrication drawings.
See "Assumption" section in Attachment 1, 2, 3, 4, 6 and 7 for any additional assumptions.

There is no open assumption in this calculation which requires later verification

6.0 DESIGN INPUT

6.1 Design Load

GTSTRUDL force output results are documented in Attachment 7, 8 and 10 of calculation S09-0036 (Ref. 4.4.5) and shall be used as new design forces for the evaluation. Existing connection forces are taken from the steel structural drawings (Ref. 4.3.1 to 4.3.5). Existing connection type is determined using the shop drawings (Ref. 4.3.6 to 4.3.12).

Any additional design inputs required for the evaluation are addressed along the body of each Attachment.

7.0 METHODOLOGY

An analysis model for the Auxiliary Building has been developed using GTSTRUDL. The forces generated from the GTSTRUDL model have been compared to the original design forces developed by Gilbert Associates (Ref. 4.4.1 to 4.4.4). Any member connections that have new forces exceeding the original forces, have been evaluated, and modified where necessary.

The evaluation and modification of the existing member connections have been analyzed and designed according to the requirements set forth in the 6th edition of the AISC Manual of Steel Construction (Ref. 4.2.2).

Per Section 1.5.6 of AISC 6th edition, the allowable stresses were increased by one third when the design forces were produced from load combinations involving wind or seismic.

There are two exceptions to using the AISC 6th edition. The first occurs when analyzing or designing a connection for prying action. Because the AISC 6th edition does not address prying action, the 9th edition of AISC (Ref. 4.2.4) was used for prying action.

The other exception involves different types of bolts being offered today, versus what was offered when the AISC 6th edition was written. The AISC 6th edition offers high strength bolt types of A325 and A354 Gr. BC. Because AISC has replaced type A354 Gr. BC bolts with type A490 bolts, type A490 bolts were used for connection modifications when necessary. The A490 bolt allowable stress values were taken from the AISC 9th edition (Ref. 4.2.4).

See Attachment 1, 2, 3, 4, 6 and 7, “Methodology” section for any additional methodology to perform an evaluation of Moment Connection, Vertical and Horizontal Bracing Connection, Beam Shear/Axial Connection, Crane Connection, and Column Base Connection.

8.0 CALCULATIONS

See the following Attachments for evaluation of various connection types.

- Attachment 1: Member Connection Evaluation (Beam Moment Connections, Vertical and Horizontal Bracing Connections)
- Attachment 2: Member Connection Evaluation (Beam, Girder and Purlin End Connections)
- Attachment 3: Crane Girder Connection Evaluation
- Attachment 4: Steel Column Base Connection Evaluation
- Attachment 6: Column 301-K Baseplate Connection Evaluation
- Attachment 7: Old and New Load Comparison at Column Base Plates