	PROYECTO: **"SERVICIO DE MODELADO Y CALCULO ESTRUCTURAL DE SOPORTES DE ESFERAS DE LA RCBA"**	CÓDIGO DE DOCUMENTO: **YPFBR-ING40-MCE-AN-005**
	TITULO: **ANÁLISIS DE ELEMENTOS FINITOS PARA LA VALIDACIÓN ESTRUCTURAL DE LA ESFERA DE GLP (1TK-2940)**	HOJA: 1 de 67

ÍNDICE DE REVISIONES

Fecha	Revisión	Observaciones
24-Dic-21	0	Aprobado

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ELABORADO POR	**REVISADO POR**	**APROBADO POR**

GENERAL INDEX

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1. INTRODUCTION

The following document presents the Finite Element Analysis for the structural validation of the 1TK-2940 sphere with updated seismic and wind loads from the original design documents.

2. SCOPE

The scope of this report is to perform the structural assessment and validation of the 1TK-2940 against current load and code specifications (referred to as the Current Assessment). Assumptions

3. ASSUMPTIONS

The following assumptions were defined for the analysis work presented in this report:

1. Validity of any engineering calculations depends upon, and is limited to, the accuracy and completeness of data provided to KHE and validated by AA BOLPEGAS.
2. This effort does not include the certification of the tank itself, just the support structures. All analysis and methodology will be per KHE's experience with this type of project. This effort does not include any analysis or review of the foundation. The foundation scope is by others.
3. In developing this report, KHE assumes that plant operation and maintenance of subject equipment and interconnecting equipment is in accordance with generally accepted industry standards and that all related equipment is designed and installed in accordance with applicable codes and standards.

4. STANDARDS AND REFERENCES DOCUMENTS

The finite element analysis was done per ASME VIII 2019 Div2 Part 5 Elastic-Plastic LRFD Analysis Method (referred to here as Part 5 EP). Part 5 EP was used as a basis for the following reasons:

- The spheres are pressure vessels. Part 5 is better suited for this application where the connections to the tops of the columns are connected via the pressure vessel as opposed to beams.

- Specific geometry for the supports is not provided for in ASME VIII Div1 (2019), ASME VIII Div2 (2019) Part 4, AISC 360 (2016), AISC 341 (2016), or UBC 97 (1997), thereby requiring “Analysis Methodology”
- Part 5 EP “stress analysis provides a more accurate assessment of the protection against plastic collapse of a component … because the actual structural behavior is more closely approximated. The redistribution of stress that occurs as a result of inelastic deformation (plasticity) and deformation characteristics of the component are considered directly in the analysis.”[1]
- Part 5 EP includes load combinations accounting for earthquake loads.
- Part 5 EP has well established acceptance criteria.

The following codes were utilized where supplemental input/data/methodologies were required or were otherwise used as reference.

- **ASCE 7-16 (2017):** Design loads and Associated Criteria for Buildings and Other Structures.
- **UBC-97 (1997):** Uniform Building Code
- **NBIC Part 2 (2021) Supplement 11:** National Board Inspection Code
- **ANSI/AISC 360-16 (LRFD) (2016):** Specification for Structural Steel Buildings
- **ANSI/AISC 341-16 (2016):** Seismic Provisions for Structural Steel Buildings

ASCE 7-16 was utilized to calculate the wind loads, UBC-97 was used in conjunction with the geotechnical study ING40-4A-125-RL-502 to develop the seismic parameters (see Seismic Load Development Per UBC 97). NBIC Part 2, supplement 11.4 (where applicable) was used as a guide for reporting the boundary conditions, loading, material properties and other modeling parameters in the report. The AISC standards were used to determine the size of notional loads,

[1] ASME VIII Div2 2021 Part 5

compressive design strength for the bracing, anchor bolt strength, and section properties for Ordinary Concentrically Braced Frame (OCBF) members.

5. DATA COLLECTION AND VERIFICATION

To support this work, a field survey was conducted by photo documentation, dimension verification, and visual observations of the supports of the RCBA Spheres. Operating history, construction history, construction drawings, geotechnical studies, and inspection reports were also compiled to support this work

6. FINITE ELEMENT ANALYSIS DEVELOPMENT

The objective of the analysis was to assess the sphere's support structure against seismic loads based on the latest ASCE/UBC codes and geotechnical surveys. The commercial software ABAQUS CAE version 2021[2] was used to perform the structural analysis.

Part 5 EP was used to conduct the assessment.

6.1. Geometry and Boundary Conditions

The model geometry was developed from drawing Y-UU047C. The model consisted of shell and beam elements. Shell elements were of type S8R and the beam elements were of type B31OS. There are three main components of the model, namely, the sphere, the columns, and the braces. There are also gussets that connect the braces to the columns. The columns, gussets, and sphere were made from shell elements whereas the braces were made of beam elements. Figure 1 and Figure 2 show the geometry and mesh of the structure. To simulate welded connections, the braces and gussets were tied at the nodes as shown in Figure 5 and Figure 7. The column shell elements and sphere shell elements were tied via shared nodes.

The model is symmetrical about the XY plane; therefore, a half model was utilized with a symmetry boundary condition as shown in Figure 3. Each column was fixed at its base in all three

[2] Abaqus Software Site ID: 200000000053861

direction and rotations as shown in Figure 4. The effects of nonlinear geometry were considered and unless otherwise specified, the corroded condition[3] was used.

Only the braces that acted in tension were included in the respective analyses as the braces in compression buckled under the factored loading. This numerically simplifies the analysis as the buckling mechanism adds local instabilities. It also adds a degree of conservatism as the compressive reaction load provides some reduction in load on the braces acting in tension.

To compensate for irregularities in the as-built geometry (verticality, concentricity, straightness, etc.), notional loads were applied to the model per AISC 360-16 Equation C2-1. The notional loads represent the effects of initial system imperfections in the position of points of intersection of members and points along members.

$\alpha := 1.0$ For LRFD

$N_c := 0.002 \cdot \alpha \cdot \dfrac{W}{n_c} = 342.2\ \mathrm{lbf}$ Notional Load added to each column (Equation C2-1)

[3] ASME VIII Div2 4.1.4.1: "The dimensional symbols used in all design equations and figures throughout this Division represent dimensions in the corroded condition."

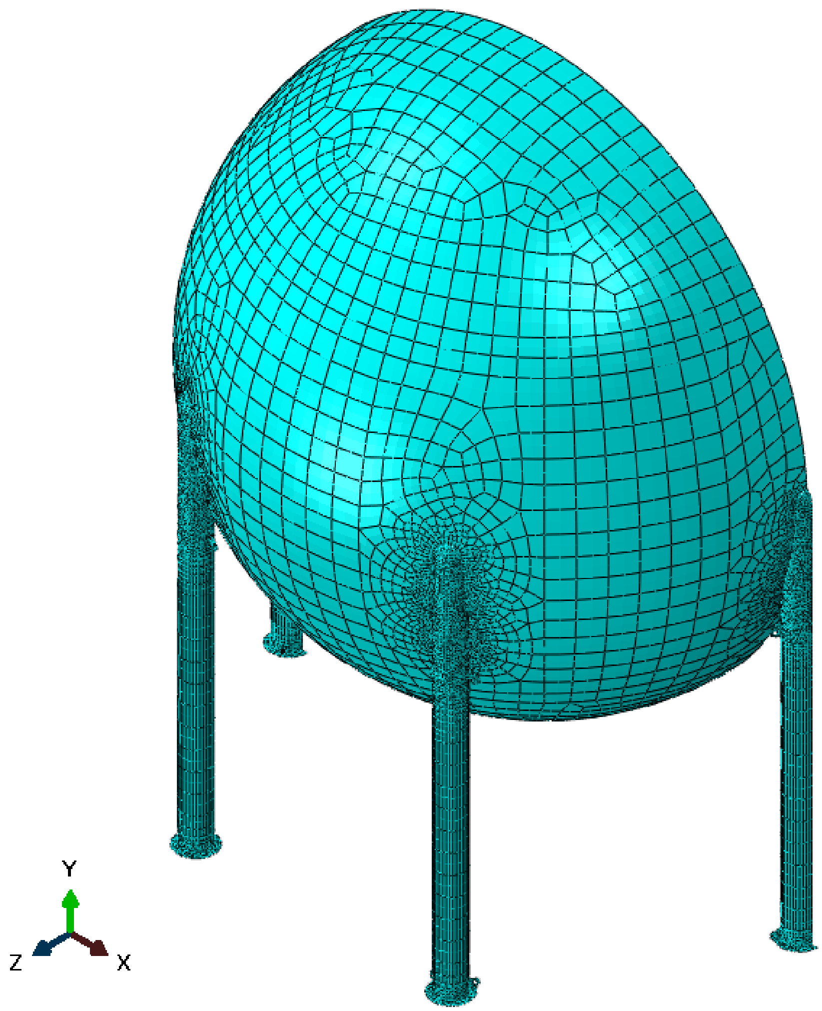


Figure 1: Mesh of the Model

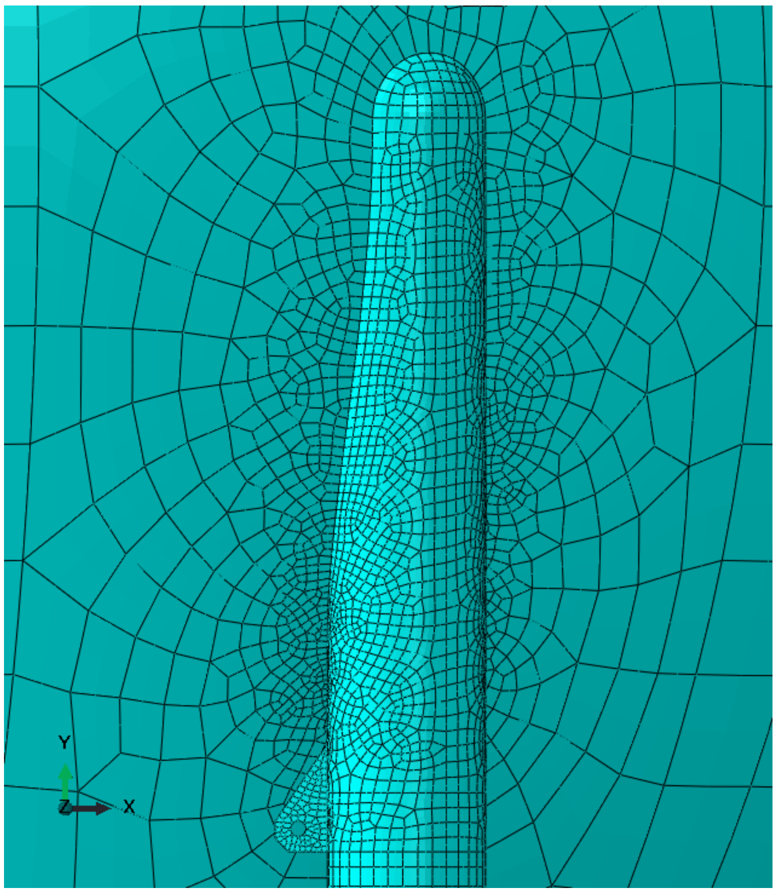


Figure 2: Mesh at the Column to Sphere Region

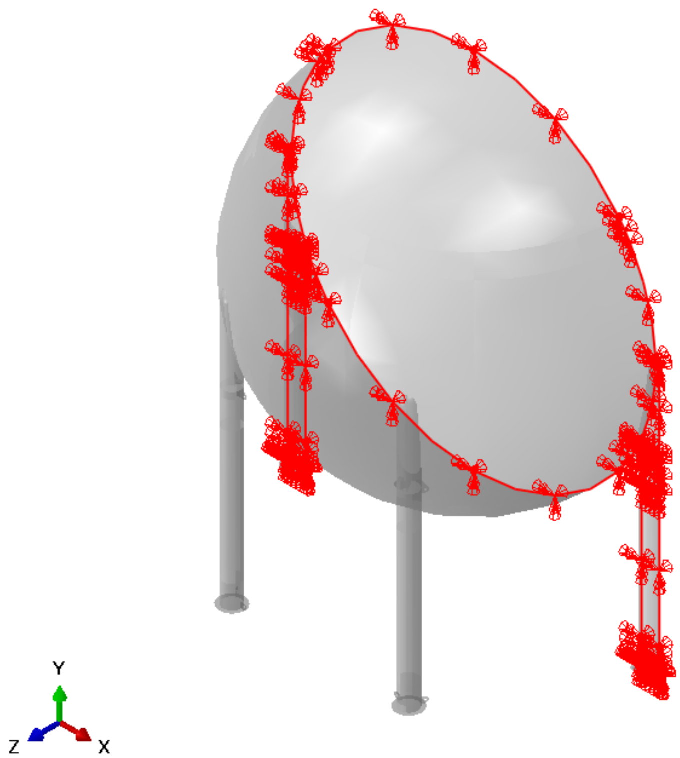


Figure 3: Symmetry Boundary Condition of the Vessel

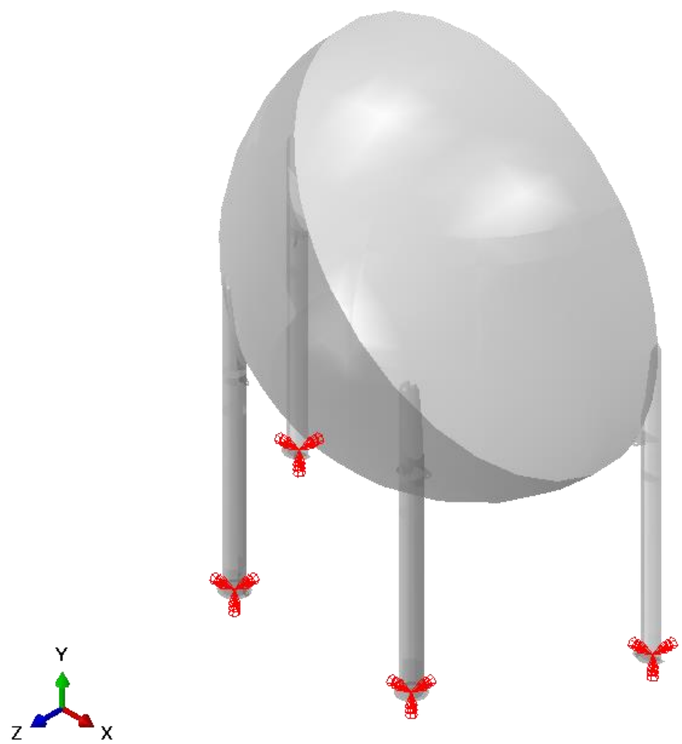


Figure 4: Fixed Boundary Condition at the Column Bases

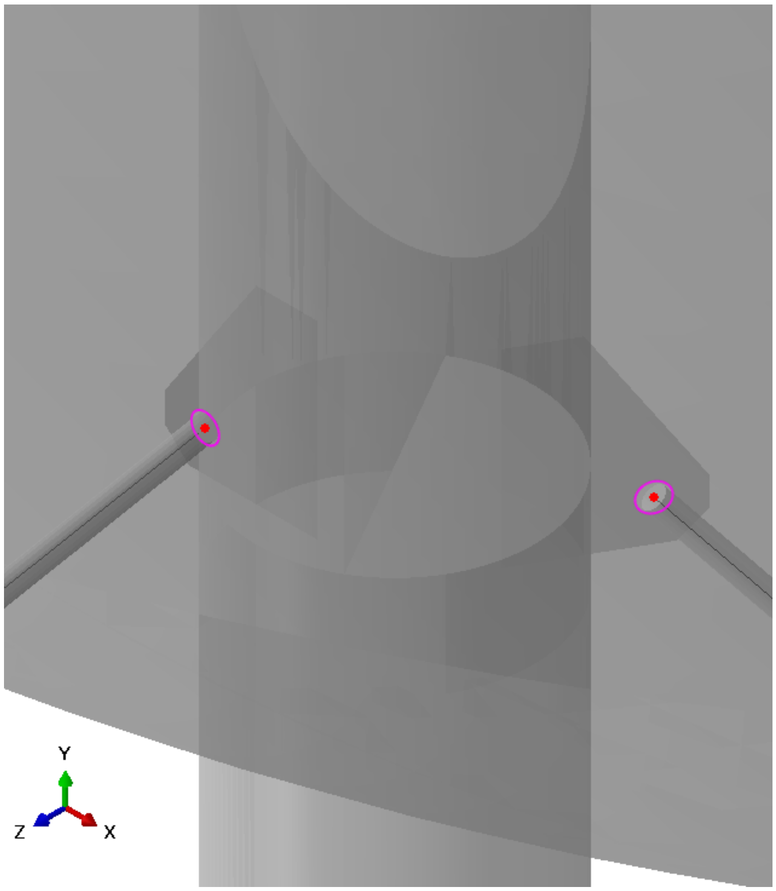


Figure 5: Node Ties at the Brace to Top Gusset Interface

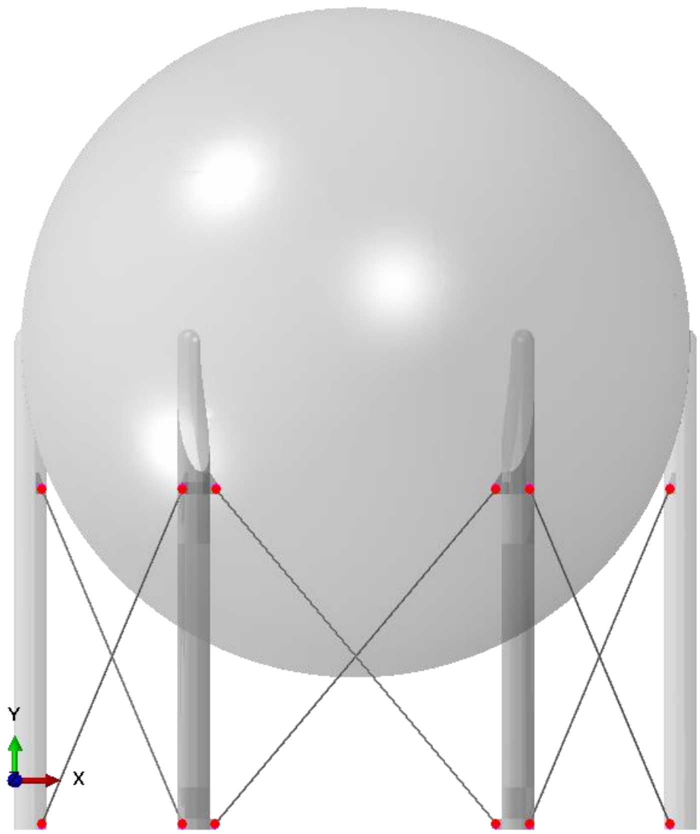


Figure 6: Ties on Model

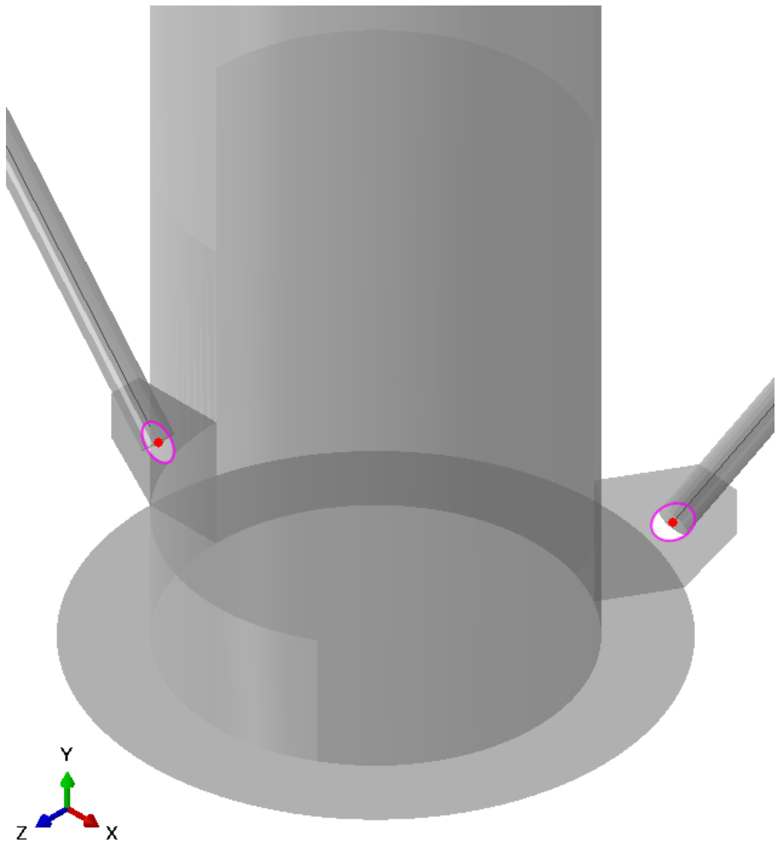


Figure 7: Node Ties at the Brace to Bottom Gusset Interface

6.2. Material Properties

Table 1 shows some of the material properties used in the model. Figure 8 shows the sections of the model where these properties were applied to the shell elements. All beam members had SA 36 properties.

Table 1: General Material Properties

PROPERTY	SA 36	A 283C	SA 516 GR70
YOUNG'S MODULUS	29,400 ksi	29,400 ksi	29,400 ksi
POISSON'S RATIO	0.3	0.3	0.3
YIELD STRENGTH	36 ksi	30 ksi	38 ksi
UTS	58ksi	55 ksi	70ksi
DENSITY	7750 kg/m^3	7750 kg/m^3	7750 kg/m^3

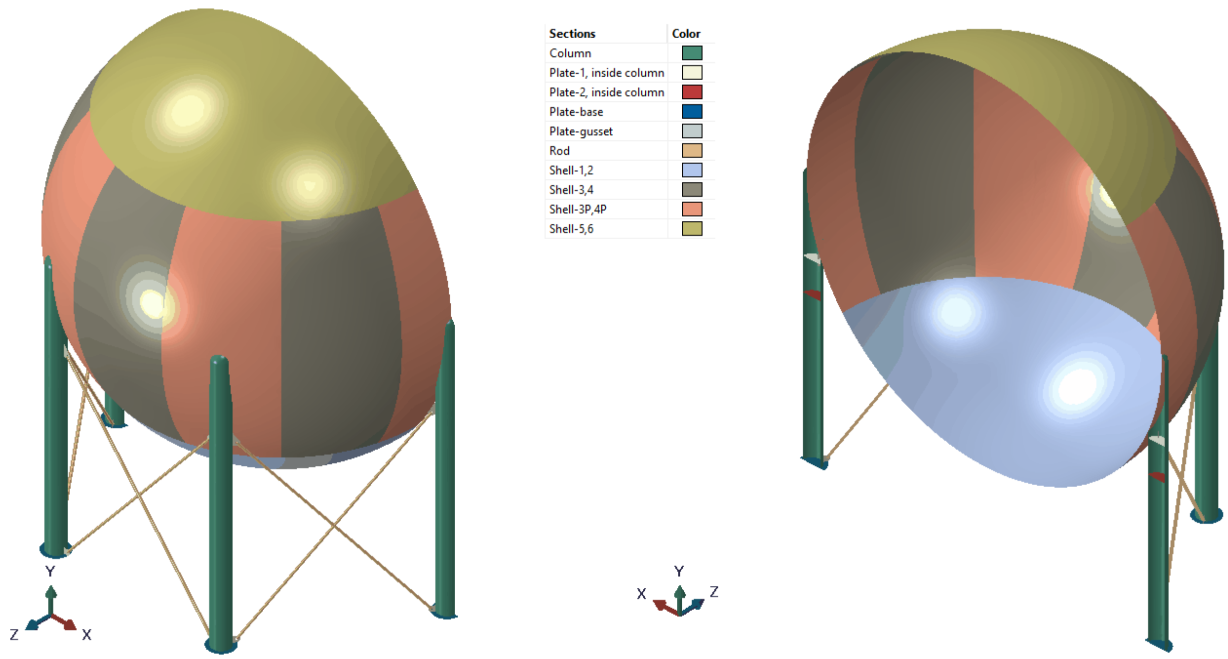


Figure 8: Section Assignments to the Model

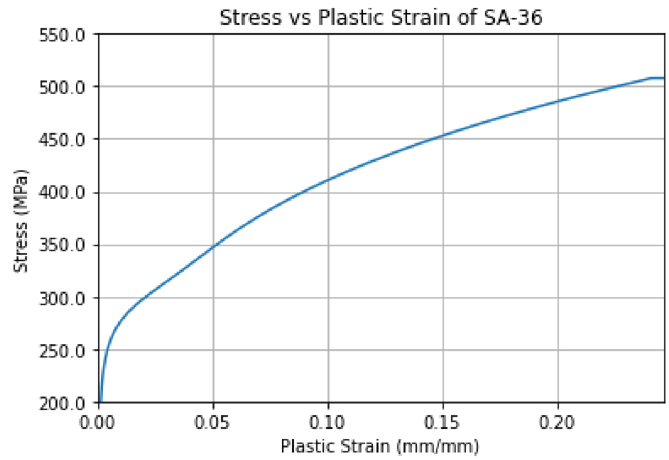


Figure 9: Elastic Plastic Material Curve for SA-36

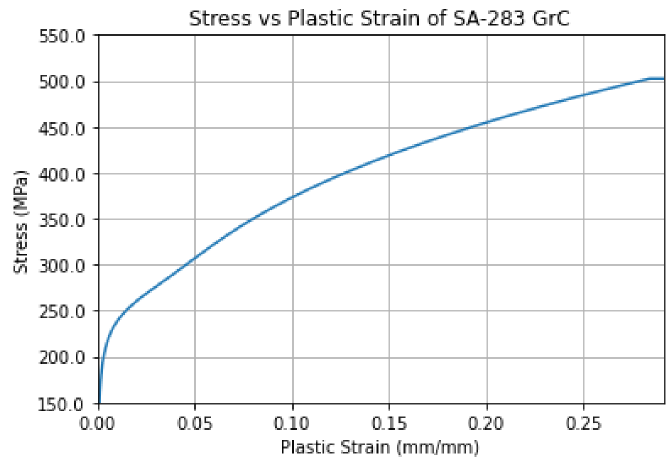


Figure 10:Elastic-Plastic Material Curve for SA-283C

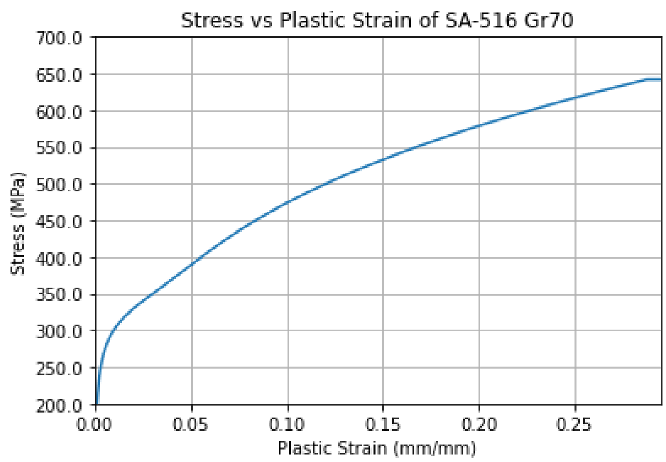


Figure 11: Elastic-Plastic Material Curve for SA-516 Gr70

6.3. Loads

Table 2 lists a description of the loads applied in the analyses. Derivation of the Earthquake, Wind, and Hydrotest Pressure loads are shown in Section 10 (Appendix B: Load Calculations). The hydrotest pressure is calculated per ASME VIII Div1 UG-99.b. Some of the parameters and characteristics of the members are shown in Section 9 (Appendix A: Sphere and Component Characteristics). Table 3 lists the load cases for this three assessment along with passing criteria and status. Where the load case fails, the point in the analysis where divergence occurred is indicated next to the status. Note that there are failing load cases. The failure point indicates the percentage of the maximum load for that case where the analysis cannot find equilibrium and therefore diverges. This failure point should not be used to extrapolate an acceptable fill height for the sphere as separate analysis would be required.

It should also be noted that the posted hydrotest pressure (350 psi) is higher than 1.5 x P = 337.5. A calculated hydrotest pressure value is calculated in the Appendix labeled, “Hydrotest Load Development Per ASME VII Div1”

Table 2: Loads Applied to the Model

Load Parameter	Description	Value	Units
S	Snow Loads	0	kN
L	Live Loads	0	kN
T	Thermal Loads	0	kN
P	Design Pressure	225	psig
Pt	Hydrotest Pressure	350	psig
Po	Operating Pressure	177.8	psig
Ps	Static Head from Liquid	6.3	psig
PsT	Static Head from Hydrotest	15.2	psig
D	Dead Weight	1137	kN
Dc	Dead Weight Corroded	calculated	kN
DI	Weight of Contents	2972	kN
Eh	Current Horizontal Earthquake	1360	kN
Ev	Current Vertical Earthquake	600	kN
E_1978	Original Earthquake	400	kN
W	Current Wind	43.3	kN
W_1978	Original Wind	67.6	kN
Wpt	Hydrotest Wind	43.6	kN

Table 3: LRFD Combinations ASME VIII Div. 2 2019 Table 5.5

Load Case	Assessment	Description	Load Combination	Criteria	Status
1	1978	Global Check	2.4(P+Ps+Dc+DI)	Convergence	Fail @ 99%
2	1978	Wind Global Check	2.1(P+Ps+Dc+DI) + 1.7W_1978	Convergence	Pass
3	1978	Seismic Global Check	2.1(P+Ps+D+DI) + 1.7E_1978	Convergence	Pass
4	1978	Local Check	1.7(P+Ps+Dc+DI)	LTSR	Pass
5	1978	Hydrotest Global Check	1.7(Pt+PsT+Dc+DI) + Wpt	Convergence	Fail @ 73%
6	Current	New seismic Global Check	2.1(P+Ps+Dc) + 1.7Eh + 1.7Ev	Convergence	Fail @ 49%
7	Current	New Seismic 27% Fill	2.1(P+Ps+Dc+0.32DI) + 1.7Eh + 1.7Ev	Convergence	Pass
1-A	Current	Global Check	2.4(P+Ps+Dc+DI)	Convergence	Fail @ 99%
2-A	Current	Wind Global Check	2.1(P+Ps+Dc+DI) + 1.7W	Convergence	Pass
4-A	Current	Local Check	1.7(P+Ps+Dc+DI)	LTSR	Pass
5-A	Current	Hydrostest global check	1.7(Pt+PsT+Dc+DI) + Wpt	Convergence	Fail @ 73%

Figure 12 shows the loads applied to the model. As load case 6 fails, load case 7 was added to show that the sphere passes code with a maximum fill ratio of 27% as to the 85% in load cases 1-6. Figure 13 shows the loads applied to the sphere for this case.

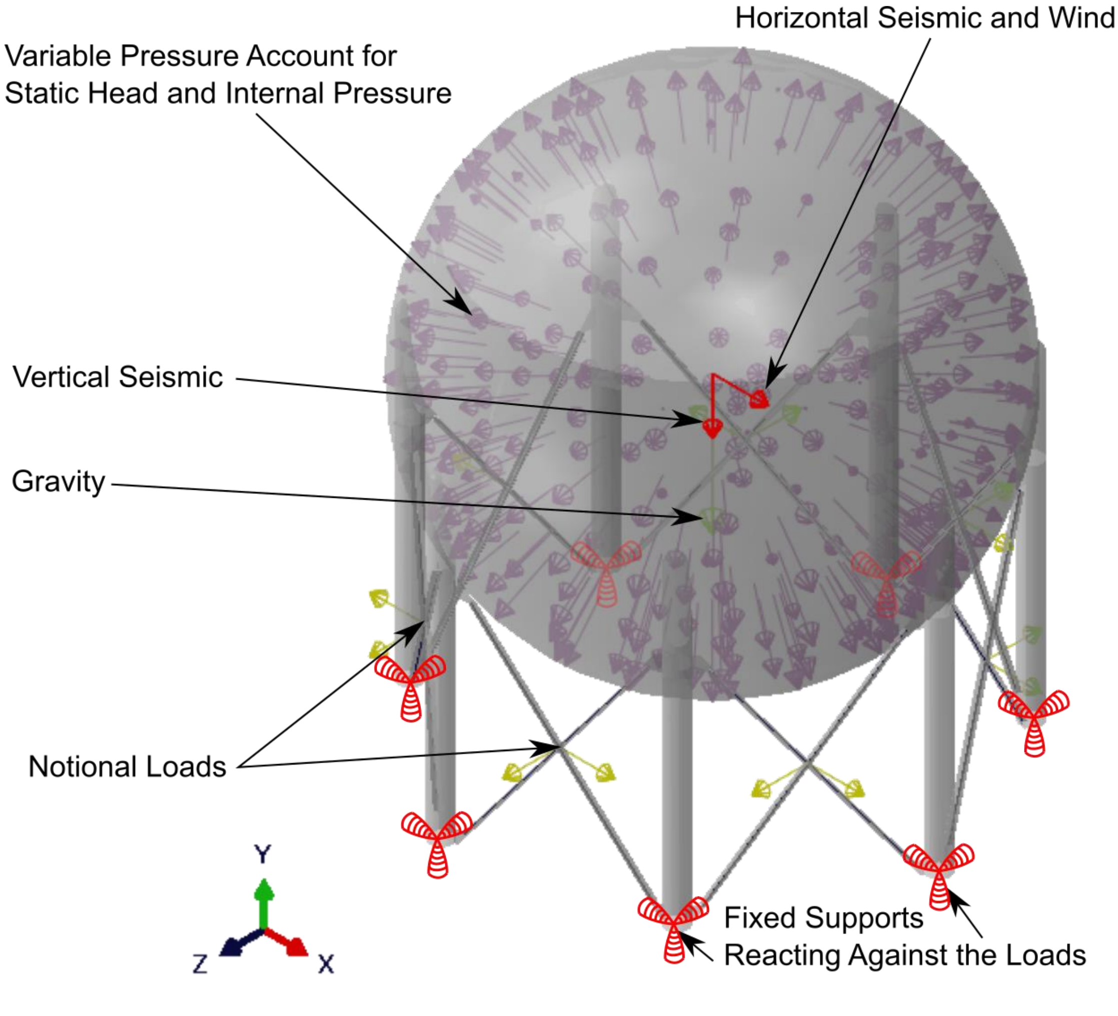


Figure 12: Loads Applied to Model

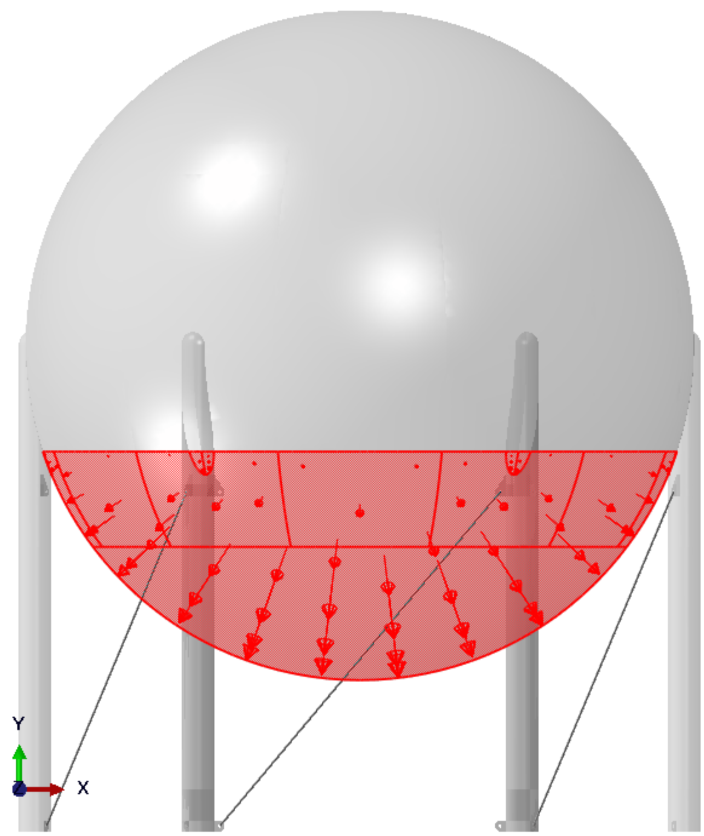


Figure 13: Pressure Profile Applied to Load Case 7 (Maximum Fill Height)

7. RESULTS

Two criteria are used in the evaluation of a Part 5 EP analysis, Protection from Global Collapse[4] and Protection from Local Failure[5]. The Protection from Global Collapse criteria is met if the analysis converges on a solution under the given LRFD case loading[6].

The Protection from Local Failure criteria is met if the Limiting Triaxial Stress Ratio (LTSR) is less than one at each element for the converged results as shown in the relationship below:

$$\frac{\varepsilon_{peq}+\varepsilon_{cf}}{\varepsilon_{Lu}\cdot \mathbf{e}^{\left(-\left(\frac{\alpha_{sl}}{1+m_2}\right)\cdot\left(\left(\frac{(\sigma_1+\sigma_2+\sigma_3)}{3\cdot\sigma_e}\right)-\frac{1}{3}\right)\right)}}\leq 1.0$$

Where :

ε_{Lu}	Uniaxial Strain Limit (ASME VIII Div2 Part 5 Table 5.7)
m_2	Uniaxial Strain Limit Factor (ASME VIII Div2 Part 5 Table 5.7)
α_{sl}	Material Factor for the Multiaxial Strain Limit (ASME VIII Div2 Part 5 Table 5.7)
σ_1	Maximum Principal Stress
σ_2	Middle Principle Stress
σ_3	Minimum Principle Stress
σ_e	Equivelant Stress
ε_{peq}	Equivelant Plastic Strain
ε_{cf}	Cold Forming Strain

Table 3 (shown previously) presents a list of the LRFD combinations run with the resulting status. The following subsections show graphical results from each load case. For each global collapse

[4] ASME VIII Div2 Part 5.2
[5] ASME VIII Div2 Part 5.3
[6] ASME VIII Div2 Part 5.2.4.4.e

case there is an equivalent stress plot (units are MPa) and an equivalent plastic strain plot indicating whether the model converged (Passed) or diverged (Failed). For each local failure case there is a plot showing the Limiting Triaxial Strain Ratio. If the maximum LTSR is less than one, the criteria are met. There are plots in each subsection showing closeups of the location of the welds of the column to sphere connections. There are also plots showing the reaction forces in the braces that are in tension for the load cases where a side load is applied (Seismic or Wind).

The analysis clearly shows one of the consistent weak points in the sphere support system is at the "armpit" or at the underside of the column to sphere connection (see Figure 16). This area buckles under load and causes divergence in load cases 1, 5, and 6. In cases 1 and 5 the failure is due to buckling in the "armpit" due to vertical loading only. Case 6 vertical loads are exacerbated by the new seismic loading.

Table 4 and Table 5 show the reaction loads and moments respectively at the base of each column for each load case. The factored (1.7) horizontal earthquake load is the summation of the Fx forces for load case 3 (shown in the "Totals" column) The maximum shear force at the column occurs in load case 3. The maximum shear force formulation is shown in Figure 14. The calculations shown in 11.1 "Column Base Support (CBS) Shear Calculation" show the column base support (including the anchor bolts) is sufficient for all load cases as the maximum load is used in the calculations.

The strength of the brace to column connection is shown section 11.2 and meets criteria

i	Specific Column Number
n	number of columnns
F_{xi}	Reaction force in the x direction at column number "i"
F_{yi}	Reaction force in the y direction at column number "i"
F_{zi}	Reaction force in the z direction at column number "i"

Maximum Column Shear Load = $\max\left(\left[\sqrt{{F_{x1}}^2+{F_{z1}}^2}, \sqrt{{F_{x2}}^2+{F_{z2}}^2} .. \sqrt{{F_{xn}}^2+{F_{zn}}^2}\right]\right)$

Figure 14: Maximum Shear Force Calculation

Table 4: Reaction Loads (N) at Base of Columns

LOAD CASE		COLUMN 1	2	3	4	5	6	TOTAL	MAXI COLUMN SHEAR
1	Fx	1.19E+02	-2.36E+02	-2.85E+02	-5.63E+01	-2.85E+02	-2.36E+02	-9.81E+02	7.32E+02
	Fy	1.59E+06	1.59E+06	1.60E+06	1.60E+06	1.60E+06	1.59E+06	9.58E+06	
	Fz	0.00E+00	-6.45E+02	-6.74E+02	0.00E+00	6.74E+02	6.45E+02	0.00E+00	
2	Fx	-1.47E+04	-8.39E+03	-4.35E+04	4.24E+03	-4.35E+04	-8.39E+03	-1.14E+05	4.54E+04
	Fy	1.43E+06	1.45E+06	1.35E+06	1.37E+06	1.35E+06	1.45E+06	8.39E+06	
	Fz	0.00E+00	-1.18E+04	-1.30E+04	0.00E+00	1.30E+04	1.18E+04	0.00E+00	
3	Fx	1.38E+04	-2.51E+04	-2.09E+05	-2.18E+05	-2.09E+05	-2.51E+04	-6.73E+05	2.18E+05
	Fy	1.70E+06	1.59E+06	1.32E+06	8.73E+05	1.32E+06	1.59E+06	8.39E+06	
	Fz	0.00E+00	6.33E+04	-2.35E+03	0.00E+00	2.35E+03	-6.33E+04	0.00E+00	
4	Fx	-1.97E+04	-9.83E+03	9.86E+03	1.97E+04	9.86E+03	-9.83E+03	9.21E+01	1.97E+04
	Fy	1.13E+06	1.13E+06	1.13E+06	1.13E+06	1.13E+06	1.13E+06	6.79E+06	
	Fz	0.00E+00	-1.71E+04	-1.71E+04	0.00E+00	1.71E+04	1.71E+04	0.00E+00	
5	Fx	-5.90E+03	-5.25E+03	-4.29E+03	-4.68E+03	-4.29E+03	-5.25E+03	-2.97E+04	6.04E+03
	Fy	1.54E+06	1.53E+06	1.51E+06	1.50E+06	1.51E+06	1.53E+06	9.13E+06	
	Fz	0.00E+00	-2.98E+03	-3.77E+03	0.00E+00	3.77E+03	2.98E+03	0.00E+00	

6	Fx	-1.04E+04	-1.33E+05	-2.86E+05	-2.87E+05	-2.86E+05	-1.33E+05	-1.14E+06	2.87E+05
	Fy	1.35E+06	9.89E+05	5.75E+05	1.41E+05	5.75E+05	9.89E+05	4.62E+06	
	Fz	0.00E+00	1.78E+05	-4.25E+03	0.00E+00	4.25E+03	-1.78E+05	0.00E+00	
7	Fx	-3.37E+04	-1.47E+05	-2.82E+05	-2.72E+05	-2.82E+05	-1.47E+05	-1.17E+06	2.83E+05
	Fy	1.20E+06	7.89E+05	4.07E+05	-2.94E+04	4.07E+05	7.89E+05	3.56E+06	
	Fz	0.00E+00	1.68E+05	-1.36E+04	0.00E+00	1.36E+04	-1.68E+05	0.00E+00	

Table 5: Reaction Moments (N-mm) at Base of Columns (In Reference to Global Coordinate System)

LOAD CASE		COLUMN					
		1	2	3	4	5	6
1	Mx	0.00E+00	-1.02E+07	-1.07E+07	0.00E+00	1.07E+07	1.02E+07
	My	0.00E+00	-1.13E+06	-9.44E+05	0.00E+00	9.44E+05	1.13E+06
	Mz	-1.90E+06	-5.50E+06	-1.49E+07	-2.08E+07	-1.49E+07	-5.50E+06
2	Mx	0.00E+00	-2.09E+07	-3.68E+07	0.00E+00	3.68E+07	2.09E+07
	My	0.00E+00	2.31E+06	1.19E+06	0.00E+00	-1.19E+06	-2.31E+06
	Mz	4.98E+07	3.80E+07	1.68E+06	-7.01E+06	1.68E+06	3.80E+07

3	Mx	0.00E+00	-2.66E+07	-8.56E+06	0.00E+00	8.56E+06	2.66E+07
	My	0.00E+00	3.79E+06	7.26E+06	0.00E+00	-7.26E+06	-3.79E+06
	Mz	1.03E+07	-5.63E+06	-3.82E+07	2.00E+07	-3.82E+07	-5.63E+06
4	Mx	0.00E+00	-4.27E+07	-4.26E+07	0.00E+00	4.26E+07	4.27E+07
	My	0.00E+00	5.77E+03	1.29E+04	0.00E+00	-1.29E+04	-5.77E+03
	Mz	4.92E+07	2.45E+07	-2.47E+07	-4.93E+07	-2.47E+07	2.45E+07
5	Mx	0.00E+00	-1.10E+07	-1.14E+07	0.00E+00	1.14E+07	1.10E+07
	My	0.00E+00	1.28E+06	1.96E+06	0.00E+00	-1.96E+06	-1.28E+06
	Mz	4.39E+07	4.02E+07	3.64E+07	3.73E+07	3.64E+07	4.02E+07
6	Mx	0.00E+00	-6.08E+07	-7.76E+06	0.00E+00	7.76E+06	6.08E+07
	My	0.00E+00	3.66E+06	1.63E+07	0.00E+00	-1.63E+07	-3.66E+06
	Mz	5.37E+07	9.84E+07	1.46E+08	1.84E+08	1.46E+08	9.84E+07
7	Mx	0.00E+00	-7.46E+07	-3.00E+07	0.00E+00	3.00E+07	7.46E+07
	My	0.00E+00	8.47E+06	1.74E+07	0.00E+00	-1.74E+07	-8.47E+06
	Mz	1.20E+08	1.30E+08	1.51E+08	1.65E+08	1.51E+08	1.30E+08

7.1. Case 1: 1978 Assessment Global Check

The analysis did not have a converged solution. Convergence stopped at 99%.

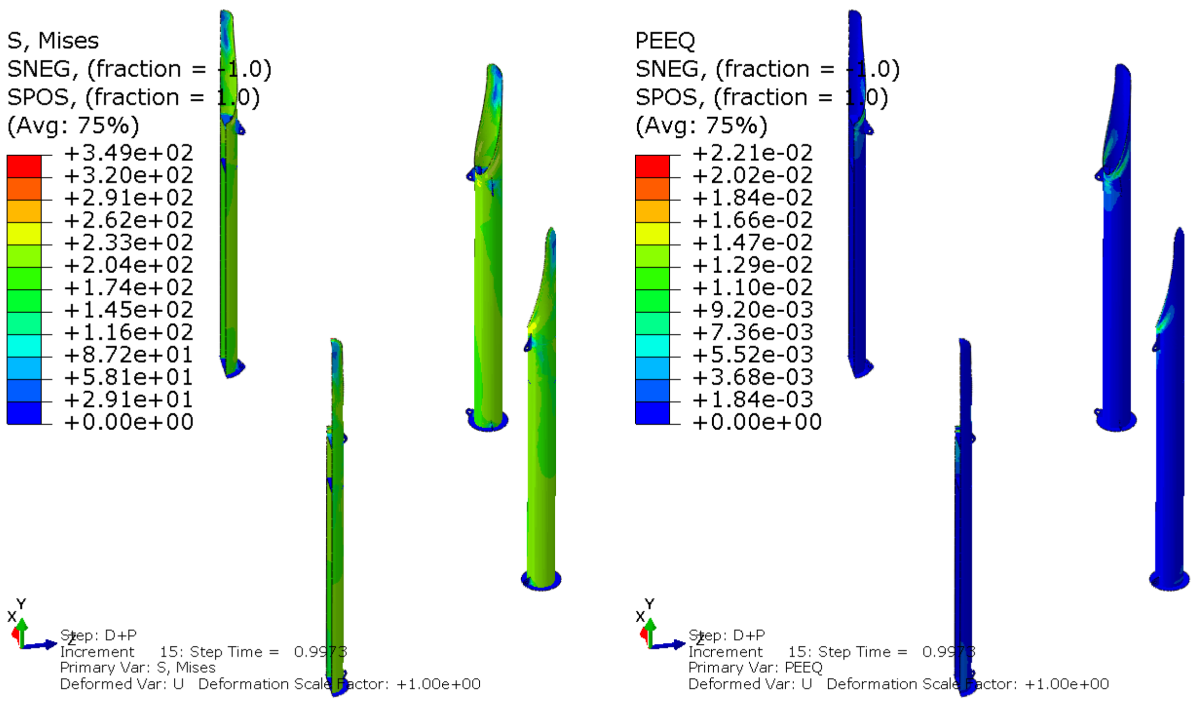


Figure 15: Case 1 Equivalent Stress and Plastic Strain (Failed)

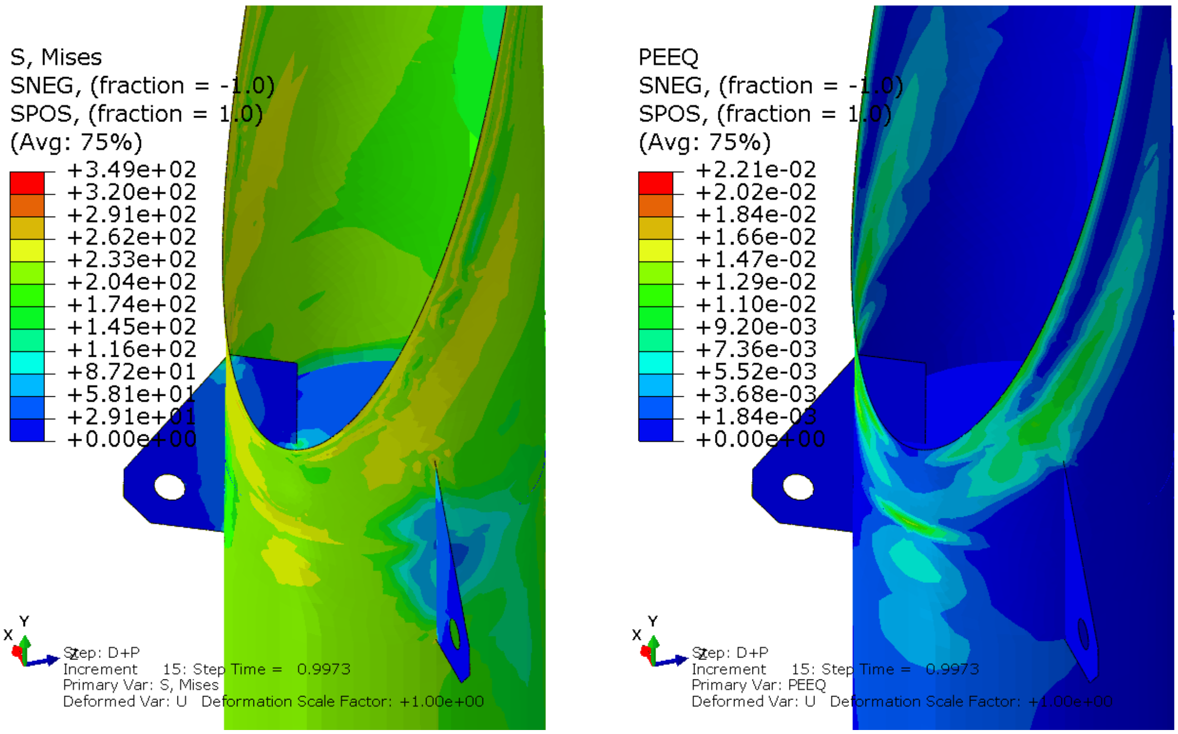


Figure 16: Case 1 Equivalent Stress and Plastic Strain Closeup (Failed)

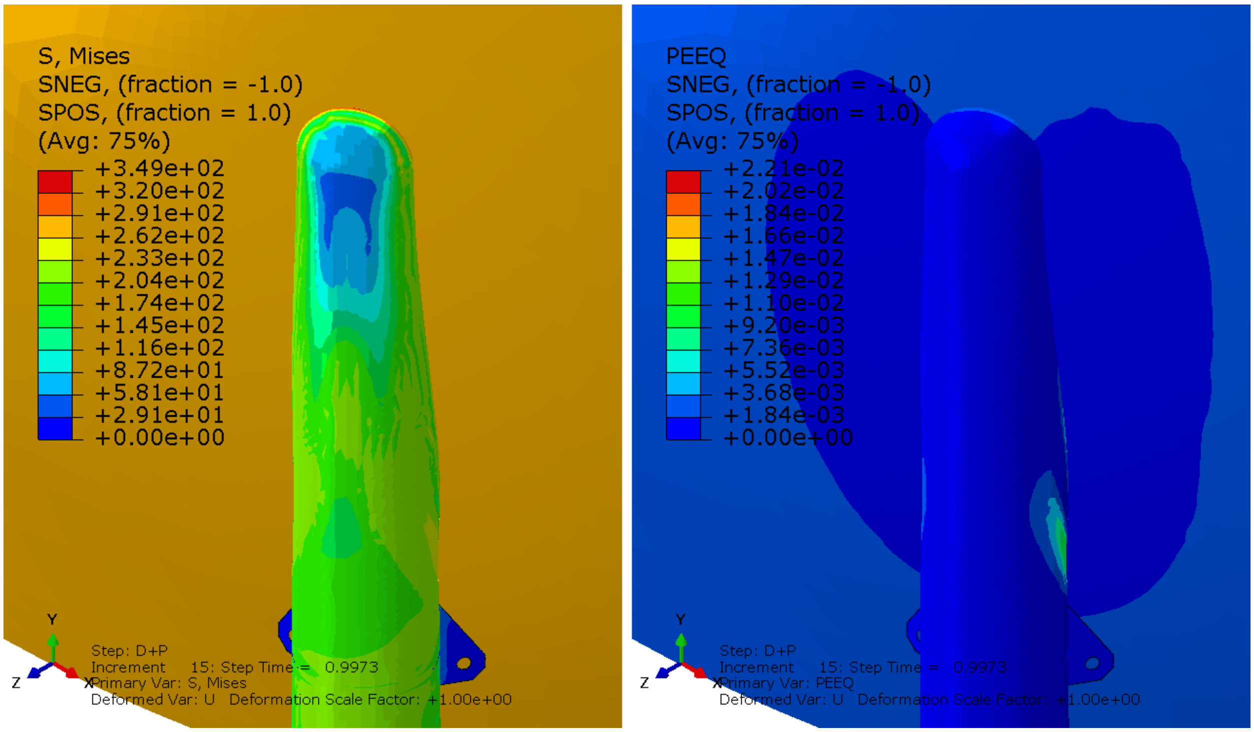


Figure 17: Case 1 Sphere to Column Weld Region

7.2. Case 2: 1978 Assessment w/ Wind Global Check

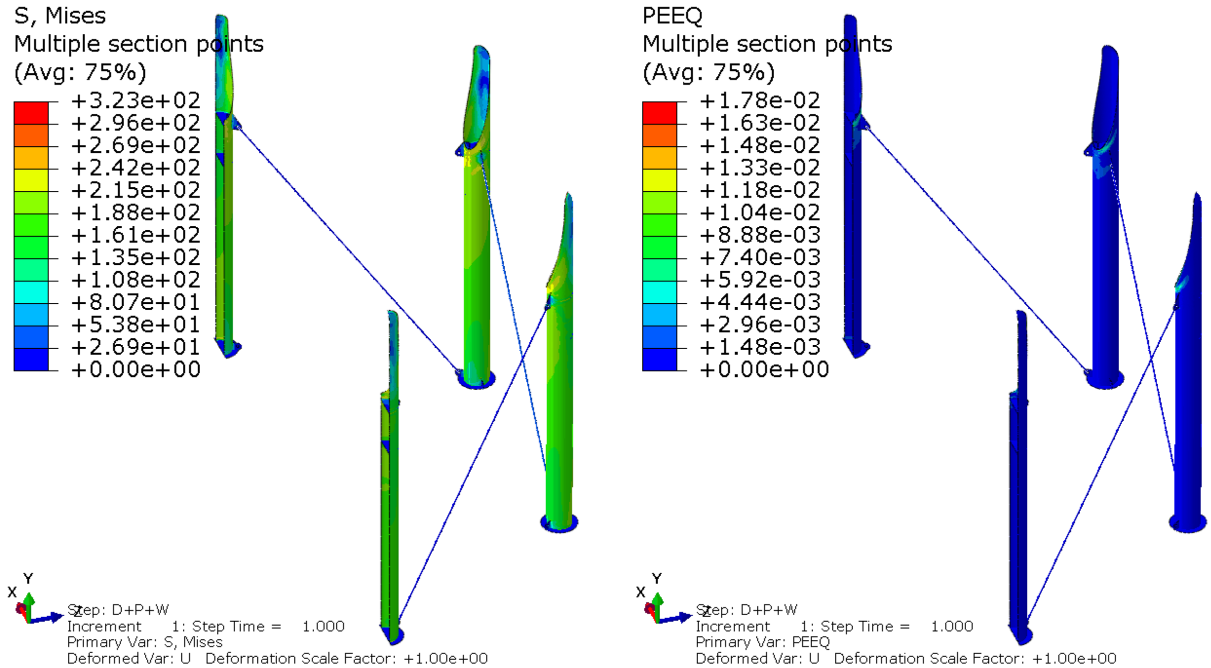


Figure 18: Case 2 Equivalent Stress and Plastic Strain (Passed)

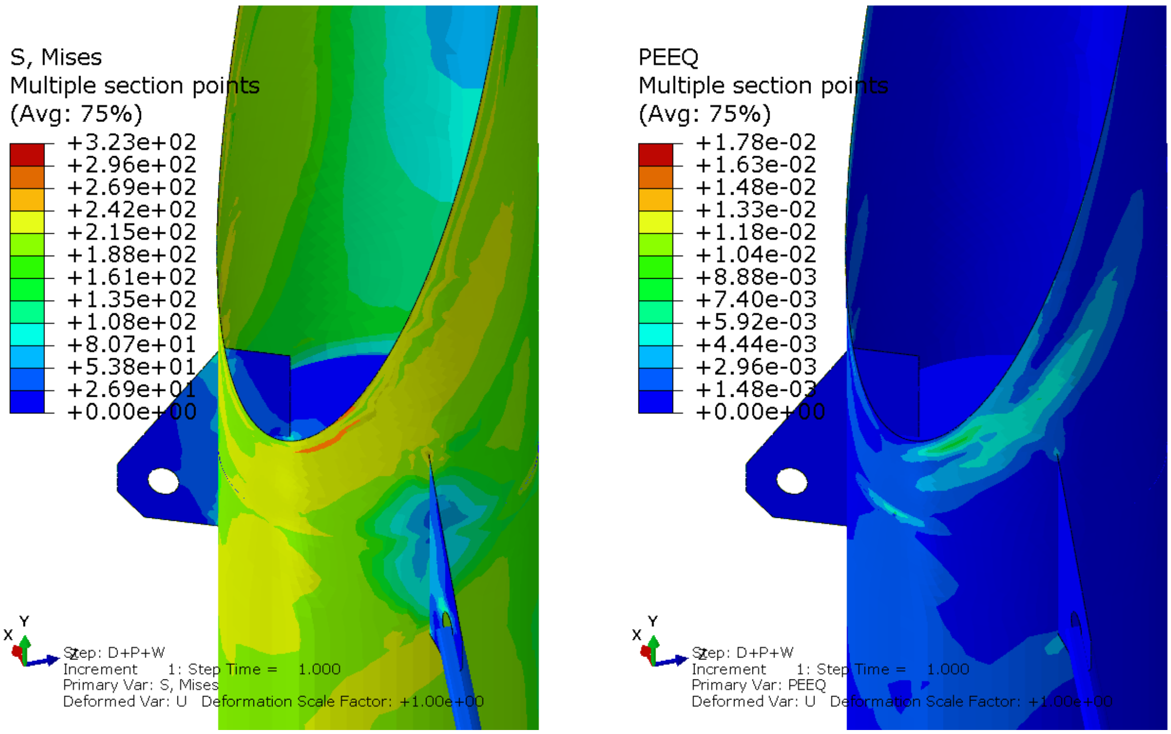


Figure 19: Case 2 Equivalent Stress and Plastic Strain Closeup (Passed)

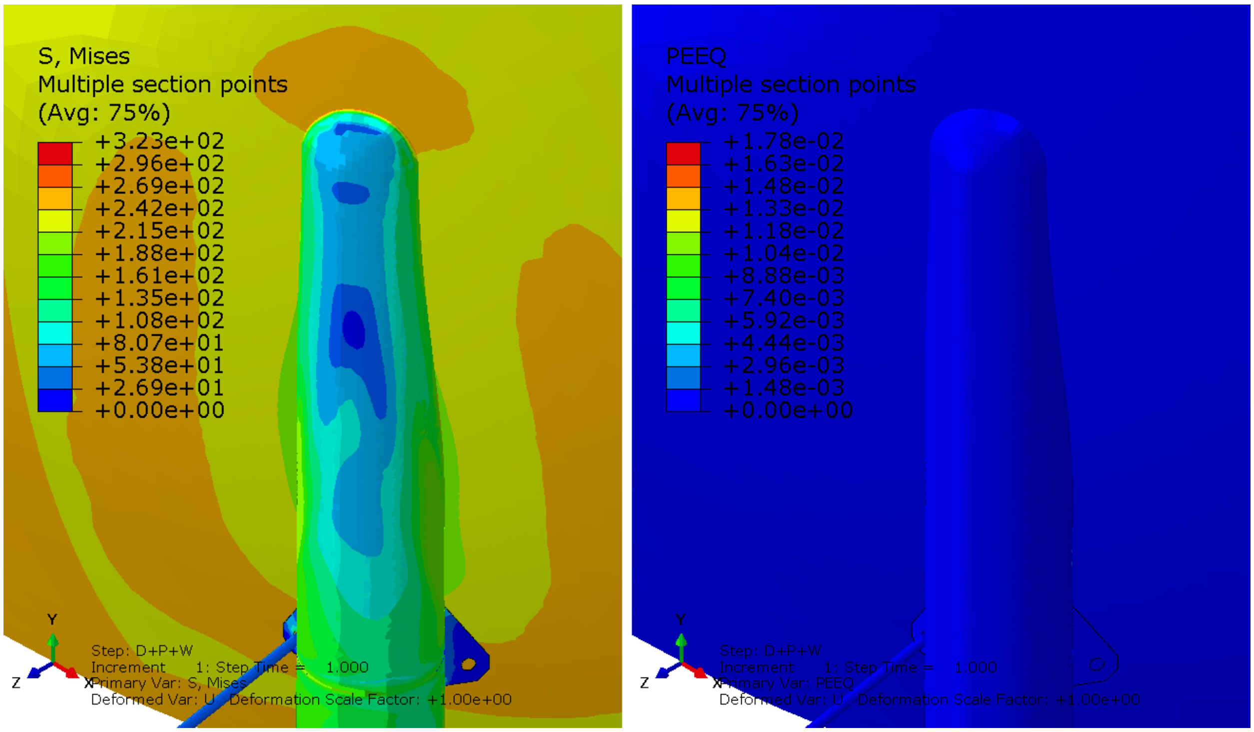


Figure 20: Case 2 Sphere to Column Weld Region

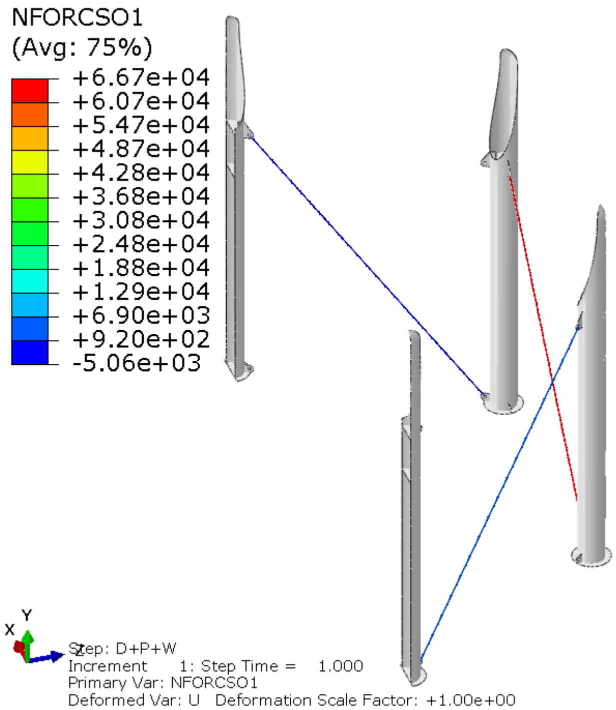


Figure 21: Case 2 Brace Reaction Loads

7.3. Case 3: 1978 Assessment w/ Seismic Global Check

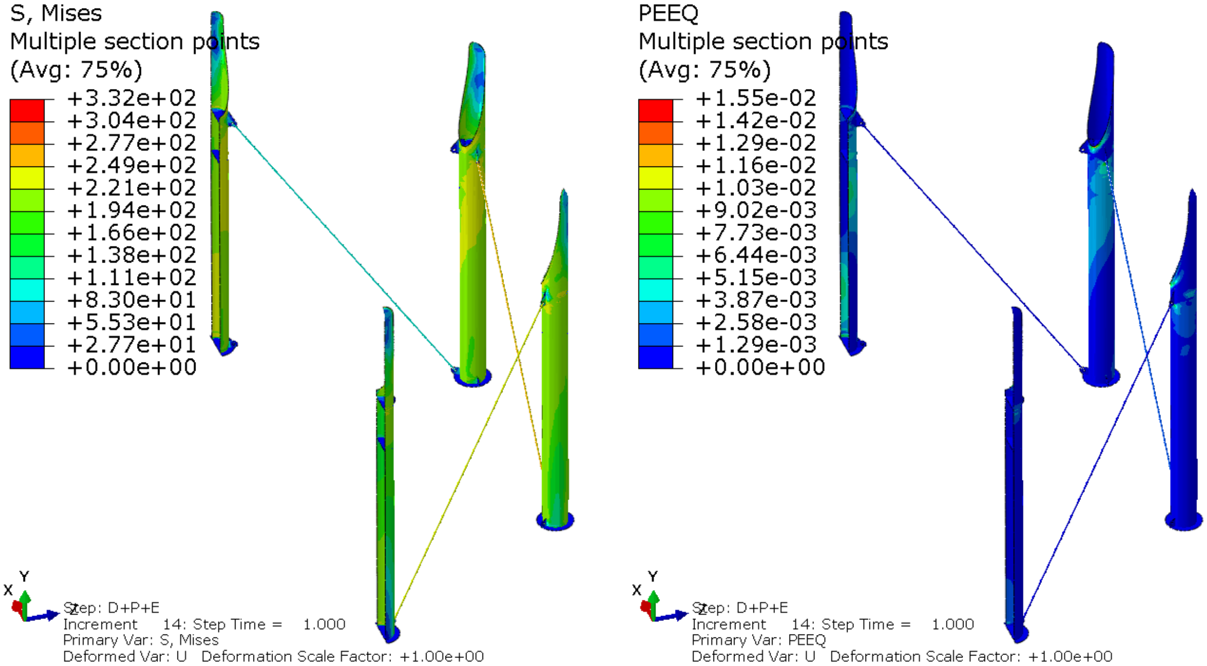


Figure 22: Case 3 Equivalent Stress and Plastic Strain (Passed)

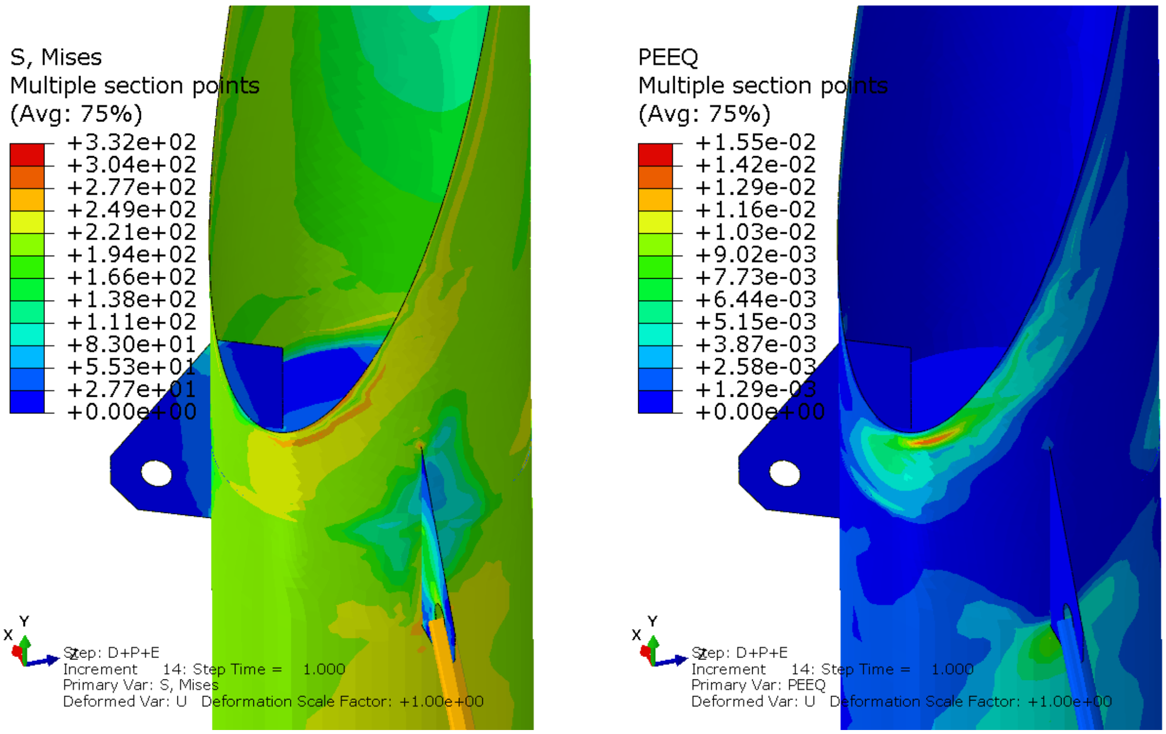


Figure 23: Case 3 Equivalent Stress and Plastic Strain Closeup (Passed)

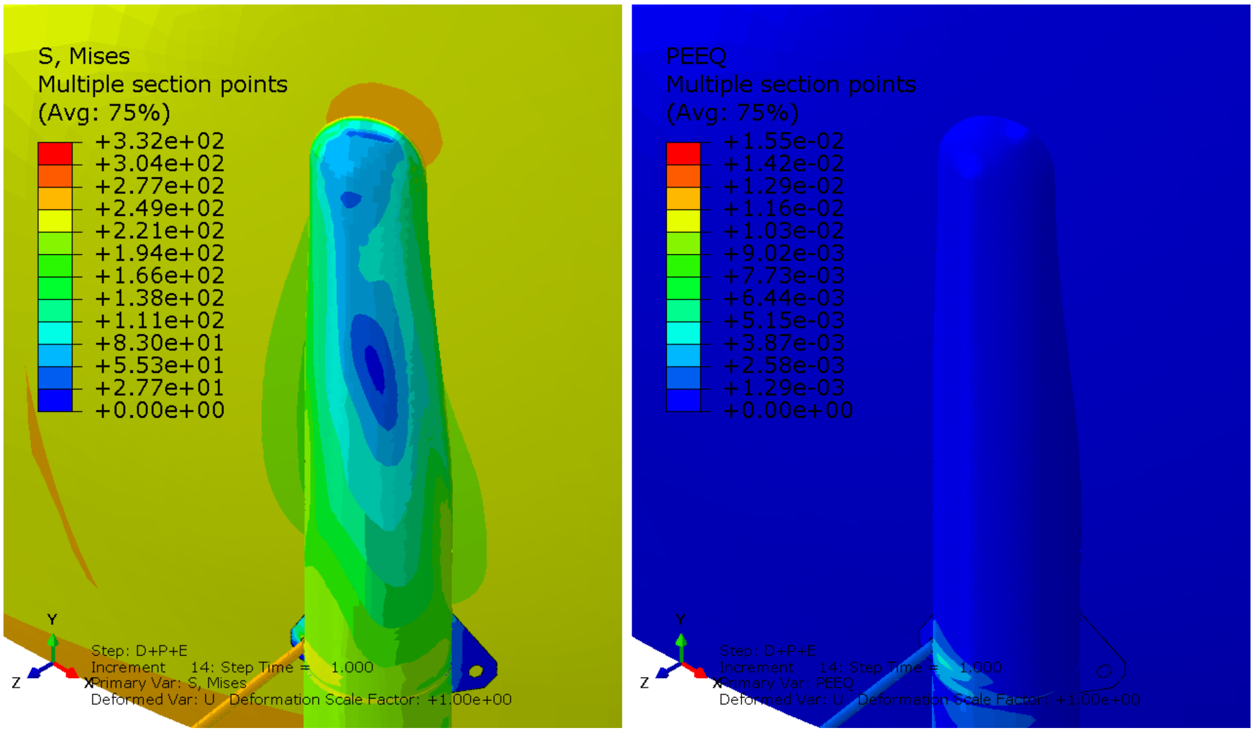


Figure 24: Case 3 Sphere to Column Weld Region

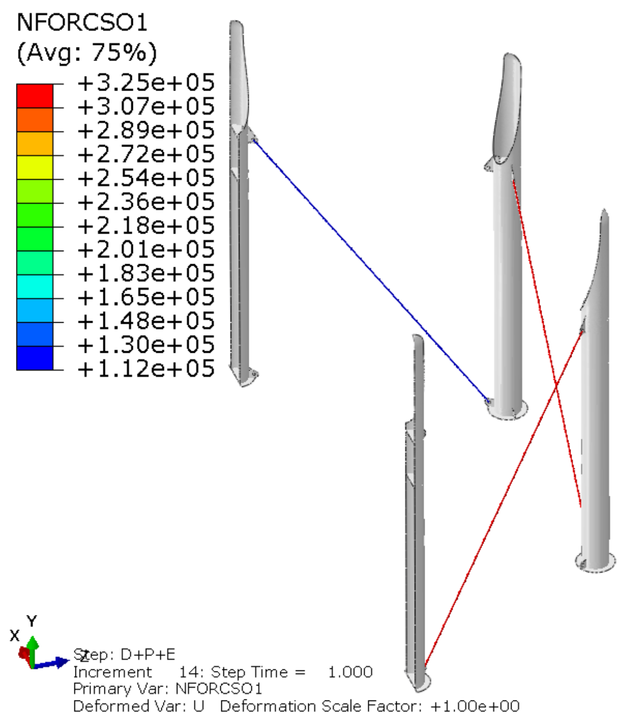


Figure 25: Case 3 Brace Reaction Loads

7.4. Case 4: 1978 Assessment Local Check

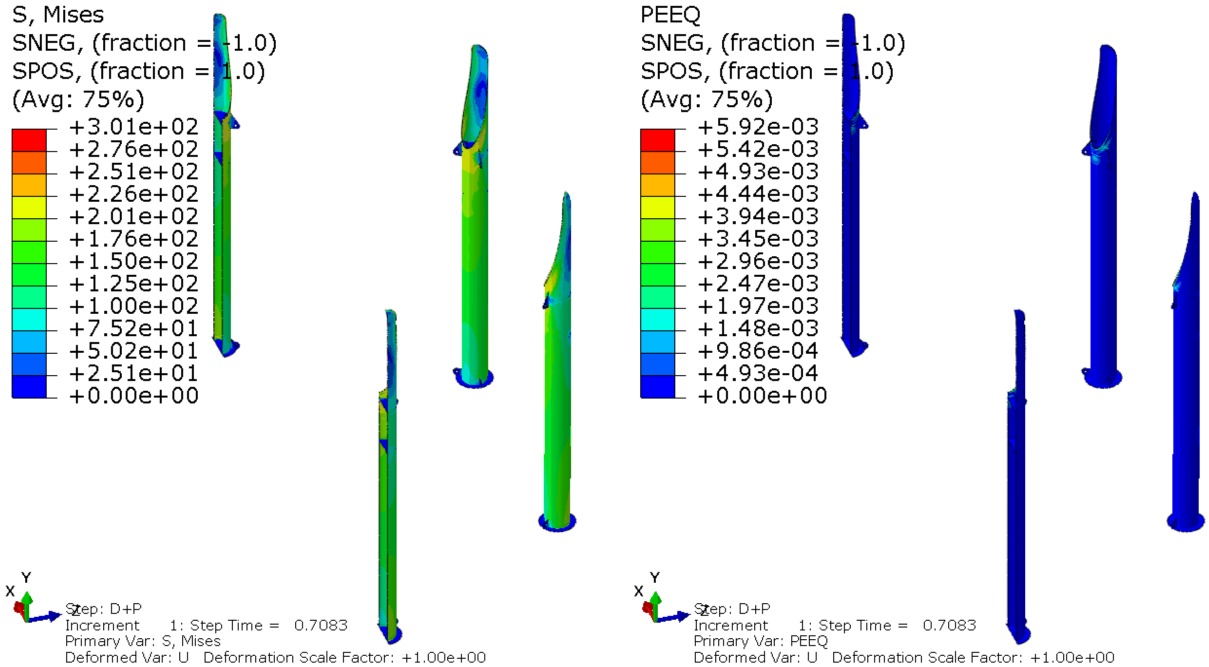


Figure 26: Case 4 Equivalent Stress and Plastic Strain (Passed)

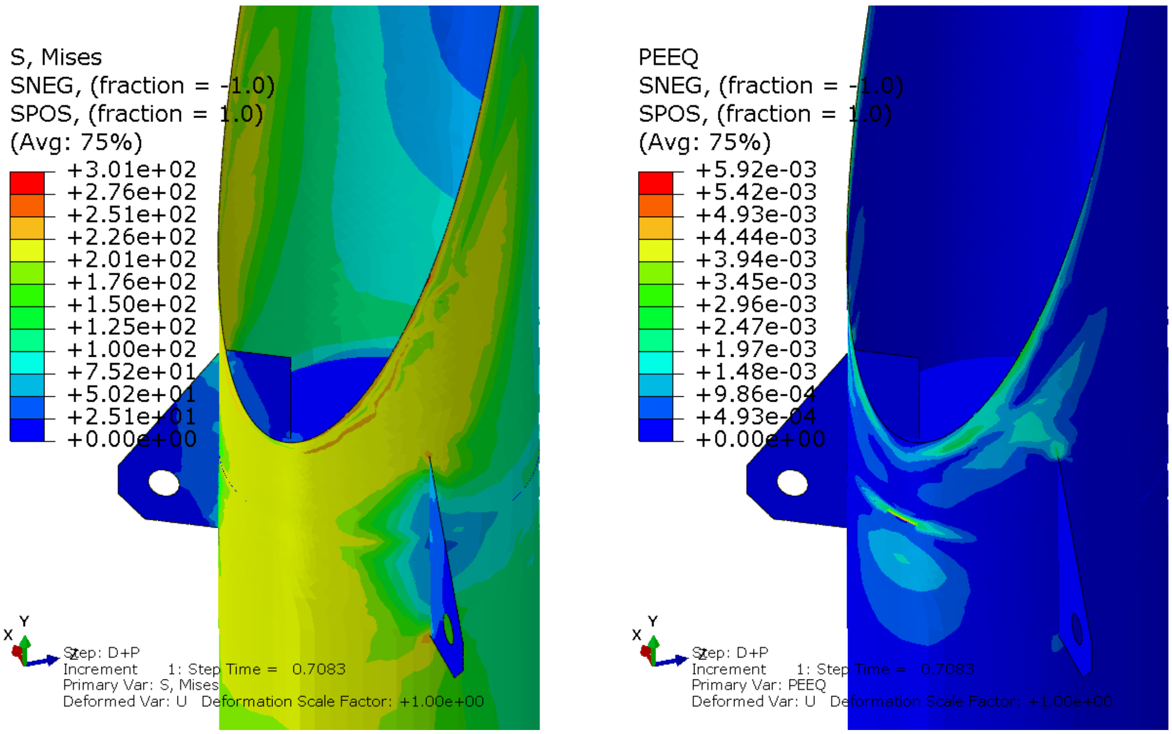


Figure 27: Case 4 Equivalent Stress and Plastic Strain Closeup (Passed)

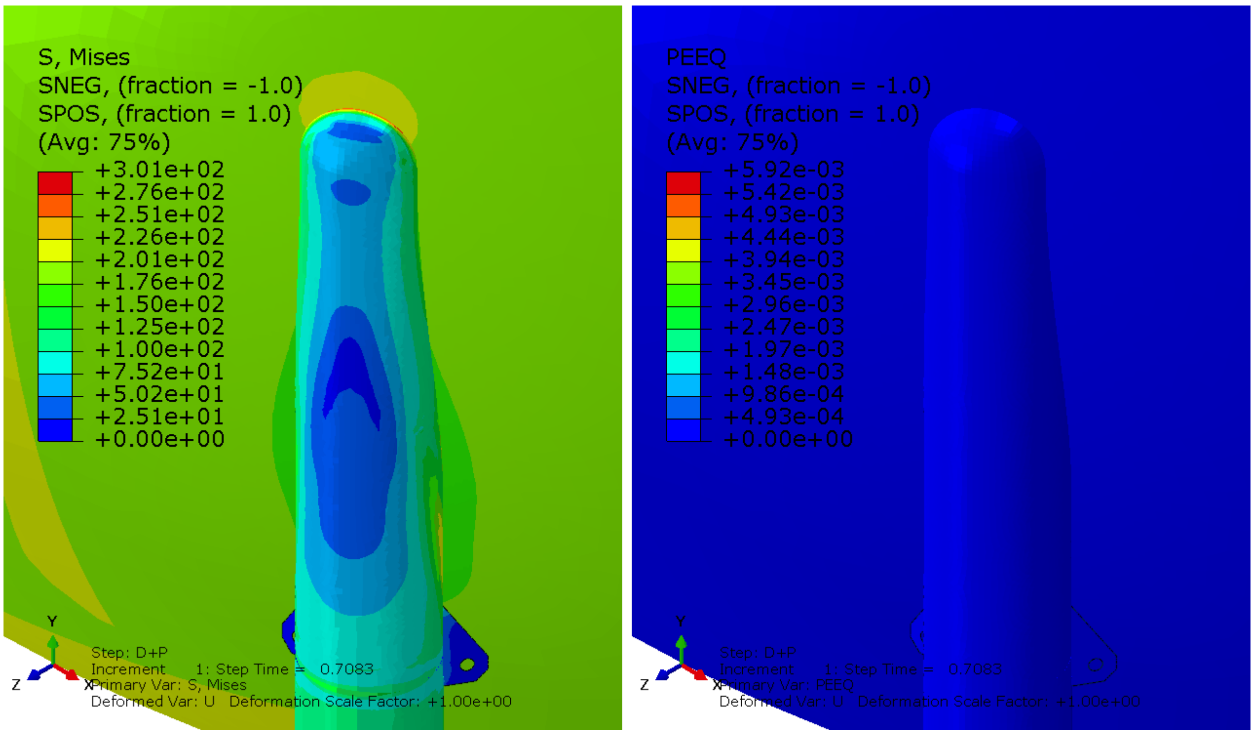


Figure 28: Case 4 Sphere to Column Weld Region

7.5. Case 5: 1978 Assessment Hydrotest Global Check

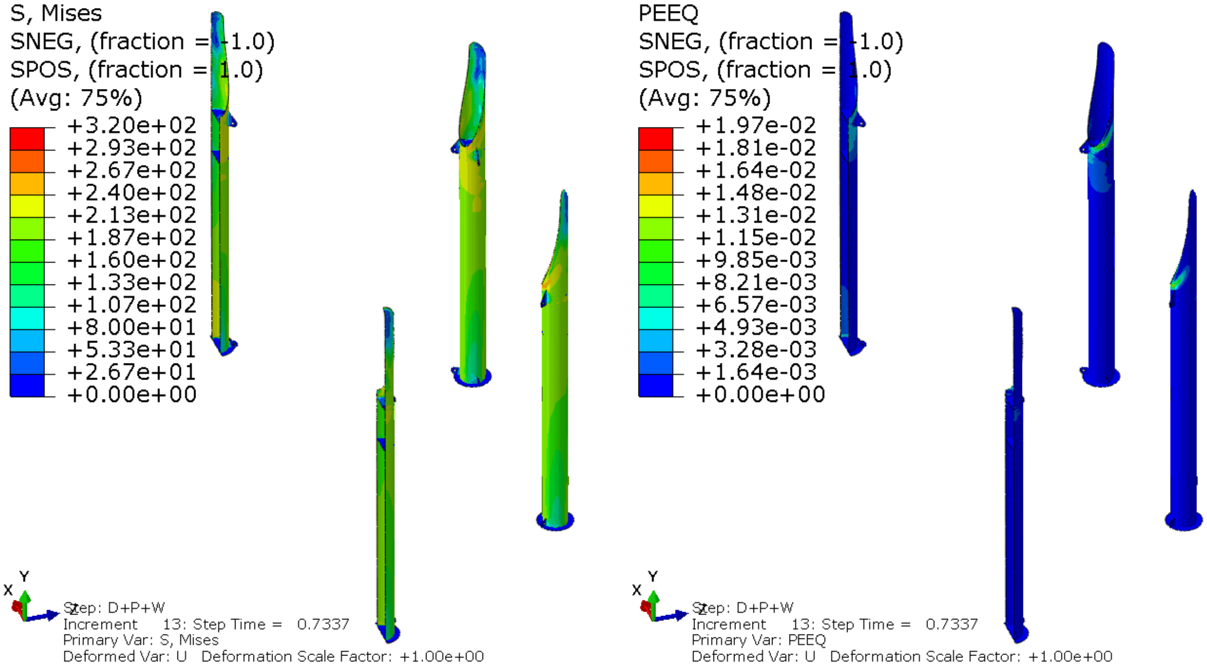


Figure 29: Case 5 Equivalent Stress and Plastic Strain (Failed)

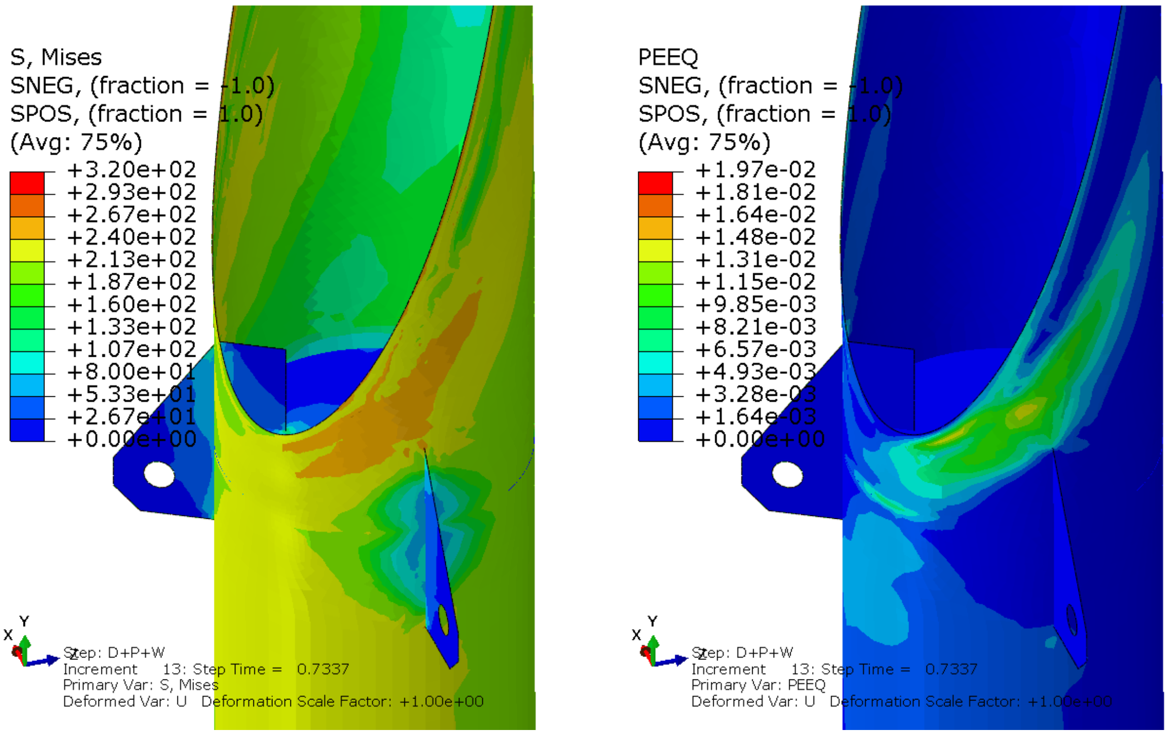


Figure 30: Case 5 Equivalent Stress and Plastic Strain Closeup (Failed)

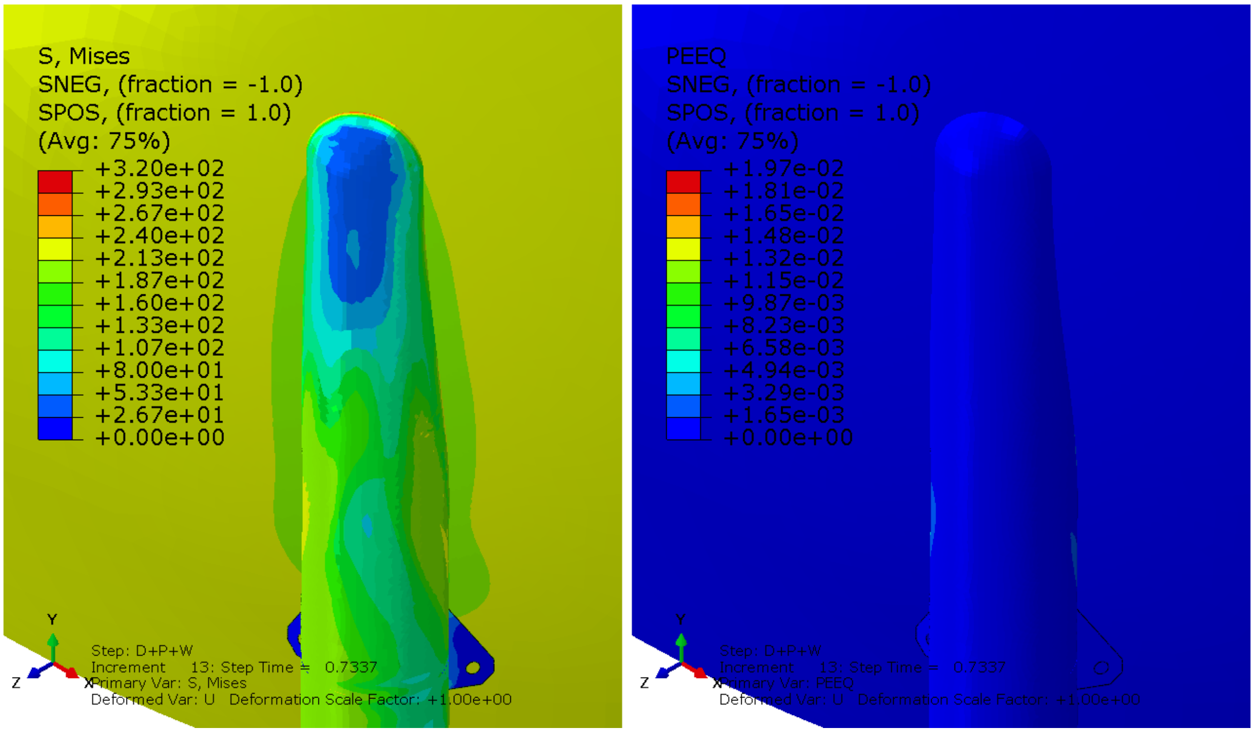


Figure 31: Case 5 Sphere to Column Weld Region

7.6. Case 6: Current Assessment

The analysis did not have a converged solution. Convergence stopped at 49%.

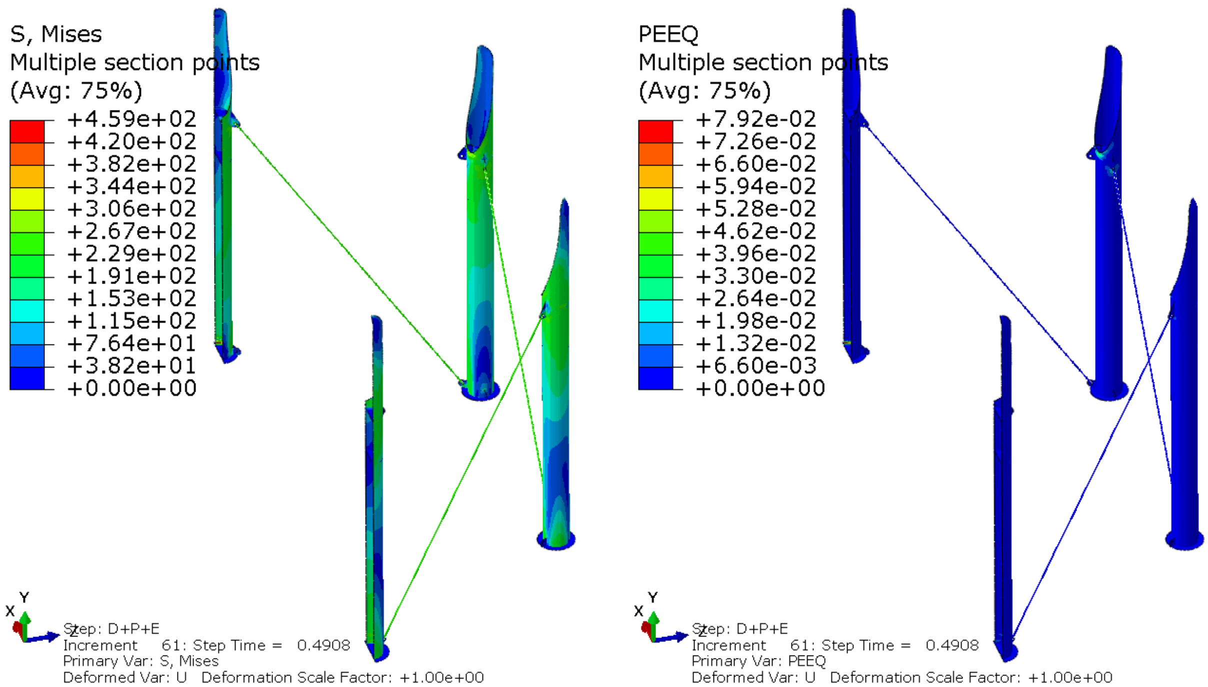


Figure 32: Case 6 Equivalent Stress and Plastic Strain (Failed)

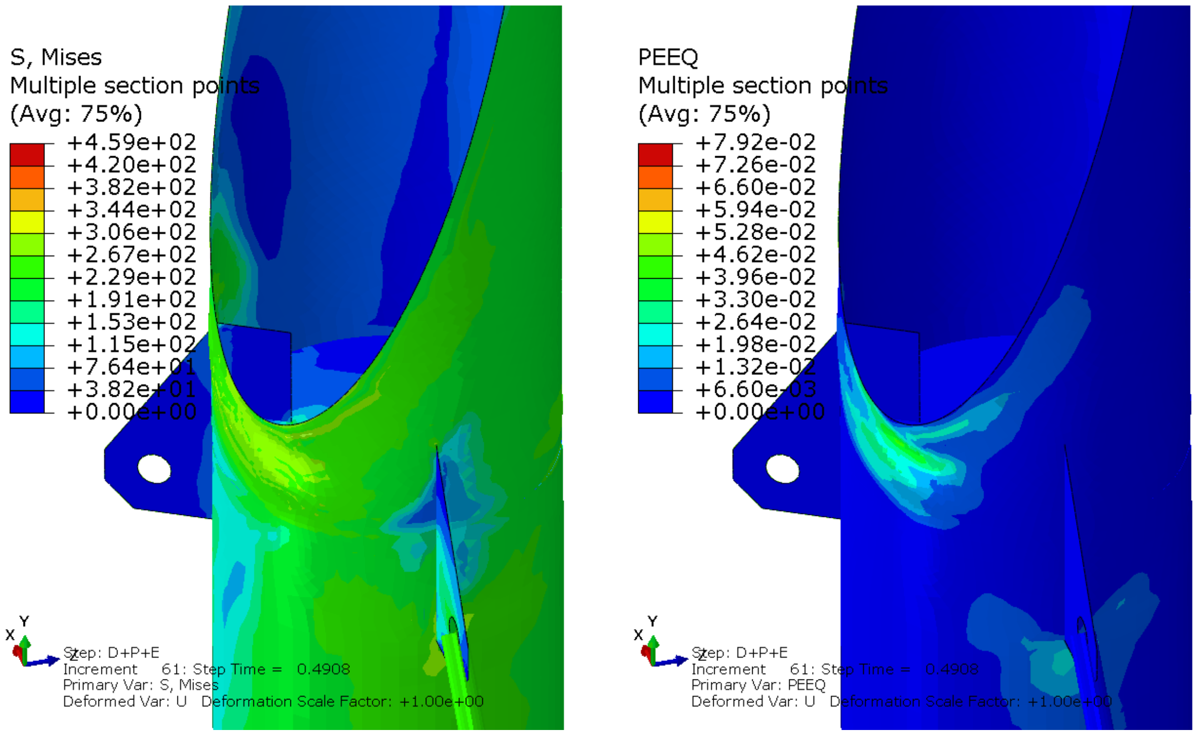


Figure 33: Case 6 Equivalent Stress and Plastic Strain Closeup (Failed)

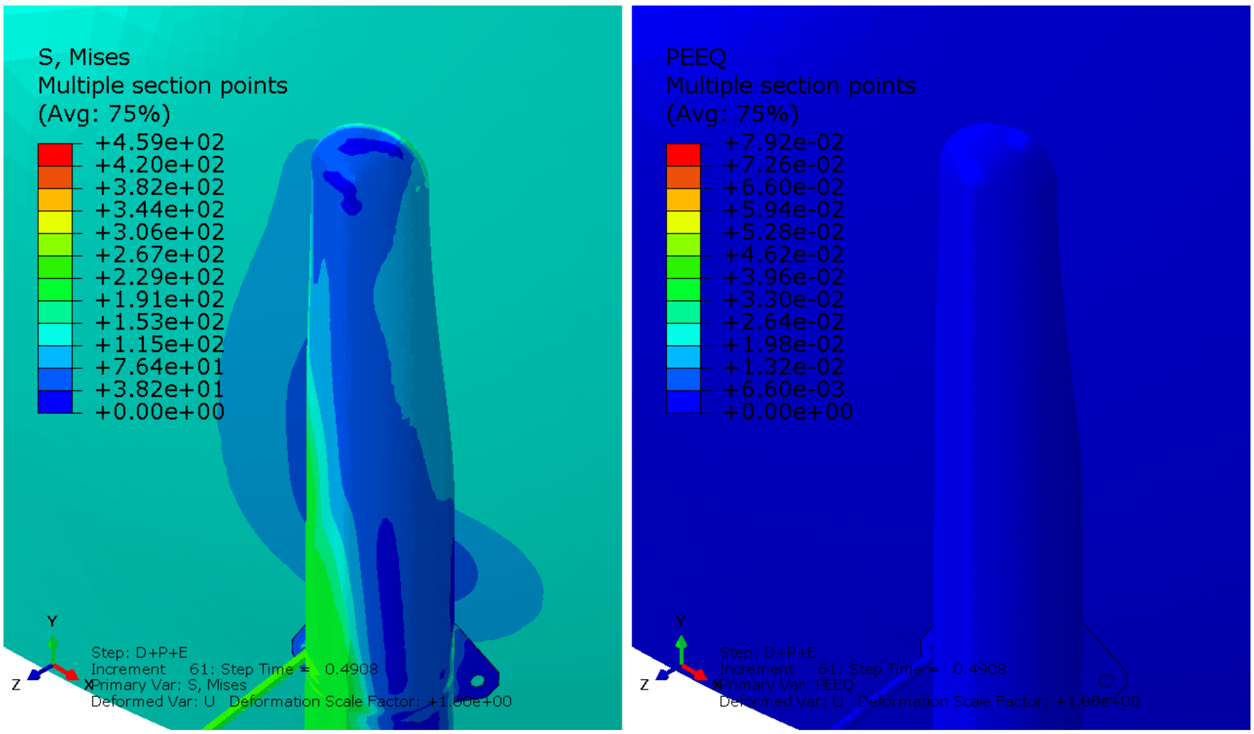


Figure 34: Case 6 Sphere to Column Weld Region

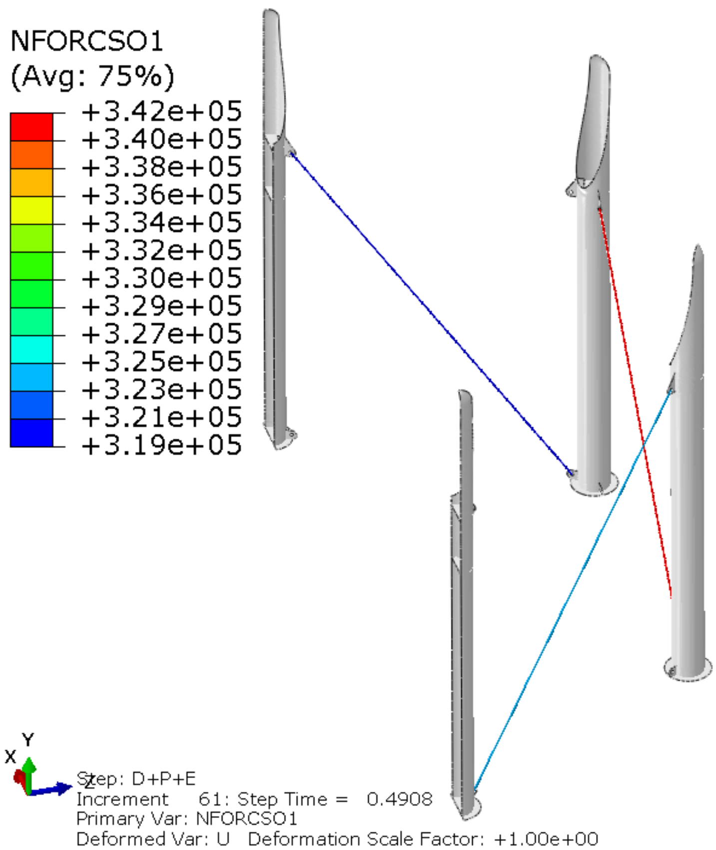


Figure 35: Case 6 Brace Reaction Loads

7.7. Case 7: Current Assessment Reduced Fill

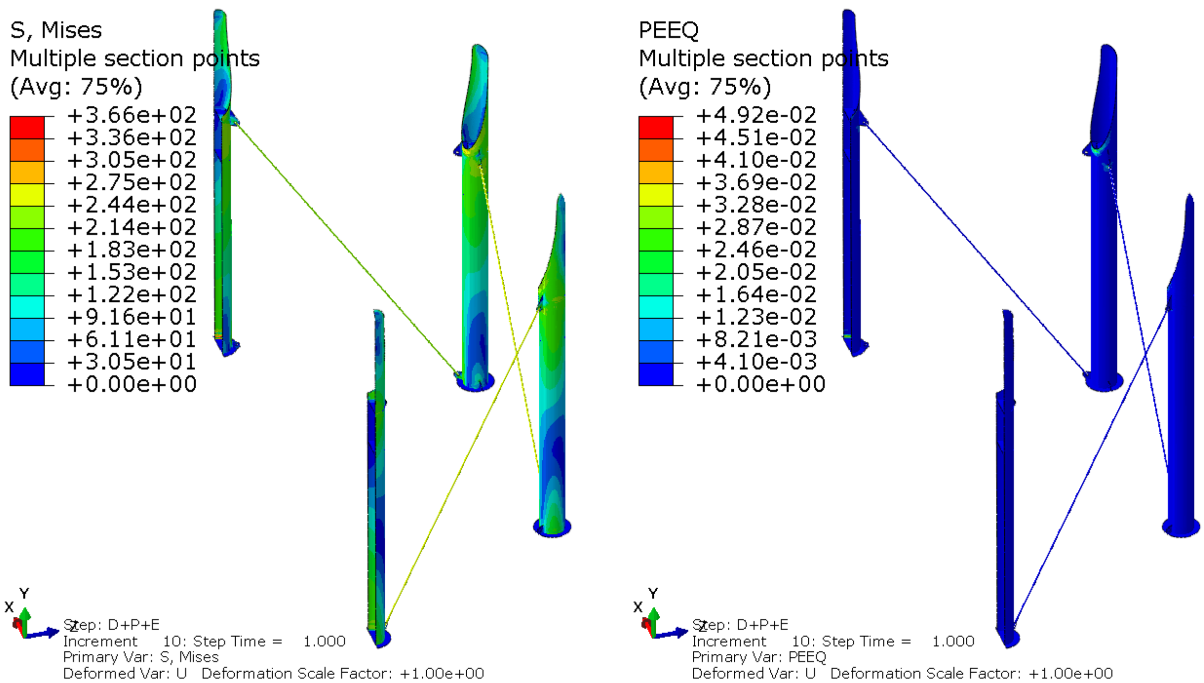


Figure 36: Case 7 Equivalent Stress and Plastic Strain

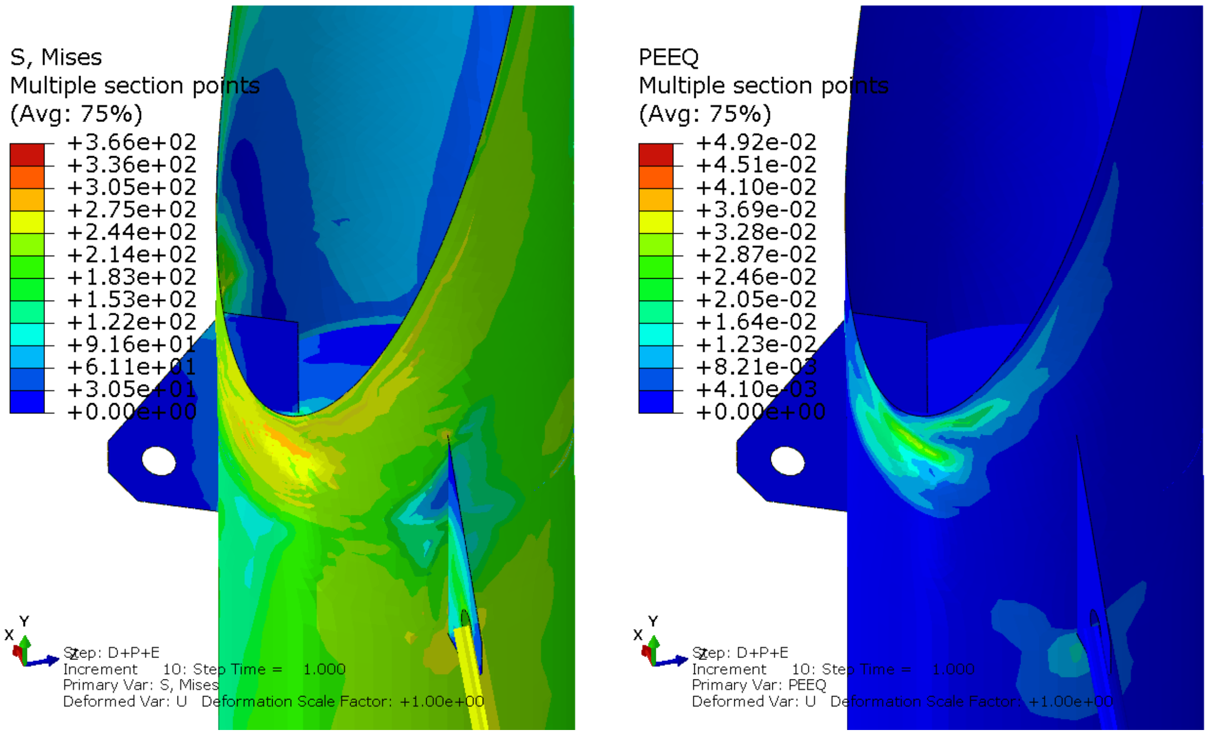


Figure 37: Case 7 Equivalent Stress and Plastic Strain Closeup

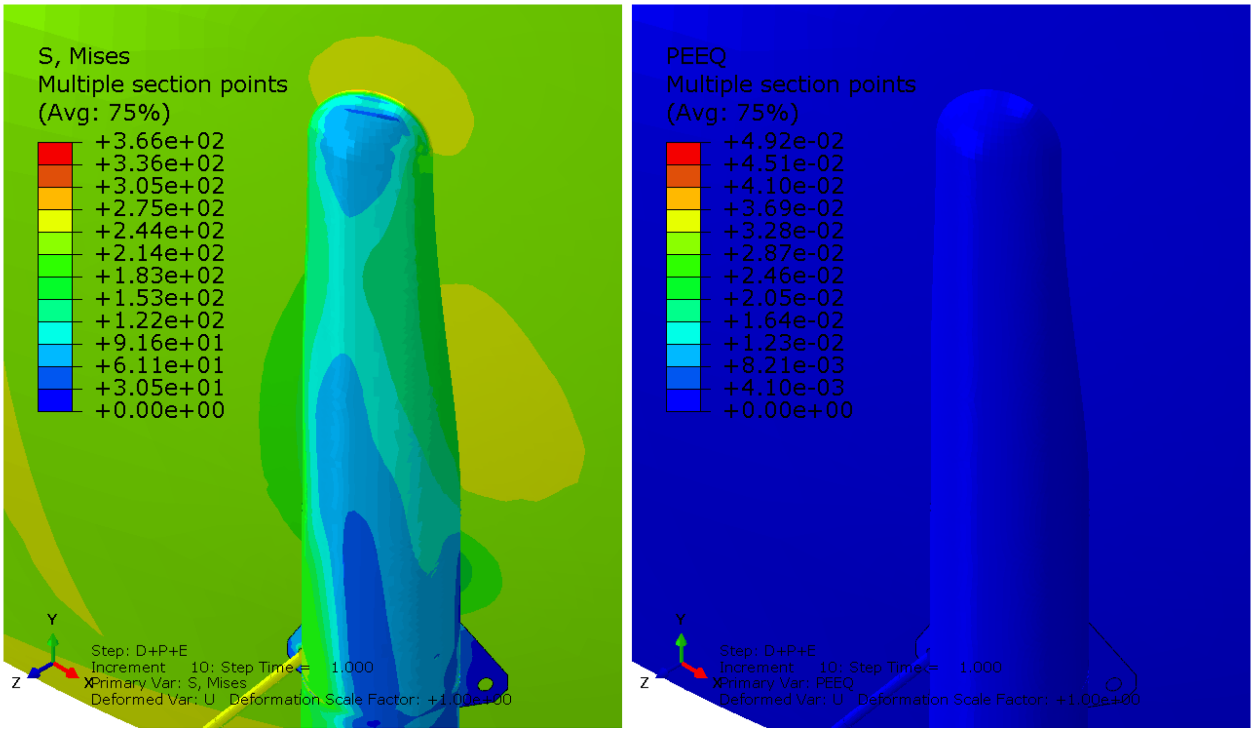


Figure 38: Case 7 Sphere to Column Weld Region

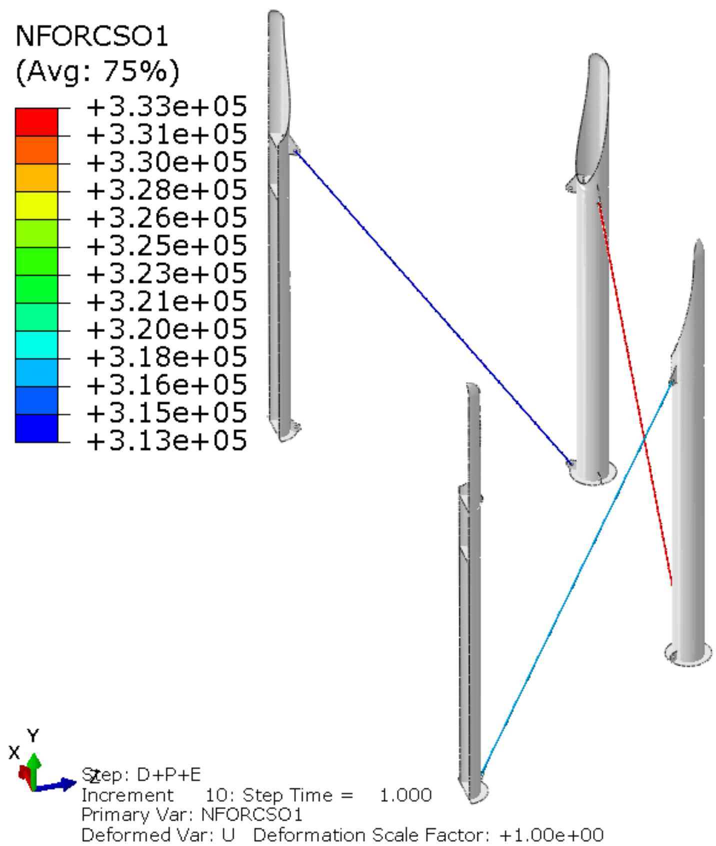


Figure 39: Case 7 Brace Reaction Loads

8. CONCLUSIONS AND RECOMMENDATIONS

The current design of the sphere support structures do not meet code criteria for the design conditions per ASME VIII Div2 Part5 Design By Analysis. Failing load cases 1 (global check from weight and pressure), 5 (global check for hydrotest conditions), and 6 (global check with new seismic loads). The support structure does pass the applied loads if the fill capacity is reduced from 85% to 27%. We recommend to modify the support structure as outlined in the reports in order to meet ASME VIII Div2 design by analysis criteria. It should be noted that passing the code criteria for seismic loading seeks to ensure that a collapse does not occur while significant yielding can take place. It is further recommended to perform a Fitness For Service (FFS) assessment per ASME FFS-1 / API 579 considering the historical loading of the tanks.

Increasing the number of bolts wouldn't be necessesary as the existing anchor bolts and bars provide a sufficient column base strenght to withstand the maximum lateral load at column bases. See Appendix 11.1

Respectfully submitted,	Approved,
Yang Gao, Ph.D. Senior Advisor Specialty Engineering Group	Erik Howard, PE Senior Consultant KnightHawk Engineering, Inc. TX Registration – 101110 TX Firm - 1720

9. Appendix A: Sphere and Component Characteristics

UBC Seismic Zone: 2B Per client specification — $Z := 0.2$ Per UBC 97 Table 16-I

Seismic Importance Factor: $I_e := 1.25$ — Per UBC 97 Table16-K

$s_g := 0.55$ — Specific gravity of propane

$T := 0.316\ s$ — Approximate natural frequency of structure

$T_L := 6\ s$ — Long period for location per Figure 22-14

$g := 9.8\ \frac{m}{sec^2}$ — Acceleration due to gravity

$h_0 := 7772\ mm$ — Sphere equator height

$E_s := 29400000\ \frac{lbf}{in^2}$ — Young's Modulus of Steel

$\rho_s := 0.28365\ \frac{lbf}{in^3}$ — Density of steel

9.1. Sphere Characteristics

$d_s := 10668\ \mathrm{mm}$	Sphere ID
$t_s := 36.35\ \mathrm{mm}$	Sphere thickness
$t_{sB} := 37.2\ \mathrm{mm}$	Sphere thickness at bottom
$D_s := d_s + 2 \cdot t_s = 10.7407\ \mathrm{m}$	Sphere OD
$\xi_s := 0.85$	Fill Ratio
$W_s := \pi \cdot D_s^{\ 2} \cdot t_s \cdot \rho_s = 1014\ \mathrm{kN}$	Dead Weight of Sphere
$W_I := \frac{4}{3} \cdot \pi \cdot \left(\frac{D_s}{2}\right)^3 \cdot 1000\ \frac{\mathrm{kg}}{\mathrm{m}^3} \cdot s_g \cdot g \cdot \xi_s = 2972\ \mathrm{kN}$	Weight of contents, design and operating
$W_{IT} := \frac{W_I}{s_g \cdot \xi_s} = 6358\ \mathrm{kN}$	Weight of contents hydrotest
$f(x) := x^3 - x^2 \cdot 3 \cdot \left(\frac{D_s}{2\ \mathrm{m}}\right) + 4 \cdot \left(\frac{D_s}{2\ \mathrm{m}}\right)^3 \cdot \left(1 - \xi_s\right)$	Equation for cap height of sphere
$h_s := \left(\frac{d_s}{\mathrm{m}} - \mathrm{roots}\left(f(\_x), \_x, 3\ \mathrm{m}\right)\right) \mathrm{m} = 8.0429\ \mathrm{m}$	Height of contents
$P_s := s_g \cdot 1000\ \frac{\mathrm{kg}}{\mathrm{m}^3} \cdot g \cdot h_s = 6.3\ \mathrm{psi}$	Static head from contents
$P_{sT} := 1000\ \frac{\mathrm{kg}}{\mathrm{m}^3} \cdot g \cdot d_s = 15.2\ \mathrm{psi}$	Static head from hydrotest

9.2. Column Characteristics

$d_c := 508\ \text{mm}$ — Column ID

$t_c := 6.4\ \text{mm}$ — Column thicknes

$D_c := d_c + 2 \cdot t_c = 0.5208\ \text{m}$ — Column OD

$D_{CD} := 12802\ \text{mm}$ — Column Circle diamter

$n_c := 6$ — Number of columns

$$A_c := \frac{\pi}{4} \cdot \left(D_c^{\ 2} - d_c^{\ 2}\right) = 0.0103\ \text{m}^2$$ Area

$$I_c := \frac{\pi \cdot \left(D_c^{\ 4} - d_c^{\ 4}\right)}{64} = 0.0003\ \text{m}^4$$ Area moment of inertia

$$Z_c := \frac{\pi \cdot \left(D_c^{\ 4} - d_c^{\ 4}\right)}{32 \cdot D_c} = 0.0013\ \text{m}^3$$ Section modulus

$$Z_{pc} := \frac{D_c^{\ 3} - d_c^{\ 3}}{6} = 0.0017\ \text{m}^3$$ Plastic section modulus

$$i_c := \sqrt{\frac{I_c}{A_c}} = 0.1819\ \text{m}$$ Radius of giration

$$W_c := \frac{\pi}{4} \cdot \left(D_c^{\ 2} - d_c^{\ 2}\right) \cdot h_0 \cdot \rho_s \cdot n_c \cdot 2 = 74\ \text{kN}$$ Column weight

$$\lambda_c := \frac{D_c}{t_c} = 81.375$$ slenderness ratio

$$\lambda_{chd} := 0.053 \cdot \frac{E_s}{R_{Y283} \cdot Y_{283}} = 34.6267$$ Minimum requirement for high ductility

$$\lambda_{md} := 0.062 \cdot \frac{E_s}{R_{Y283} \cdot Y_{283}} = 40.5067$$ Minimum requirement for medium ductility

```
if λc ≤ λchd                                   = "Not Highly Ductile. Cannot be part of a SCBF"
  "Highly Ductile"
else
  "Not Highly Ductile. Cannot be part of a SCBF"
```

9.3. Bracing Characteristics

$D_b := 1.625\ \mathrm{in}$ — Diameter of Rod

$L_b := 7002\ \mathrm{mm}$ — Length of Brace

$n_b := n_c \cdot 2 = 12$ — Number of Braces

$y_b := \dfrac{D_b}{2} = 21\ \mathrm{mm}$ — Distance to centroid

$A_b := \dfrac{\pi}{4} \cdot D_b^{\ 2} = 1338\ \mathrm{mm}^2$ — Section Area

$I_b := \dfrac{\pi}{4} \cdot \left(\dfrac{D_b}{2}\right)^4 = 1.4247 \cdot 10^{-7}\ \mathrm{m}^4$ — 2nd Moment of Area

$Z_b := \dfrac{I_b}{y_b} = 6.9034 \cdot 10^{-6}\ \mathrm{m}^3$ — Section Modulus

$i_b := \sqrt{\dfrac{I_b}{A_b}} = 0.0103\ \mathrm{m}$ — Radius of gyration

$K_b := 0.699$ — Specified by client

$L_{cb} := K_b \cdot L_b = 4.8944\ \text{m}$ — Critical Length of bracing

Brace must be moderatley ductile per F1.5a (Exception made for Tension only braces)

$$\text{if } \frac{D_b}{Y_b} \le 0.062 \cdot \frac{E_s}{R_{Y36} \cdot Y_{36}} = \text{"IS moderatley ductile"}$$

"IS moderatley ductile"
else
"is NOT moderatley ductile"

$$F_{eb} := \frac{\pi^2 \cdot E_s}{\left(\frac{L_{cb}}{i_b}\right)^2} = 1.3\ \text{ksi}$$

Elastic Buckling Stress (AISC 360-16 E3-4)

$$F_{crb} := \text{if } \frac{Y_{36}}{F_{eb}} \le 2.25 = 1.1\ \text{ksi}$$

Local Buckling Sress (AISC 360-16 E3)

$$Y_{36} \cdot 0.658^{\left(\frac{Y_{36}}{F_{eb}}\right)}$$

Equation E3-2

else

$$0.877 \cdot F_{eb}$$

Equation E3-3

Effective Area of Beam - AISC 360-16 E.7

$A_{ecb} := A_b = 0.0013\ \text{m}^2$

$\phi_c := 0.9$ — LRFD Factor

$P_{ncb} := F_{crb} \cdot A_{ecb} = 10\ \text{kN}$ — Nominal compressive strength of bracing

$\phi P_{ncb} := \phi_c \cdot P_{ncb} = 9\ \text{kN}$ — Design Compressive Strength of bracing

$$\phi\sigma_{cb} := \frac{\phi P_{ncb}}{A_b} = 1.02\ \text{ksi}$$

Design Compressive Stress of bracing

10. Appendix B: Load Calculations

10.1. Seismic Load Development Per UBC 97

Seismic Coefficient Per UBC V.2 97 Table 16-Q

$C_a := 0.24$

Seismic Coefficient Per UBC V.2 97 Table 16-R

$C_v := 0.32\ \mathrm{sec}$

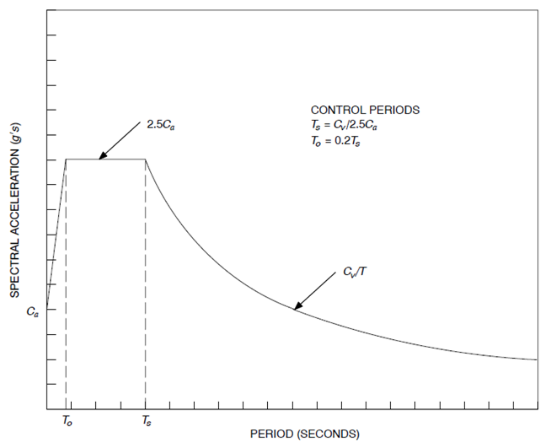


FIGURE 16-3—DESIGN RESPONSE SPECTRA

$R_{UBC} := 2.2$
$\Omega_{0UBC} := 2.0$

Factors for Non-Building Structures Per Table 16-P: Structure Type 1 (Vessels, including tanks and pressurized spheres, on braced or unbraced legs.)

$T_h := 0.316\ \mathrm{s}$ Horizontal Natural Period

Design Base Shear Coefficient

$$CV_{UBC} := \max\left(\left[\min\left(\left[\begin{matrix}\dfrac{C_v \cdot I_e}{R_{UBC} \cdot T_h} \\ \dfrac{2.5 \cdot C_a \cdot I_e}{R_{UBC}}\end{matrix}\right]\right) \\ 0.56 \cdot C_a \cdot I_e\right]\right) = 0.3409$$

Equations (30 - 4,5, and 34 -2)

$r_i := .2$ approximate Ratio of maximum horiztonal load on member / total shear. Based on analysis results

$A_B := 135\ \mathrm{m}^2$

$$\rho_{UBC} := \min\left(\left[\max\left(\left[\begin{matrix}2 - \dfrac{6.1\ \mathrm{m}}{r_i \cdot \sqrt{A_B}} \\ 1\end{matrix}\right]\right) \\ 1.5\right]\right) = 1$$

Reliability / Redundancy Factor

$C_{sUBC} := CV_{UBC} \cdot \rho_{UBC} = 0.3409$ Horizontal Load Multiplier (Multiplied by dead load including contents)

$C_{EvUBC} := 0.5 \cdot C_a \cdot I_e = 0.15$ Vertical Load Multiplier (Multiplied by dead load including contents)

Convective (sloshing) forces are assumed to be negligible based on research done on a similar application.[7]

[7] Fiore, A., Demartino, C., Greco, R. et al. Seismic performance of spherical liquid storage tanks: a case study. Int J Adv Struct Eng 10, 121130 (2018). https://doi.org/10.1007/s40091-018-0185-1

10.2. Wind Load Development Per ASCE 7-16

Current and Hydrotest Wind Load

$D_s := 10704\ \mathrm{mm}$ — Sphere Diameter

$V_w := 80\ \mathrm{mph}$ — Wind speed per Drawing ING40 - 4TK - 125 - HD - 301

$K_d := 1.0$ — Wind directionality factor per 26.6

Exposer : Surface Roughness C per 26.7.2 and Exposure Category C per 26.7.3

$K_{zt} := 1.0$ — Togographic factor per 26.8

$z_g := 2616\ \mathrm{m}$ — Height of location above sea level

$K_e := \mathbf{e}^{-0.000119 \cdot \frac{z_g}{\mathrm{m}}} = 0.7325$ — Ground elevation factor per 26.10

$K_z := 0.98$ — Velocity pressure exposure coefficient per Table 26.10 - 1

$q_z := 0.613\ \frac{\mathrm{kg}}{\mathrm{m}^3} \cdot K_z \cdot K_e \cdot K_{zt} \cdot K_d \cdot V_w^{\,2} = 562.8\ \mathrm{Pa}$ — Velocity pressure per equation 26.10 - 1.si

$G := 0.85$ — Gust effect factor per 26.11.1

Enclosure Classification: "Closed" per 26.12

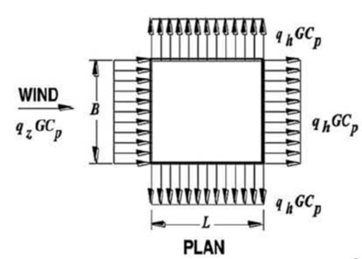
$GC_{pia} := 0.18$ — Internal pressure coefficient per Table 26.13 - 1

$GC_{pib} := -0.18$

$C_{pwindward} := 0.8$ — Wall pressure coefficients per Figure 27.3 - 1

$C_{pleeward} := -0.3$

$q := q_z = 562.8\ \mathrm{Pa}$



Design wind pressure per 27.3.1

$$p := \max\left(\begin{bmatrix} q \cdot G \cdot C_{pwindward} - q \cdot GC_{pia} \\ q \cdot G \cdot C_{pwindward} - q \cdot GC_{pib} \\ q \cdot G \cdot C_{pleeward} - q \cdot GC_{pia} \\ q \cdot G \cdot C_{pleeward} - q \cdot GC_{pib} \end{bmatrix}\right) = 484\ \mathrm{Pa}$$

The pressure is horizontally applied to windward side of sphere.

$p = 10.1\ \mathrm{psf}$ $\qquad p = 0.0702\ \mathrm{psi}$

$F_w := \frac{\pi}{4} \cdot D_s^{\,2} \cdot p = 43.6\ \mathrm{kN}$ — Current effective lateral force on sphere

$V_{w1974} := 100\ \text{mph}$ Original wind speed per drawing Y-UU047B Dwg1

$q_{z1974} := 0.613\ \frac{\text{kg}}{\text{m}^3} \cdot K_z \cdot K_e \cdot K_{zt} \cdot K_d \cdot V_{w1974}^{\ 2} = 879.4\ \text{Pa}$ 1974 Velocity Pressure

$q_{1974} := q_{z1974}$

$$p_{1974} := \max\left(\begin{bmatrix} q_{1974} \cdot G \cdot C_{pwindward} - q_{1974} \cdot GC_{pia} \\ q_{1974} \cdot G \cdot C_{pwindward} - q_{1974} \cdot GC_{pib} \\ q_{1974} \cdot G \cdot C_{pleeward} - q_{1974} \cdot GC_{pia} \\ q_{1974} \cdot G \cdot C_{pleeward} - q_{1974} \cdot GC_{pib} \end{bmatrix}\right) = 756.3\ \text{Pa}$$

$p_{1974} = 15.8\ \text{psf}$ $p_{1974} = 0.1097\ \text{psi}$

$F_{w1974} := \frac{\pi}{4} \cdot D_s^{\ 2} \cdot p_{1974} = 68.1\ \text{kN}$ Original effective lateral force on sphere

10.3. Hydrotest Load Development Per ASME VII Div1

$$P_{mawp} := \frac{2 \cdot S_{70} \cdot E_{je} \cdot \left(t_{sB} - ca\right)}{\frac{D_s}{2} - 0.8 \cdot \left(t_{sB} - ca\right)} - P_s = 260.4\ \text{psi}$$

$$P_t := \left(\frac{2 \cdot S_{70} \cdot E_{je} \cdot \left(t_{sB} - ca\right)}{\frac{D_s}{2} - 0.8 \cdot \left(t_{sB} - ca\right)} - P_{sT}\right) \cdot 1.3 = 327\ \text{psi}$$

Calculated Hydrostatic Test Pressure per UG-99, 3 - 2, and UG-21

11. Appendix C: Strength Checks

11.1. Column Base Support (CBS) Shear Calculation

$P_{cbase} := \frac{287}{0.49}\ \text{kN} = 585.7\ \text{kN}$ — Maximum lateral load at column base assuming extapolated load from failure point

CBS Anchor Bolts

$F_{\tau a} := 0.45 \cdot T_{36} = 26100\ \text{psi}$ — Shear strength per Table J3.2

$D_a := 1.5\ \text{in}$ — Diameter of Anchor Bolt

$A_a := \frac{\pi \cdot D_a^2}{4} = 0.0011\ \text{m}^2$ — Area anchor bolt

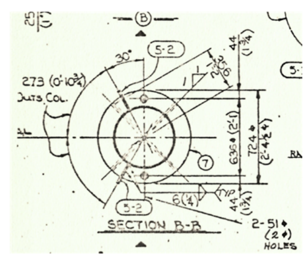
$n_a := 2$ — Number of anchor bolts per column

$\phi_{va} := 0.75$

$R_{va} := A_a \cdot F_{\tau a} = 205\ \text{kN}$ — Nominal shear strength of anchor bolt (Equation J3-1)

$\phi R_{va} := \phi_{va} \cdot R_{va} = 154\ \text{kN}$ — Design shear strength of anchor bolt

$V_a := \phi R_{va} \cdot n_a = 308\ \text{kN}$ — Shear strength of anchor bolts per column



CBS Bar

$F_{\tau bar} := 0.45 \cdot T_{36} = 26100\ \text{psi}$ — Shear strength per Table J3.2

$t_{bar} := 13.5\ \text{mm}$ — Bar thickness for the CBS

$L_{bar} := D_c = 0.5208\ \text{m}$ — Bar Length

$A_{bar} := t_{bar} \cdot L_{bar} = 0.007\ \text{m}^2$ — Shear Area of the Bar

$\phi_{vbar} := 0.75$

$R_{vbar} := A_{bar} \cdot F_{\tau bar} = 1265\ \text{kN}$ — Nominal shear strength of the bar (Equation J3-1)

$\phi R_{vbar} := \phi_{vbar} \cdot R_{vbar} = 949\ \text{kN}$ — Design shear strength the bar

CBS Total

$V_{cbs} := \phi R_{vbar} + V_a = 1257\ \text{kN}$ — Shear strength of the CBS and Anchor Bolts per column

```
if  V_cbs < P_cbase        = "CBS sufficient"
  "CBS NOT Sufficient"
else
  "CBS sufficient"
```

11.2. Clevis Pin Shear Strength at Brace Connection

$F_{\tau a} := 0.45 \cdot T_{36} = 26100\ \text{psi}$ — Shear strength per Table J3.2

$D_a := 2.0\ \text{in}$ — Diameter of Clevis Pin

$A_a := \dfrac{\pi \cdot D_a^{\,2}}{4} = 0.002\ \text{m}^2$ — Area of the clevis pin

$\phi_{va} := 0.75$

$R_{va} := A_a \cdot F_{\tau a} = 365\ \text{kN}$ — Nominal shear strength of the clevis (Equation J3-1)

$\phi R_{va} := \phi_{va} \cdot R_{va} = 274\ \text{kN}$ — Design shear strength of the clevis pin

$n_{planes} := 2$

$V_a := \phi R_{va} \cdot n_{planes} = 547\ \text{kN}$ — Shear strength of the clevis pin

$P_a := 325\ \text{kN}$ — Load on the clevis pin (load case 3)

```
if V_a > P_a                       = "Clevis pin sufficient"
  "Clevis pin sufficient"
else
  "Clevis pin NOT sufficient"
```