REPORT OF STRUCTURAL EVALUATION

5m Diameter Observatory Dome Structure
Montgomery Hill Observatory, Evergreen Valley College
San Jose, California

prepared for

Observa-Dome Laboratories, Inc.
Jackson, Mississippi

September 6, 2002

ADVANCED ENGINEERING RESOURCES, INC. Madison, Mississippi

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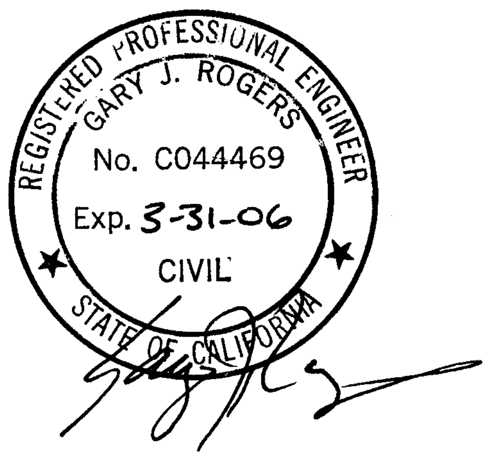
SECTION I

CERTIFICATION

CERTIFICATION

This report and the supporting analysis have been prepared by or under the direct supervision of Gary J. Rogers, P.E., California Professional Engineering License No. 44469 whose seal and signature appear below.



SECTION II

SUMMARY

EXECUTIVE SUMMARY

At the request of Observa-Dome Laboratories, Inc. of Jackson, Mississippi, an evaluation of the observatory dome structure proposed Montgomery Hill Observatory at Evergreen Valley College in San Jose, California has been conducted. As described in the detailed report which follows this Executive Summary, a finite element model of the structure was prepared and subjected to various extreme environmental loading conditions specified for the facility. The stresses in the various components making up the structure have been computed in this manner and compared to allowable stresses as prescribed by nationally recognized authoritative standards for aluminum and steel structures. It is the conclusion of this evaluation that the design of the structure is adequate to withstand the expected loading conditions.

SECTION III

REPORT

INTRODUCTION

This report presents the findings of a structural evaluation of the 5 meter diameter observatory dome structure to be constructed by Observa-Dome Laboratories, Inc. and located at Montgomery Hill Observatory at Evergreen Valley College located at 3095 Yerba Buena Road in San Jose, California. This evaluation has been undertaken at the request of Mr. Randy Clark of Observa-Dome Laboratories, Inc., Jackson, Mississippi.

STRUCTURE DESCRIPTION

The dome structure has a nominal outside diameter of 5 meters (196.5 inches) and an aperture approximately 55" wide extending from the bottom to about 27.5" beyond the zenith with a pair of horizontally actuated shutters. The dome is made up of aluminum 45° radial sections assembled on site and a structural frame consisting of a steel tension ring at the base and parallel steel arches on each side of the aperture. The 45° radial sections are a shop assembly of five 9° radial sections (gores). The surface of the dome is made up of 0.063 inch thick aluminum sheet joined together by bending the edges up 90° and welding adjacent sections along the top edge. A 1/4 inch by 3 9/16 inch aluminum plate is fastened to the edges of the shop assemblies to form a flange for bolted assembly on site. The dome includes a 12" high cylindrical section below the springline. The aperture is reinforced on three sides by a 0.1875 inch by 10 inch aluminum plate (slitframe) standing vertically relative to the curvature of the dome. The shutters are made up of 0.08 inch thick aluminum sheet attached to 0.1875 inch thick aluminum sidewalls. The shutter halves are attached to the dome structure by 2"x2"x3/16" rectangular tube steel runners which engage track members attached to the dome structural frame. Two "I" shaped track members, one located at the bottom of the aperture and the other positioned near the zenith, permit the horizontal operation of the shutter. The dome is supported on 8 spring loaded vertical casters and prevented from uplift and horizontal translation by 8 horizontal caster/hold-down assemblies. The vertical and horizontal casters permit a full 360° rotation of the entire structure. Figure 1 and Figure 2 display a rendering of the complete dome in both the closed and open configurations.

STRUCTURE LOADINGS

The loadings to which the structure will be subjected are its own weight, the force of the wind, the weight of snow accumulation, seismic acceleration and operational loads. The shape and configuration of the dome structure render it inaccessible to personnel; therefore, live loads of this nature have not been applied in this evaluation. Experience with these observatory dome structures has shown that when proportioned adequately for all other environmental loads, the structures perform adequately under the wide variety of operational loads that may be expected to occur. Because of such experience and due to the difficulty in application of such loads to the dome's peculiar shape, operational loads have not been specifically considered in this study.

Wind Loading

The ability to open the shutter thereby creating a single, large opening leads to the necessity to consider two wind loading scenarios. It is reasonable to expect the shutter to be closed during the extreme wind event ordinarily used for structure design, however, it is clearly possible for wind load to occur during normal operation with the shutter open creating a load which is not insignificant and must be considered.

This dome has been designed for wind speeds up to 120 mph. Reference 1 provides procedures for the determination of forces on structures due to wind speeds expressed in 3 second gusts. These procedures have been used to determine the magnitude of forces exerted on the dome by the specified wind velocity. The external pressures resulting from this wind velocity can be exerted on the closed dome in any horizontal direction and would be coupled with internal pressures associated with an enclosed structure. It is reasonable to assume that a wind gust in advance of a severe storm might occur while the shutter is open and with insufficient warning to complete a closing operation. A 3 second gust wind velocity of 60 miles per hour is considered appropriate for such a scenario. In this case, more severe internal pressures are created on the partially enclosed structure which may produce higher component stresses than would be created by the higher wind velocity with the shutter closed.

Clearly, the wind can come from any direction relative to the orientation of the dome. The symmetry of the structure allows the variety of directions to be reduced to three for the purpose of structural evaluation. Wind directions into the face of the shutter as well as 90 degrees and 180 degrees to this direction are considered adequate to envelope the maximum component stresses which would be produced by wind loads.

Snow Loading

This dome is designed to function after exposure to a ground snow load of up to 30 pounds per square foot. Reference 1 provides procedures for determining the magnitude and distribution of snow loads on structures and roofs based on ground snow loads.

While it is unlikely that the observatory dome will be open and in use during a snowstorm, it is possible that it might be opened and used after an accumulation of snow. This possibility has been considered in this study.

Seismic Loading

The dome is designed for operation in Seismic Zone 4 as defined by the California Building Code (CBC, Reference 3). To comply with this requirement, a dynamic analysis has been performed utilizing a response spectrum constructed in accordance with Figure 16-3 in Reference 3. Lacking site specific information, Soil Profile Type S_D (Reference 3, Table 16-J), Seismic Source Type A (Reference 3, Table 16-U) and a distance to a known seismic source less than 5 kilometers was used in the construction of the design response spectrum. This response spectrum is reproduced in this report as Figure 3. The possibility that the design seismic event might occur while the shutters are in either of the open or closed positions has been considered in this evaluation.

The manner of supporting the dome on vertical and horizontal casters introduces a degree of isolation from ground acceleration forces. While this isolation has not been considered due to the complexity of quantifying it, isolation would have the effect of reducing the stresses experienced by the structure, therefore this approach is considered conservative.

Load Combinations

The structure was evaluated for the combined effects of the above described loads using the basic load combinations prescribed in article 1612.3.1 of Reference 2 as follows:

1. D
2. D + S
3. D + W
4. D + E/1.4
5. 0.9D ± E/1.4
6. D + 0.75[S + W]

Where: D = Dead load
S = Snow load
W = Wind load (3 horizontal directions)
E = Seismic load (2 horizontal directions)

EVALUATION APPROACH

Analysis Procedure

In order to ascertain the expected maximum stresses in the various components of the dome structure, a finite element model was developed. In the model, the aluminum sheet and plate materials were modeled as plate elements and the dome tension ring, arches, shutter tracks and track tubes were modeled as line elements or members. The stiffness of plate elements is represented only by their thickness while conventional strengths of materials techniques have been employed to compute the stiffness of the members. Vertical only supports were provided at the approximate locations of the vertical casters and horizontal casters were modeled as supports capable of resisting only horizontal forces. The supports were arranged so as to produce the maximum stresses in materials. It was necessary to create two separate models, one representing the dome in the closed configuration and another representing the dome with the shutter opened. The structure finite element model has been developed using STAADPro® computer software as published by Research Engineers International. of Yorba Linda, California. A graphical representation of each finite element model produced directly by the computer program is shown on Figures 4 and 5 respectively.

The analysis software has the capability of computing a wide range of stresses and forces within elements and members and also provides tools for the extraction and sorting of the great number of stresses calculated by the program. This feature was used to extract the critical values identified and discussed herein. In addition, color stress contours can be developed and printed out for review. All of these features were used extensively to review the results of the analysis.

Allowable Stresses

All aluminum components of the dome structure are alloy 3003-H14. Allowable stresses for the aluminum materials have been calculated in accordance with Reference 4. The allowable tensile stress for alloy 3003-H14 is 10,500 pounds per square inch (psi). For material within 1 inch of a weld, the allowable tensile stress is reduced to 4,200 psi. Allowable compressive stress varies significantly for thin elements depending on whether the element is stiffened or unstiffened and the width of unstiffened elements in relation to their thickness. Because the mode of failure in compression is most often buckling, allowable compressive stresses are generally considerably lower than allowable tensile stress. The critical allowable compressive stresses are given in Tables 1 to 3. Table 1 shows the allowable compressive stresses in all of the aluminum elements except the sheet elements that make up the surface of the dome. Tables 2 and 3 show the allowable compressive stresses in the sheet elements that make up the surface of the dome which increase as the distance above the springline increases. This is due to the configuration of the shop seams and field splices which stiffen the skin and become closer together above the springline thus reducing the width-to-thickness ratio (b/t) which increases the allowable stress.

All steel components of the dome structure are mild steel with a yield stress of 36,000 pounds per square inch. Allowable stresses for the steel materials have been calculated in accordance with Reference 5. The critical allowable stresses are given in Table 4.

RESULTS

The results of the stress analysis have been reviewed in detail and the maximum stresses have been extracted and are tabulated in Tables 1 to 4. The distribution of some of these stresses is shown through the use of color coded stress contours on Figures 6 through 12.

An initial review of the element stresses was conducted considering the absolute value of the stress level without consideration of the direction of stress. Elements whose absolute stress exceeded the allowable for compression were then examined to determine the direction of the maximum absolute stress, that is, whether the stress was a tensile stress or a compressive stress. In this manner, tensile stresses could be considered separately from all other stresses and compared to the higher allowable stress of 4,200 psi.

In Tables 1, 2 and 3 it is demonstrated that all aluminum element stresses reported are less than the applicable allowable stress. Two sets of elements, the dome elements below the springline and the dome sheet elements, contained elements whose absolute value applied stress exceeded the compression allowable stress requiring determination of the direction of stress. These elements were examined more closely and the determination was made that the maximum stresses were tensile stresses for which the higher allowable stress is applicable.

Table 4 demonstrates that all steel component stresses produced by the loads are within the allowable stresses dictated by Reference 5.

TABLE 1 - Aluminum Element Stresses

ALUMINUM ELEMENT STRESSES Allowables per Specification for Aluminum Structures						
	Stresses (psi)					
	WIND		SNOW		SEISMIC	
Element Description	Allowable	Applied	Allowable	Applied	Allowable	Applied
Outer membrane - cylindrical elements below springline	1521	1122*	1141	520	1521	266
Field splice plates	3233	536	2425	298	3233	155
Shop splice standing seams	5600	3332	4200	453	5600	304
Slit frame	6529	2429	4897	471	6529	253
Shutter outer membrane elements	1085	736	814	324	1085	123
Shutter sidewalls	6915	2058	5186	870	6915	908

* Excludes element numbers 1 and 38 singled out for consideration of direction of stress, see Table 3

TABLE 2 – Dome Sheet Stresses

OUTER MEMBRANE STRESSES Allowables per Specification for Aluminum Structures Alloy: 3003-H14 Specification No. 9									
THICKNESS =		0.063 inches		STRESSES (psi)					
Elevation Above Springline (in)	Diameter (in)	"b" (in)	"b/t"	WIND		SNOW		SEISMIC	
				Allowable	Applied	Allowable	Applied	Allowable	Applied
0.00	196.90	15.46	245	1521	928*	1141	306	1521	147
15.40	194.48	15.27	242	1540	606*	1155	186	1540	87
30.42	187.26	14.71	233	1599	1162	1199	115	1599	67
44.70	175.44	13.78	219	1707	451	1280	102	1707	34
57.87	159.30	12.51	199	1880	455	1410	123	1880	43
69.61	139.23	10.94	174	2151	694	1613	160	2151	70
79.65	115.73	9.09	144	2588	732	1941	343	2588	156
87.72	89.39	7.02	111	3350	2151	2513	792	3350	369
93.63	60.85	4.78	76	4922	952	3691	265	4922	127

* Excludes element numbers 57, 58, 95, and 96 singled out for consideration of direction of stress, see Table 3

TABLE 3 – Highly Stressed Elements

HIGHLY STRESSED ELEMENTS Allowables per Specification for Aluminum Structures						
Element Number	Stresses (psi)					Stress Ratio
	Maximum Absolute	Axial Tension		Plate Bending		
		Max. Applied	Allowable*	Net Applied	Allowable*	
1	2475	1420	5600	1055	7333	0.40
38	2440	1385	5600	1055	7333	0.39
57	2226	1223	5600	1003	7333	0.36
58	2252	1223	5600	1029	7333	0.36
95	1617	334	5600	1283	7333	0.23
96	1631	336	5600	1295	7333	0.24

* Applied stresses are derived from load case 3 which includes wind therefore allowables are increased by one third

TABLE 4 - Steel Component Stresses

STEEL COMPONENT STRESSES Allowables per American Institute for Steel Construction			
Element Description	Allowable Stress (psi)		Maximum Stress Ratio
	Strong axis	Weak axis	
Dome tension ring	21522	21522	0.21
Lower shutter track	11612	21600	0.32
Upper shutter track	18404	21600	0.71
Shutter track tubes	21600	21600	0.66
Front Arch	9288	9288	0.93
Back Arch	24000	21600	0.10

CONCLUSIONS

Stress levels in all components produced by all of the loading configurations considered fall below the appropriate allowable stress and therefore all portions of the dome are considered acceptable and meeting the requirements of Reference 1.

REFERENCES

1. Minimum Design Loads for Buildings and Other Structures; ASCE 7-98; American Society of Civil Engineers

2. Uniform Building Code; 1997 Edition; International Conference of Building Officials

3. California Building Code; 1998 Edition; California Building Standards Commission

4. Specifications for Aluminum Structures; 5th Edition; December, 1996; The Aluminum Association, Inc.

5. Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design; June 1, 1989, American Institute of Steel Construction

SECTION IV

DOME RENDERINGS AND FINITE ELEMENT MODELS

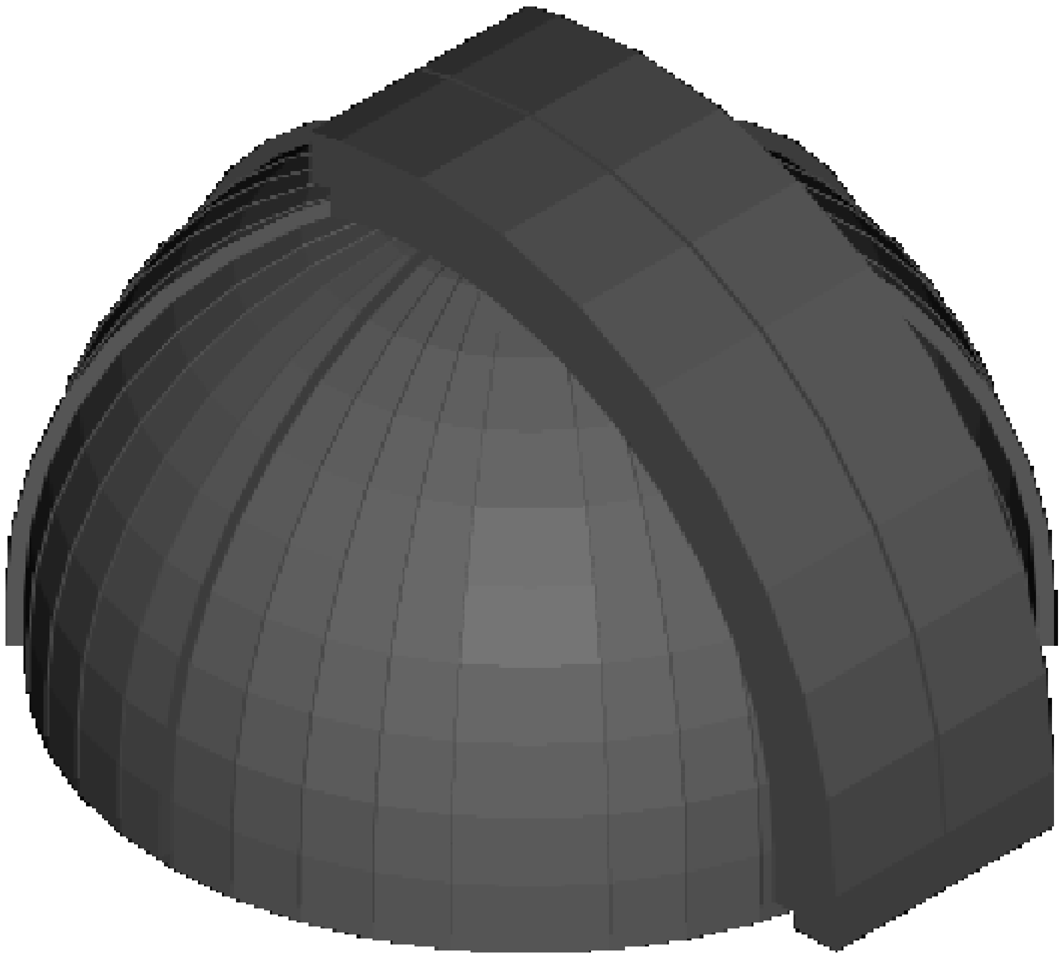


FIGURE 1 – Rendering of Closed Observatory Dome

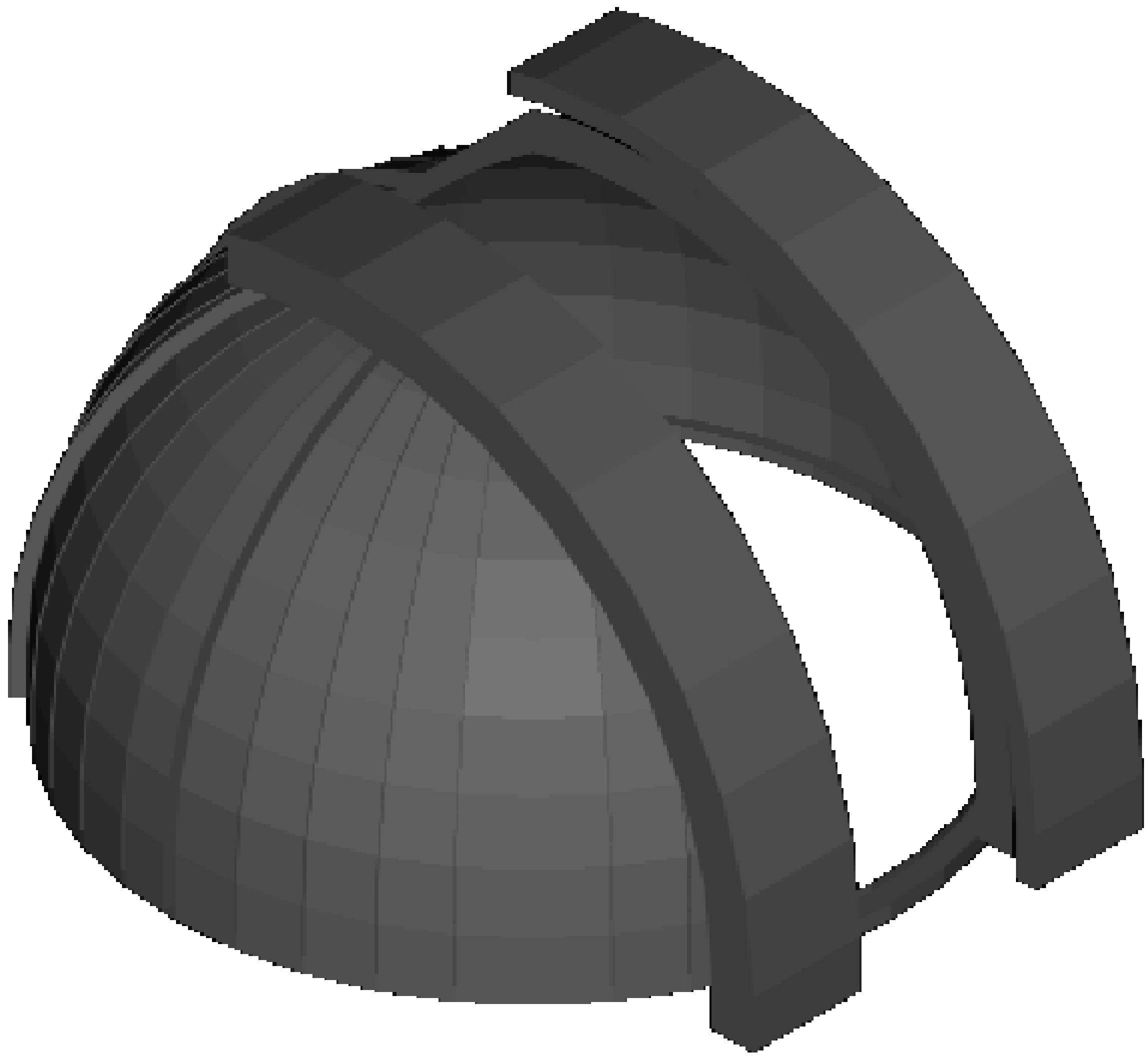


FIGURE 2 – Rendering of Open Observatory Dome

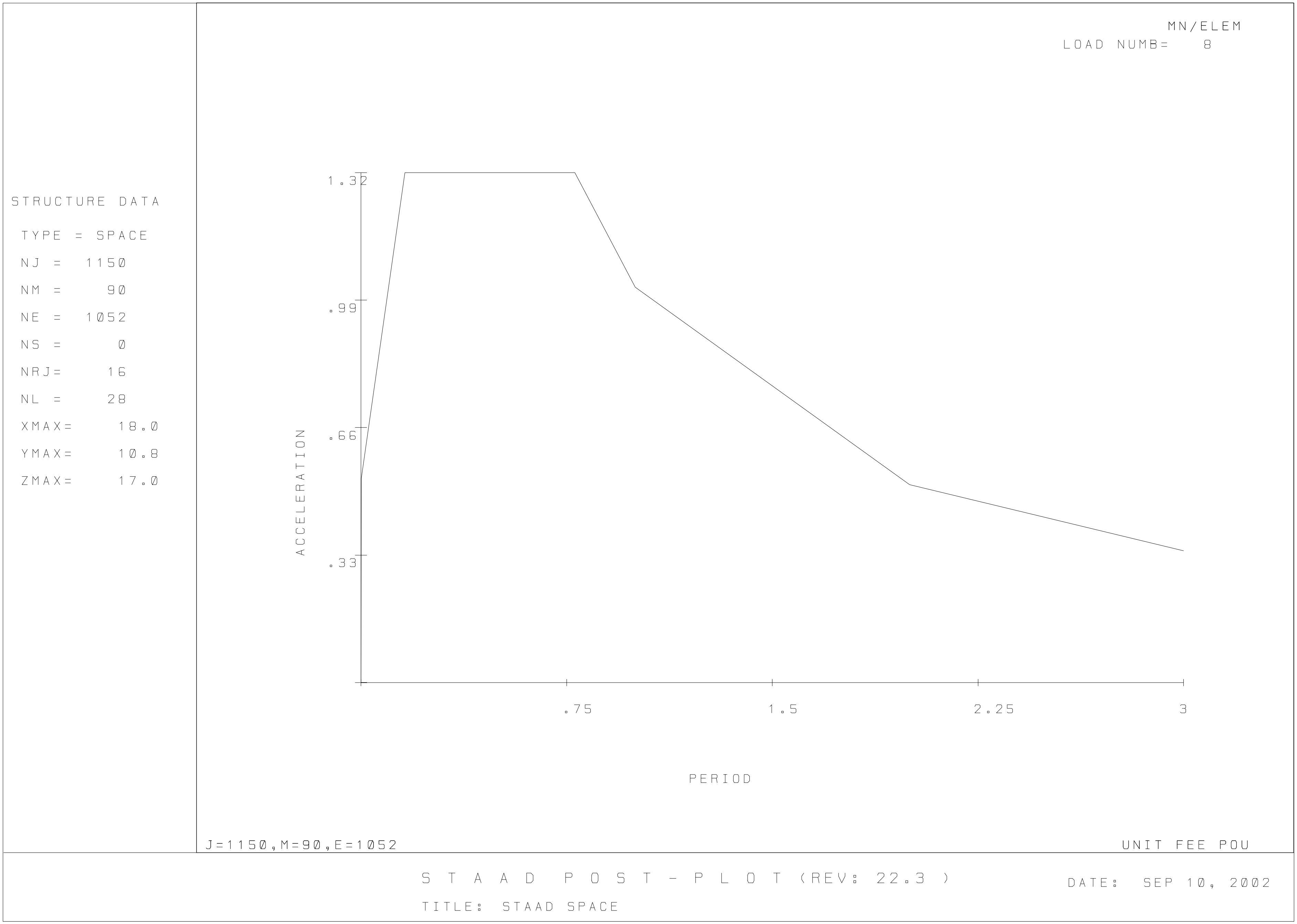


FIGURE 3 – CBC Response Spectrum

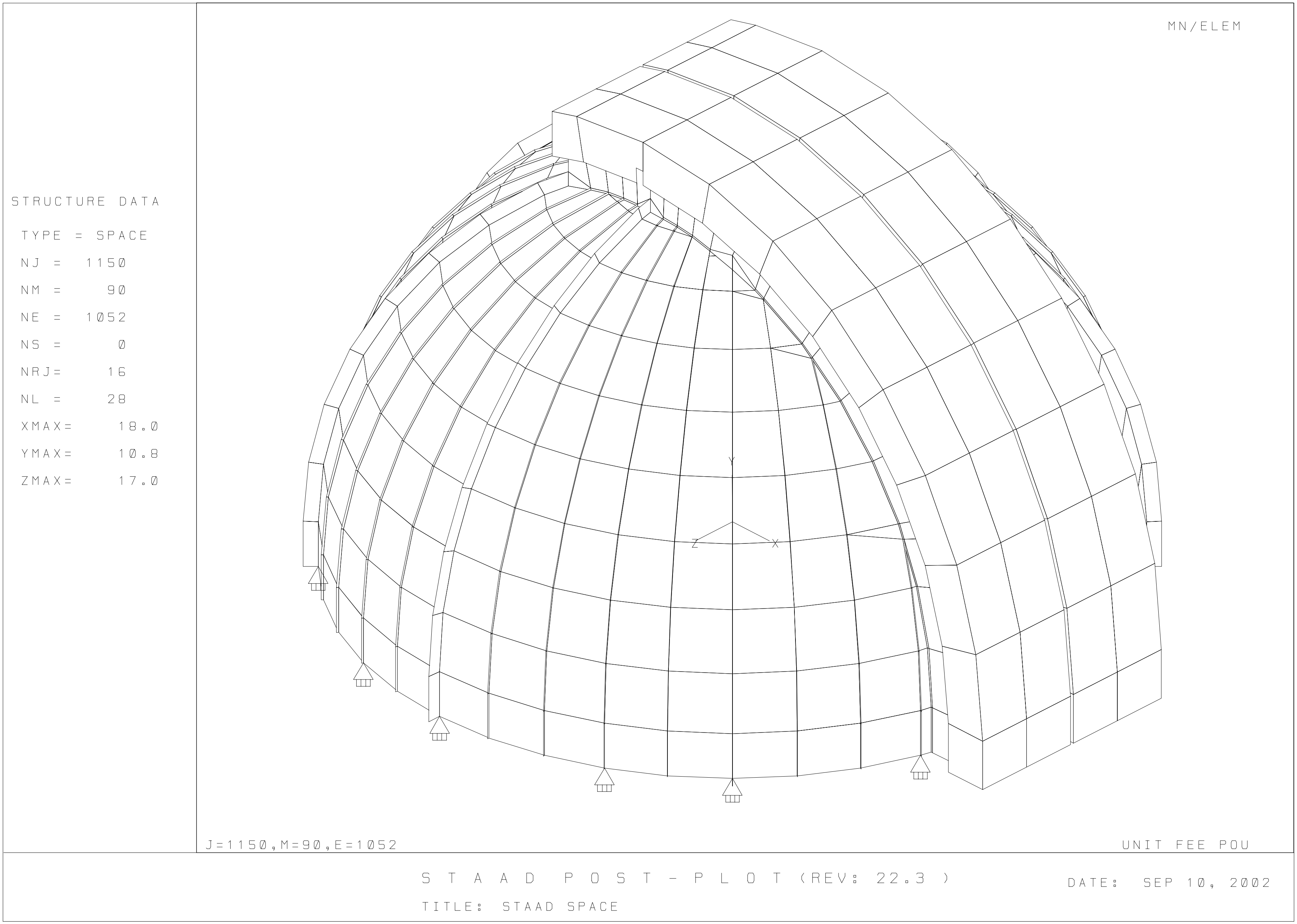


FIGURE 4 - Finite Element Model of Closed Observatory Dome

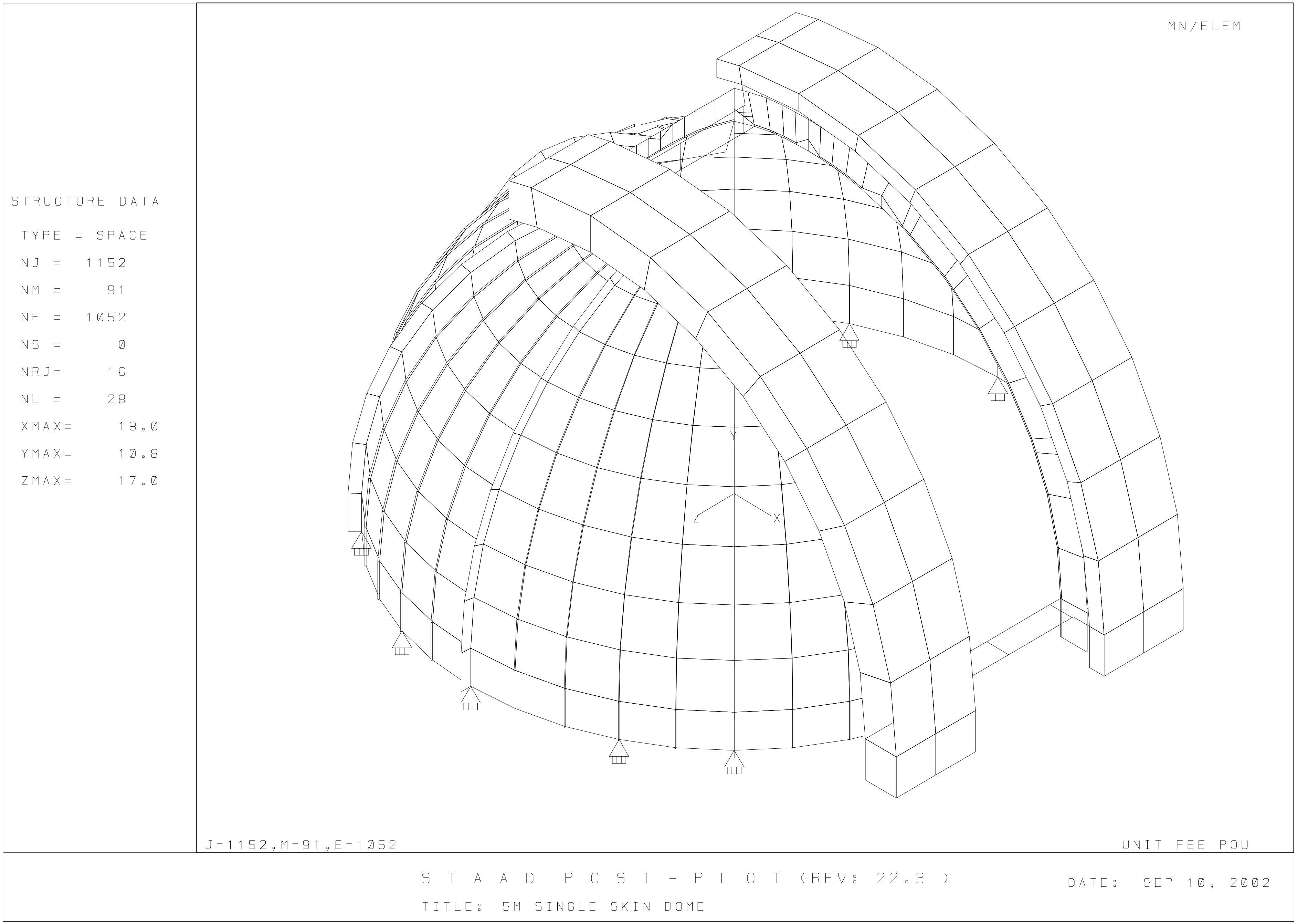


FIGURE 5 - Finite Element Model of Open Observatory Dome

<u>SECTION V</u>

STRESS CONTOUR GRAPHICS

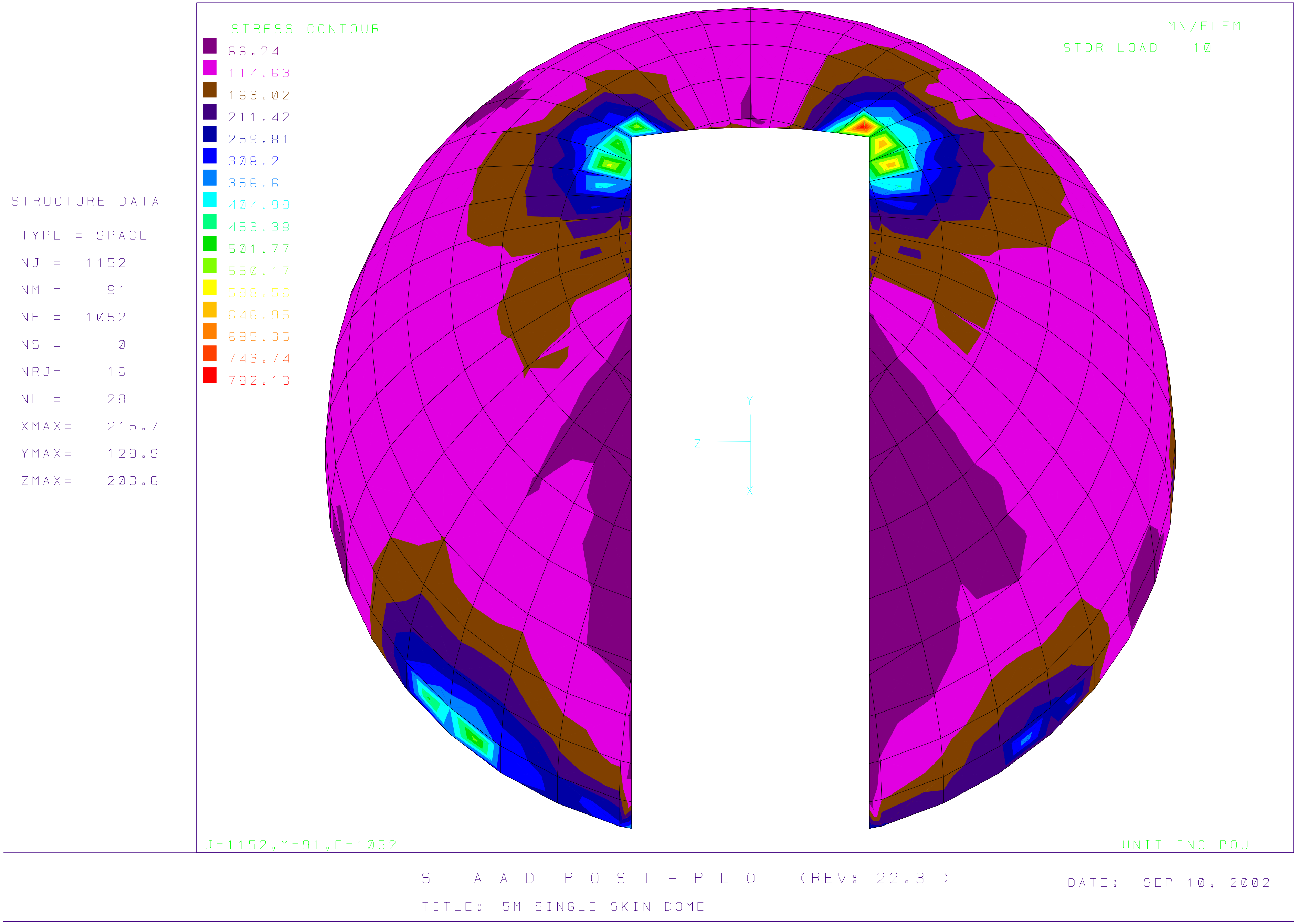


FIGURE 7a – Outside Dome Sheet Stress Contours

(Load Combination 2)

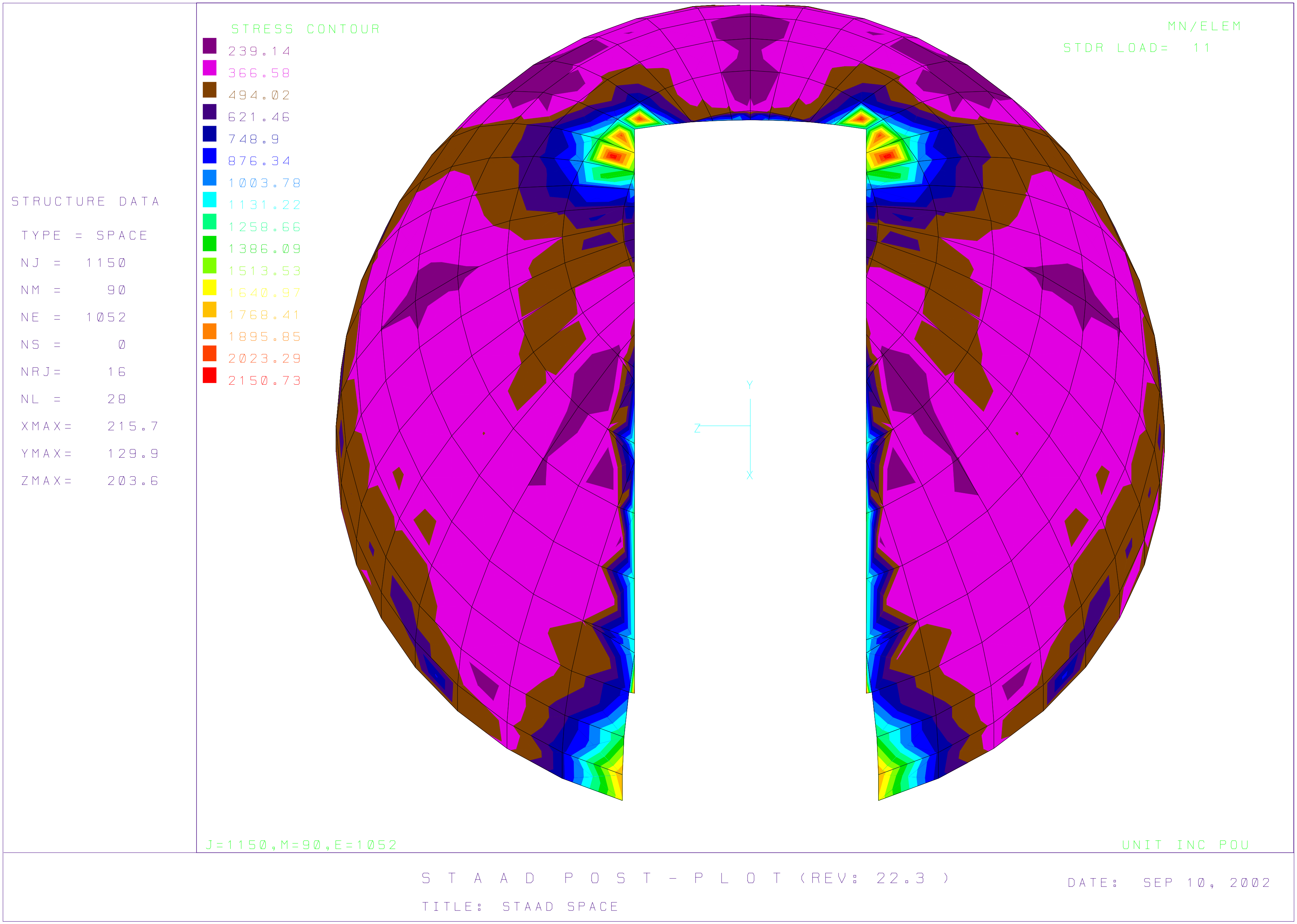


FIGURE 7b – Outside Dome Sheet Stress Contours
(Load Combination 3, Wind in +X Direction)

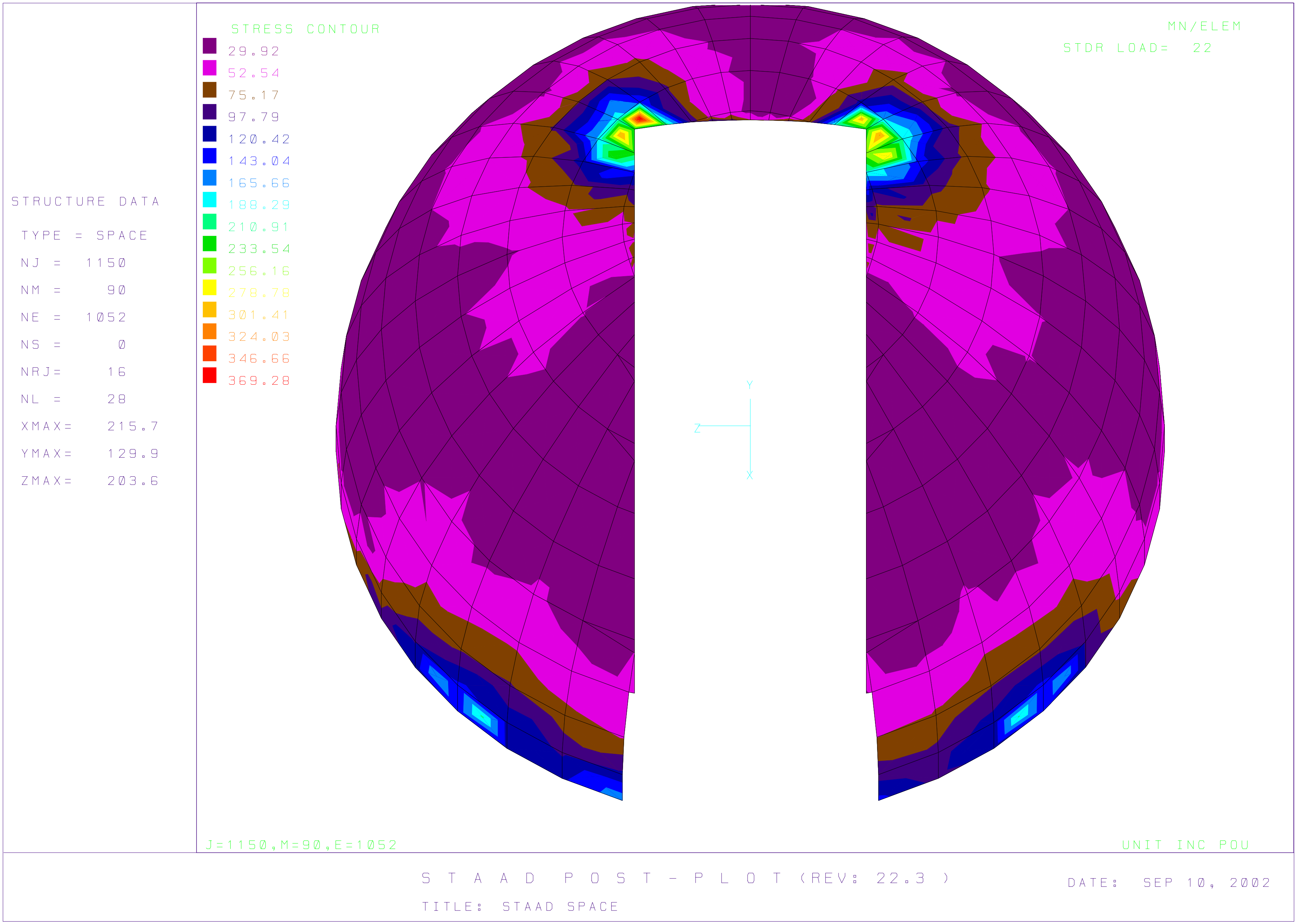


FIGURE 7c – Outside Dome Sheet Stress Contours
(Load Combination 5, Seismic in Z Direction)

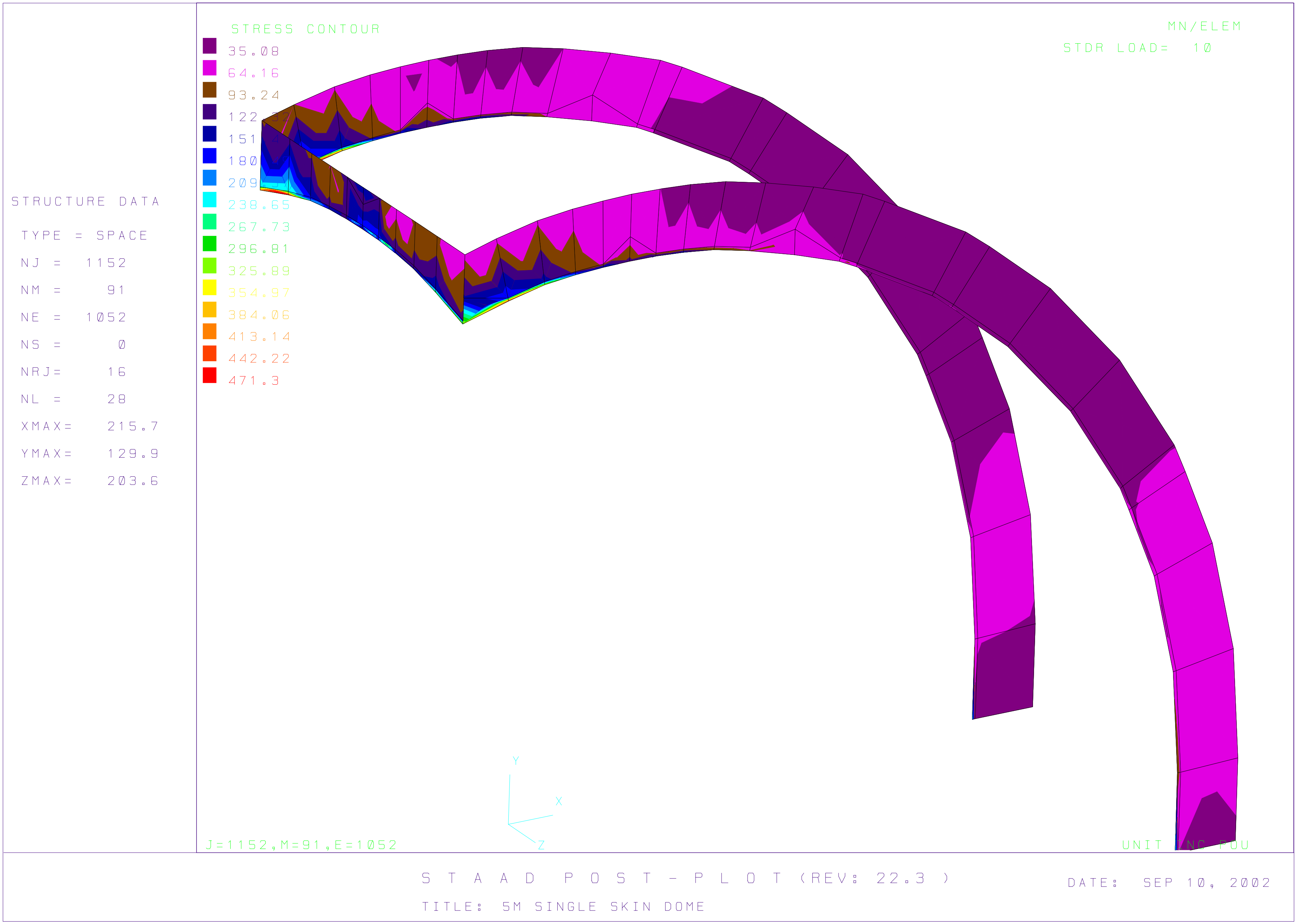


FIGURE 8a – Slitframe Element Stress Contours
(Load Combination 2)

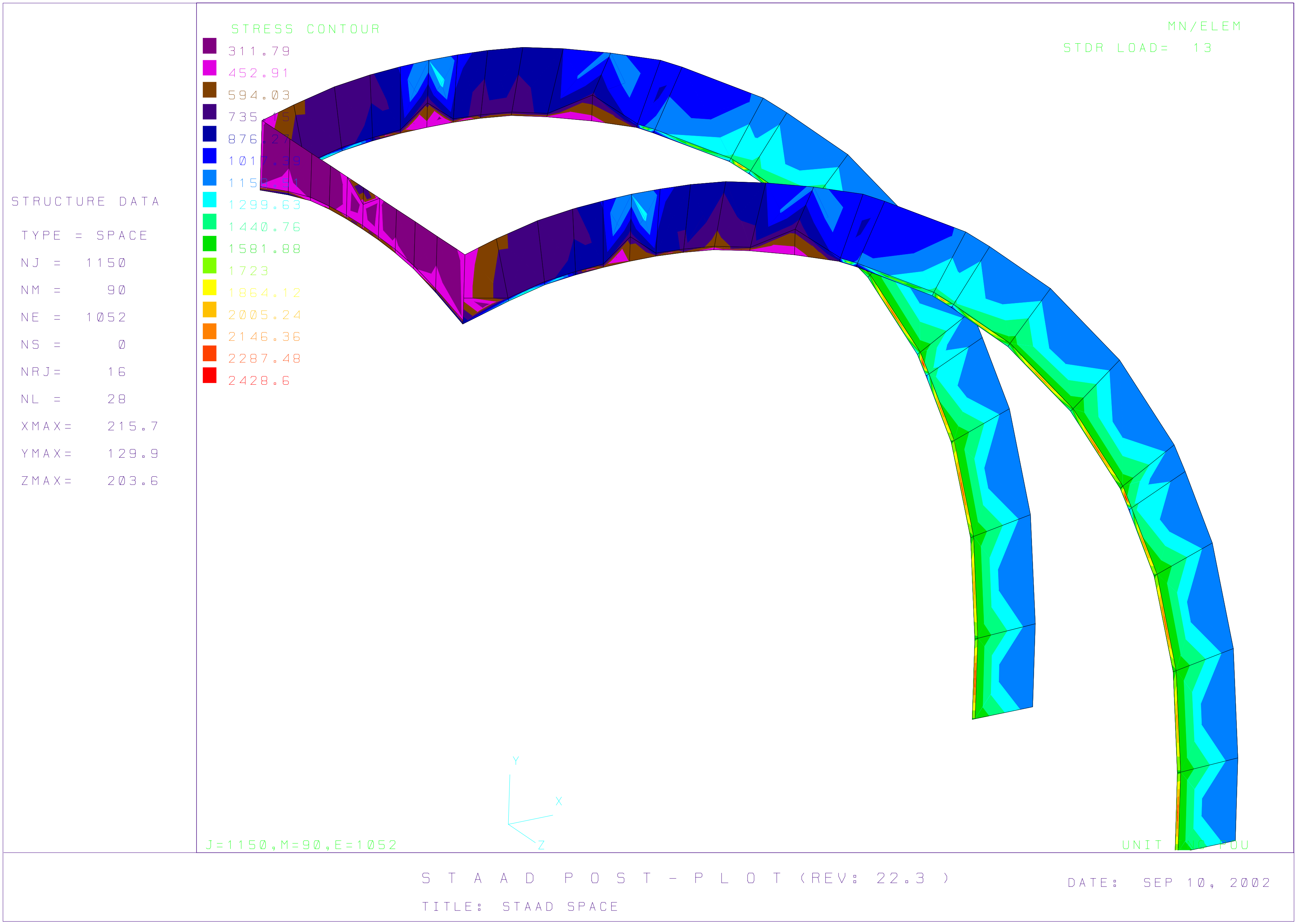


FIGURE 8b – Slitframe Element Stress Contours
(Load Combination 3, Wind in -X (right to left) Direction)

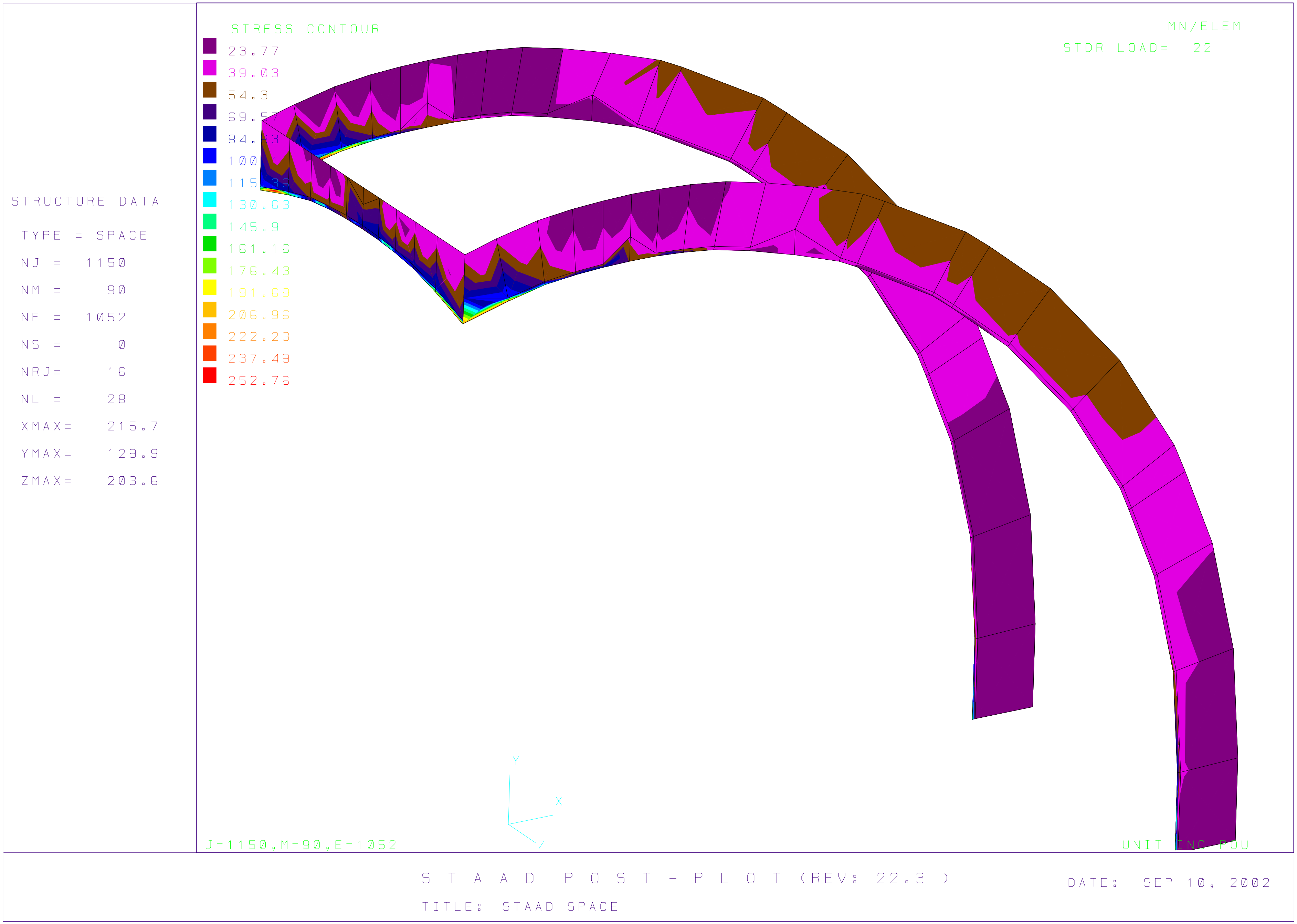


FIGURE 8c – Slitframe Element Stress Contours

(Load Combination 5, Seismic in Z Direction)

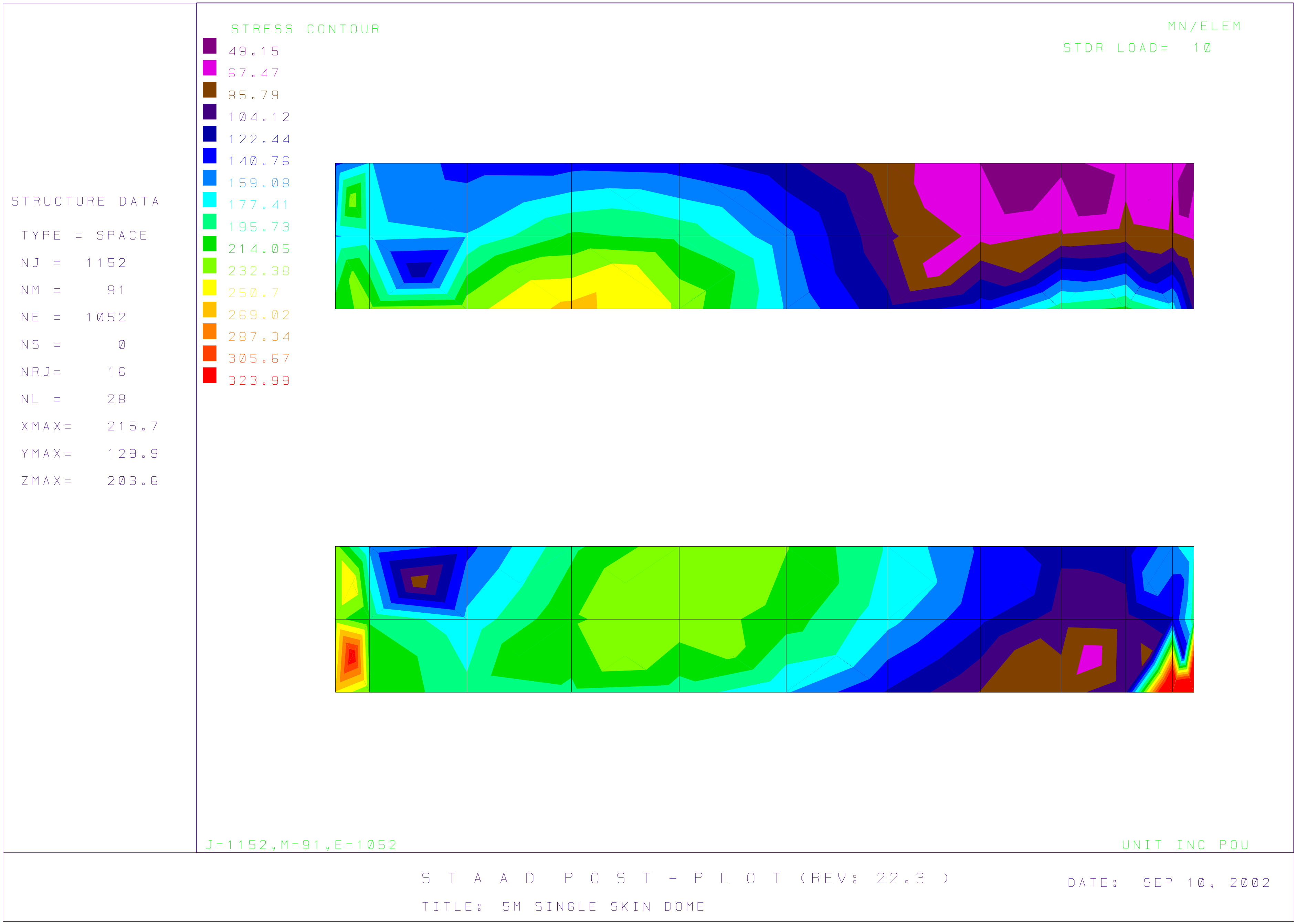
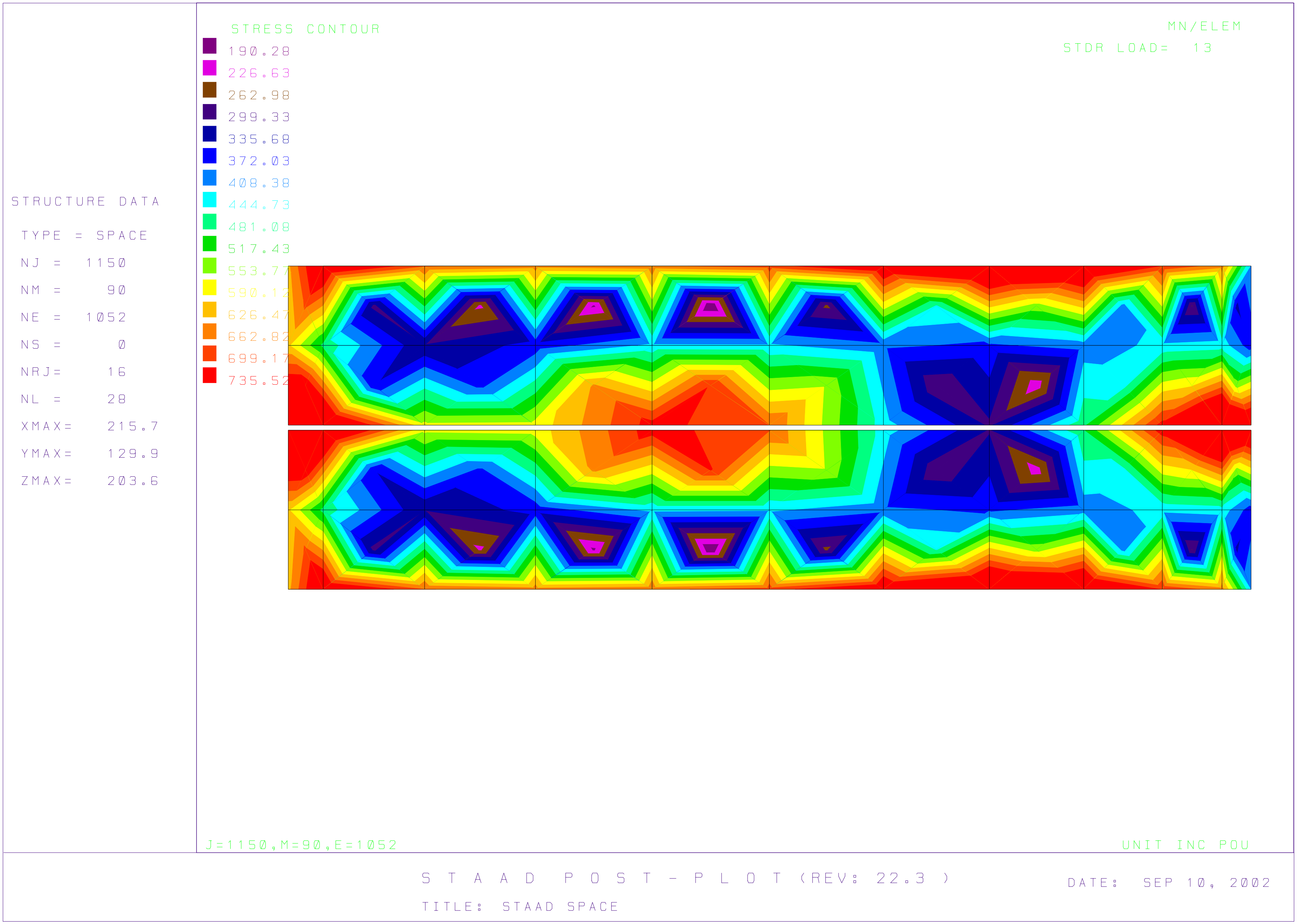


FIGURE 9a – Outside Shutter Sheet Stress Contours
(Load Combination 2)



<u>FIGURE 9b – Outside Shutter Sheet Stress Contours</u>
(Load Combination 3, Wind Load in -X (right to left) direction)

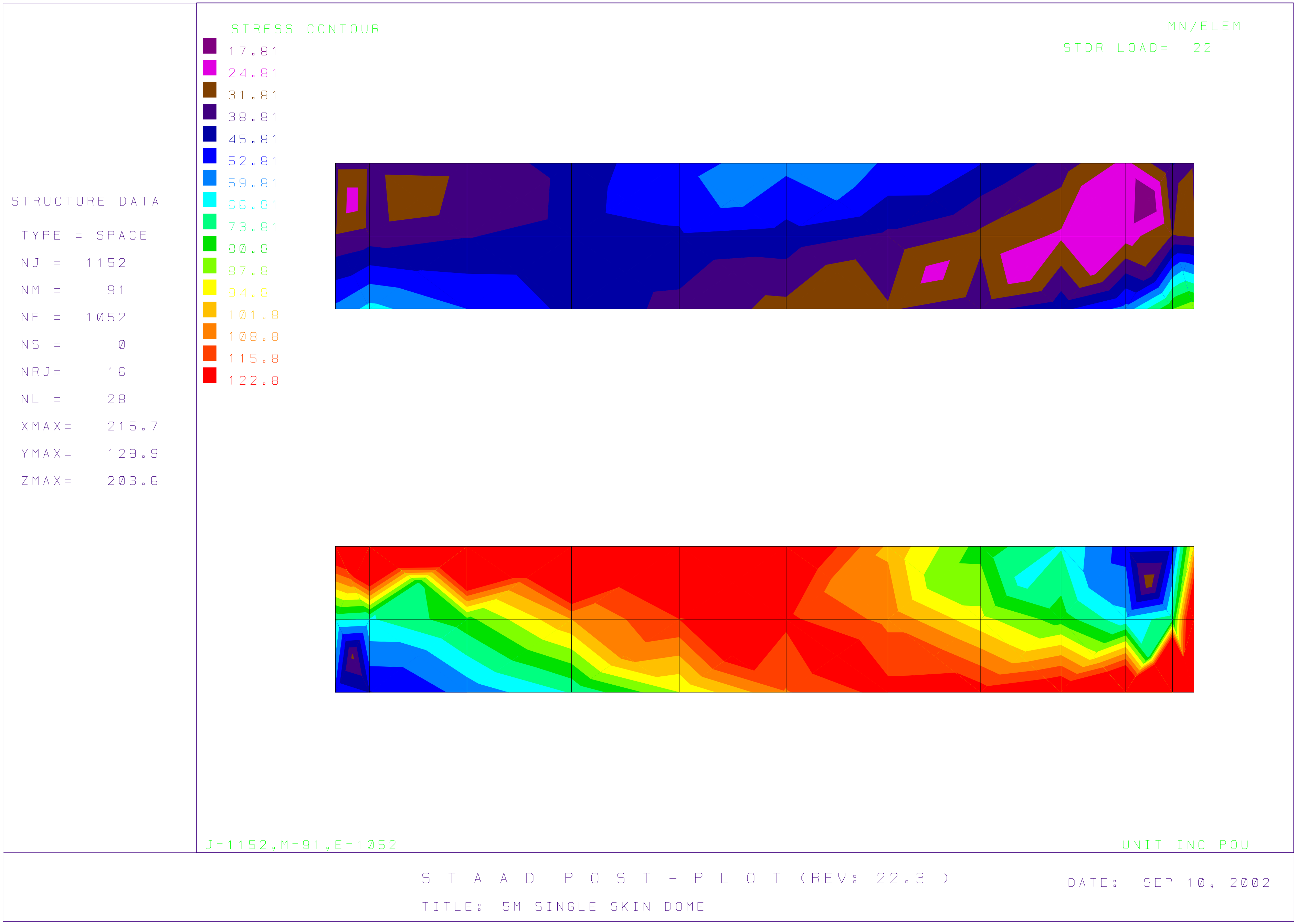


FIGURE 9c – Outside Shutter Sheet Stress Contours
(Load Combination 5, Seismic in Z Direction)

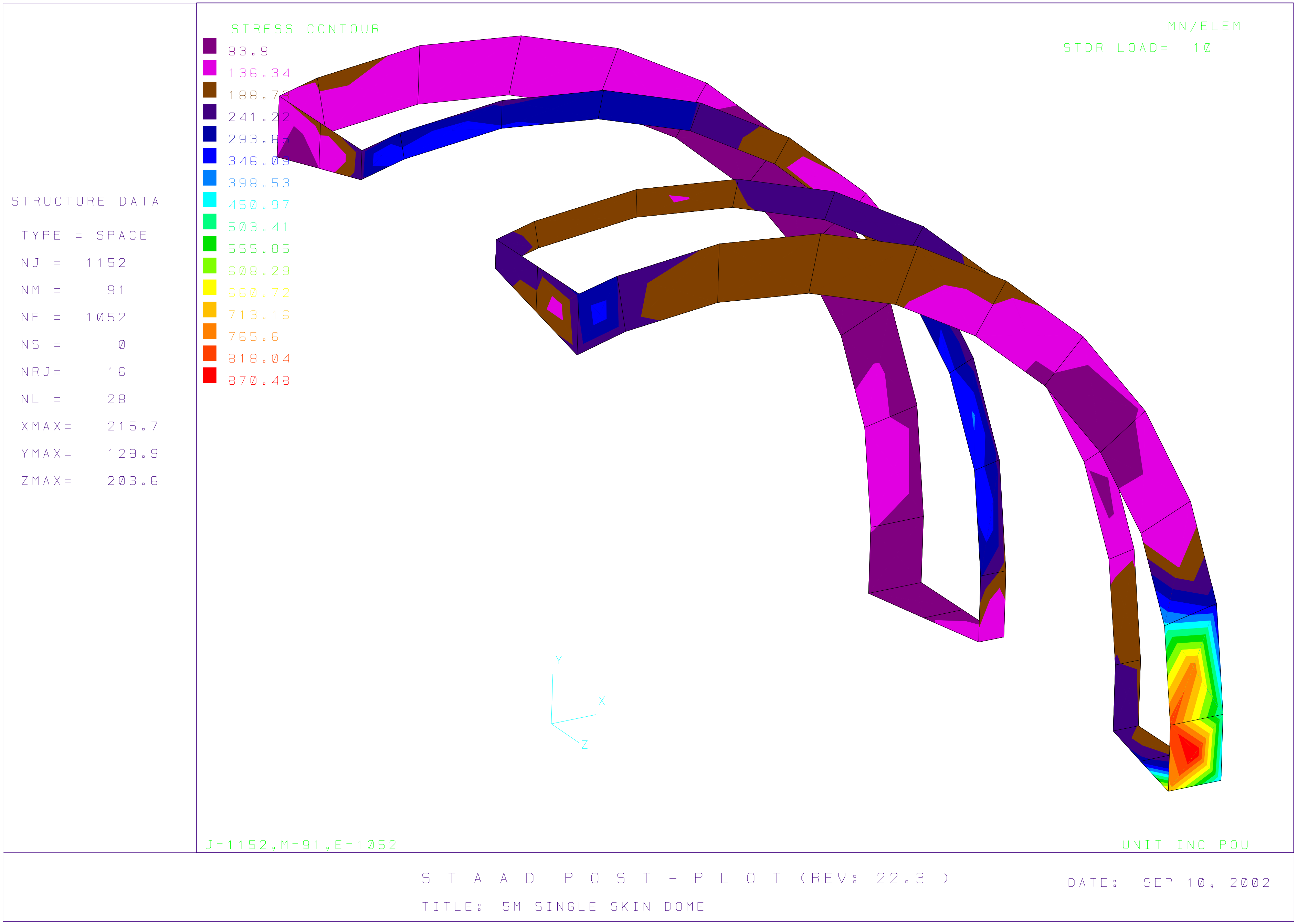


FIGURE 10a –Shutter Side Element Stress Contours
(Load Combination 2)

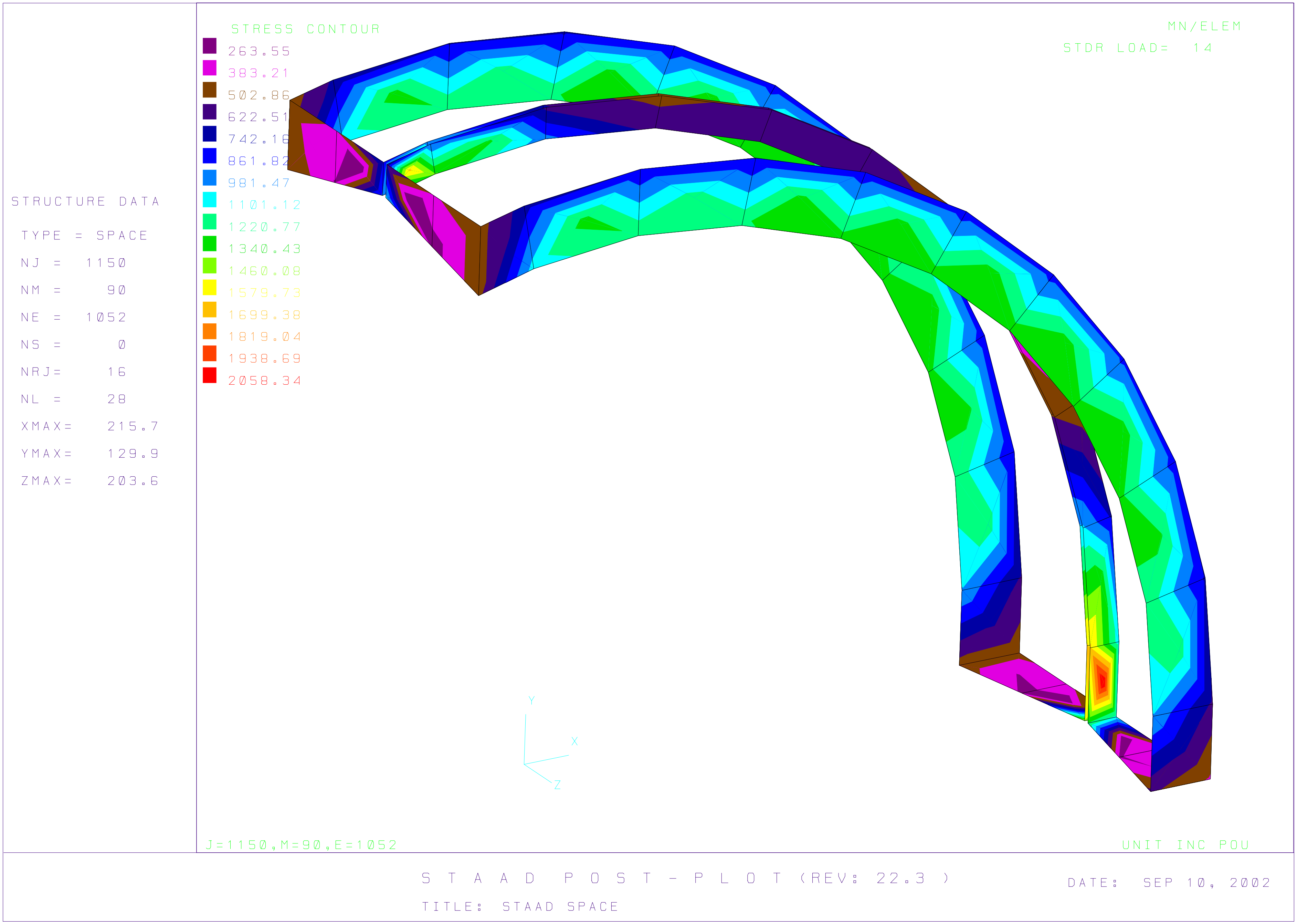


FIGURE 10b –Shutter Side Element Stress Contours

(Load Combination 3, Wind Load in +X (left to right) direction)

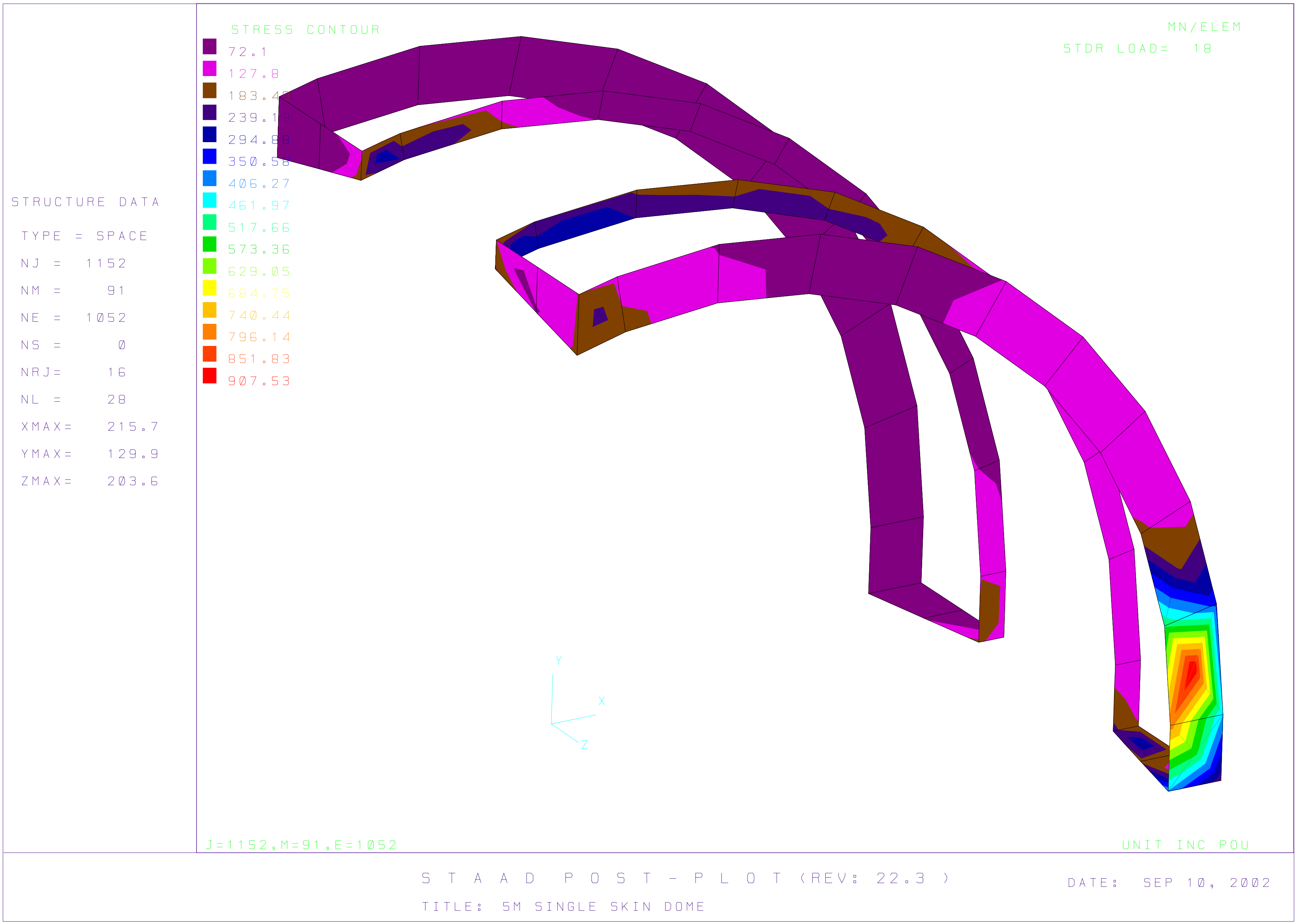


FIGURE 10c –Shutter Side Element Stress Contours
(Load Combination 4, Seismic in Z Direction)

SECTION VI

APPENDIX A – MANUAL CALCULATIONS

WIND LOADS

REQUIREMENT IS SURVIVAL OF 120 MPH WIND

ASSUME WIND IS THE MAXIMUM CORRESPONDING TO A 3-SEC GUST AND APPLY ASCE 7-98

$V = 120$ MPH, CATEGORY II $\therefore I = 1.0$

$K_d = 0.85$ (TABLE 6-6) FOR ARCHED ROOFS BUT USE A FACTOR OF 1.0 TO BE CONSERVATIVE

TOPOGRAPHY IS UNKNOWN, USE $K_{zt} = 1.0$

SINCE HEIGHT ABOVE GRADE IS UNKNOWN, USE $H = 30'$ $\therefore$

$K_z = 0.98$

ENCLOSURE SPECIFICATION:

WITH SHUTTERS CLOSED, CLASSIFICATION IS ENCLOSED

WITH SHUTTERS OPEN CLASSIFICATION IS PARTIALLY ENCLOSED. HOWEVER 120 MPH WIND SPEED APPLIES TO CLOSED DOME. USE WIND GUST OF 60 MPH FOR OPEN DOME WIND LOADING.

CLOSED DOME:

$$q_z = 0.00256(0.98)(1.0)(1.0)(120)^2(1.0) = 36.13 \text{ PSF}$$

OPEN DOME

$$q_z = 0.00256(0.98)(1.0)(1.0)(60^2)(1.0) = 9.03 \text{ PSF}$$

FOR PRESSURE COEFFICIENTS, CONSIDER DOME ANALOGOUS TO AN ARCHED ROOF

WHERE RISE TO SPAN RATIO, $r = 1:2 = 0.5$
(FOR A SPHERICAL DOME)

WINDWARD QUARTER

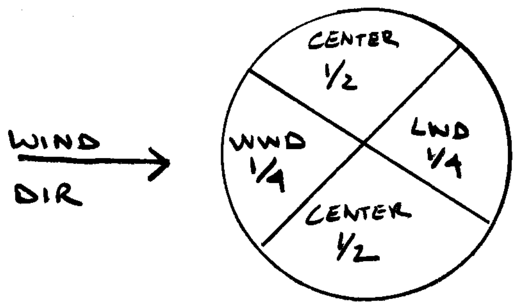
$$C_p = (2.75)(0.5) - 0.7 = 0.675$$

CENTER HALF

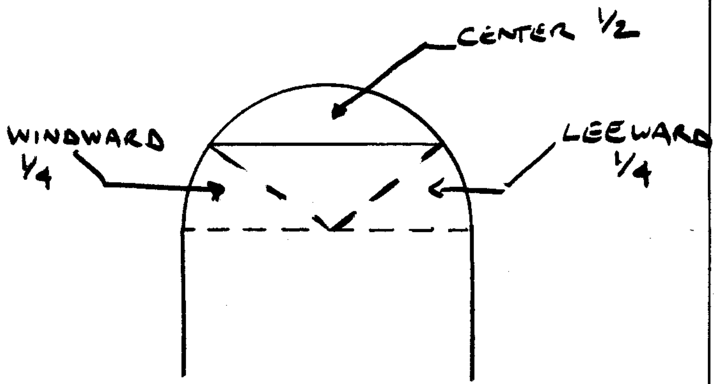
$$C_p = -0.7 - 0.5 = -1.2$$

LEEWARD QUARTER

$$C_p = -0.5$$



PLAN



ELEVATION

INTERNAL PRESSURE COEFFICIENTS:

CLOSED DOME, $GC_{pi} = \pm 0.18$

OPEN DOME, $GC_{pi} = \pm 0.55$

EXTERNAL PRESSURE ON CLOSED DOME:

WWD 1/4 → $P = (36.13)(0.85)(0.675) = 20.7$ PSF

CENTER 1/2 → $P = (36.13)(0.85)(-1.2) = -36.9$ PSF

LWD 1/4 → $P = (36.13)(0.85)(-0.5) = -15.4$ PSF

INTERNAL PRESSURE ON CLOSED DOME:

$P = (36.13)(\pm 0.18) = \pm 6.5$ PSF

EXTERNAL PRESSURE ON OPEN DOME

WWD 1/4 → $P = (9.03)(0.85)(0.675) = 5.2$ PSF

CENTER 1/2 → $P = (9.03)(0.85)(-1.2) = -9.2$ PSF

LWD 1/4 → $P = (9.03)(0.85)(-0.5) = -3.8$ PSF

INTERNAL PRESSURE ON OPEN DOME

$P = (9.03)(\pm 0.55) = \pm 5.0$ PSF

SNOW LOAD

ASSUME 30 PSF GROUND SNOW LOAD

UNHEATED STRUCTURE $\therefore C_T = 1.2$

CATEGORY II $\therefore I = 1.0$

EXPOSURE IS UNKNOWN, ASSUME FULLY EXPOSED, ABOVE THE TREE LINE IN WINDSWEPT MOUNTAINOUS REGIONS $\therefore$

$C_e = 0.7$

$P_f = 0.7\, C_e\, C_T\, I\, p_g$

$P_f = 0.7(0.7)(1.2)(1.0)(30) = 17.64$ PSF

SEISMIC LOAD (UBC 1997, CBC 1998)

ASSUME SEISMIC ZONE 4 $\therefore$ $Z = 0.4$ SAN JOSE, CA

ASSUME SOIL PROFILE S_D

MISC. STRUCTURE $I = 1.00$

$R = 2.9$ FOR "ALL OTHER SELF SUPPORTING STRUCTURES"

$C_a = 0.44 N_a$ $N_a = 1.2$, $N_v = 1.6$ WITHIN 5km OF HAYWARD FAULT AND 15km OF SAN ANDREAS FAULT

$C_v = 0.64 N_v$

$C_a = 0.44 N_A = 0.44(1.2) = 0.528$

$C_v = 0.64 N_v = 0.64(1.6) = 1.024$

S/

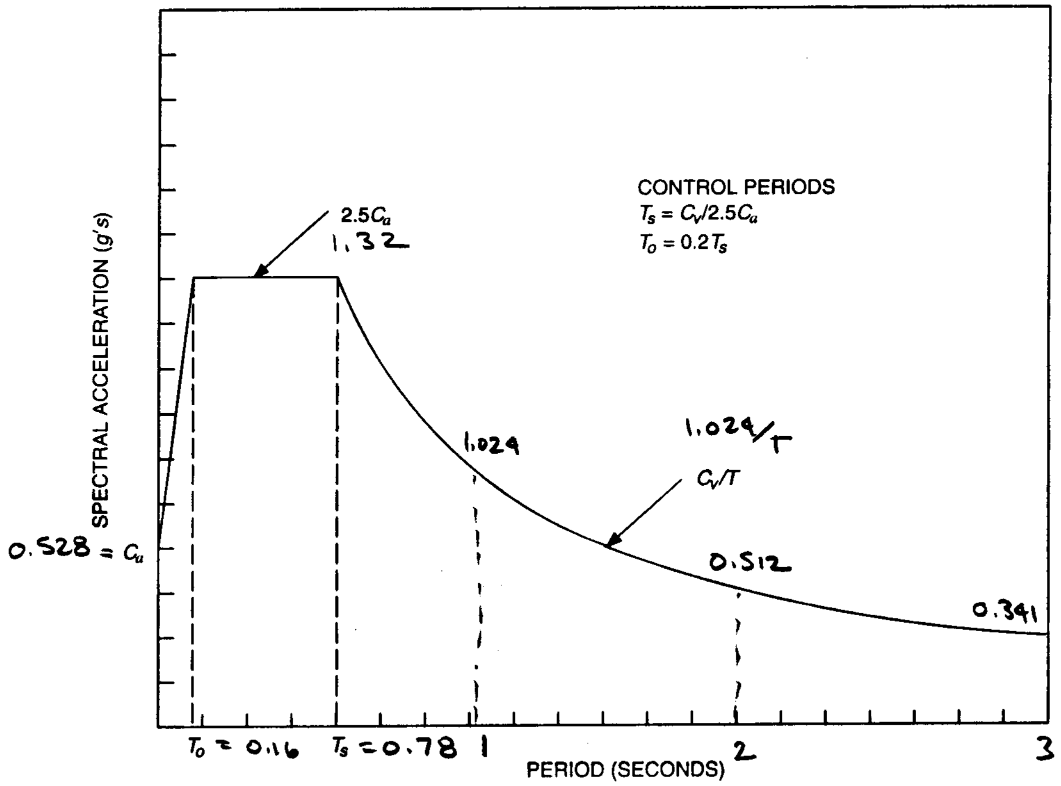


FIGURE 16A-3—DESIGN RESPONSE SPECTRA

GIVEN: SEISMIC ZONE 4

ASSUME: SOIL PROFILE S_D, $N_a = 1.2$, $N_v = 1.6$ (HAYWARD FAULT & SAN ANDREAS WITHIN 5 km)

$\therefore$ $C_a = 0.528$, $C_v = 1.024$

$$2.5C_a = 2.5(0.528) = 1.32$$

$$T_s = \frac{C_v}{2.5C_a} = \frac{1.024}{2.5(0.528)} = 0.78$$

$$T_o = 0.2T_s = 0.2(0.78) = 0.16$$

No. 937 811E Engineer's Computation Pad

STAEDTLER®

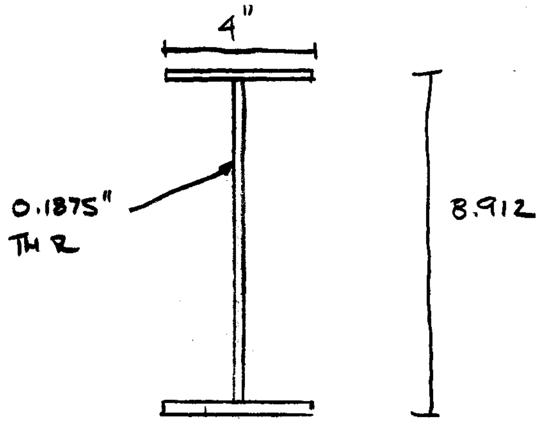
UPPER SHUTTER TRACK :

$A = 2(4)(0.1875) + (8.912)(0.1875)$

$A = 3.171 \text{ IN}^2$

$A_y = (8.912)(0.1875) = 1.671 \text{ in}^2$

$A_z = (2)(4)(0.1875) = 1.5 \text{ in}^2$

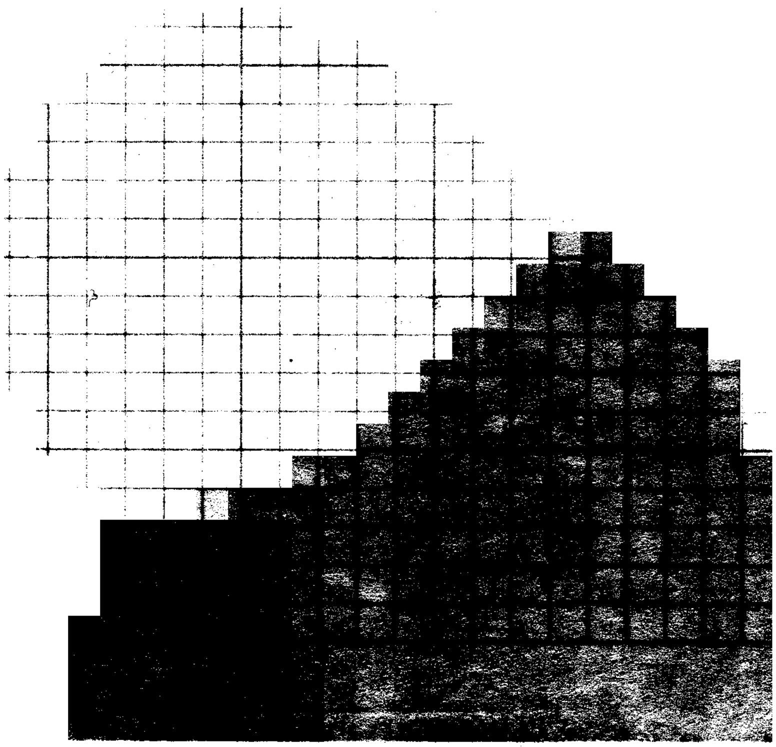


$$I_z = \frac{(4)(8.912)^3}{12} - \frac{(3.8125)(8.662)^3}{12} = 29.5 \text{ IN}^4$$

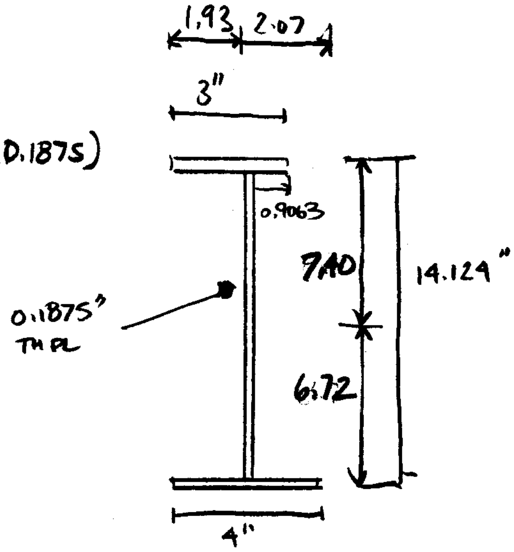
$$I_y = \frac{2(0.1875)(4)^3}{12} + \frac{(8.662)(0.1875)^3}{12} = .2 \text{ IN}^4$$

$I_x = 1.0 \text{ in}^4$

$y_o = 8.912"$ $\quad z_o = 4"$



LOWER SHUTTER TRACK:



$A = (3)(0.1875) + (4)(0.1875) + (13.749)(0.1875)$

$A = 3.89 \text{ in}^2$

$A_y = (14.124)(0.1875) = 2.65 \text{ in}^2$

$A_z = (3+4)(0.1875) = 1.31 \text{ in}^2$

$$\bar{y} = \frac{(3)(0.1875)(0.0938) + (13.749)(0.1875)(7.062) + (4)(0.1875)(14.03)}{3.89}$$

$\bar{y} = 7.40''$

$$\bar{x} = \frac{(4)(0.1875)(2) + (13.749)(0.1875)(2) + (3)(0.1875)(1.5)}{3.89}$$

$\bar{x} = 1.93''$

$$I_z = \frac{0.1875(7.2125)^3}{3} + \frac{0.1875(6.5325)^3}{3} + \frac{3(0.1875)^3}{12} +$$
$$3(0.1875)(7.3063)^2 + (4)(0.1875)(6.6263)^2 + \frac{4(0.1875)^3}{12}$$

$I_z = \underline{\underline{103.83 \text{ in}^4}}$

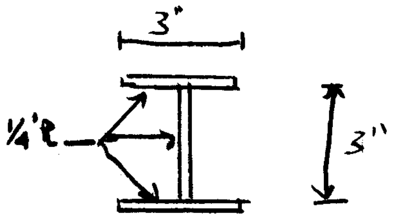
$$I_y = \frac{0.1875(1.93)^3}{3} + \frac{0.1875(1.07)^3}{3} + \frac{0.1875(1.93)^3}{3} + \frac{0.1875(2.07)^3}{3}$$
$$+ \frac{13.749(0.1875)^3}{12} + (13.749)(0.1875)(0.07)^2$$

$I_y = 1.55 \text{ in}^4$

$I_x = 1$

$y_o = 13.4 \qquad z_o = 3.9$

BACK ARCH:



$A_x = (3)(2)(0.25) + (2.5)(0.25) = 2.125 \text{ in}^2$

$A_y = (3)(0.25) = 0.75 \text{ in}^2$

$A_z = (3)(2)(0.25) = 1.5 \text{ in}^2$

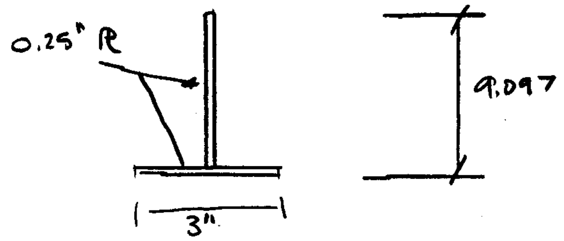
$I_x = 1$

$I_y = \dfrac{(2)(0.25)(3)^3}{12} + \dfrac{(2.5)(0.25)^3}{12} = 1.128 \text{ in}^4$

$I_z = \dfrac{3(3)^3}{12} - \dfrac{2.75(2.5)^3}{12} = 3.169 \text{ in}^4$

$y_D = 3 \qquad z_d = 3$

FRONT ARCH:



$A_x = 3(0.25) + 8.847(0.25) = 2.96 \text{ in}^2$

$A_y = (9.097)(0.25) = 2.27 \text{ in}^2$

$A_z = 3(0.25) = 0.75$

$$\bar{y} = \frac{(3)(0.25)(0.125) + 8.847(0.25)(4.6735)}{2.96}$$

$\bar{y} = 3.52 \text{ in}$

$$I_z = \frac{(0.25)(5.577)^3}{3} + \frac{(0.25)(3.27)^3}{3} + \frac{(3)(0.25)^3}{12} + (3)(0.25)(3.145)^2$$

$I_z = 24.79 \text{ in}^4$

$$I_y = \frac{0.25(3)^3}{12} + \frac{8.847(0.25)^3}{12} = 0.574 \text{ in}^4$$

$I_x = 1$

$y_d = 7.04 \quad z_d = 3$

[redacted]

STEEL ALLOWABLE STRESSES

UPPER SHUTTER TRACK:

$$b/t = \frac{1.9063}{0.1875} = 10.167 < 10.83 = 65/\sqrt{F_y}$$

∴ FLANGE IS COMPACT

$$d/t = \frac{8.912}{0.1875} = 47.53 < 106.7 = \frac{640}{\sqrt{F_y}}$$

∴ WEB IS COMPACT

$L_b = 54.874"$

$$\frac{76 b_f}{\sqrt{F_y}} = \frac{76(4)}{\sqrt{36}} = 50.667"$$

$$\frac{20000}{d/A_f \; F_y} = \frac{20000}{\left(\frac{8.912}{(4)(0.1875)}\right)36} = 46.8"$$

$$L_b = 54.874 > L_c = 46.8"$$

$$\therefore F_b = \frac{12000}{l d/A_f} = \frac{12000}{(54.874)\left(8.912/(4)(0.1875)\right)}$$

$$F_{bz} = 18.404 \text{ ksi} = \underline{18404 \text{ psi}}$$

$$F_{by} = 0.6F_y = \underline{21600} \text{ psi}$$

LOWER SHUTTER TRACK:

$$b/t = \frac{1.9063}{0.1875} = 10.167 < 10.83 = 65/\sqrt{F_y}$$

∴ FLANGE IS COMPACT

$$d/t = \frac{14.124}{0.1875} = 75.33 < 106.7 = \frac{640}{\sqrt{F_y}}$$

∴ WEB IS COMPACT

$L_b = 54.874$

$$\frac{76\, b_f}{\sqrt{F_y}} = \frac{76(3)}{\sqrt{36}} = 38''$$

$$\frac{20000}{(d/A_f)F_y} = \frac{20000}{\frac{14.124}{(3)(0.1875)}(36)} = 22.125''$$

$L_b = 54.874 > 22.125'' = L_c$

$$F_{bx} = \frac{12000}{(54.874)\left(\frac{14.124}{(3)(0.125)}\right)} = 11.612 \text{ KSI}$$

$$= \underline{11612 \text{ PSI}}$$

$F_{by} = 0.6 F_y = \underline{21600} \text{ PSI}$

DOME RING:

$b/t = 4/0.25 = 16 > 15.8 = 95/\sqrt{F_y}$

$\therefore$ SECTION IS SLENDER

$h/t = \frac{5.75}{0.25} = 23 < 70 \quad \therefore k_c = 1$

$195/\sqrt{36} = 32.5$

$\therefore \quad Q_s = 1.293 - 0.00309\,(b/t)\sqrt{F_y/k_c}$

$Q_s = 1.293 - 0.00309\,(16)(\sqrt{36})$

$Q_s = 0.9964$

$F_b = 0.6 F_y Q_s = 0.6(36)(0.9964) = 21.522$ KSI

USE $\underline{21522 \text{ PSI}}$ FOR ALL STRESSES IN DOME RING ELEMENTS

[redacted]

FRONT ARCH SECTION:

STEM $d/t = 9/0.25 = 36 > 21 = 127/\sqrt{F_y}$

SECTION IS SLENDER

$176/\sqrt{F_y} = 29 < 36$

$Q_s = \dfrac{20000}{(36)(36)^2} = 0.43$

$F_b = (0.6)(36)(0.43) = 9.288$ KSI $= \underline{9288 \text{ PSI}}$

BACK ARCH SECTION:

$b/t = 1.375/0.25 = 5.5 < 10.8 = 65/\sqrt{F_y}$

$\therefore$ FLANGE IS COMPACT

$d/t = 3/0.25 = 12 < 107$

$\therefore$ WEB IS COMPACT

$L = 48''$ $\qquad \dfrac{76\, b_f}{\sqrt{F_y}} = \dfrac{76(3)}{\sqrt{36}} = 38''$

$\dfrac{20000}{\left(\frac{d}{A_f}\right) F_y} = \dfrac{20000}{\frac{3}{(3)(0.25)}(36)} = 139''$

$48'' > 38'' \therefore F_{bt} = \dfrac{12000}{\frac{ld}{A_f}} = \dfrac{12000}{\frac{(48)(3)}{(3)(0.25)}} = 62.5$

USE $0.6 F_y = \underline{21600 \text{ PSI}}$

ALUMINUM ALLOWABLE STRESSES:

DOME SKIN ELEMENTS

MATERIAL - ALUMINUM ALLOY 3003 - H14

REFERENCE ALUMINUM ASSOCIATION TABLE 3.3.9

TENSION ALLOWABLE : 10.5 ksi 1" OR MORE FROM WELD

4.2 ksi WITHIN 1" OF WELDS

COMPRESSION ALLOWABLE:

SPECIFICATION NO. 9 APPLIES TO WHERE ELEMENTS ARE SATISFIED BY SHOP STANDING SEAMS OR FIELD SPLICES AND b = SPACING BETWEEN SEAMS AT POINT OF INTEREST.

SEAMS BECOME CLOSER TOGETHER AS THE DISTANCE ABOVE THE SPRINGLINE INCREASES SO b DECREASES RESULTING IN A VARIABLE ALLOWABLE STRESS - SEE TABLE IN REPORT FOR SKIN ALLOWABLE STRESSES

SAMPLE COMPUTATION

OUTSIDE SKIN, AT SPRINGLINE, 40 EQUALLY SPACED SEAMS, $\phi = 196.85"$

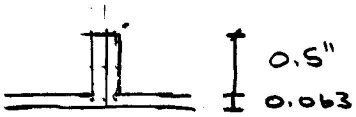
$$b = \frac{\pi(196.85)}{40} = 15.46$$

$$b/t = \frac{15.46}{0.063} = 245.4$$

$$F_a = \frac{280}{(b/t)} = \frac{280}{245.4} = 1.141 \text{ ksi} \Rightarrow \underline{\underline{1141 \text{ PSI}}}$$

NOTE: F_c APPLIES REGARDLESS OF PROXIMITY TO WELD

STANDING SEAMS:

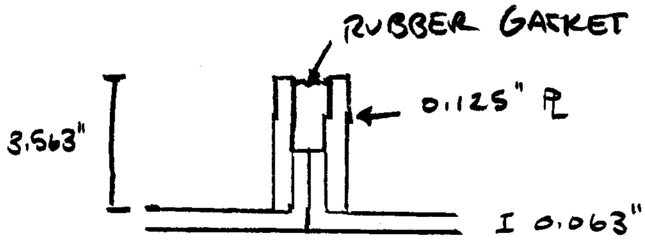


SPEC NO. 8 APPLIES

$$b/t = \frac{0.5}{(2)(0.063)} = 4$$

$$\therefore F_a = 4.2 \text{ KSI} = \underline{4200 \text{ PSI}}$$

FIELD SPLICE:



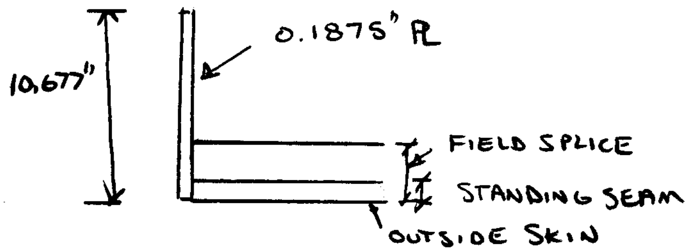
SPEC NO. 8 APPLIES

$$b/t = \frac{3.563}{0.125} = 28.5 > 25$$

$$\therefore F_a = \frac{1970}{(b/t)^2} = \frac{1970}{(28.5)^2} = 2.425 \text{ KSI}$$

$$\underline{2425 \text{ PSI}}$$

SLITFRAME



SPEC NO. 9 APPLIES

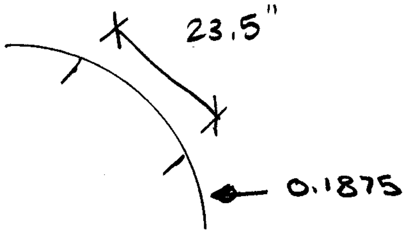
$b/t = 10.677 / 0.1875 = 57 \quad > 24$

$\quad\quad\quad\quad\quad\quad\quad\quad\quad\quad < 60$

$F_a = 9.4 - 0.079(b/t) = 9.4 - 0.079(57) = 4.897 \text{ KSI}$

$= \underline{4897 \text{ PSI}}$

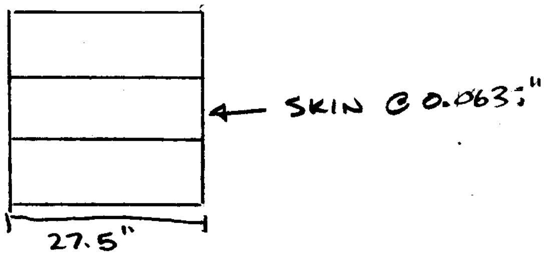
SHUTTER SKIN



PER ABOVE, b = 23.5"

$b/t = \dfrac{23.5"}{0.08} = 293.75$

$F_a = 280 / 293.75 = .953 \text{ KSI} = \underline{953 \text{ PSI}}$

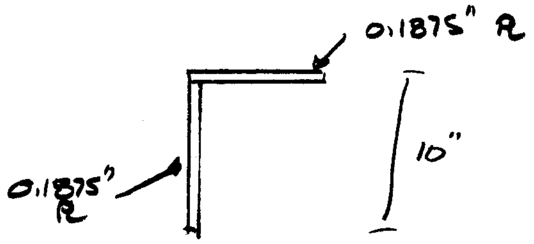


PER ABOVE

$b/t = 27.5 / 0.08 = 343.75 \quad\quad F_a = 280 / 343.75 = 0.814 \text{ KSI}$

CONTROLS → $\underline{814 \text{ PSI}}$

SHUTTER SIDE WALLS



SPEC NO. 9 APPLIES

$$b/t = \frac{10}{0.1875} = 53.333 \quad > 24$$
$$< 60$$

$$F_a = 9.4 - 0.079(53.333) = 5.186 \text{ KSI} = \underline{5186 \text{ PSI}}$$

MIDDLE DOOR SIDES AND INSIDE SHUTTER SIDES ARE THE SAME AS ABOVE EXCEPT b = 5" AND b = 4". USE SAME ALLOWABLES