Appendix 8A.1 Structural Analysis of DTS

8A.1.1 Structural Specifications

The design bases for the DTS are described in Chapter 3.0. The structure will be constructed from reinforced concrete and structural steel work, the design of which complies with the following principle specifications:

- American Concrete Institute ACI 349-85: Code Requirements for Nuclear Safety Related Concrete Structures.

- American Concrete Institute ACI 318-89: Building Code Requirements for Reinforced Concrete with Commentary.

- American Institute of Steel Construction: AISC Specification for the Design, Fabrication and Erection of Structured Steel for Buildings. June, 1989.

8A.1.2 General Description of the DTS Structure

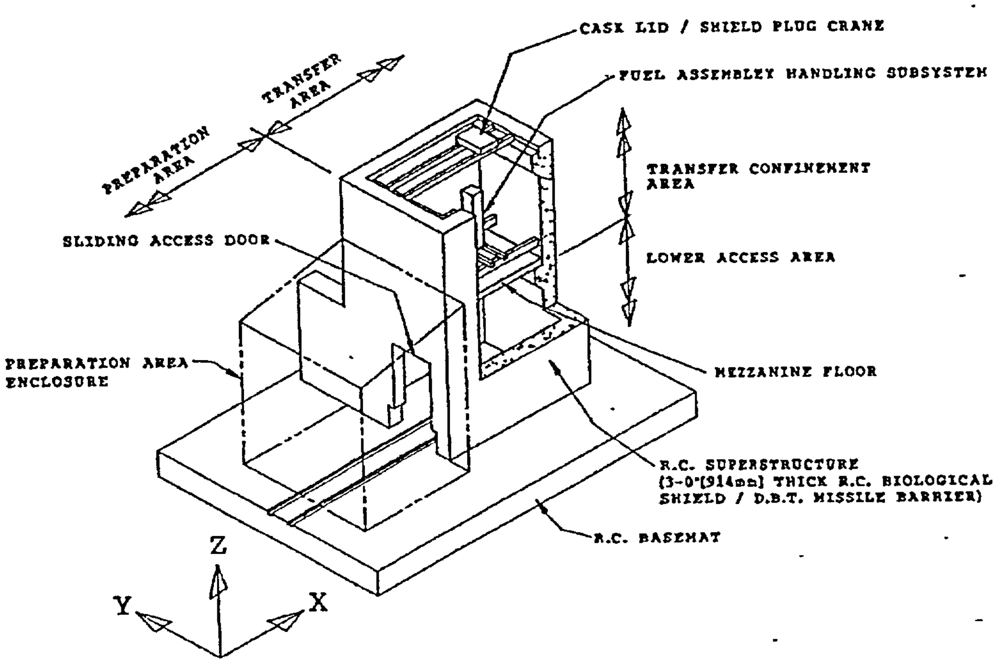
The primary functions of the DTS structure are to provide radiation shielding, tornado missile protection, and confinement of radioactive material. The Dry Transfer System Overview is shown on Figure 8A.1-1.

The DTS structure includes the following major elements:

- Reinforced concrete basemat;
- Reinforced concrete superstructure which encloses the Transfer Confinement Area and Lower Access Area, provides a radiation shield, tornado missile barrier, and primary confinement, and contains the fuel transfer equipment;
- Embedments for support of the fuel handling crane;
- Protective cover;
- Structural steel roof plate that supports the upper crane;
- Mezzanine plate that supports the Cask Mating Subsystem;
- Sliding door between the Lower Access Area and the Preparation Area; and
- Preparation Area Enclosure.

Figure 8A.1-1

Dry Transfer System Overview



8A.1.3 Design Loadings and Input Parameters

In preparing the design of the DTS, the loadings and other input parameters have been based upon the following principle codes and standards:

- American National Standards Institute, Design Criteria for an Independent Spent Fuel Storage Installation. ANSI/ANS 57.9-1992.
- American National Standards Institute, Minimum Design Loads for Buildings and Other Structures. ANSI/ASCE 7-95-1996.
- U.S. Nuclear Regulatory Commission, Regulatory Guide 1.60. Design Response Spectra for Seismic Design of Nuclear Power Plants, 1973.
- U.S. Nuclear Regulatory Commission, Regulatory Guide 1.61. Damping Values for Seismic Design of Nuclear Power Plants, 1973.
- U.S. Nuclear Regulatory Commission, Regulatory Guide 1.76. Design Basis Tornado for Nuclear Power Plants, 1974.
- "Missile Generated by Natural Phenomena." U.S. Nuclear Regulatory Commission, NUREG-0800. Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants, July 1981. Para. 3.5.1.4.

Since the design presented in this report is based on a non-site specific location, conservative assumptions have been made for the selection of input parameters (wind loading, seismic loading, soil conditions, missiles, etc.).

8A.1.4 Design Basis and Philosophy

The reinforced concrete structure forms a heavy rigid box structure with substantial stiff cross walls in both directions.

A. Seismic Analysis

The seismic analysis is performed assuming the structure is founded on a hard rock site. A response spectrum analysis was used to verify the structural design. The seismic input was taken from 10 CFR 72.102.

B. Tornado Missiles

The design of the structure has taken into account the missiles produced by the design basis tornado. The most damaging missiles are the automobile, wood utility pole and 12" (305 mm) diameter pipe. The automobile and utility pole impacts are limited to a height of no more than 30' (9.1 m) above grade.

The automobile does not create local damage since it is a "soft" missile and crushes on impact. It is considered with respect to overall barrier stability and energy absorption. Similarly, the wood utility pole is subject to considerable deformation and will not locally deform the structure. Thus, the 12" diameter (305 mm) Schedule 40 steel pipe is considered the missile that can produce the worst local damage effects.

The DTS wall thickness is primarily-dictated by shielding requirements. The massive walls adequately protect the DTS against tornado missiles and other adverse natural phenomena. The tornado generated missile impacts are considered to bound all other reasonable impact-type accidents.

8A.1.5 Normal Operation Structural Analysis

Table 8A.1-1 shows the normal operating loads for which the DTS structural components are designed. The table also lists the individual components which are affected by each loading. The magnitude and characteristics of each load are described in Section 8A.1.5.1.

The method of analysis and analytical results for each load are described in sections 8A.1.5.2 through 8A.1.5.6. The mechanical properties of materials employed in the structural analysis of the DTS system components are presented in Table 8A.1-2.

Table 8A.1-1

DTS Normal Operating Loads

Load Type	Affected Component: Reinforced Concrete Structure	Protective Cover	Roof Plate	Mezzanine Plate	Sliding Door
Dead Loads	X	X	X	X	X
Operational Handling Loads	X		X	X	
Live Loads	X	X	X	X	
Normal Thermal Loads	X	X	X	X	X
Internal Pressure	X	X	X	X	X
Design Basis Wind Pressure	X	X			

Table 8A.1-2A

Mechanical Properties of Material - Protective Cover & Roof Plate

Component	Material	Temp. (^{0}F)	Yield Strength S_y (ksi)	Ultimate Strength S_u (ksi)	Remark
Protective Cover	A-36	70	36	58	
Protective Cover Beams	A-36	70	36	58	
Protective Cover Bolts	A-193-B7	70	105	125	
Roof Plate	A-105	70	36	70	
Roof Plate Beams	A-36	70	36	58	
Roof Plate Bolts	A-193-B7	70	105	125	
Roof Plate Corbel Connection Bolts	A-325[1]	70	56	73	

Notes:

1. For 1 in. bolt

Table 8A.1-2B

Mechanical Properties of Material - Mezzanine Plate

Component	Material	Temp. (^{0}F)	Yield Strength S_y (ksi)	Ultimate Strength S_u (ksi)	Remark
Mezzanine Plate	A-36	70	36	58	
Mezzanine Plate Beams	A-36	70	36	58	
Mezzanine Plate Bolts	A-193-B7	70	105	125	
Mezzanine Plate Corbel Connection Bolts	A-325[(1)]	70	56	73	

Notes:

1. For 1 in. bolt

Table 8A.1-2C

Mechanical Properties of Material - Sliding Door

Component	Material	Temp. (^{0}F)	Yield Strength S_y (ksi)	Ultimate Strength S_u (ksi)	Remark
Sliding Door	A-105	70	36	70	
Sliding Door Wheels	Drop Forged Steel	70	----	64	
Sliding Door wheel Axle	A-564 Type 630 H 1100	70	115	140	
Axle Bracket	A-514	70	100	110	
Axle Bracket Bearing	A-514	70	22	45	
Sliding Door Support Bracket	A-514	70	100	110	
Sliding Door Shear Pin	A-36	70	36	58	
Sliding Door Rail	A-514	70	100	110	

Table 8A.1-2D

Mechanical Properties of Reinforced Concrete and Rebar

Component	Material	Temp. (^0F)	Yield Strength S_y (ksi)	Ultimate Strength S_u (ksi)	Remark
Reinforcing Steel	A-615 Grade 60	70	60	90	

Component	Density (lbs/ft^3)	28 days Compress. Strength (ksi)	Modulus of Elasticity (ksi)	Remark
Reinforced Concrete	150	3	3000	

8A.1.5.1 Normal Operating Loads

The normal operating loads are described in detail in the following paragraphs.

A. Dead Loads

The calculation of the weights of the DTS reinforced concrete structure and equipment is provided below. The dead weight of each component is determined based on nominal component dimensions.

Weight of Reinforced Concrete Structure:

Assume density of concrete with Re-Bars = 150 lbs./cu. ft.

Concrete Volume above Elevation 0-0 excluding wing wall:

$$(280 \times 379 - 208 \times 307)\ 562.8 - (240 \times 109.5 + 48 \times 40.5)\ 36$$
$$+ (40 \times 20 + 20 \times 20/2)\ (4 \times 307 + 3 \times 208)$$
$$= 24{,}622{,}115 \text{ in}^3$$

Weight of concrete above E1evation 0-0 excluding wing wall:

$$=(24{,}622{,}115)\ 150/(12)^3$$
$$= 2{,}137{,}336 \text{ lbs.}$$

Weight of Wing Wall above Elevation 0-0

$$= 300 \times 120 \times 36 \times 150/(12)^3 = 112{,}500 \text{ lbs.}$$

Total Concrete above E1.0-0

$$= 2{,}137{,}336 + 112{,}500$$
$$= 2{,}249{,}836 \text{ lbs.}$$

Concrete Basemat below E1.0-0

$$= 867 \times 592 \times 60 \times 150/(120)^3$$
$$= 2{,}673{,}250 \text{ lbs.}$$

Total concrete Weight Including Wing Wall & Basemat

$$= 2{,}136{,}336 + 112{,}500 + 2{,}673{,}250$$
$$= 4{,}923{,}086 \text{ lbs.}$$

Weight of Structural Steel:

Protective Cover:

1.5" thick cover PL
= 379 x 280 x 1.5 x 0.284 + 2 (369 + 270) 105. 5 x 1.5 x 0.284
= 102,644 lbs

Structural Steel Beams & Columns (W12 x 120)
= 120/12 {2 x 369 + 2 x 270 + 8 x (107-12)}
= 20,380 lbs.

Total Structural Steel & 1.5" PLs. @ Roof Level
= 102,644 + 20,380 = 123,024 lbs.

Roof Plate:

7" Thick Cover Pl.
= (343 x 244 - 2 x 31.5 x 23.62) 7 x 0.284
= 163,421 lbs.

Structural Steel Beams (W14 x 550)
550/12 {3(244 - 2 x 12) + 2(343 - 2 x 12)}
= 59,492 lbs.

Total Structural Steel & 7" Pl.
= 163,421 + 59,492
= 222, 913 lbs.

Mezzanine Floor:

1.5" Pl.
$= \{(307-0.5)\ (208 - 0.5) - \pi/4\ (70.9^2 + 49.25^2)\}\ 1.5 \times 0.284$
= 24,600 lbs.

Structural Steel Beams (W12 x 120)
= 120/12 (3 x 204 + 2 x 303)
= 12,180 lbs.

Total 1.5" Pl. + Structural Steel
= 24, 600 + 12,180 = 36,780 lbs.

Equipment Weights

At Roof Plate:

Equipment	Load
Motor Driven Trolley (1)	10,000 lbs
Shield Trap Door (2)	12,000 lbs
MPC Shield Plug	5,000 lbs
Total	27,000 lbs

Loading at Mezzanine Plate Level

Equipment	Load
Source Cask Shield Trap Door	2,000 lbs
Receiving Cask Shield Trap Door	8,000 lbs
Overlid on Jack Up Platform: 2 x 2000	4,000 lbs
Jack Up Platform 2 x 4000	8,000 lbs
Source Cask Lid Wt.	2,500 lbs
MPC Shield Plug Wt.	5,000 lbs
Total	49,500 lbs

Fuel Handling Crane	22,000 lbs
Sliding Door	85,000 lbs.

Table 8A.1-3 shows the weights of various components of the DTS.

B. Operational Handling Loads

The operational handling loads are included in the weight of the equipment presented in Table 8A.1-3. For simplification of the analysis the operating handling loads are lumped together with dead loads.

C. Live Loads

A maximum live load of 250 lbs/ft^2 (11,970 Pa), based on ANSI/ASCE 7-95, Table 4-1, is to be used for floor design, which includes the effects of snow and ice.

The total live load on 7 inch Plate = $(343 \times 244 - 2 \times 31.5 \times 23.6) \times (250/144) = 142{,}715$ lb.

Total live load on mezzanine floor= $[306.5 \times 207.5 - (\pi/4)(70.9^2 + 49.25^2)] \times (250/144)$

$$= 100{,}253 \text{ lb.}$$

D. Normal Thermal Loads

The DTS is subject to thermal expansion loads associated with normal operating conditions. The range of normal operating temperature used for the design of the DTS is 60°F to 100°F (16°C to 38°C) in the Preparation Area and 40°F to 130°F (4°C to 54°C) in other areas.

E. Internal Pressure

The internal pressures (created by the HVAC system) during operation are as follows:

- TCA: 1 in (25.4 mm) H_20 less than ambient.
- Lower Access Area: 0.5 in (12.7 mm) H_20 less than ambient.
- Preparation Area: 0.25 in (6.4 mm) H_20 less than ambient.

These pressure loads are small and bounded by pressure loads generated by the 150 mph wind load. Consequently, these internal pressure loads are not evaluated.

F. Design Basis Wind Loads

Wind pressure effects on the DTS concrete wall are evaluated using both the simplified method and moment distribution method to calculate the sagging and hogging moments. The forces and moments acting on the concrete walls due to wind loads are calculated using equations and coefficients given in Section 6.4 of ASCE 7-95. A bounding wind speed of 150 mph is used to perform the analysis.

Table 8A.1-3

DTS Component Weights

Component Description		Calculated Weight
Reinforced Concrete Structure		2,249,836 lbs (1,020,508 kg)
R. C. Basemat		2,673,250 lbs (1,212,566 kg)
Protective Cover & Beams		123,024lbs (55,803 kg)
Roof Plate Level	Roof Plate	163,421 lbs (74,127 kg)
	Support Beam	59,492 lbs (26,985 kg)
	Equipment (including handling loads)	27,260 lbs (12,365 kg)
Fuel Handling Crane		22,000 lbs (10,000 kg)
Mezzanine Plate Level	Mezzanine Plate	24,600 lbs (11,158 kg)
	Support Beam	12,180 lbs (5,525 kg)
	Equipment (including handling loads)	49,500 lbs (22,400 kg)
Sliding Door		85,000 lbs (38,560 kg)

8A.1.5.2 Reinforced Concrete (Structure) Structural Analysis

Chapter 4, Section 4.2.3.1 provides the detailed description of the reinforced concrete structure. The configuration details including dimensions are shown on Drawings Nos.3039-15, Rev. 0, 3039-16, Rev. 0, and 3039-18, Rev. 0. The structure is designed to withstand a number of different loads and combinations of loads. The relevant normal operating loads are as follows (Refer to Table 8A.1-1):

A. Dead loads (including Operational handling loads)
B. Live loads
C. Normal thermal loads
D. Design basis wind pressure

The DTS reinforced concrete wall thickness is primarily dictated by shielding requirements. For calculation of stresses, the design is dominated by the design basis tornado (DBT) and safe shutdown earthquake (SSE) loads (Reference Section 8A.1.6.1). Other loads are much smaller. In general, loads which are clearly not limiting are not evaluated; brief checks are included on less obviously unimportant loads.

A summary of the results of the following analysis is provided in Table 8.2-2. The results for the combinations of the following loads are provided in Table 2.2-3. The stress results for the various bolts listed in Tables 8.2-2 and 8.2-3 are not calculated here, but are calculated using the same methods described below.

Section Properties

The following table summarizes the section properties of the DTS concrete structure. All dimensions used to compute these properties are taken from the engineering drawings listed above. The coordinate system used is such that the x direction is parallel to the rail tracks, the y direction is perpendicular to the rail tracks, and the z direction is vertical. These section properties are used throughout structural evaluation of the DTS concrete superstructure.

Table 8A.1-4
Section Properties of the Concrete Structure

Component	Base of DTS excluding Wing Wall	Base of DTS including Wing Wall	Base of Basemat
Min Cross Sectional Area	36,864 in.^{2}	41,184 in.^{2}	513,264 in.^{2}
Shear Area for Lateral Load in x direction	27,288 in.^{2}	27,288 in.^{2}	-
Shear Area for Lateral Load in y direction	14,706 in.^{2}	19,080 in.^{2}	-
Moment of Inertia, I_{xx}	451,902,856 in.^{4}	608,800,377 in.^{4}	-
Moment of Inertia, I_{yy}	586,059,999 in.^{4}	736,020,024 in.^{4}	-
Eccentricity (between CG of DTS and CG of basemat)	-	-	126 in.

A. Dead Loads (D)

Base of DTS Structure (excluding wing wall) (D)

The compressive stress of the reinforced concrete wall due to dead weights and operating loads are calculated in the following way. Individual weights are taken from Table 8A.1-3.

Weight of concrete excluding wing wall	= 2,137,024 lb.
Weight of roof structure	= 575,824 lb.
Weight of protective cover	= 222,913 lb.
Weight of mezzanine floor structure	= 36,780 lb.
Equipment weight at cover plate	= 27,260 lb.
Equipment weight at mezzanine	= 49,500 lb.
Weight of fuel handling crane	= 22,000 lb.
Weight of sliding door	= 85,000 lb.
Total axial load	= 2,703,813 lb.

Minimum cross section area (excluding wing wall):

$$A = 379\times280 - 307\times208 - 150\times36 = 36,864 \text{ in.}^2$$

S = compression stress = 2,703,813 lb. / 36,864 in.^{2} = 73.35 psi

Base of Wing Wall (D)

Weight of wing wall	= 112,500 lb.
Weight of sliding door	= 85,000 lb.
Total axial load	=197,500 lb.

Axial Stress in wing wall due to vertical load

$$= 197,500 / (120\times36) = 45.72 \text{ psi.}$$

Assuming an eccentricity of 6 inches from the face of the wall,

Axial stress due to eccentricity = $85,000(6 + 18)\times6 / (120\times36^2) = 78.70$ psi.

Maximum axial stress due to (D) = 45.72 + 78.70 = 124.4 psi.

Foundation Pressure (D)

Total Weight including basemat = 5,377,063 lb.

Moment due to eccentricity of DTS = 340,680 Kin.

Uniform pressure = 10.467 psi. (1,509 psf.)

Distributed pressure due to moment = 4.593 psi.

Max foundation pressure = 15.07 psi. (2,170 psf.)

Minimum foundation pressure = 847.1 psf.

B. Live Loads (L)

Base of DTS Structure (excluding wing wall) (L)

Live load on cover plate = 142,715 lb.

Live load on mezzanine floor = 100,253 lb.

Total live load = 142,715 lb. + 100,253 lb. = 316,662 lb.

Maximum axial stress due to live load = 316,662 / 36,864 = 8.59 psi.

Base of Wing Wall (L)

There are no live loads that act on the wing wall.

Foundation Pressure (L)

Total live load from DTS superstructure = 316,662 lb. (316.7 Kip)

Live load on basemat = 513,264$\times$(250/144) = 891,083 lb.

Moment due to live load = 316,662$\times$126 = 39,904.2 Kin.

Uniform pressure due to live loads = 2.351 psi. (338.8 psf.)

Distributed pressure due to moment = 0.538 psi. (77.5 psf.)

Max foundation pressure = 2.889 psi. (416.3 psf.)

C. Thermal Loads (T)

The thermal analysis of the concrete building is also evaluated. Thermal loads within the structure due to the presence of the fuel assemblies and operating equipment induce two effects in the concrete walls.

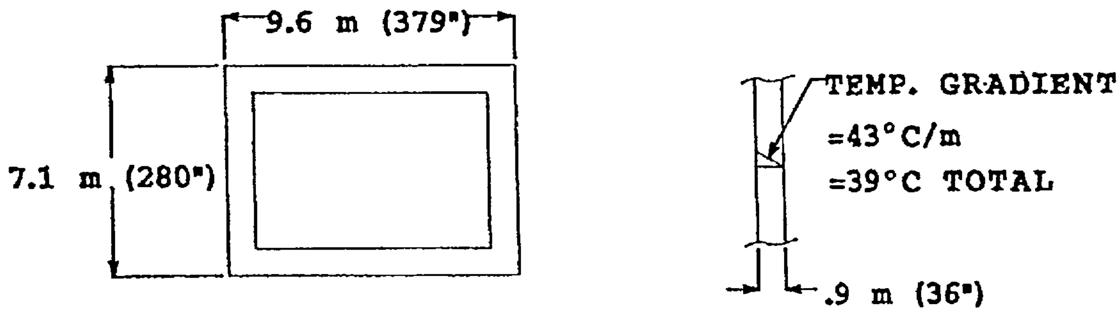
- Bending due to temperature gradients across the walls
- Expansion due to rise in bulk temperature above base (setting) temperature

Thermal loading is considered to induce horizontal forces/moments in the walls due to restraint provided by adjacent orthogonal walls.

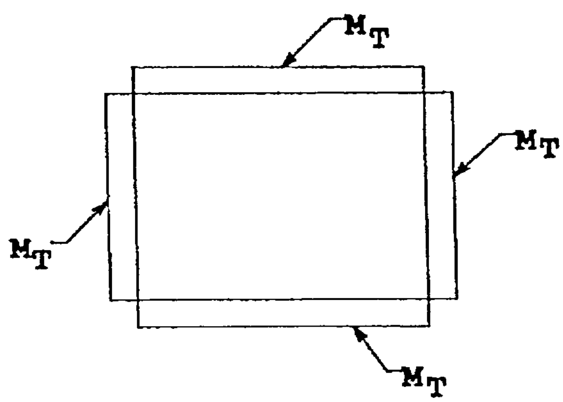
Thermal loads are assessed in accordance with ACI-349-1R (Reference 8A.1.8-1) and are calculated below. These calculated bending moments are to be combined with other loads for reinforced concrete wall design. The wall expansion due to bulk rise in temperature is found to be negligible.

Table 8A.1-5
Thermal Stress Analysis Diagram

(a) TEMPERATURE CROSS WALL EFFECTS



(b) BENDING MOMENT DIAGRAM DUE TO THERMAL LOAD



$$M_T = E\,\alpha\,\Delta T\,\frac{bd^2}{12}$$

39 °C×1.8 + 32 = 102.2 °F
$E = 3.12\times10^6$ psi.
$\alpha = 5.5\times10^{-6}$ in/in/°F
$T = 102.2$ °F
$d = 36 - 3 = 33$ in.

$M_T = (3.12\times10^6)(5.5\times10^{-6})(102.2)(12)(33^2)/12 = 1{,}909{,}836$ lb.in./ft. = 1,909.8 Kin./ft.

D. Internal Pressure

E. Wind Load (150 mph) (W)

Concrete Walls (W)

Moment calculation using simplified method (150 mph wind at the 40' critical elevation):

Design Pressure $p = q_z\, G\, C_p - q_h\, G\, C_{pi}$
(q_z, G, C_p, and C_{pi} are defined in ANSI/ASCE 7-95, Table 6-1)

Therefore, $p = 46.85 + 73.53 \times 0.18 = 60.09$ psf

Maximum Moment $= w\, L^2/12 = 60.09\,(28.6)^2/12 = 4{,}095.9$ ft-lbs/ft
$= 49.15$ in-kip/ft

Maximum moment at the midspan $= 60.09 \times 28.6^2/24 = 2047.95$ lbsft/ft $= 24.58$ Kin/ft.

Maximum shear $= 60.09 \times 25.6/2 = 769.2$ lb/ft

Maximum Shear Stress $= 769.2/12/(36\text{-}3) = 1.94$ psi.

Total horizontal load in the wall panel (307 in. $\times$ 224 in, panel)
(conservatively assume uniform pressure of 60.09 psf)
$= 60.09 \times 307 \times 224/144/1000 = 28.7$ Kip

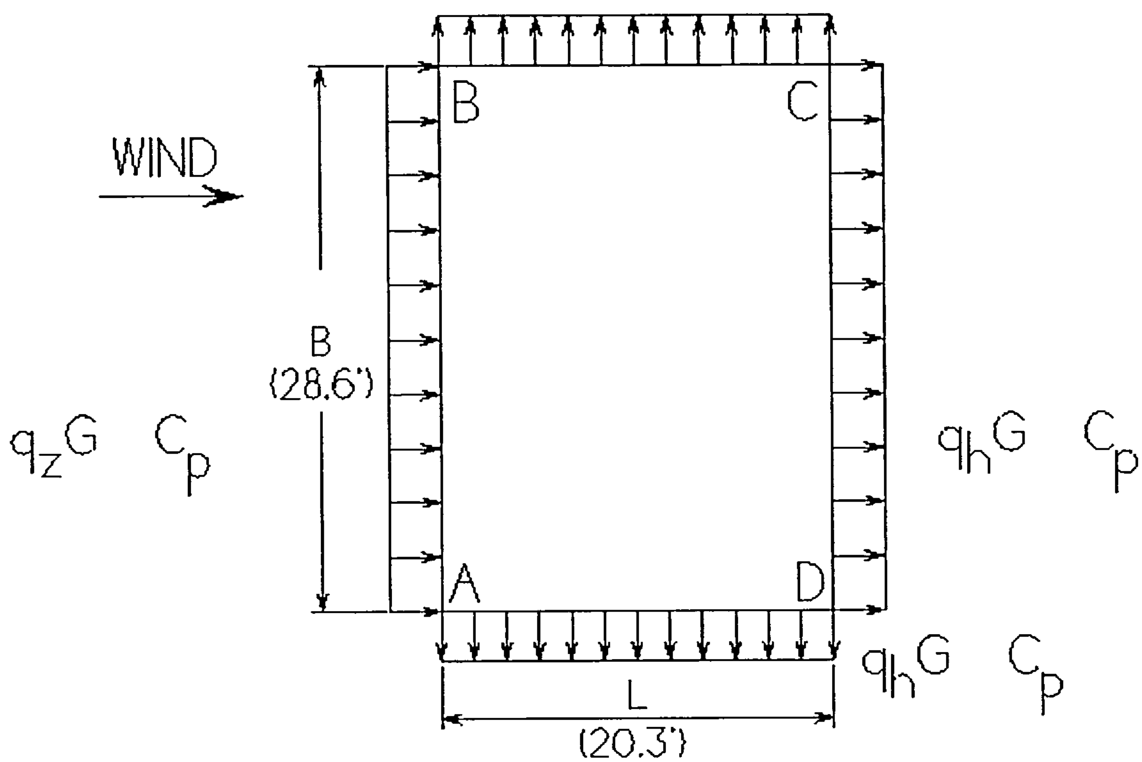
Moment calculation using moment distribution method
(initially calculated for 140 mph wind at 40 ft. elevation)

Windward wall pressure $= q_z\, G\, C_p = 60.01 \times 0.85 \times 0.8 = 40.81$ psf

Leeward wall pressure $= q_h\, G\, C_p = -64.05 \times 0.85 \times 0.43 = -23.41$ psf

Side wall pressure $= q_h\, G\, C_p = -64.05 \times 0.85 \times 0.7 = -38.11$ psf

The following sketch depicts the wind loading configuration.



Fixed end moment $M_{AB} = M_{BA} = wL^2/12 = 40.81(28.6)^2/12 = 2781.7$ ft-lb

Fixed end moment $M_{BC} = M_{CB} = M_{AD} = M_{DA} = -38.11(20.3)^2/12 = -1308.7$ ft-lb

Fixed end moment $M_{CD} = M_{DC} = -23.41(28.6)^2/12 = -1595.7$ ft-lb

Stiffness values of wall $\propto$ 1/span length

$K_{AB} = K_{CD} \propto 1/28.6 = 0.035$

$K_{AD} = K_{BC} \propto 1/20.3 = 0.0493$

Distribution factor at joint A

$D_{AB} = 0.035/(0.035 + 0.0493) = 0.415$

$D_{AD} = 0.049/(0.035 + 0.0493) = 0.585$

Based on the moment distribution calculated in the following table, the maximum moment at fixed end is:

$M = 1996.9$ ft-lb/ft = 23.6 in-kip/ft

Considering internal pressure; p=52.34 psf vs. 40.81 psf

The maximum hogging moment at fixed end is:

$M_{max\ hogging} = 23.6\ (52.34/40.81) = 30.27$ in-kip/ft

The maximum sagging moment at middle span is:

$M_{sagging} = (wL^2/8) - 1573.8 = 40.81\ (28.6)^2/8 - 1573.8 = 2598.8$ ft-lb/ft
= 31.19 in-kip/ft

$M_{max.\ sagging} = 31.19\ (52.34/40.81) = 40.0$ in-kip/ft

For 150 mph. wind:

Total sagging moment = $M_{max\ sagging} = (150/140)^2 \times 40.00 = 45.92$ Kin./ft.

Total moment at ends (hogging moment) = $M_{max\ hogging}$
= $(150/140)^2 \times 30.27 = 34.75$ Kin./ft.

Moment Distribution

Joints	A		B		C		D	
Member	AD	AB	BA	BC	CB	CD	DC	DA
Stiffness	0.0493	0.035	0.035	0.0493	0.0493	0.035	0.035	0.0493
Distribution Factor	0.585	0.415	0.415	0.585	0.585	0.415	0.415	0.585
Fixed End Moment	1308.7	2781.7	-2781.7	-1308.7	1308.7	-1595.7	1595.7	-1308.7
Balance	-2392.3	-1698.1	1698.1	2392.3	167.9	119.1	-119.1	-167.9
Carry forward	-83.95	849.1	-849.1	83.95	1196.2	-59.6	59.6	-1196.2
Balance	-447.5	-317.6	317.6	447.5	-664.8	-471.8	471.8	664.8
Carry forward	332.4	158.8	-158.8	-332.4	223.8	235.9	-235.9	-223.8
Balance	-287.3	-203.9	203.9	287.3	-268.8	-190.8	190.8	268.8
Carry forward	134.4	102.0	-102.0	-134.4	143.7	95.4	-95.4	-143.6
Balance	-138.3	-98.1	98.1	138.3	-139.8	-99.2	99.2	139.8
Final Moment	-1573.8	1573.8	-1573.9	1573.9	1966.9	-1966.9	1966.9	-1966.9

Concrete Structure at Base (W)

For 150 mph wind along the x-direction

Maximum shear stress = 91,512/27,288 = 3.35 psi.

Maximum axial compressive/tensile stress
for face A-D = $2{,}624{,}071 \times 12/2{,}730{,}660 = \pm 11.53$ psi.

for face B-C = $2{,}624{,}071 \times 12/3{,}565{,}320.5 = \pm 8.83$ psi.

For 150 mph wind along the y-direction

Maximum shear stress = 123,880/14,760 = 8.39 psi.

Maximum axial stress = $3{,}551{,}868 \times 12/3{,}184{,}133 = \pm 13.39$ psi.

Foundation Pressure (W)

150 mph wind along x-direction

Maximum foundation pressure $3{,}081{,}676 \times 12/74{,}166{,}648$
$= \pm 0.050$ psi. $= \pm 71.8$psf.

150 mph wind along y-direction

Maximum foundation pressure = $4{,}171{,}269 \times 12/50{,}642{,}048$
$= \pm 0.99$ psi. $= \pm 142.3$ psf.

8A.1.5.3 Protective Cover Structural Analysis

Chapter 4, Section 4.2.3.2 provides the detailed description of the protective cover structure. The configuration details including dimensions are shown on Drawing No. 3039-17, Rev. 0.

A summary of the results of the following analysis is provided in Table 8.2-4a. The results for the combinations of the following loads are provided in Table 2.2-4b. The stress results for the various bolts listed in Tables 8.2-4a and 8.2-4b are not calculated here, but are calculated using the same methods described below.

A. Dead Loads (D)

1.5 inch Roof Plate (D)

The shell stress in the plate is evaluated using Roark, Table 26, assuming the plate is fixed on all sides with a uniform load over the entire surface (Reference 8A.1.8- 10).

$$\text{Max}\,\sigma = \frac{-\beta_1 q b^2}{t^2} \qquad\qquad \text{Max}\,y = \frac{\alpha q b^4}{E t^3}$$

where:

q = design load
$a = 120$ in.
$b = 90$ in.
$a/b = 1.33$
$\beta_1 = 0.4182$
$E = 29\times10^6$

For dead loads, $q = 0.29\times1.5 = 0.435$ psi., therefore,

$$Max\sigma = \frac{(0.4182)(0.435)(90)^2}{(1.5)^2} = 654.9 \text{ psi.}$$

$$\text{Maximum Deflection, } y = \frac{(0.0213)(0.435)(90)^4}{(29\times10^6)(1.5)^3} = 0.0062 \text{ in.}$$

Protective Cover Support Beams (D)

The maximum span of the W12×20 beams is 369 – 12 = 357 inches.

Tributary load, $w = 0.435{\times}(90/3){\times}2 + 120/12 = 36.1$ lb./in.
$= 36.1{\times}12 = 433.2$ lb./ft.

Maximum moment $= wL^2/8 = 433.2(29.752)/8 = 47{,}926.1$ ft.lb.
$= 47{,}926.1{\times}12/1000 = 575.1$ Kin. $= M$

Maximum reaction load = 433.2×29.75/2 = 6,444.0 lb.

For W12×120, $S_x = 163$ in.^{3} and $I_x = 1070$ in.^{4}

Maximum Bending Stress = 575.1/163 = 3.53 Ksi.

Maximum Deflection $= \delta = (5wL^4) / (384EI)$
$= (5{\times}433.3{\times}(29.75{\times}12)^4) / (12{\times}384{\times}29{\times}106{\times}1070) = 0.246$ in.

W12×120 Columns (D)

Column height, $H = 107$ in. = 8.92 ft.

Maximum reaction load at top of column = 6,440.0 lb.

Maximum reaction load at bottom of column = 6,440 +1.5×0.29×107×125 +8.92×120
= 13,333 lb. = 13.33 Kip.

For W12×120: $A = 35.3$ in.^{2}, $I_y = 345$ in.^{4}, $S_y = 56.0$ in.^{3}

Maximum axial stress = 13.33/35.3 = 0.38 Ksi.

B. Live Loads (L)

1.5 inch Roof Plate (L)

For live loads, $q = 250$ psf. = 1.736 psi., therefore,

$$Max\sigma = \frac{(0.4182)(1.736)(90)^2}{(1.5)^2} = 2{,}613.8 \text{ psi.} < 21{,}600 \text{ psi.}$$

$$\text{Maximum Deflection, } y = \frac{1.7361(654.9)}{0.435} = 0.0247 \text{ in.}$$

Protective Cover Support Beams (L)

w = 250 psf. = 1.7361×90×2/3 = 104.17 lb./in. = 1,250 lb./ft.

Maximum bending stress = (1,250/433.2)(3.53) = 10.19 Ksi.

Maximum Deflection = 1250×0.246/433.2 = 0.710 in.

W12×120 Columns (L)

Reaction loads from ceiling beams = 1,250.0×29.75/2 = 18,594 lb. = 18.59 Kip.

Maximum axial stress = 18.59/35.3 = 0.53 Ksi.

C. Thermal Loads (T)

The protective cover is a free standing structure which permits free thermal expansion. Therefore, there are no significant thermal stresses.

D. Wind Loads (W)

1.5 inch Roof Plate (150 mph) (W)

Wind load = $q_h GC_p$ = -73.53×1.0 = -73.53 psf. (see Chapter 3)

Maximum bending stress = 73.53 / (144×0.435) × 654.9 = -768.8 psf.

Maximum deflection = (73.53/144)×(1/0.435)×0.0062 = -0.0073 in.

Wind internal negative pressure = $q_h GC_p$ = 73.53×0.18 = 13.24 psf.

Maximum bending stress = 13.24 / 64.05 × 669.6 = 138.4 psi.

Protective Cover Support Beams (W)

Since upward wind loads are balanced by the dead weight of the plate and the beam, only downward pressure is evaluated. The downward pressure = 13.24 psf. (see Chapter 3)

Tributary load = 13.24/144 × 90×(2/3) = 5.52 lb./in. = 66.2 lb./ft.

Maximum bending stress = -2,117.6/433.2 × 3.53 = - 17.26 ksi.

1.5 inch Wall Plate (W)

From ANSI/ASCE 7-95, the design pressure, $p = qGC_p - q_h(Gc_{pi})$

Q_zGC_p = 50.00 psf. For 150 mph wind

$GC_{pi} = \pm 0.18$, q_h = 73.53 psf.

Design pressure = 50.00 + 0.18×73.53 = 63.24 psf.

For a = 125 in., b = 107 in., a/b = 1.17

From Table 26 of Reference 8A.1.8- 10,

$\beta_1 = 0.3714$, $\beta_2 = 0.1729 < \beta_1$, $\alpha = 0.018$

Maximum bending stress at edge = 63.24 /55.08 × 723 = 830 psi.

Maximum bending stress at center = 63.24/55.08 × 337 = 387 psi.

Maximum deformation = $(0.018 \times 63.24 \times 107^4) / (144 \times 30 \times 10^6 \times 1.5^3) = 0.010$ in.

W12×120 Columns (W)

Tributary load = 125/12 × 63.24 = 658.8 bl./ft.

Maximum moment = $658.8 \times 8.92^2 / 8$ = 6,552.3 ft.lb. = 78.63 Kin.

Maximum bending stress = 78.63/56 = 1.40 ksi.

Maximum deformation = $(5 \times 658.8 \times 107^4) / (384 \times 12 \times 30 \times 10^6 \times 345) = 0.009$ in.

8A.1.5.4 Roof Plate Structural Analysis

Chapter 4, Section 4.2.3.3 provides the detailed description of the roof plate structure. The configuration details including dimensions are shown on Drawing No.3039-17, Rev. 0.

A summary of the results of the following analysis is provided in Table 8.2-4a. The results for the combinations of the following loads are provided in Table 2.2-4b. The stress results for the various bolts listed in Tables 8.2-4a and 8.2-4b are not calculated here, but are calculated using the same methods described below.

A. Dead Loads (D)

7 inch Plate (D)

The shell stress in the plate is evaluated using Roark, Table 26, assuming the plate is fixed on two sides with a uniform load over the entire surface (Reference 8A.1.8- 10).

$$\text{Max}\,\sigma = \frac{-\beta q b^2}{t^2} \qquad\qquad \text{Max}\,y = \frac{-\alpha q b^4}{Et^3}$$

where:

q = design load
a = 102 in.
b = 100.2 in.
a/b = 1.02
β = 0.4272
$E = 29\times10^6$ psi.

From Table 8A.1-3, total equipment loads = 27,260 lb.

Therefore, for dead loads, $q = 0.29\times7.0 + 27{,}260/(343\times2544) = 2.356$ psi.

$$Max\sigma = \frac{(0.4272)(2.356)(100.2)^2}{(17.0)^2} = 206.2 \text{ psi.}$$

$$\text{Maximum deflection, } y = \frac{-(0.0222)(2.356)(100.2)^4}{(29\times10^6)(7)^3} = 0.0005 \text{ in.}$$

Roof Plate Support Beams (D)

Maximum Span = 307 in. = 25.58 ft.

Tributary Load = 2.356(56/3 + 102/3) + 550/12 = 169.9 lb./in.
= 12×169.9 = 2039.0 lb./ft.

Maximum moment = M = 2,039.0(25.58^2)/8 = 166,774 ft.lb.
= 2,001.3 Kin.

For W14×550, A = 162 in.^{2}, I_x = 9,430 in.^{4}, I_y = 2,039 in^4. S_x = 931.0 in.^{3}

Maximum bending stress = 2,001.3/931 = 2.15 Ksi.

Maximum vertical deflection = δ= (5×2039×(25.58×12)^4) / (384×12×29×10^6×2,039)
= 0.072 in.

B. Live Loads (L)

7 inch Plate (L)

For live loads, q = 250 psf. = 1.736 psi., therefore,

$$Max\sigma = \frac{(0.4272)(1.736)(100.2)^2}{(7.0)^2} = 152.0 \text{ psi.} < 21{,}600 \text{ psi.}$$

$$\text{Maximum deflection, } y = \frac{1.736 \times 0.0005}{2.356} = 0.0004 \text{ in.}$$

Roof Plate Support Beams (L)

Tributary load = 1.736(56/3 + 102/3) = 91.4 lb./in.

Maximum bending stress = 1,097/2,039×0.072 = 0.039 in.

Maximum vertical deflection = 1097×0.072/2,039 = 0.039 in.

C. Thermal Loads (T)

The thermal expansion between the roof plate and reinforced concrete wall is calculated as follows:

Assume roof plate temperature = 130^{0}F
Assume concrete temperature = 70^{0}F

$\alpha_{steel} = 6.5 \times 10^{-6}$ in./^{0}F

$\delta_{steel} = 307\ (130\text{-}70) \times 6.5 \times 10^{-6} = 0.1197$ in.

$\delta_{concrete} = 0$ (conservative)

Both the roof plate and beam are bolted to the reinforced concrete. 1-1/4 in. diameter oversized holes (1-in. bolt) are provided at the plate and beam connection points to allow free thermal expansion.

D. Wind Loads (W)

7" Plate (W)

Wind internal negative pressure = qhGCpi = 73.53×0.18 = 13.24 psf. = 0.092 psi.

Maximum bending stress = 0.092/2.356 × 206.2 = 8.0 psi.

Roof Plate Support Beams (W)

Wind internal negative pressure = 0.092 psi.

Maximum bending stress = 0.092/1.736 × 1.16 = 0.06 ksi.

8A.1.5.5 Mezzanine Plate Structural Analysis

Chapter 4, Section 4.2.3.4 provides the detailed description of the mezzanine plate structure. The configuration details including dimensions are shown on Drawing No. 3039-17, Rev. 0.

A summary of the results of the following analysis is provided in Table 8.2-4a. The results for the combinations of the following loads are provided in Table 2.2-4b. The stress results for the various bolts listed in Tables 8.2-4a and 8.2-4b are not calculated here, but are calculated using the same methods described below.

A. Dead Loads (D)

1.5 inch Thick Floor Plate (D)

The shell stress in the plate is evaluated using Roark, Table 26, assuming the plate is fixed on three sides with a uniform load over the entire surface (Reference 8A.1.8- 10).

$$\sigma = \frac{-\beta_3 q b^2}{t^2}$$

where:

q = design load
a = 112 in.
b = 102 in.
a/b = 1.098
β_3 = 0.6212

From Table 8A.1-3, total equipment loads = 49,500 lb.

Therefore, for dead loads, $q = 0.29 \times 1.5 + 49{,}500/(307 \times 208) = 1.2102$ psi.

$$Max\sigma = \frac{(0.6212)(1.2102)(102)^2}{(1.5)^2} = 3{,}476.2 \text{ psi.}$$

Mezzanine Floor Support Beams (D)

Effective Span = 307 - 2×7.5 = 292 in. = 24.33 ft.

Tributary load = 1.2102×(50/2 + 112/2) + 120/2 = 108.02 lb./in.
= 1296.3 lb./ft.

Maximum moment = $1296.3(24.33^2)/8$ = 95,944 ft.lb = 1,151.3 Kin.

For W12×120, I_x = 1,070 in.^{4}, S_x = 163 in.^{3}

Maximum bending stress = 1151.3/163 = 7.06 Ksi.

Maximum vertical deflection = $\delta = (5 \times 1296.3 \times 292^4) / (384 \times 12 \times 29 \times 10^6 \times 1070) = 0.33$ in.

B. Live Loads (L)

1.5 inch Thick Floor Plate (L)

Live load = 250 psf. = 1.736 psi.

Maximum bending stress = 1/736/1.2102×3476.2 = 4986.9 psi.

Mezzanine Floor Support Beams (L)

Tributary Load = 1.736(50/2 + 112/2) = 140.6 lb./in. = 1,687.4 lb./ft.

Maximum bending stress = 1,687.4/1,296.3×7.06 = 9.19 Ksi.

C. Thermal Loads (T)

For thermal expansion, the required minimum clearance between the end of the plate and the inside surface of the concrete wall is approximately 0.125"(3 mm) (Reference Section 8A.1.5.4). An adequate clearance is provided between the plate and the concrete wall to permit free thermal expansion under the maximum differential temperatures expected during normal operation. The mezzanine plates are bolted to the support beams and the beams are bolted to the reinforced concrete. The 1-1/4 in. diameter oversized hole has been provided in the support beams and will permit free thermal expansion of the support beams and thus minimize thermal stress.

E. Wind Loads (W)

1.5 inch Thick Floor Plate (W)

Wind internal negative pressure = 0.092 psi.

Maximum bending stress = 0.092/1.2102 $\times$ 3476.2 = 264 psi.

Mezzanine Floor Support Beams (W)

Maximum bending stress = 0.092/1.736 $\times$ 9.19 = 0.49 ksi.

8A.1.5.6 Sliding Door Structural Analysis

Chapter 4, Section 4.2.3.5 provides the detailed description of the sliding door structure. The configuration details including dimensions are shown on Drawing No.3039-5, Rev. 0. The design of the sliding door is based on shielding requirements. For the dead load analysis, the most limiting conditions are considered. By considering the sliding door to be supported at the rails, the weight of the sliding door is conservatively increased by a factor of 1.5.

The weight of the sliding door is 85,000 lbs (Reference Table 8A.1-3). This load is increased by 1.5 to include the handling load. The total design load becomes:

$W = 85{,}000 \times 1.5 = 127{,}500$ lbs

S = Tension Stress = $127{,}500/(133.5 \times 9) = 106$ psi

This is much less than the allowable stress of $0.6 \times S_y = 0.6 \times 36{,}000 = 21{,}600$ psi. Other loads are much smaller; for example, the internal pressure is 2.59 lb/ft^2 (124 Pa), and is much less than the suction on the door due to the tornado wind, which is 419 lb/ft^2 (20,100 Pa). In general, loads which are clearly non-limiting are not considered explicitly; brief checks are included on less obviously unimportant loads. The stress calculations due to the DBT and SSE loads are described in Section 8A.1.6.5)

The sliding door is a free standing structure which permits free thermal expansion. Therefore, there are no significant thermal stresses.

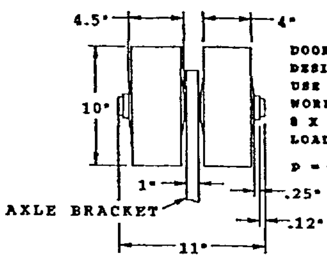
The other components of the sliding door affected by the normal handling loads are door wheels and door wheel axle, axle bracket, door rail, and support bracket. The stresses of these components are calculated in:

a. Door wheels and door wheel axle - Table 8A.1-10
b. Axle bracket - Table 8A.1-11
c. Door rail - Table 8A.1-12
d. Support bracket - Table 8A.1-13

The summary of the analytical results and comparisons with the acceptance criteria define in Chapter 3 are also presented in Table 8A.1-14.

Table 8A.1-10

Sliding Door Wheels and Axle Stress Calculations



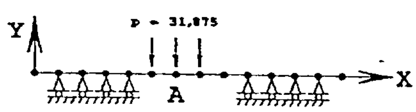
DOOR WEIGHT = 85,000 #
DESIGN WEIGHT = 1.5 X 85,000 = 127,500 #
USE 8" - 10" DROP FORGED STEEL WHEELS
WORKING CAPACITY = 20,000 #
8 X 20,000 # = 160,000 # $>$ 127,500 #
LOAD AT EACH SET OF WHEELS

$$P = \frac{127{,}500}{4} = 31{,}875\ \#$$

(a) CALCULATE THE SHEAR STRESS - AXLE

$$S = \frac{31875}{2(\pi)(0.75)^2} = 9{,}019\text{psi} < 46{,}000\text{psi}$$

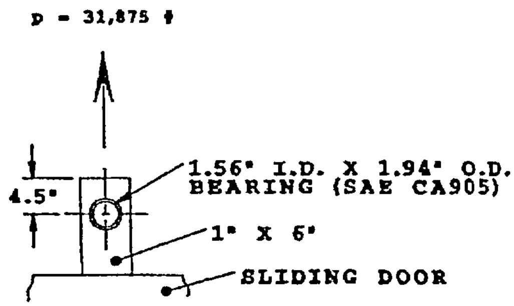
(b) CALCULATE THE BENDING STRESS - AXLE
A ANSYS FINITE ELEMENT MODEL WAS
DEVELOPED USING STIF 16



THE MAXIMUM BENDING STRESS AT LOCATION A
IS 19,108psi, WHICH IS LESS THAN
THE ALLOWABLE STRESS OF 69,000psi

Table 8A.1-11

Sliding Door Axle Bracket Stress Calculations



(a) CALCULATE THE BEARING STRESS

$$S = \frac{31875}{1 \times 1.5} = 21{,}250\text{psi} < 45{,}000\text{psi}$$

(b) CALCULATE THE SHEAR STRESS

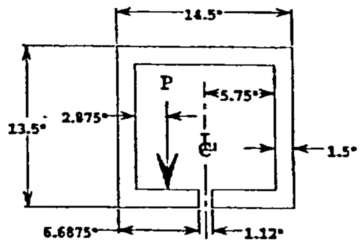
$$S = \frac{31875}{(4.5-2) \times 1 \times 2} = 6{,}375\text{psi} < 40{,}000\text{psi}$$

(c) CALCULATE THE TENSION STRESS

$$S = \frac{31875}{1 \times (6-2)} = 7{,}969\text{psi} < 60{,}000\text{psi}$$

Table 8A.1-12

Sliding Door Rail Stress Calculations



TOTAL LENGTH OF DOOR RAIL = 355" (REF. DWG. 1051-5)
ASSUME ONLY 133.5" (SAME AS THE WIDTH OF THE DOOR)
WILL SUPPORT THE WEIGHT OF THE DOOR.

$P = \frac{1.5 \times 85{,}000}{2} = 63{,}750$ #

M = BENDING MOMENT = $63{,}750 \times (2.875 + 0.75) = 231{,}094$ IN-#

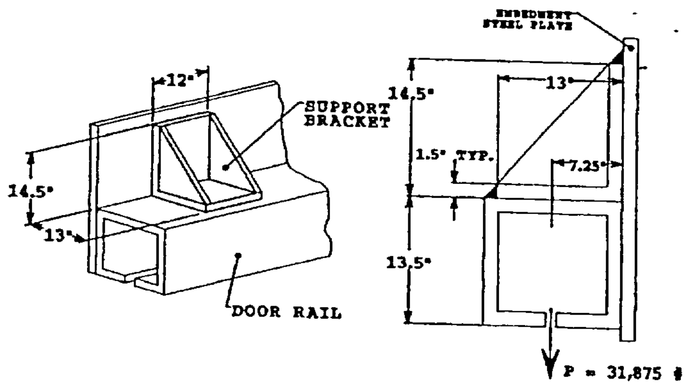
$I = \frac{bh^3}{12} = \frac{133.5\,(1.5)^3}{12} = 37.5$ IN^4

S = BENDING STRESS $= \frac{Mc}{I} = \frac{231{,}094 \times 0.75}{37.5}$

$= 4{,}622\text{psi} < 69{,}000\text{psi}$

Table 8A.1-13

Sliding Door Support Bracket Stress Calculations



ASSUME EACH SET OF DOOR WHEELS WILL BE SUPPORTED BY ONE SET OF THE BRACKET

(a) TENSION STRESS AT WELD (3 SIDES) BETWEEN BASE PLATE AND DOOR RAIL

$$\sigma = \frac{31{,}875}{(13 \times 2 + 12) \times 1.5 \times 0.707} = 791\text{psi}$$

(b) SHEAR STRESS BETWEEN BRACKET AND EMBEDMENT STEEL PLATE

$$\sigma = \frac{31{,}875}{(14.5 \times 2 + 12) \times 1.5 \times 0.707} = 733\text{psi}$$

(b) BENDING STRESS AT SUPPORT BRACKET

$$I = I_1 + I_2 = \frac{12 \times 1.5^3}{12} + 2\left(\frac{1}{2} \times \frac{1.5 \times 13^3}{12}\right) = 278 \text{ IN}^4$$

$$\sigma = \frac{Mc}{I} = \frac{31{,}875 \times 7.25 \times 7.25}{278} = 6{,}027\text{psi} < 60{,}000\text{psi}$$

Table 8A.1-14

Sliding Door and Major Components Stress Analysis Results Summary (Dead Loads and Handling Loads)

Comp.	Calculated Stress			Allowable Stress		
	Tension	Bending	Shear	Tension	Bending	Shear
Door	106			21,600		
Door Wheel	127,500 (Load)			160,000 (Capacity)		
Wheel Axle		18,108	9,019		69,000	46,000
Axle Bracket	7,969		6,375	60,000		40,000
Bracket Bearing		21,250 (Bearing)			45,000 (Bearing)	
Support Bracket	791	6,027	733	60,000	60,000	40,000
Door Rail		4,622			69,000	

8A.1.6 Accident Loads Structural Analysis

Table 8A.1-15 shows the accident loads for which the DTS structural components are designed. The table also lists the individual components which are affected by each loading. In the following sections, each accident condition is analyzed to demonstrate that the requirements of the applicable codes are met and that adequate safety margins exist for the DTS design.

Table 8A.1-15

DTS Accident Loads Identification

	Affected Component				
Load Type	**Reinforced Concrete Structure**	**Protective Cover**	**Roof Plate**	**Mezzanine Plate**	**Sliding Door**
Seismic Load	X	X	X	X	X
Tornado Wind Load	X	X			X
Tornado Missiles	X	X			X

8A.1.6.1 Reinforced Concrete (Building) Structural Analysis

As described in Section 8A.1.5.2, the design of the concrete structure is dominated by shielding requirements. Analyses are also performed to evaluate the effects of tornado wind, tornado missiles and seismic loads.

A. Tornado Wind Load

The 360 mph tornado wind load analysis is based on the methods used to evaluate the 150 mph normal wind load. The force, moment and stress results from the 150 mph wind load analysis are multiplied by a factor of $(360/150)^2 = 5.76$ to acquire the results for the 360 tornado wind loads. These results are listed in Table 8.2-2.

A differential pressure of 3 psi generated by the tornado winds is also assumed to act on the DTS structure. A structural analysis for a 3 psi differential pressure is performed using the same classical engineering methods used to evaluate the normal wind loads. These results are also listed in Table 8.2-2.

B. Tornado Missiles

1. Local Effects of Tornado Generated Missiles

The side walls of the reinforced concrete are 36 inches thick (914 mm). The walls are

designed to provide adequate radiation shielding and they easily meet the minimum acceptable barrier thickness requirements for local damage against tornado generated missiles, specified in Chapter 3.0. Nevertheless, in order to demonstrate the adequacy of the DTS design for tornado missiles, detail analysis of the concrete wall has been performed and presented in Section 3.2.1.4. The items evaluated include the resistance to penetration, spalling, scabbing and perforation for a postulated missile impact.

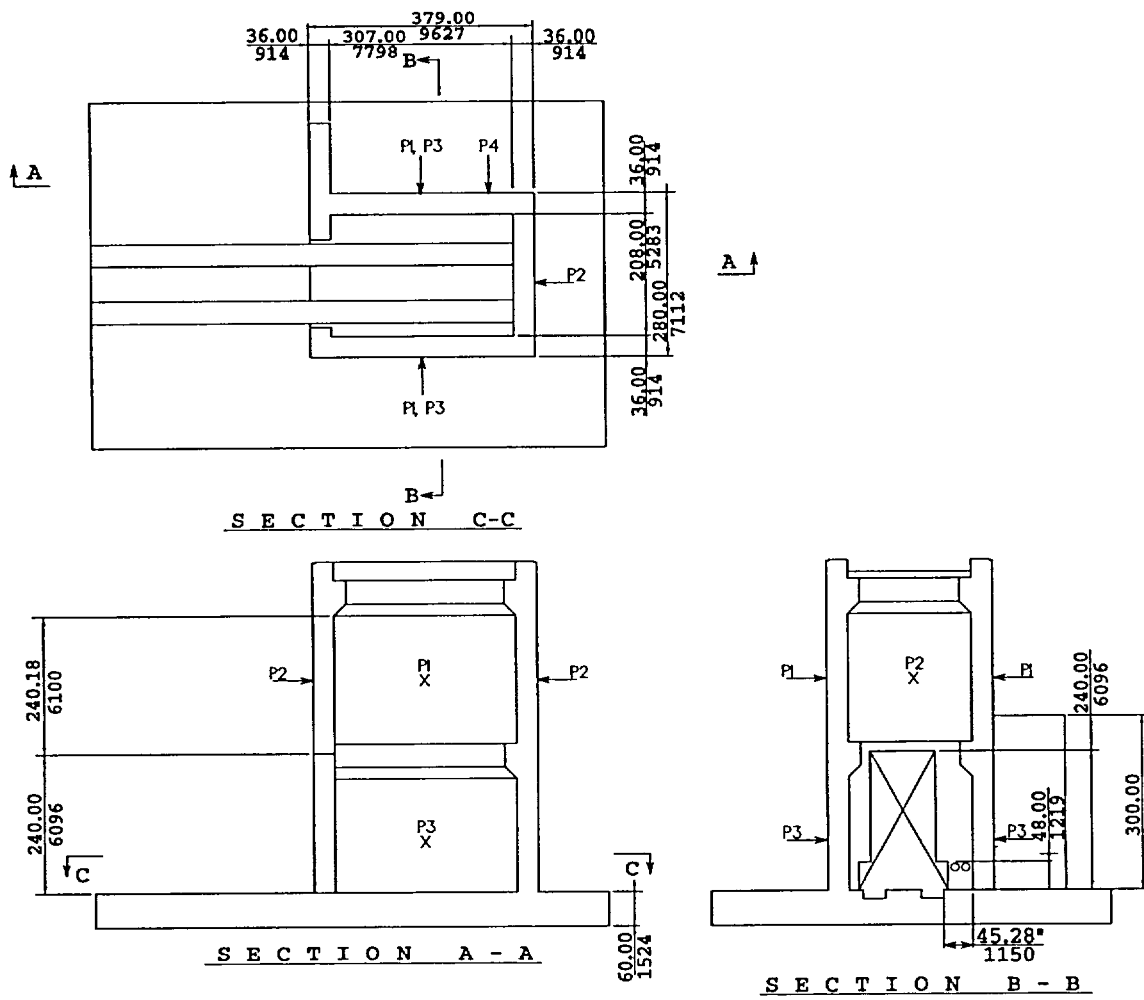
Based on the analysis shown on Section 3.2.1.4, the actual wall thickness of DTS (36 inches) is greater than the thickness required for protection against penetration, perforation and scabbing. Therefore during a tornado generated missile event, there will be penetration of the concrete walls but no perforation or scabbing of it. Local damage to the outer surfaces of the structure will not compromise their confinement capability. Local repair to the structure will be performed if required after a missile impact.

2. Global Effects of Tornado Generated Missiles

Using the maximum tornado generated missile impact force, the structural integrity of the DTS concrete structure has been evaluated for global structural effects.

Refer to Figure 8A.1-2 for critical missile impact locations. P1 through P4 are the critical (typical) missile impact locations.

Figure 8A.1-2 Critical Tornado Missile Impact Locations



Missile Impact at P1:

Max. Impact Force, $P = 704$ kips (for 12" Sch. 40 Pipe Impact)

From R.J. Roark "Formulas for Stress Strain" fifth Edition, Table 26, Case 8b (Reference 8A.1.8-10).

For $a = 307$ in. $b = 240$ in. $a/b = 1.279$ $\beta_1 = -0.04285$ $\beta_2 = 0.9210$

$r = 12.75/2 = 6.375$ in. $\nu = 0.24$

Max. stress at center $= f_b = 3P/2\pi t^2\{(1+0.24)\, ln(2 \times 240/\pi \times 6.375) - 0.0429\}$
$= 1.8603\ P/t^2 = 1.8603 \times 704/t^2 = 1309.7/t^2$

Max. stress at long edge $= 0.921\ P/t^2 = 0.921 \times 704/t^2 = 648.4/\ t^2$

Section Modulus, $S = 1.0\ (t)^2/6$

Equivalent Moment at Center $= 1309.7/t^2 \times t^2/6 = 218.28$ kip-in/in
$= 218.28$ Kip ft/ft $= 2619.3$ kip-in/ft

Equivalent Moment at Edge $= 648.4/\ t^2 \times t^2/6 = 108.1$ kip-ft/ft $= 1297$ kip-in/ft

Missile Impact at P2:

Max. Impact Force, $P = 704$ kips (for 12" Sch. 40 Pipe Impact)

For $a = 240$ in. $b = 208$ in. $a/b = 1.154$ $\beta_1 = -0.1149$ $\beta_2 = 0.8617$

Max Stress at Center $= 3P/2\pi\ t^2\ \{(1+0.24)\ ln(2 \times 208\ /\ \pi \times 6.375) - 0.1149\}$
$= 1.7412P/\ t^2 = 1.7412 \times 704/\ t^2 = 1225.8/\ t^2$

Max Stress at Edge $= 0.8617 \times P/\ t^2 = 0.8617 \times 704/\ t^2 = 606.7/\ t^2$

Equivalent Moment at Center $= 1225.8/\ t^2 \times t^2/6 = 204.3$ kip-in/in or kip- ft/ft
$= 2451.6$ kip-in/ft < 2619.3 kip-in/ft

Equivalent Moment at Edges $= 606.7/\ t^2 \times t^2/6 = 101.11$ kip-in/in
$= 1213.3$ kip-in/ft < 1297.0 kip-in/ft

Missile Impact at Locations P4:

(Distance $d = 36$" from inside edge of the walls)

Max. Impact Load = 704 kip

Effective Width of Resistance = (12.75 + 2 $\times$ 36 + 36) = 120.75"

Effective Depth, d = 33 in.

Effective Shear Area = 120.75 $\times$ 33 = 3984.75 in^2

Max Shear Reaction Load at Edge = (307-36)/307 $\times$ 704 = 621.4 kip

Max Shear Stress in Concrete = 621.4/3984.75 = 0.156 ksi = 156 psi

Allowable Shear Stress = $2\ \phi (f'c)^{0.5} = 2 \times .085\ (3000)^{0.5}$ = 93.1 psi < 156 psi

There will be local crack in the vicinity of missile impact location. However, as indicated below, the factor of safety > 1 for punching shear stress, therefore, there will not be any missile punching through or collapse of the wall.

Check punching shear stress:

Effective punching shear area = π (36 + 12.75)33 = 5054.0 in^2

Punching Shear Stress = 704/5054.0 = 0.139 ksi = 139 psi

Allowable punching shear stress = $4\ \phi (f'c)^{0.5}$ = 186.2 psi > 139.0 psi ...o.k.

Collapse (Resistance) Load for Wall Section:

From Table 4-3 of BC-TOP-9A Rev.2, "Topical Report Design of Structures for Missile Impact" Bechtel Power Corporation, San Francisco, CA (Reference 8A.1.8-9).

Collapse (Resistance Load) for wall panel = $R = 2\pi\ (M_u^+ + M_u^-)$

Where M_u^+ = Ultimate positive moment capacity
M_u^- = Plastic negative moment capacity

$M_u^+ = M_u^- = \phi \times A_s \times fy \times (d - a/2)$

$\phi = 0.9$
$A_s = A_s$ = #10@ 8" = 1.91 in^2/ft
B = 12"; d = 36-3.0 = 33 in.
f_y = yield stress for Rebar = 60.0 ksi.
$f'c$ = 3.0 ksi
a = 1.91 $\times$ 60 / 0.85 $\times$ 3.0 $\times$12 = 3.74"

Ultimate Moment Capacity $= M_u^+ = M_u^- = 0.9 \times 1.91 \times 60\ (33.0 - 3.74/2)$
$= 3{,}202.6$ kip-in/ft

Collapse (Resistance) load, $R = 2\,\pi\,(3{,}202.6 \times 2)1/12$
$= 3{,}353.7$ kip >> 704 kip

Factor of Safety against collapse of wall due to tornado missile = 3353.7 / 704 = 4.76 > 1.0. o.k

Residual Ultimate Moment Capacity of Wall after Missile Penetration:

Max. depth of penetration = X = 6.26 in (for 12" sch. 40 pipe impact)
Effective depth d = 33 – 6.26 = 26.74 in.

Residual Ultimate Moment Capacity at Mid Span

$M_{ur} = 0.9 \times 1.91 \times 60\,(26.74 - 3.74/2)$
$= 2565.1$ kip-in/ft

Residual Collapse Load Capacity = $2\,\pi\,(2565.1 + 3202.6)\ 1/12$
= 3020 kip >> 704 kip o.k.

Additional structural evaluation using the following loads and load combinations, subsequent to a tornado missile impact, have been considered as per US NRC standard Review Plan "NUREG-0800" Para. 3.3.2:

(i) Wt = Ww
(ii) Wt = Wp
(iii) Wt = Wm
(iv) Wt = Ww + .5 Wp
(v) Wt = Ww + Wm
(vi) Wt = Ww + .5 Wp + Wm

Where: Wt: total tornado load,
Ww: tornado wind load,
Wp: tornado differential pressure load, and
Wm: tornado missile load.

For each particular structure or portion thereof, the most adverse of the above combinations as appropriate, have been used.

These combined effects constitute the total tornado load (Wt) which is then combined with other loads as specified in the TSAR.

Tornado Wind Load (Ww):

For critical section at elevation 40',
Design pressure $p = q_z\, GC_p - q_h GD_{pi} = 269.86 + 76.26 = 346.13$ psf

For fixed end boundary conditions at ends A and B, Max. Moment at Ends of Wall Panel
$= pl^2/12 = 346.12\,(28.6)^2/12 = 23{,}592.7$ lb- ft/ft. = 283.1 kip-in/ft

Maximum Moment at Mid Span $= pl^2/24 = 1/2 \times 283.1 = 141.6$ kip-in/ft

Max Shear Force = 346.12 $\times$ 28.6 / 2 = 4949.5 lb.

Max Shear Stress = 4949.5/12(36-3) = 12.5 psi

Total horizontal Load on Wall Panel = (307 $\times$ 224 / 144) $\times$ (346.12 / 1000) = 165.3 kip

Tornado Differential Pressure Load (Wp):

Design Pressure p = 3 psi = 432 psf

Max. Moment at ends = 432 $(28.6)^2/12$ = 29446.6 lb-ft/ft=353.4 kip-in/ft

Max. Moment at mid span = 1/2 (353.4) = 176.7 kip-in/ft

Max shear force = 432 $\times$ 28.6 / 2 = 6177.7 lb.

Max shear stress = 6177.6 / 12(36-3) = 15.60 psi

Total Horizontal load on Wall panel = (307 $\times$ 224 / 144) $\times$ (432 / 1000) = 206.3 kip

From the results shown in the following summary table, it can be seen that tornado missiles will cause local damage to the walls in the vicinity of the impact locations. However, there will not be any missile punching through or collapse of the wall as indicated by factor of safety > 1 for punching shear stress. The residual ultimate moment capacity of the wall damaged by missile impact is also greater than the moments resulting from the continuing tornado wind pressure effects. Therefore, it can be concluded that although there will be local damage in the vicinity of the missile impact, the missile will not punch through. The walls will continue to resist the subsequent tornado wind loadings even after the damage, and the overall structural integrity of the DTS structure will be maintained.

Table 8A.1-16
Maximum Stresses due to Tornado Loads (Wt)

Maximum Stresses due to Tornado Loads (Wt)						
Critical component	Tornado wind (Ww)	Tornado Differential pressure (Wp)	Tornado missile* (Wm)	Ww+0.5Wp	Ww+Wm	Ww+0.5Wp +Wm
Max. moment at ends (kip-in/ft)	283.1	353 4	1297	459 8	1580 1	1756.8
Ultimate (allowable) moment (kip-in/ft)	3202.6	3202.6	3202.6	3202.6	3202.6	3202 6
Factor of safety	11.31	9.06	2.47	6.97	2 03	1.82
Max moment at mid span (kip-in/ft)	141 6	176.7	2619 3	230 0	2760 9	2849.3
Ultimate (allowable) moment (kip-in/ft)	2565 1*	2565 1*	3202.6	2565.1*	3202 6	3202.6
Factor of safety	18 12	14 52	1.22	11 15	1.16	1.12
Max beam shear stress (psi)	12.5	15 6	156 0	20.3	168 5	176.3
Ultimate (allowable) shear stress (psi)	93 1	93.1	93.1	93.1	93.1	93 1
Factor of safety	7 45	5.97	0.6[1]	4 59	0 55[1]	0.53[1]
Max. punching shear stress (psi)	-	-	139 0	-	139 0	139 0
Ultimate allowable punching shear stress (psi)	-	-	186.20	-	186 2	186 2
Factor of safety	-	-	1.34	-	1 34	-
Total horizontal load (kip)	165 3	206.3	704	268 4	869.3	972.4
Collapse load (kip)	3020**	3020**	3353.7	3020**	3353 7	3353.7
Factor of safety	18 27	14 64	4 76	11 25	3 86	3 45

*Residual Ultimate moment capacity after missile damage.
**Residual Collapse load after missile damage.

Note 1:
The factor of safety < 1 indicates that there will be local cracks in the vicinity of missile impact location. However, there will not be any missile punching or collapse of the wall as indicated by factor of safety > 1 for punching shear stress. The residual ultimate moment capacity of the wall damaged by missile impact is also greater than the moments resulting from the continuing tornado wind pressure effects. Therefore, it can be concluded that although there will be local damage in the vicinity of the missile impact, the missile will not punch through. The walls will continue to resist the subsequent tornado wind loadings even after the damage, and the overall structural integrity of the DTS structure will be maintained.

C. Seismic Evaluation (SSE)

1. Discussion of the Seismic Analysis

The design basis response spectra of NRC Regulatory Guide 1.60 (Reference 8A.1.8- 6) is selected for the DTS design earthquake as defined in 10CFR72.102. From the Regulatory Guide 1.61 (Reference 8A.1.8- 7) Table 1, a damping value of seven (7) percent of critical damping is used for the reinforced concrete superstructure. The horizontal and vertical components of the response spectra (in Figures 1 and 2, respectively, of the NRC Regulatory Guide 1.60) correspond to a maximum horizontal and vertical ground acceleration of 1.0g. The maximum ground displacement is taken to be proportional to the maximum ground acceleration, and is set at 36 inches for a ground acceleration of 1.0g. The design adequacy of the DTS SSCs, for each specific plant site, will be verified by comparing the DTS design bases SSE spectra with the plant site free field spectra.

NRC regulatory Guide 1.60 also states that for sites with different acceleration values specified for the design basis earthquake, the response spectra used for design should be linearly scaled from Regulatory Guide 1.60, Figures 1 and 2, in proportion to the maximum specified horizontal grounding acceleration. The maximum horizontal ground acceleration component selected for design of the DTS superstructure is 0.25g. The maximum vertical acceleration component selected is two - thirds of the horizontal component which is 0.17g. These ground acceleration values comply with the recommendations of the 10CFR72.102 for sites underlaid by rock east of the Rocky Mountain front, except in the areas of known seismic activity. The input response spectrum for this analysis is shown on Figure 8A.1-4.

An appropriate design basis earthquake (also described as a Safe Shutdown Earthquake) can be represented by three orthogonal translational components (2 horizontal and 1 vertical) consisting of free field ground acceleration response spectra. These ground response spectra represent the maximum acceleration response to the earthquake motion of a series of single degree of freedom oscillators of natural frequency varying between 0.1 Hertz and 33 Hertz.

The seismic analysis of the DTS assumes that the structure is founded on competent rock. In this circumstance, the phenomenon of Soil Structure Interaction (SSI) in which dynamic interaction between the structure and supporting soil medium need not be considered.

Furthermore the structure can be analyzed as fully fixed at the base of the shear walls at the top of the basement.

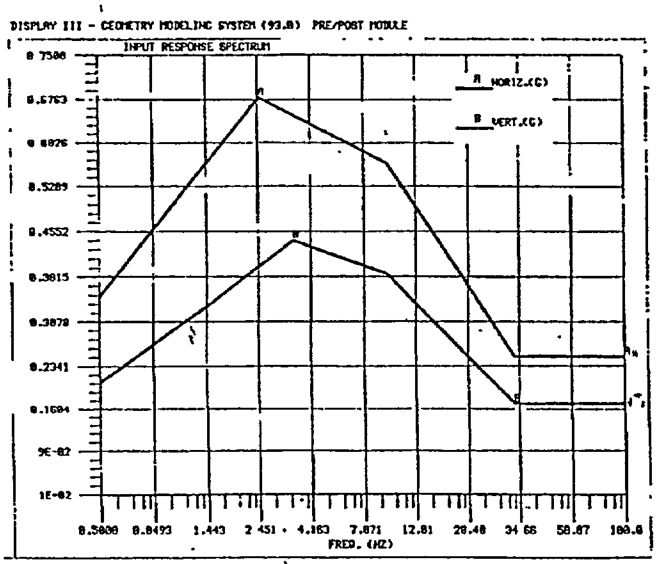
A separate modal analysis has been carried out for each earthquake direction. The results from the 3 runs have been combined using the square root of the sum of the squares method (SRSS). Results for individual earthquake direction analysis have been combined using the Complete Quadratic Combination technique (CQC).

Zero Period Accelerations (ZPA) at different locations are subsequently adjusted manually by adding base input accelerations by SRSS to correct for dynamic mass missing from the modes considered.

Seismic inertia forces at different levels were then calculated from the ZPA values at that level. Structural design analysis of the critical structural components/section was then performed using standard design methods for steel and concrete structures. In these calculations, spatial effects of earthquake components were combined on a SRSS basis. The structural configuration (symmetrical and uniform distribution of mass and stiffness) and load paths of the DTS structural system are such that this analytical procedure is considered conservative and justified.

Figure 8A.1-4

Seismic Analysis - Input Response Spectrum



2. Model Generation

The structure has been modeled for seismic analysis purposes using the computer code ANSYS 4.4A.

The superstructure of the DTS facility comprises a relatively stiff shear wall structure in reinforced concrete supporting plant items and equipment on two flexible internal structural steel floors. In common with normal practice, equipment and internal structural steel floors are assumed not to contribute to the stiffness of the supporting reinforced concrete structure.

A three dimensional plate model of concrete superstructure above top of base, including plant masses was prepared. All reinforced concrete walls have been represented by four-node shell elements with elastic material properties based on gross uncracked concrete sections. Walls have been modeled at center locations throughout.

Rigid equipment is generally represented as lumped translational mass. Internal floors (including the roof level and mezzanine floor) which support major plant items and equipment are flexible in the vertical direction and have been represented as structural beam elements supporting vertical mass elements representing equipment, self weight, and floor imposed loading. The model generation and modeling assumption are shown in the following Figures:

Figure 8A.1-5: Finite element model of reinforced concrete structure
Figure 8A.1-6: Roof floor modeling assumptions
Figure 8A.1-7: Fuel assembly crane modeling assumptions
Figure 8A.1-8: Mezzanine floor modeling assumptions
Figure 8A.1-9: Sliding door and preparation area modeling assumptions

All concrete walls assumed 36" thick. Wall overall center line dimensions assumed 343" × 244" O/A in plan by 570.8" high.

Figure 8A.1-5

Finite Element Model of Reinforced Concrete Structure

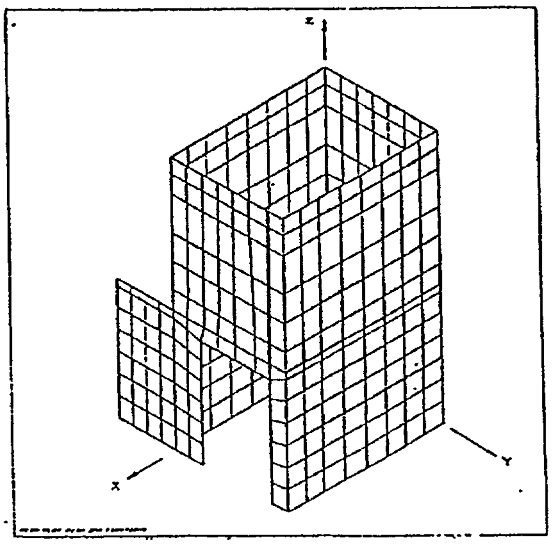


Figure 8A.1-6

Roof Floor Modeling Assumptions

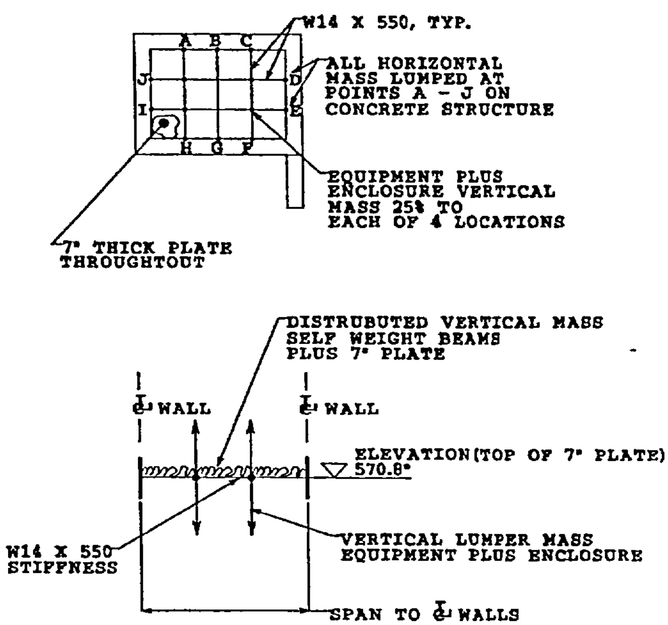
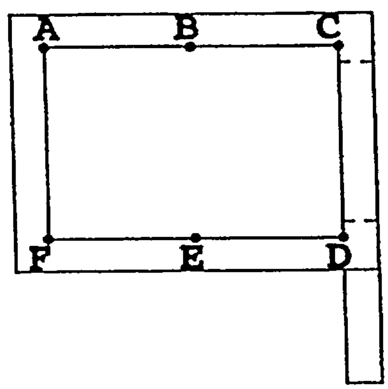


Figure 8A.1-7

Fuel Assembly Crane Modeling Assumptions
(at elevation 310.92 inches)



TOTAL EQUIPMENT MASS = 22,000 LB.
(FUEL ASSEMBLY CRANE)

MASS X,Y,Z DIRECTIONS (22000/6 = 3667 LB.)
APPLIED EQUALLY AT POINTS A-F

Figure 8A.1-8

Mezzanine Floor Modeling Assumptions

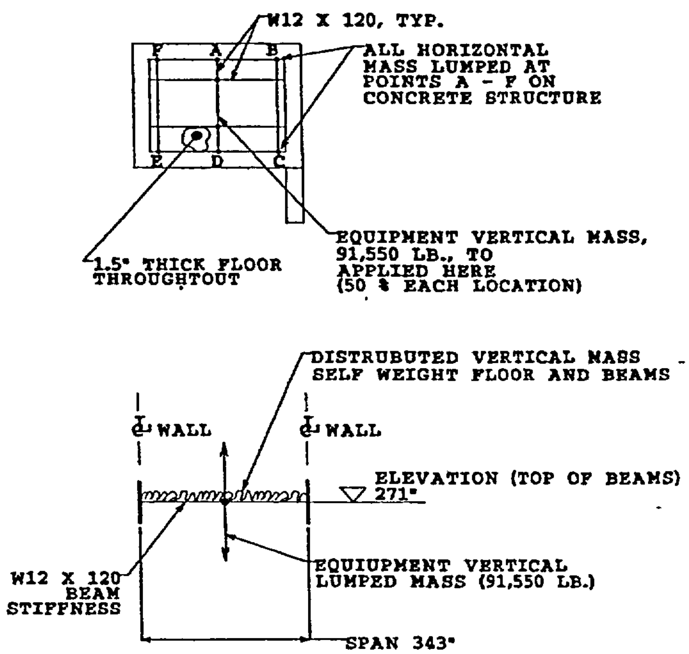
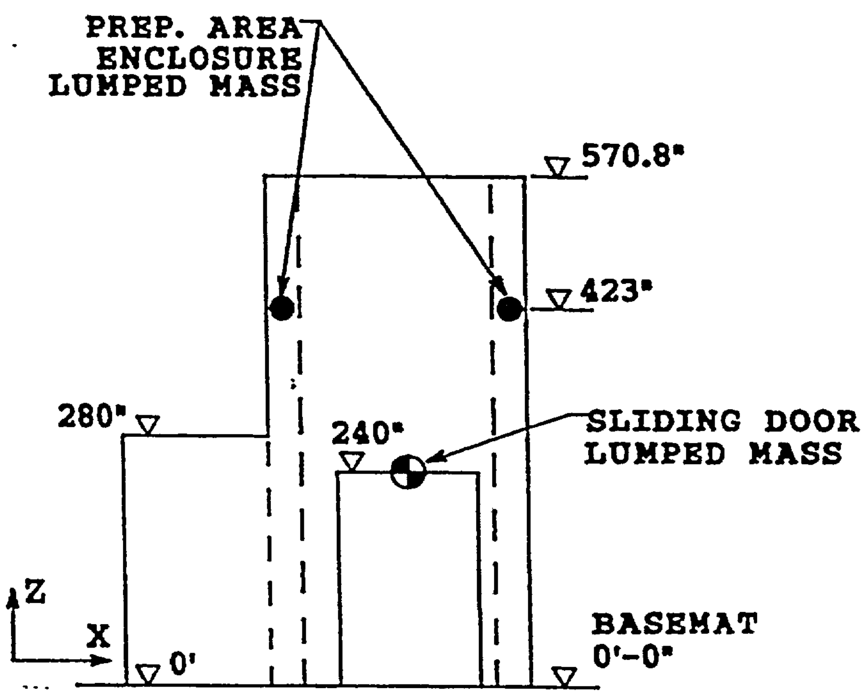


Figure 8A.1-9

Sliding Door and Preparation Area Modeling Assumptions



3. Analysis

A fixed base modal analysis technique was used to predict the structure response (in terms of acceleration) to the design earthquake input motion. A damping level of 7% was used which reflects the overall damping in a reinforced concrete structure stressed to levels approaching yield at the SSE. The DTS concrete structure finite element model includes the DTS concrete structure and the vertical effects of the structural steel beams at the roof plate and mezzanine floor level. The DTS structure dynamic model does not include the protective cover plate structure, plate structure at the roof plate and mezzanine floor level and the lateral rigidity provided by the roof plate and mezzanine structural steel. The results are listed in Table Table 8A.1-20A.

In order to calculate the modes, frequencies, and participation factors of the roof plate, mezzanine plate and their support beams, two ANSYS finite element models are constructed using BEAM4, SHELL63 and MASS21 elements to perform the modal analyses. The results are listed in Tables Table 8A.1-20B and Table 8A.1-20C.

A separate modal analysis has been carried out for each earthquake direction. The results from the 3 runs were combined using the square root sum of the squares method (SRSS). Results for individual earthquake direction analysis have been combined using the Complete Quadratic Combination technique (CQC).

Results from the modal analysis are as follows:

- Mode shapes (typical samples shown in Figures 8A.1-10A, through 8A.1-10G), frequencies and mass participation factors (Tables 8A.1-20A, 8A.1-20B, 8A.1-20C) for all structure modes of vibration up to approximately 50 Hertz. From Table Table 8A.1-20A it can be seen that the first mode at 4.03 Hz with a small participation factor is associated with the vertical vibration of the mezzanine floor beams. Second, fourth, fifth and seventh modes at 9.71, 13.41, 13.83 and 15.95 Hz with large participation factors are associated with the DTS concrete structure.

- Zero Period Accelerations (also known as rigid body accelerations) at selected locations throughout the structure. These represent the maximum acceleration response at the locations in 2 horizontal and the vertical translational directions on the structure.

Zero Period Accelerations (ZPA)are subsequently adjusted manually by adding base input accelerations by SRSS to correct for:

- Dynamic mass missing from the modes considered and

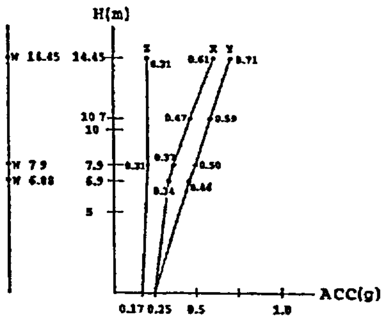
- Base input motion constant acceleration profile.

To produce design acceleration profiles from global seismic effects it is necessary to combine output profiles with the ZPA values (0.25g horiz, 0.17g vert) using the SRSS method to account for fixed base analysis with zero response at 0 m elevation, which should be ZPA value.

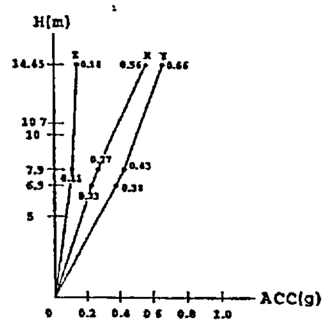
For global considerations, use acceleration values from "Middle Wall" positions. Figure 8A.1-10 shows both the global average seismic accelerations and global average seismic acceleration combined with ZPA. These accelerations are also listed in the Table 8A.1-20 and to be used for the reinforced concrete superstructure design.

Figure 8A.1-10

Wall Peak Accelerations - Global Seismic Effects



GLOBAL AV. SEISMIC ACCELERATION COMBINED WITH ZPA



GLOBAL AV. SEISMIC ACCELERATION

Figure 8A.1-10A
DTS Superstructure Mode Shape (9.709 Hz)

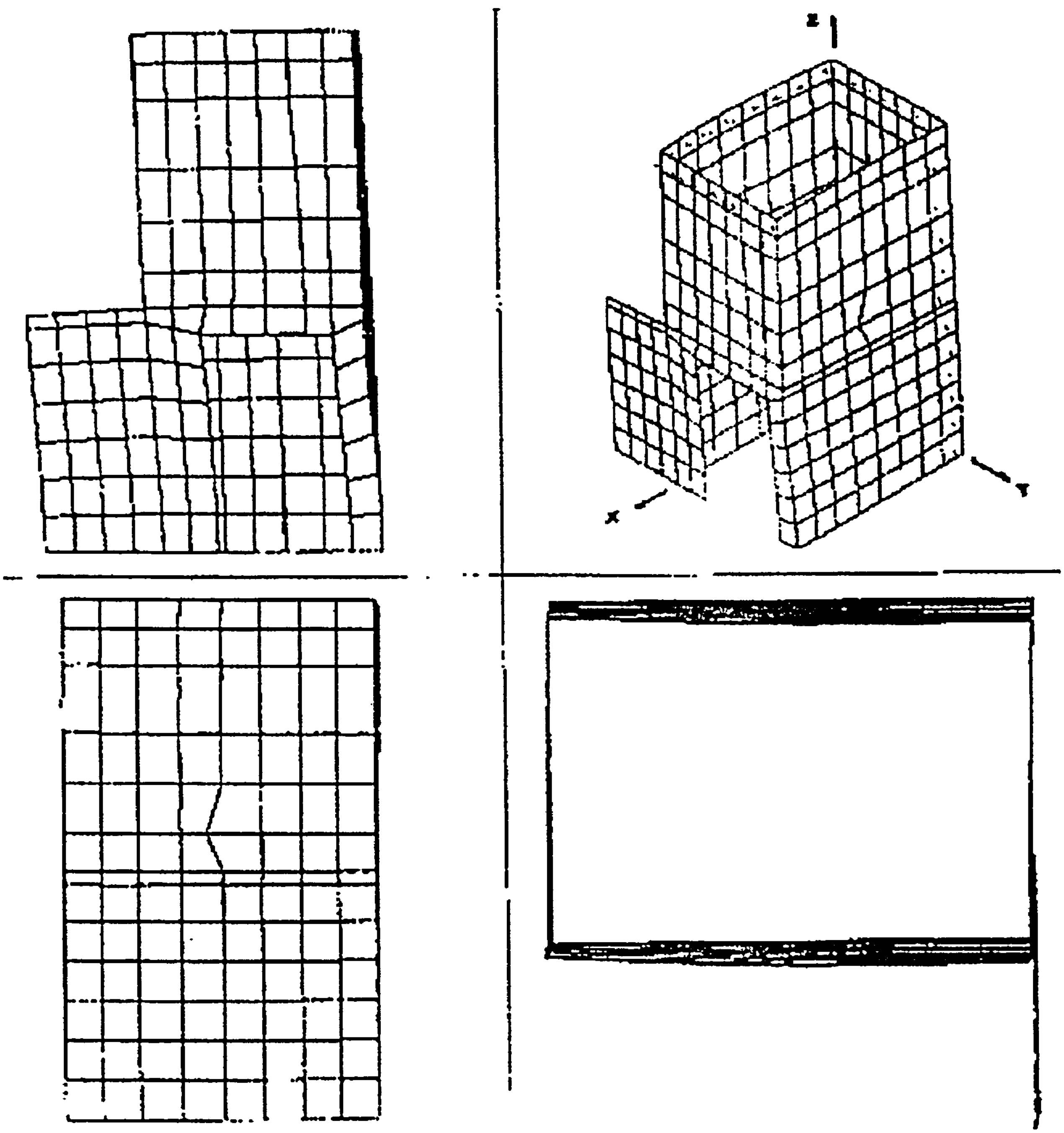


Figure 8A.1-10B
DTS Superstructure Mode Shape (13.411 Hz)

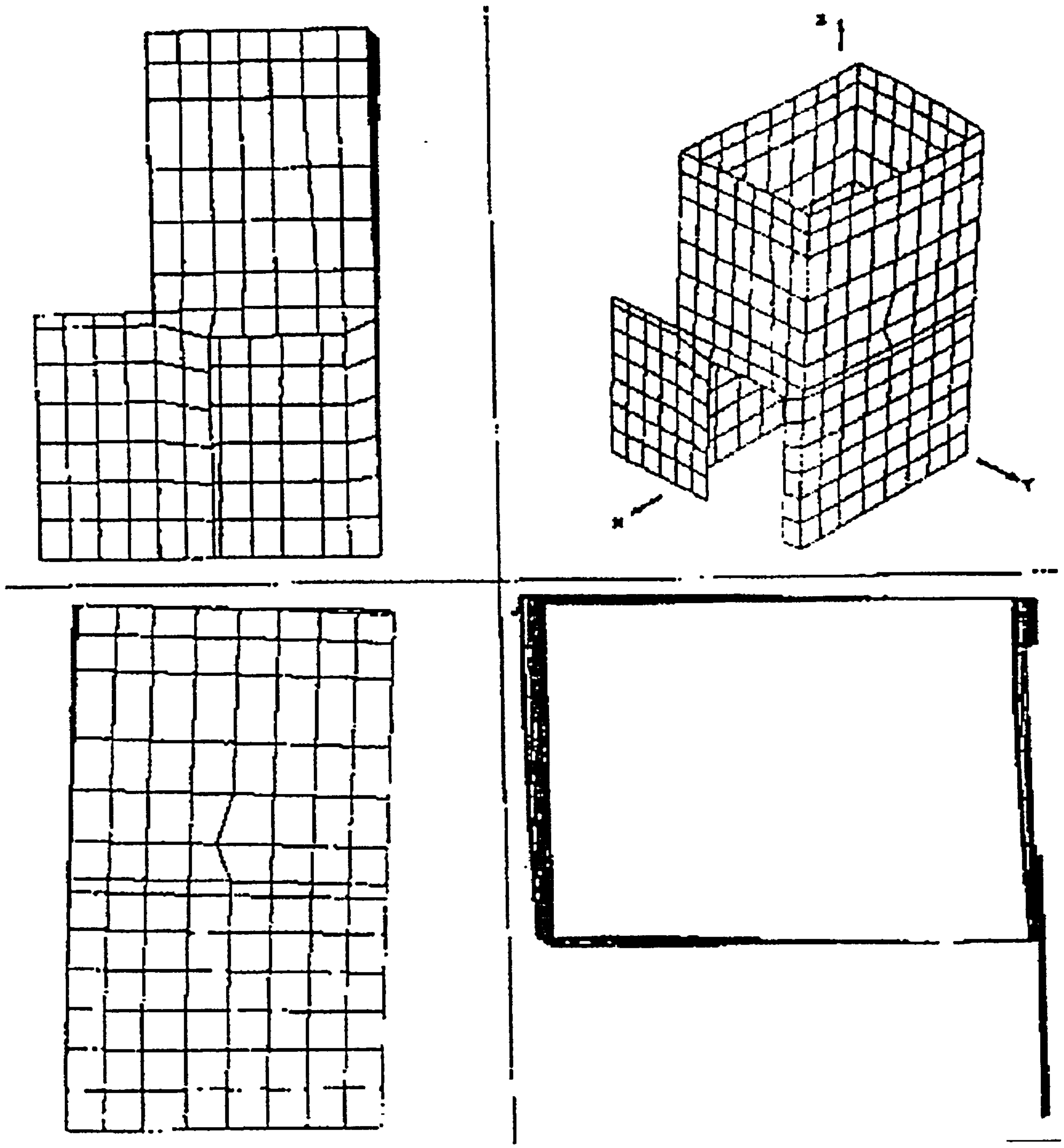


Figure 8A.1-10C
DTS Superstructure Mode Shape (20.244 Hz)

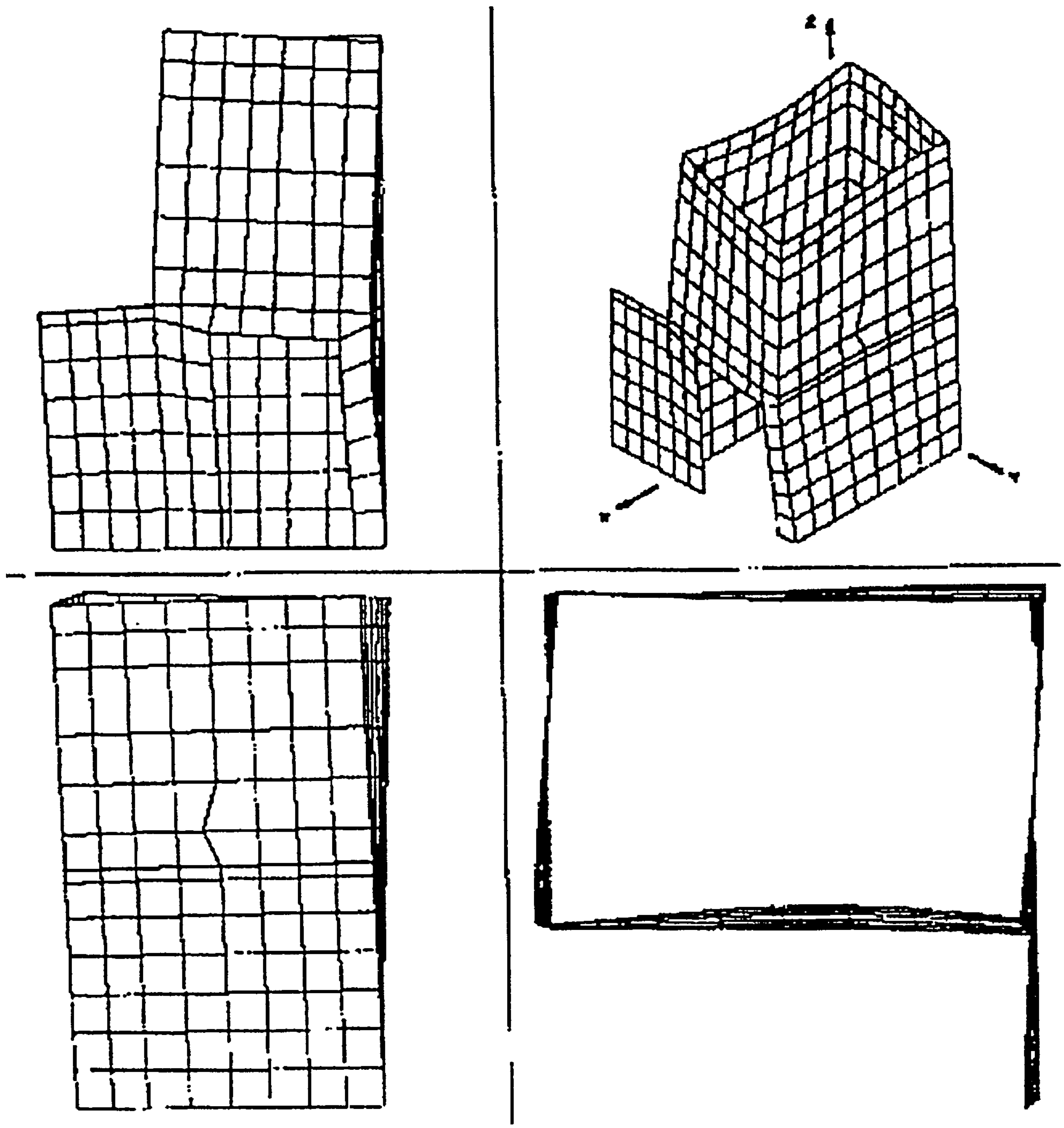


Figure 8A.1-10D
Roof Plate Mode Shape (17.417 Hz)

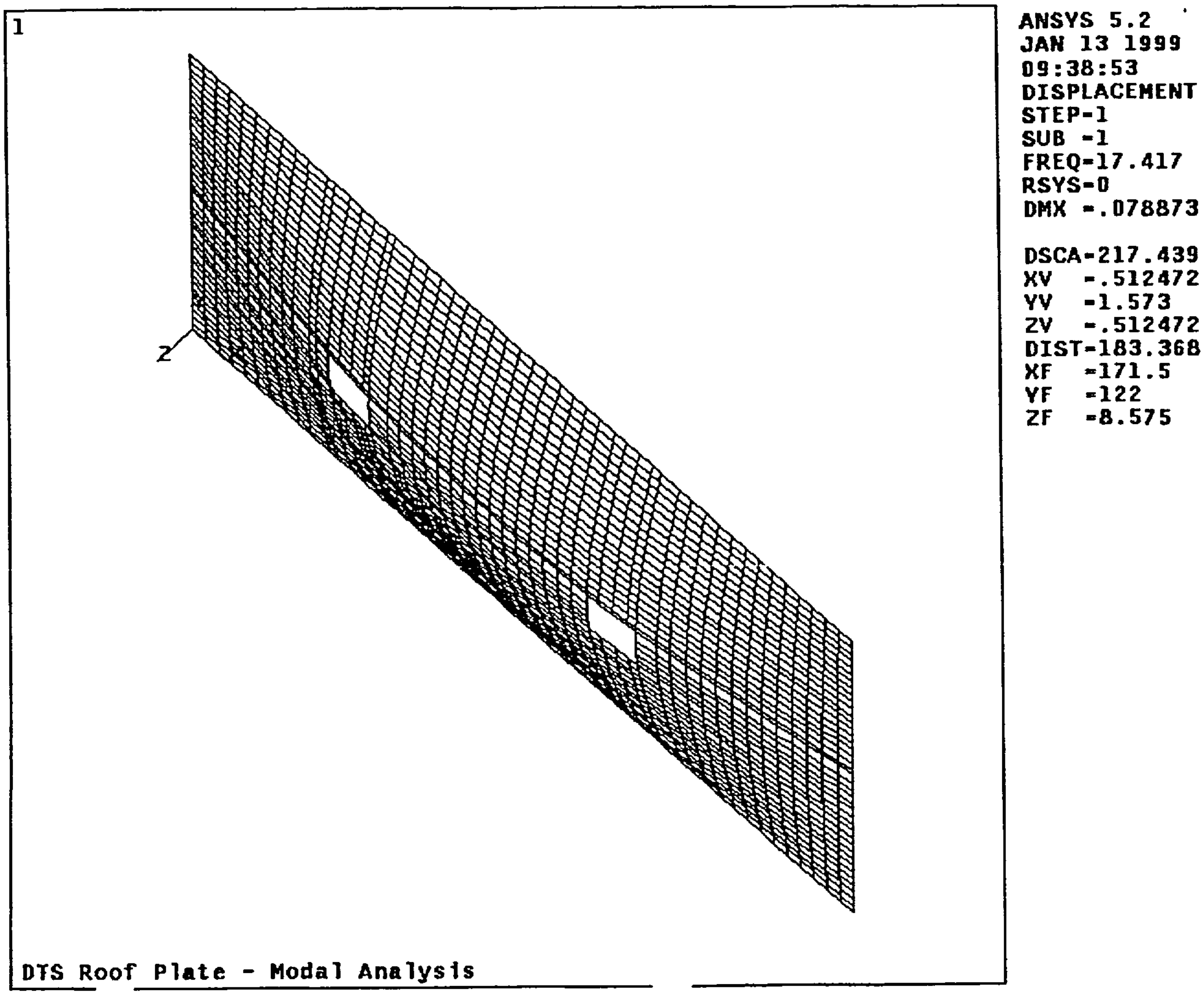


Figure 8A.1-10E
Roof Plate Mode Shape (34.622 Hz)

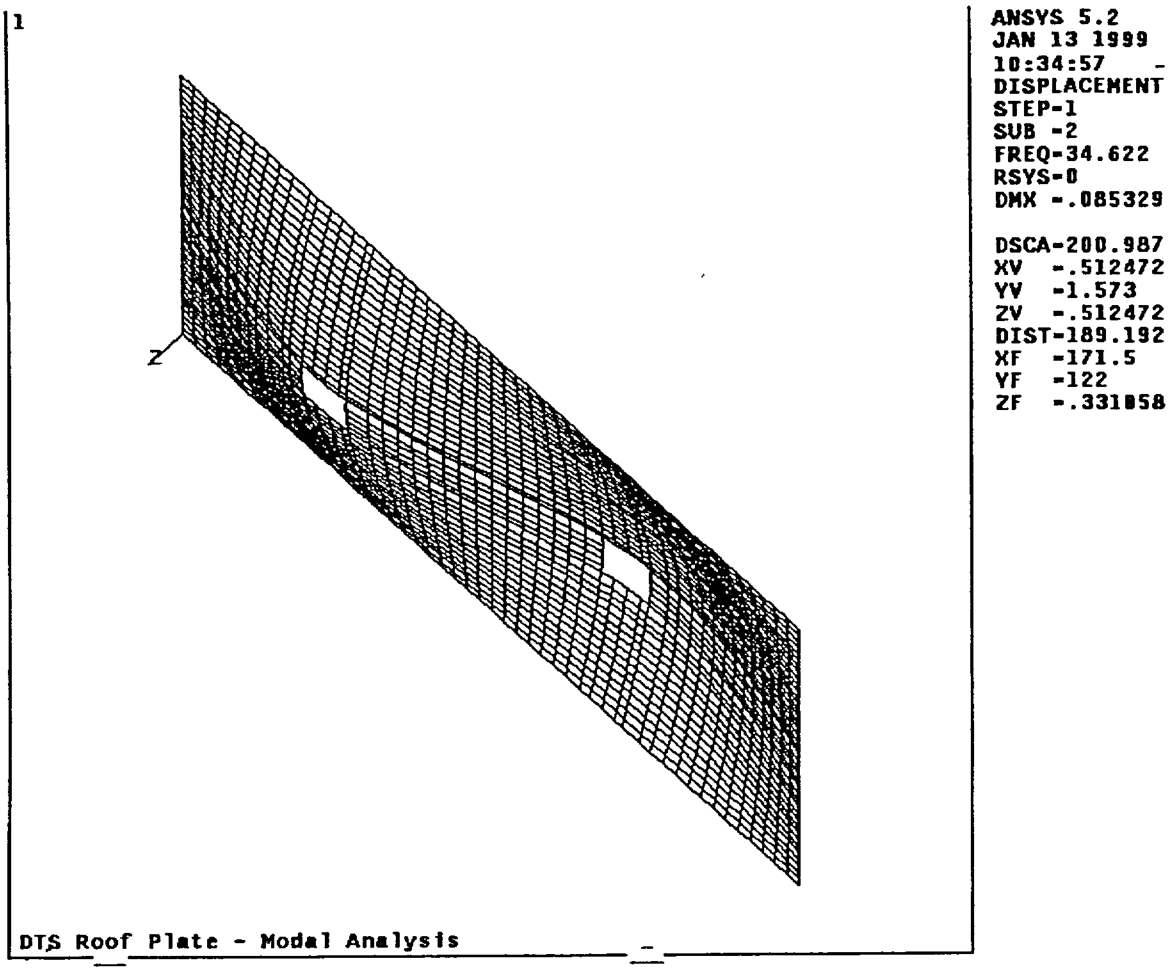


Figure 8A.1-10F
Mezzanine Plate Mode Shape (4.754 Hz)

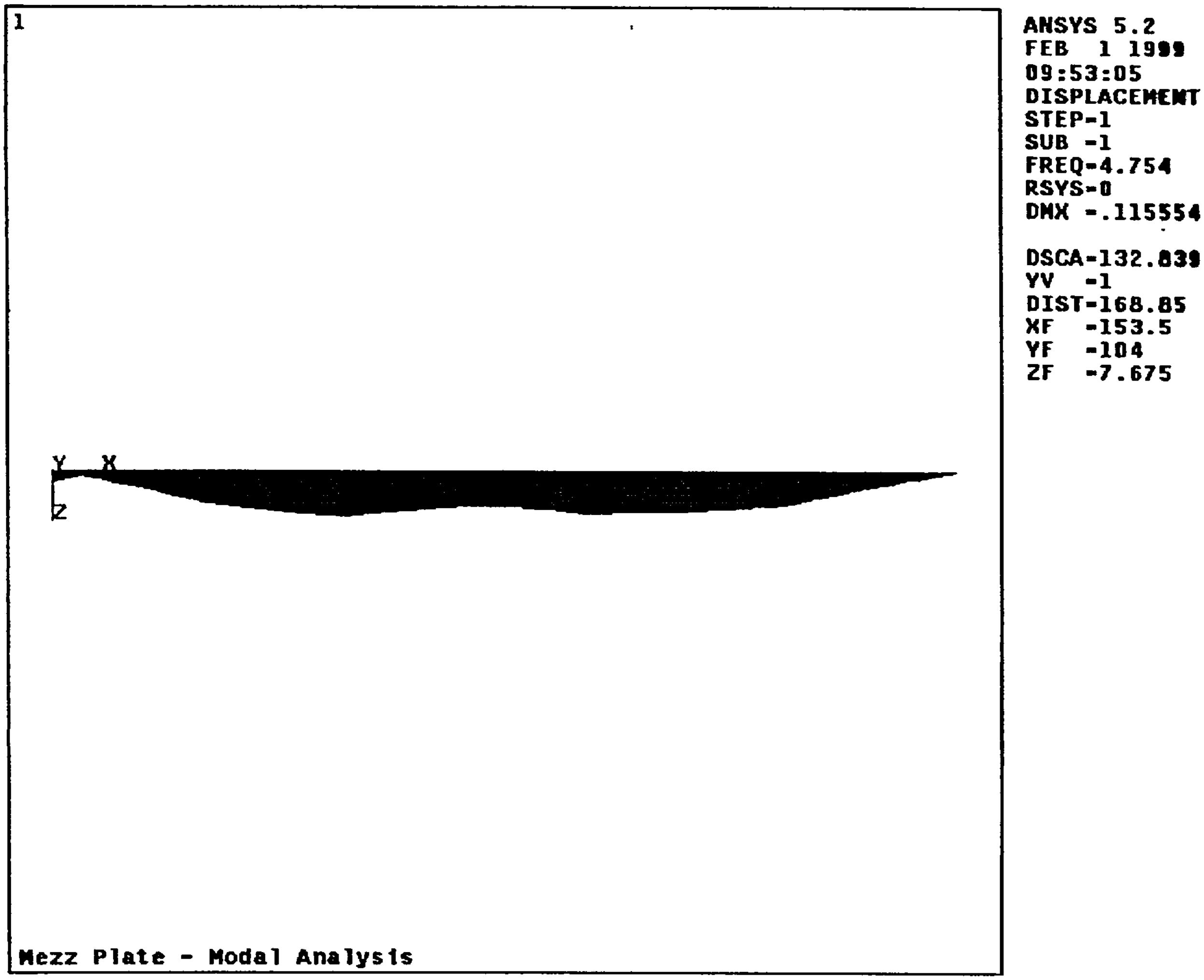
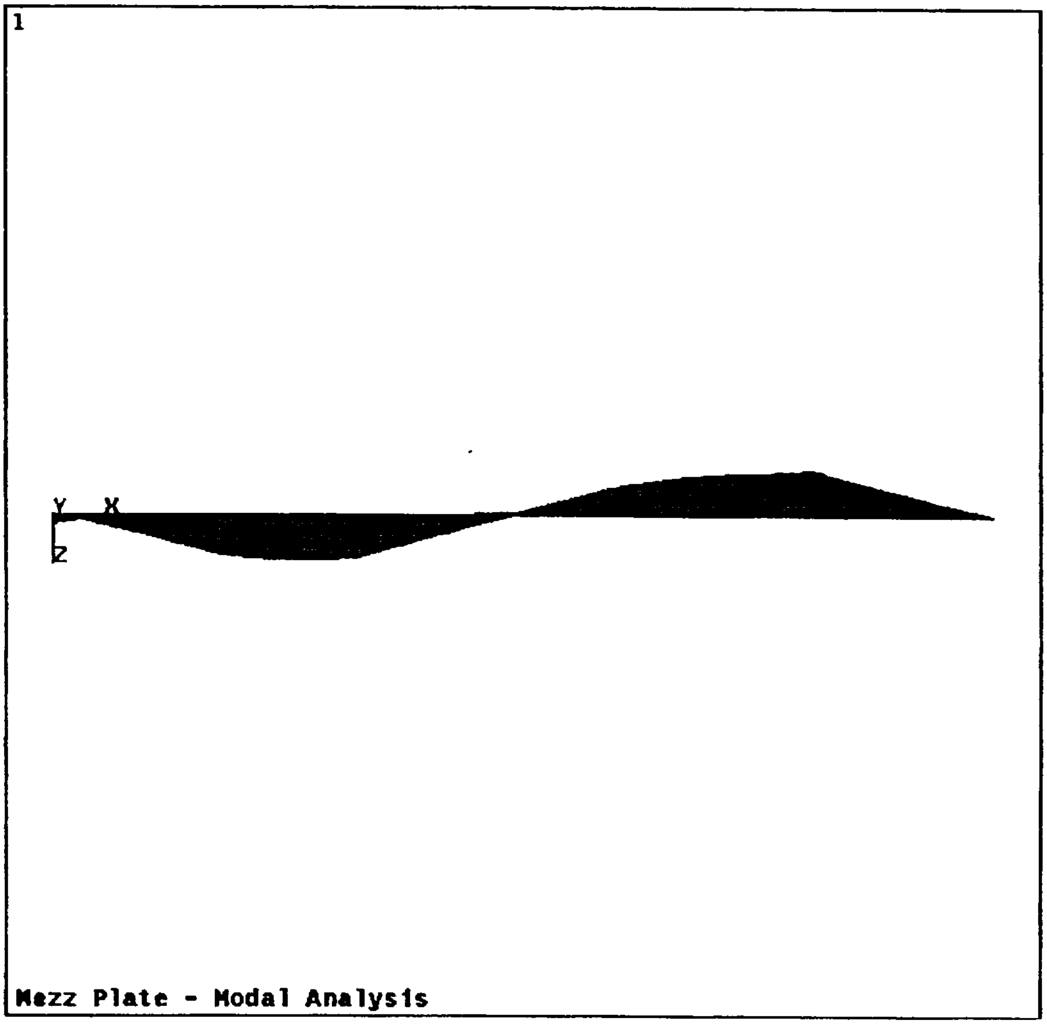


Figure 8A.1-10G
Mezzanine Plate Mode Shape (6.82 Hz)



ANSYS 5.2
FEB 1 1999
09:54:49
DISPLACEMENT
STEP=1
SUB =2
FREQ=6.82
RSYS=0
DMX =.138858

DSCA=110.545
YV =1
DIST=168.85
XF =153.5
YF =104
ZF =.384734

Table 8A.1-20

Global Average Seismic Accelerations Combined With ZPA

Location	Global Av. Seismic Acceleration Combined With ZPA			
	Acel X	Acel Y	Acel Z	
0 m	0.25g	0.25g	0.17g	
6.9 m	0.34g	0.46g	0.17g	
7.9 m	0.37g	0.50g	0.21g	
14.45m	0.61g	0.71g	0.21g	

Table 8A.1-20A
Modes, Frequencies & Participation Factors
(DTS Structure)

MODE	FREQUENCY (Hz)	PARTICIPATION FACTOR
1	4.030	0.5085
2	9.709	-183.1
3	10.52	-14.81
4	13.41	-883.7
5	13.83	-173.2
6	15.61	-55.41
7	15.95	-246.7
8	16.99	1.678
9	20.24	36.07
10	20.42	2.942
11	23.47	-113.1
12	27.61	-21.53
13	28.46	41.05
14	35.29	235.4
15	36.92	-144.5
16	39.11	-251.0
17	40.79	-45.40
18	42.82	68.27
19	44.45	-22.77
20	45.09	-153.3
21	46.19	-28.72
22	48.37	30.97
23	50.11	-1.419
24	50.82	20.00

Table 8A.1-20B
DTS Roof Plate - Modal Analysis Results
Modes, Frequencies & Participation Factors

Mode Number	Frequency (hz)	Participation Factor Z (transverse) Direction
1	17.4	20.5
2	34.6	-0.05
3	54.2	0.46
4	65.0	-6.54

Table 8A.1-20C
DTS Mezz Plate - Modal Analysis Results
Modes, Frequencies & Participation Factors

Mode Number	Frequency (hz)	Participation Factor - Z (transverse) Direction
1	4.8	13.16
2	6.8	-0.46
3	10.3	0.45
4	11.8	-0.64
5	13.0	1.21
6	14.1	0.75
7	17.1	3.19
8	18.1	0.78
9	20.4	2.95
10	21.5	0.65
11	36.9	-0.25
12	46.8	1.39
13	52.4	1.19

Calculation of SSE seismic loads:

Elevation (in.)	Components	Dead & Equipment Lumped Weight (Kip)	Horizontal Inertia Loads (Kip)		Vertical Inertia Load (Kip)
			F_x	F_y	
667	Protective Cover	123.0	75.03	87.33	25.83
560	Roof Plate	250.2	152.62	177.64	52.54
560	Concrete Walls Lumped @ El. 560	282.2	172.1	200.4	59.26
421	Concrete Walls Lumped @ El. 421	580.4	272.79	342.44	121.88
311	Fuel Assembly Crane	22.0	8.14	11.0	4.62
272	Mezzanine Floor	86.28	29.34	39.69	14.67
272	Concrete Mass Lumped @ El. 272	842.7	286.52	387.64	143.26
268.5	Sliding Door	85.0	28.9	39.1	14.45
0.0	Concrete Mass Lumped @ El. 0.0	544.4	136.1	136.1	92.55
-3.0	Foundation	2673.3	668.33	668.33	454.46
	Total	**5,489.5**	**1,830**	**2,089.7**	**983.52**

Total weight of concrete above elevation 0.0 in. = 2,249,836.0 lb.

Distributed weight of concrete above elevation 0.0 in. = 2,249,836 / 562 = 4,003.31 lb./in.

Tributary weight of concrete above elevation 0.0 in. = 4.003 Kip./in.

Concrete mass lumped at elevation 560 in. = $4.003(562\text{-}421)\times(1/2) = 282.2$ Kip.

Concrete mass lumped at elevation 421 in. = $282.2 + 4.003(421\text{-}272)\times(1/2) = 580.4$ Kip.

Concrete mass lumped at elevation 272 in. = $4.003[(421\text{-}272)\times(1/2)+272/2] = 842.7$ Kip.

Concrete mass lumped at elevation 0 in. = $4.003(272/2) = 544.4$ Kip.

Concrete mass lumped at elevation –3.0 in. = 2,673.3 Kip.

For x-direction:

Total base shear = V_x = 75.03 + 152.62 + 172.1 + 272.79 + 8.14 + 29.34 + 286.52 + 28.9 + 136.1 = 1,161.54 Kip.

Base moment = M_x = 75.03×667.0 + 152.62×560.0 + 172.1×560 + 272.79×421 + 8.14×311 + 29.34×272 + 286.52×272 + 28.9×268.5 = 442,937.9 Kip.in.

Shear at base of basemat = V_{xm} = 1,161.64 + 668.33 = 1,829.97 Kip, use 1,830 Kip.

Moment at base of basemat = M_{xm} = 442,937.9 + 1,161.64×60 + 668.33×30 = 532,680.2 Kip.in.

For y-direction

Base shear = V_y = 87.33+177.64 + 200.4 + 342.44 + 11.0 + 39.69 + 136.1 = 1,421.34 Kip.

Base moment = M_y = 87.33×677.0 + 177.64×560 + 200.4×560 + 342.44×421 + 11.0×311 + 39.69×272 + 387.64×272 + 39.1×268.5 = 544,271.9 Kip.in.

Shear at base of basemat = V_{ym} = 1421.34 + 668.33 = 2,089.7 Kip.

Moment at base of basemat = M_{ym} = 649,602.2 Kip.in.

For z-direction:

Seismic inertia load above basemat = V_z = 25.83 + 52.54 + 59.26 + 121.88 + 4.62 + 14.67 + 143.26 + 14.45 + 92.55 = ±529.06 Kip.

Seismic inertia load at base of basemat = ±(529.06 + 454.46) = ±983.52 Kip.

Table 8A.1-21
Forces and Moments Analysis Results Summary – Global Seismic Effects

	x-direction (longitudinal)		y-direction (lateral)		z-direction (vertical)	
	F_x (Kip.)	M_x (Kip.in.)	F_y (Kip.)	M_y (Kip.in.)	F_z (Kip.)	M_z (Kip.in.)
At top of basemat	1,161.6	442,938	1,421.3	544,272	±529.1	0
At bottom of basemat	1,830.0	532,680	2,089.7	649,602	±983.5	0

4. Accelerations at Selected Locations on the Concrete Structure and Flexible Steel Floors.

Accelerations appropriate for a competent rock site are listed in the Table 8A.1-22 for selected locations on the concrete structure and flexible structural steel floors.

Table 8A.1-22

Acceleration For Selected Locations

Location	Equipment	ZPA Accelerations			
		Acel X	Acel Y	Acel Z	
0 m	Cask Trolley Base of Sliding Door	0.25g	0.25g	0.17g	
6.9 m	Mezz. Floor Top of Sliding Door	0.37g	0.5g	0.7g	
7.9 m	Fuel Crane Prep Area Roof Struc.	0.37g	0.50g	0.21g	
14.45 m	Roof Floor Lid Crane Shield Plug	0.69g	0.77g	0.40g	

5. Seismic Accelerations and Estimated Secondary Response Spectra For Design of support equipment.

Figures 8A.1-11 to 19 present the secondary response spectra which is to be used for equipment design. Spectra at 7% damping have been derived using the following data from the fixed base model analysis:

- Structure zero period accelerations at the location or area of interest.
- Structure mode shapes, frequencies and mass participation factors to identify modes of interest.
- Amplifications at important frequencies above zero period accelerations have been estimated using systems of single degree of freedom oscillators attached to locations in a separate finite element model.

Figure 8A.1-11

Secondary Response Spectra - X Direction (14.45m Level)

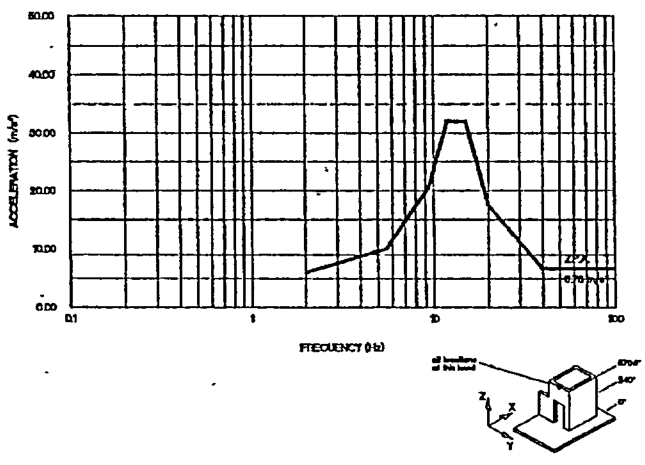


Figure 8A.1-12

Secondary Response Spectra - Y Direction (14.45m Level)

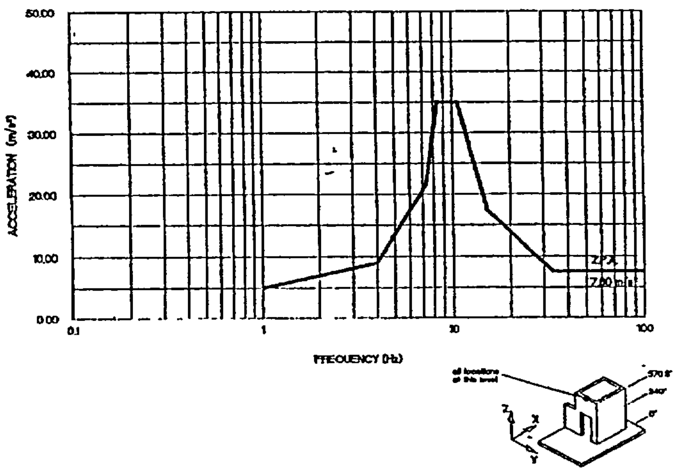


Figure 8A.1-13

Secondary Response Spectra - Z Direction (14.45m Level)

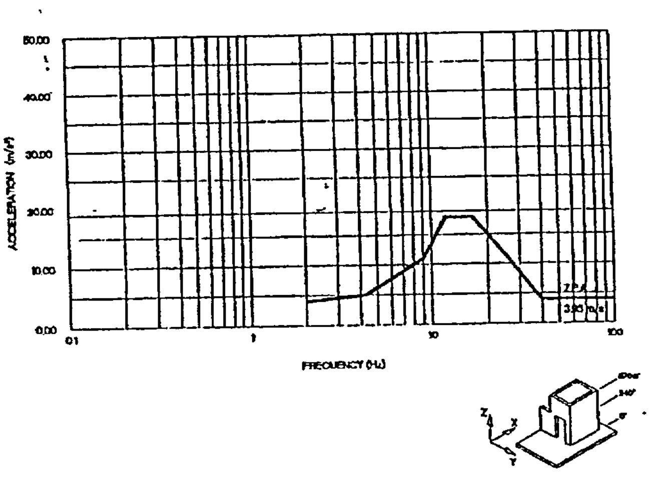


Figure 8A.1-14

Secondary Response Spectra - X Direction (7.9m Level)

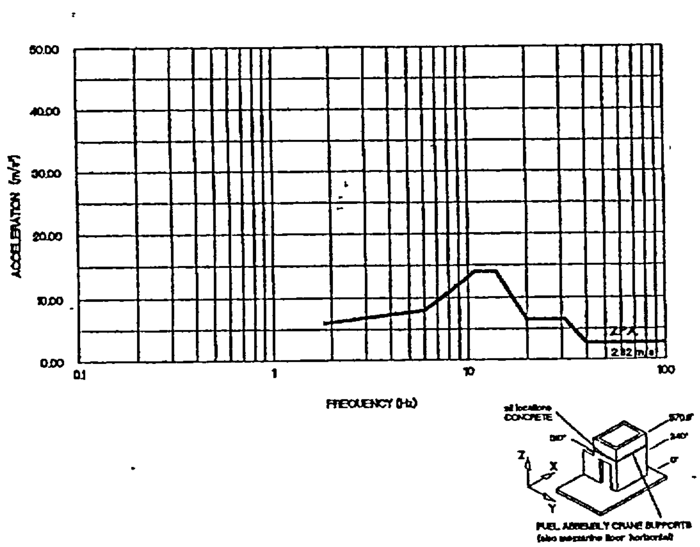


Figure 8A.1-15

Secondary Response Spectra - Y Direction (7.9m Level)

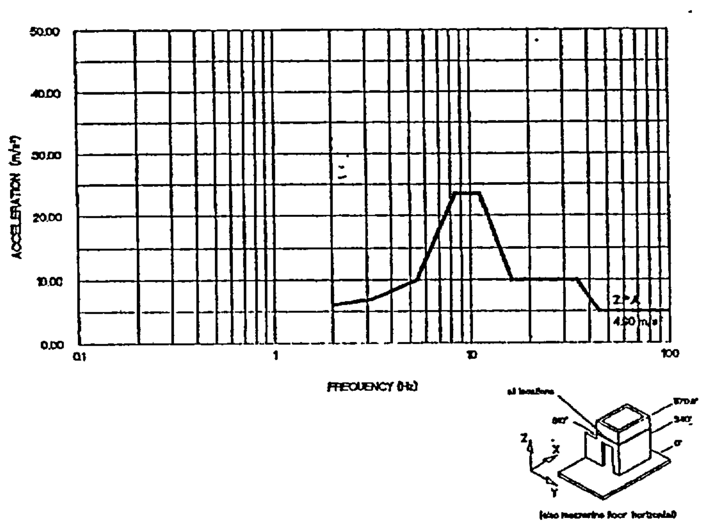


Figure 8A.1-16

Secondary Response Spectra - Z Direction (7.9m Level)

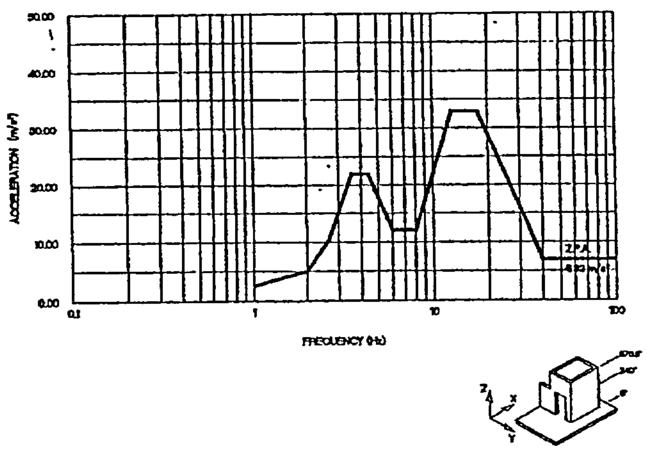


Figure 8A.1-17

Secondary Response Spectra - X Direction (6.0m Level)

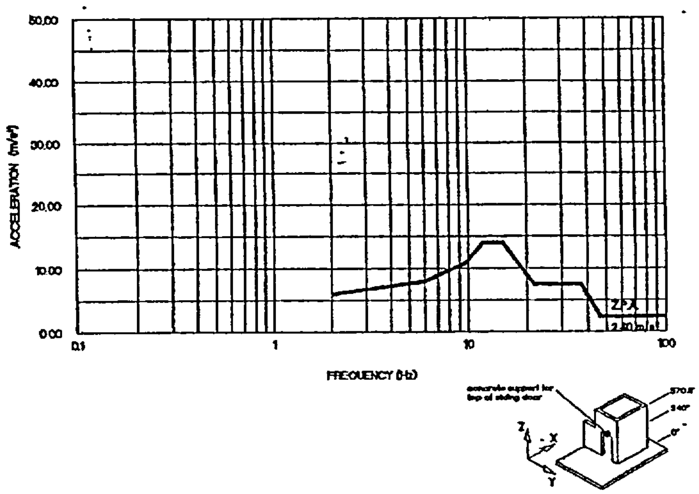


Figure 8A.1-18

Secondary Response Spectra - Y Direction (6.0m Level)

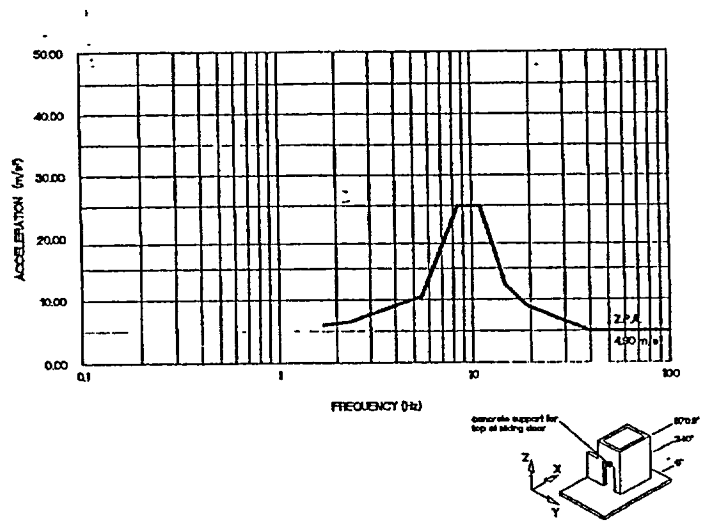
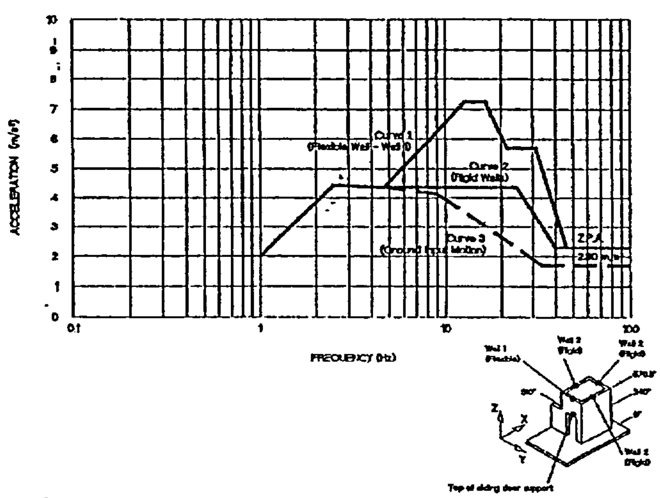


Figure 8A.1-19

Secondary Response Spectra - Z Direction (6.0m Level)



6. Basemat (SSE)

At top of basemat:

Maximum shear stress due to V_x = 1,161.5/27,288 = 0.043 Ksi.

Maximum shear stress due to V_y = 1,421.3/19,080* = 0.074 Ksi.

Maximum shear stress due to V_x & V_y = $(0.043^2 + 0.074^2)^{1/2}$ = 0.086 Ksi.

*Includes wing wall shear area. Wing wall will resist shear load because it is above the door opening in the DTS wall.

Maximum axial stress due to $M_x = \pm M_x/S_y$
$= \pm 442{,}938/2{,}730{,}660 = \pm 0.162$ Ksi. $= \pm 162$ psi.

Maximum axial stress due to $M_y = \pm M_y/S_x$
$= \pm 544{,}272/3{,}184{,}133 = \pm 0.171$ Ksi. $= \pm 171$ psi.

Maximum axial stress due to $F_z = \pm F_z/A$
$= \pm 529.1/36{,}846 = \pm 0.014$ Ksi. $= \pm 14$ psi.

Maximum axial stress due to M_x, M_y, F_z
$= \pm[162^2 + 171^2 + 14^2]^{1/2} = \pm 236.1$ psi.

At bottom of Basemat:

Maximum axial stress due to $M_x = \pm 532{,}680 \times 1000/74{,}166{,}648 = \pm 7.81$ psi.

Maximum axial stress due to $M_y = \pm 649{,}602 \times 1000/50{,}642{,}048 = \pm 12.83$ psi.

Maximum axial stress due to $F_z = \pm 983.5 \times 1000/513{,}264 = \pm 1.92$ psi.

Maximum axial stress due to M_x, M_y, $F_z = \pm[7.18^2 + 12.82^2 + 1.92^2]^{1/2} = \pm 14.83$ psi.
= 2,134.9 psf.

7. Concrete Walls (SSE)

Seismic acceleration = 0.25 g. For one foot wall height,

Ws = 0.25×0.15×3×1 = 0.1125 Kip/ft.

Span = 28.6 ft.

Maximum out of plane shear stress = 1,609/12×(36-3) = 4.06 psi.

Maximum out of plane moment at ends = $0.1125(28.6)^2/12$ = 7.668 Kip.ft./ft.
= 92.02 Kip.in./ft.

Maximum out of plane moment at midspan = wl2/24 = M2 = 0.1125(28.62)/24
= 3.83 Kip.ft./ft = 46.01 Kip.in./ft.

At elevation = 40 ft., horizontal acceleration from table 8a.1-22 = 0.77g.

Maximum moment at ends = 0.77/0.2×92.02 = 283.4 Kip.in./ft. > 92.02 Kip.in./ft.

Maximum moment at midspan = 0.77/0.25×46.01 = 141.71 Kip.in./ft. > 46.01 Kip.in./ft.

Conservatively assuming uniform seismic load of 0.77 g,

Total horizontal seismic load for 307 ft. × 224 ft. × 36 in. thick wall panel
= 0.77×307×224×36×150/1,728/1000 = 165.5 Kip.

8. Local to Equipment Anchorages

Check top of wall for roof beam horizontal seismic force F (Reference Table 8A.1-22):

$F = (1255/2 \text{ No. beams}) \times 0.77g = 483$ KN (Assume taken on one side only)

Check Shear at 14.45m Level;

$b_{eff} = 2 \times d$ (assume 45^0 load spread), $d = 800$mm, $b_{eff} = 1600$mm

$V = 483 \times 10^3/1600 \times 800 = 0.38 \text{ N/mm}^2 = 55 \text{ psi} < 93 \text{ psi} \Rightarrow$ OK

Check flexural strength as vertical nib;

$M = 483 \text{ KN} \times 575\text{mm}/2 = 138879$ KN/mm

Maximum tension in vertical reinforcement;

$T = 138870/800 \times 2/3 = 262 \text{ mm}^2 \Rightarrow$small $\Rightarrow$ Minimum rebar are adequate

I. Walls Between Base and Mezzanine Floor

Horizontal Span: As above mezzanine floor

In Plan Shear

Load Case	Shear Force (Kip)	Remark
SSE	$F_y = 1,421.3$	Table 8A.1-21
SSE	$F_x = 1,161.6$	Table 8A.1-21
DBT (Wind Load)	$F_y = 972.4$	Table 8A.1-16

$\Rightarrow F_{y\,(design)} = 1,421.3$ Kip.

Allowable shear stress

$\Phi V_c = \Phi \times 2 (f_c)^{1/2} = 0.85 \times 2 \times (3000)^{1/2} = 93$ psi

Y - direction:

Front wall

$V_i = (1,421.3 \times 1000/2 \text{ sides})/ [0.8 \times (280" + 120" - 150") \times 36"] = 0.71 \text{ N/mm}^2 = 98.7$ psi

Back wall

$V_i = (1{,}421.3 \times 1000/2 \text{ sides})/ [0.8 \times 280" \times 36"] = 0.64\ N/mm^2 = 88.1$ psi

Shear on front wall just over allowable, hence by inspection nominal reinforcement will be adequate. i.e. # 10 at 8" horizontal & # 8 at 8" vertical.

Calculate shear reinforcement required at corners of door opening.

With reference to ICE designers manual;

$A_s = V/\Phi\ f_y = 1{,}421.3 \times 1000/2 \times 0.9 \times 60{,}000 = 13\ in^2$

$\Rightarrow$use 10 - #10 at each corner $A_s = 12.7\ in^2$

The required rebar area is only slightly greater than the area afforded by the 10 - #10 rebar. Therefore, considering the additional strength of the concrete itself, the rebar is considered sufficient.

X - direction

$V_i = 1{,}161.6 \times 1000/2 \times 0.8 \times 379" \times 36" = 53.21$ psi < 93 psi OK

Axial stress due to tornado wind overturn moment

Reference Table 8A.1-17;

Maximum moment = (360/150)2×50,055 = 288,316,8 Kip.in

Check stress due to Moment;

F = 288,316,8/244 = 1,181.6 kips

$A = 343 \times 36 = 12348\ in^2$

$\sigma_x = 1{,}181.6/12348 = 95.69$ psi

Check stress due to weight;

W = 2,703,813 lbs

$A = 379 \times 280 - 307 \times 208 - 150 \times 36 = 36{,}864\ in.^2$

$\sigma_w = 2{,}703{,}813\ lb. / 36{,}864\ in.^2 = 73.35$ psi

$\sigma_{Tension} = 95.69 - 73.35 = 22.34$ psi

$\sigma_{Compression} = 95.69 + 73.35$

$= 143.04$ psi < 1785 psi ($\Phi \times 0.85 \times f'_c = 0.7 \times 0.85 \times 3000 = 1785$ psi)

Axial stress due to seismic load

From Table 8A.1-21 (at top of Basemat);

$M_x = 442{,}938$Kip.in
$M_y = 544{,}272$ Kip.in
$F_z = 529.1$ Kips

Check stress due to M_y;

$F = 544{,}272/244 = 2{,}231$ kips
$A = 343 \times 36 = 12348$ in^2
$\sigma_y = 2{,}231/12348 = 181$ psi

Check stress due to M_x;

$F = 442{,}938/343 = 1{,}291$ kips
$A = (244\text{-}150+120) \times 36 = 7704$ in^2
$\sigma_x = 1291/7704 = 168$ psi

Check stress due to F_z;

$F_z = 529.1$ Kip.
$A = ((379\text{-}72) \times 2 + (280\text{-}36) + (280\text{-}36\ \text{-}150)) \times 36 = 34272$ in^2
$\sigma_w = 529.1/34272 = 15$ psi

Check stress due to weight;

$W = 2{,}703{,}813$ lbs
$A = 379\times280 - 307\times208 - 150\times36 = 36{,}864$ in.^{2}
$\sigma_w = 2{,}703{,}813$ lb. / $36{,}864$ in.^{2} = 73 psi

$\sigma_{combined} = (181^2 + 168^2 + 15^2)^{1/2} = 247$ psi

$\sigma_{Tension} = 247 - 73 = 174$ psi
$\sigma_{Compression} = 247 + 73 = 320$ psi < 1785 psi ($\Phi \times 0.85 \times f'_c = 0.7 \times 0.85 \times 3000 = 1785$ psi)

$A_{required} = 174 \times 36 \times 12 / 0.9 \times 60000 = 1.624$ in^2/ft

$\Rightarrow$use #10 at 8" vertical up to mezzanine floor
$\Rightarrow A = \pi(1.25)^2/4 \times 1.5 = 1.84$ in^2/ft > 1.624 in^2/ft

Allowable tension stress $\Rightarrow 0.9 \times 60000 = 54000$ psi
Calculated tension stress $54000 \times 1.624/1.84 = 47{,}661$ psi

9. Roof Plate/Beam Hold Down Arrangement

From Table 8A.1-22, seismic loads at roof floor are;

Acel X = 0.69g
Acel Y = 0.77g
Acel Z = 0.40g

Check for seismic load uplift on the roof;

Weight of roof plate = 163,421 lbs (Table 8A.1-3)
Vertical seismic acceleration = 0.40g

By inspection, no uplift, small and nominal anchors would cater for it.
The hold down detail is shown in Figure 8A.1-20.

Check for seismic load uplift on the beam;

Floor steel beams are supported on two sides vertical and anchored on one side only horizontally (conservative assumption)

Weight of roof plate, beams and equipment = 250,173 lbs (Table 8A.1-3)

By inspection, there is no seismic uplift occurring on the roof and beams, hence nominal hold down connection is adequate. The hold down detail is shown in Figure 8A.1-21.

Check for horizontal shear;

The horizontal shear force $F_s = (250{,}173/2) \times 0.77 = 96{,}317$ lbs
Allowable shear stress = $0.42\ (S_u) = 0.42 \times 73000 = 30{,}600$ psi
Try 4 No. of bolts through the 36 inch wall to take floor seismic shear in direct tension.

$A_{required} = F_s/\Phi\ f_y = 96{,}317/30{,}600 = 3.148\ in^2$
i.e. use 4-1-1/4"dia. A-325 bolts ($A = 4 \times 0.969 = 3.876\ in^2$)
The hold down detail is shown in Figure 8A.1-21.

10. Mezzanine Plate/Beam Hold Down Arrangement

From Table 8A.1-21, Seismic Loads at Mezzanine roof Floor Are;

Acel X = 0.37g
Acel Y = 0.50g
Acel Z = 0.70g

By inspection use as roof details. The hold down detail is shown in Figure 8A.1-22.

Figure 8A.1-20

Roof Plate Hold Down Details

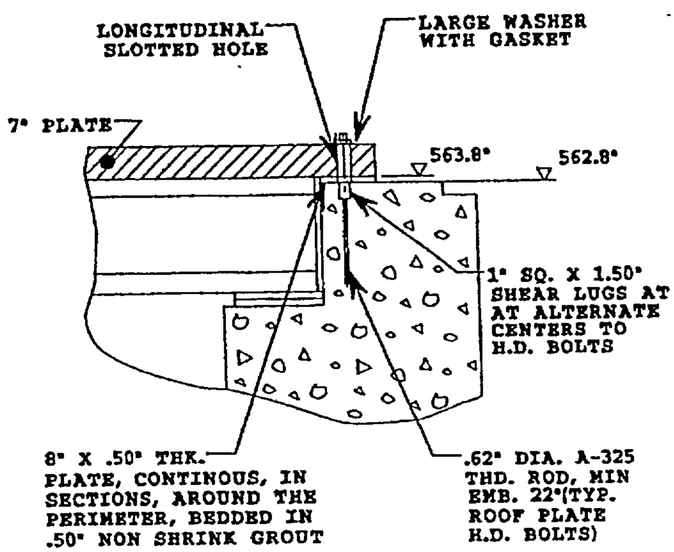


Figure 8A.1-21

Roof Plate Support Beam Connection Details

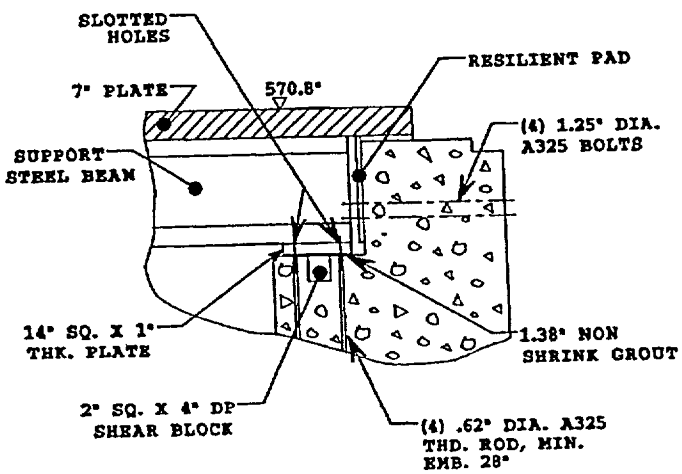
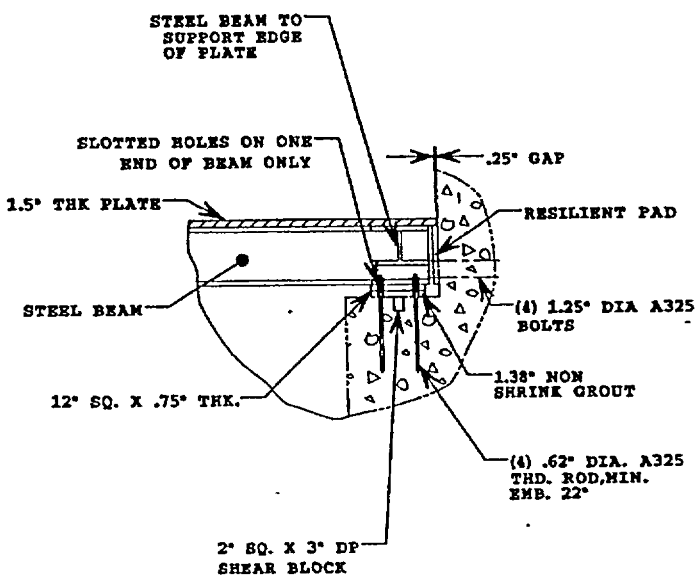


Figure 8A.1-22

Mezzanine Plate Support Beam Connection Details



11. Reinforced Concrete Superstructure Load Combination

The applicable loads for the DTS reinforced concrete superstructure include the dead weight, handling loads, tornado wind/missile loads, and seismic loads. The load combinations are based on ANSI - 57.9 as shown in Chapter 3. Table 8.2-3 shows the maximum combined stress and are compared to AISC code allowables (Reference Chapte 3).

D. Stability Analysis

Wind Load (W)

For 150 mph wind in the y-direction (critical)

Shear force at the base of the foundation mat = 123.88 Kip.

Overturning moment at base of foundation mat = 4,171,269 lb.ft.
= 50,055 Kip.in.

Roof wind load = -73.53×280×379/144 = -54,187.5 lb. = -54.19 Kip.

Minimum vertical load = 5,489.5 – 54.19 = 5,435.3 Kip.

Minimum frictional resistance load = 0.65×5,435.3 = 3,532.9 Kip.

Minimum stabilizing moment = 5,435.3×296 = 1,608,852 Kip.in.

Factor of safety against sliding = 3,532.9/123.88 = 28.5

Factor of safety against overturning = 1,608,852.5/50,055 = 32.14

Tornado Missile (Wm)

Maximum shear sliding load = 704 Kip.

Maximum overturning moment at base of foundation mat = 433,664 Kip.in.

Factor of safety against sliding = 5,489.5×0.65/704 = 5.07

Factor of safety against overturning = 1,624,892/433,664 = 3.75

Tornado Wind (Ww)

Factor of safety against sliding = $0.65[5{,}489.5-(360/150)^2 \times 54.19] / [(360/150)^2 \times 123.88]$
= 4.72

Factor of safety against overturning = $[5{,}489.5-(360/150)^2] \times 296 / [(360/150)^2 \times 50{,}055]$
= 5.32

Tornado Differential Pressure (Wp)

Maximum sliding shear load = 759.3 Kip.

Overturning moment = 299,083.8 Kip.in.

Factor of safety against sliding = $5{,}489 \times 0.65 / 759.3 = 4.70$

Factor of safety against overturning = 1,624,892 / 299,083.8 = 5.43

Seismic Load (SSE)

Maximum horizontal load due to x & y horizontal loads
= $[1{,}830^2 + 2089.7^2]^{1/2} = 2777.7$ Kip.

For minimum vertical load, use 40% of vertical seismic component

The minimum vertical load = $5{,}489.5 - 983.5 \times 0.4 = 5{,}096.1$ Kip.

For sliding resisted by base friction. Assumed reasonable friction angle between basemat and subsoil is 33° (rock, gravel or sand).

Coefficient of friction = tan33° = 0.65

Minimum frictional resistance force = $5{,}096.1 \times 0.65 = 3{,}312.5$ Kip.

Factor of safety against sliding = 3,312.5/2,777.7 = 1.19 >1 … o.k.

For equilibrium in x-direction:

Overturning moment = M_o = 532,680 Kip.in.

For minimum vertical load, use 40% of vertical seismic component.

Effective vertical load above elevation 0 in.= $(5{,}489.5–2{,}673.3)–0.4 \times 529.1 = 2{,}604.6$ Kip.

Effective vertical load below elevation 0in. = $2{,}673.3-0.4 \times 454.5 = 2{,}491.5$ Kip.

Stabilizing moment = M_s = 2,604.6×307.5 + 2,491.5×433.5 = 1,880,967.5 Kip.in.

Factor of safety against overturning = M_s/M_o = 1,880,967.5/532,680 = 3.53 > 1.5 … o.k.

For equilibrium in y-direction:

Overturning moment = M_o = 649,602 Kip.in.

Effective vertical load downward load = 5,489.5 – 0.4×983.5 = 5,096.1 Kip.

Eccentricity = e = 592/2 = 296 in.

Stabilizing moment = M_s = 5,096.1×296 = 1,508,445.6 Kip.in.

Factor of Safety against overturning = M_s/M_o = 1,508,445.6/649,602 = 2.32 > 1.5 … o.k.

The results of the stability analysis are summarized in Table 8.2-3. The minimum factor of safety is shown to be 1.19.

E. Basemat Design

1. Approach

The foundation mat has been designed using the commonly accepted engineering methods of concrete foundation/foundation mat design. The maximum forces and moments are calculated in x, y and z directions due various loads and are summarized in the following table. Based on these loads, the foundation pressure and pressure distribution for each applicable dead, live, wind, tornado and seismic loads have been determined. The foundation pressure calculations include effects of eccentricity between center line of the foundation mat and center of gravity of dead and live loads, direction for wind, tornado, and seismic loads and different area section properties of the foundation mat in x and y directions.

Foundation pressures for each component of seismic load have been calculated separately and then combined based on the SRSS (square-root-of-the-sum-of-square) method of combination for spatial component of an earthquake. For the long cantilever (overhang), foundation pressure is based on SRSS combination of seismic load in the longitudinal (x) direction and vertical (z) direction. From the foundation pressures at critical locations and pressure distributions, the maximum design moments have been calculated for various exterior cantilever parts and the internal slab using commonly used methods of foundation mat design

SUMMARY OF FORCES & MOMENTS

Forces and moments due to various dead, live, wind, tornado, and seismic loads are taken from detailed calculations and summarized below.

Applied Load	x - Direction (Longitudinal)		y – Direction (lateral)		z – Direction (Vertical)	
	Fx (kip)	Mx (kip-in)	Fy (kip)	My (kip-in)	Fz (kip)	Mz (kip-in)
Dead Weight DTS Super Structure	-	340,680			2703.8	
Dead Weight Foundation Mat	-	-			2673.3	
Total Dead Weight		340,680			5377.1	
Live Load DTS Super Structure		39904.2			316.7	
Live load on Mat		-			891.1	
Total Live Load		39904.2			1207.8	
Wind Load	91.52	36980.1	123.9	50,055.20		
Tornado Missile Load	704	433664	704	433664		
Tornado Wind Load	527.2	213,005	713.6	288,318		
Tornado Wind Differential Pressure	560.95	220959	759.3	299084		
Seismic Load	1830	532680	2089.7	649602	+/-983.5	

2. Area (Section) Properties at Base of Basemat

Refer to Figure 8A.1-23 for dimensional data.

Area $A_b = 867 \times 592 = 513{,}264\ in^2 = 3{,}564.3\ ft^2$

Section Modulus $S_{xx} = 867\ (592)^2/6 = 50{,}642{,}048\ in^3 = 29{,}306.7\ ft^3$

Section Modulus $S_{yy} = 592\ (867)^2/6 = 74{,}166{,}648\ in^3 = 42{,}929.5\ ft^3$

3. Foundation Pressures

Refer to Figure 8A.1-23 for designation of pressure.

Dead Load (D):

Uniform Pressure due to Axial Load = $F_z/A_b = (5{,}377.1/513{,}264) \times 1000 = 10.48$ psi

Distributed Pressure due to Moment = $M_x/S_y = 340{,}480 \times 1000/74{,}166{,}648 = 4.59$ psi

Max. Foundation Pressure = $p_1 = p_4 = 10.48 + 4.59 = 15.07$ psi

Min. Foundation Pressure = $p_3 = 10.49 - 4.59 = 5.89$ psi

Live Load (L):

Uniform Pressure = $F_z/A_b = 1207.8 \times 1000/513{,}264 = 2.35$ psi

Distributed Pressure = $M_x/S_y = 39904.2 \times 1000/74{,}166{,}648 = 0.54$ psi

Max. Foundation Pressure = $p_1 = p_4 = 2.35 + 0.54 = 2.89$ psi

Min. Foundation Pressure = $p_3 = 2.35 - 0.54 = 1.82$ psi

Wind Load (W):

Distributed Pressure due to Wind along x direction = $M_x/S_y = p_7$
$= 36{,}980.1 \times 1000/74{,}166{,}648 = 0.50$ psi

Distributed Pressure due to Wind along y direction = $M_y/S_x = p_2$
$= 50055.2 \times 1000/50{,}642048 = 0.99$ psi

Tornado Missile (Wm):

Distributed Pressure due to Missile Impact in x direction $= M_x/S_y = p_7$
$= 433{,}664 \times 1000/74{,}166{,}648 = 5.85$ psi

Distributed Pressure due to Missile Impact in y direction $= M_y/S_x = p_2$
$= 433664 \times 1000/50{,}642{,}048 = 8.56$ psi

Tornado Wind (Ww):

Distributed Pressure due to Tornado Wind in x direction $= M_x/S_y = p_7$
$= 213{,}005 \times 1000/74{,}166{,}648 = 2.87$ psi

Distributed Pressure due to Tornado Wind in y direction $= M_y/S_x = p_2$
$= 288{,}318 \times 1000/50{,}652{,}048 = 5.69$ psi

Tornado Wind Differential Pressure (Wp):

Distributed pressure due to tornado along x direction $= M_x/S_y = p_7$
$= 220{,}959 \times 1000/74{,}166{,}648 = 2.98$ psi

Distributed pressure due to tornado along y direction $= M_y/S_x = p_2$
$= 299084 \times 100/50{,}642{,}048 = 5.91$ psi

SSE Seismic (Ess):

Distributed pressure due to Earthquake in x direction $= M_x/S_y$
$= 532{,}680 \times 1000/74{,}166{,}648 = 7.18$ psi

Distributed pressure due to Earthquake in y direction $= M_y/S_x$
$= 649{,}602 \times 1000/50{,}642{,}048 = 12.83$ psi

Uniform pressure due to Earthquake in z (vertical) direction $= F_z/A_b$
$= 983.5 \times 1000/513{,}264 = 1.92$ psi

Max. pressure due to Earthquake in x, y & z directions $= \{(7.18)^2+(12.83)^2+(1.92)^2\}^{0.5}$
$= p_2 = p_7 = 14.83$ psi

4. Maximum Design Moments

Cantilever Moment along Longitudinal Face:

From Figure 8A.1-23, for case 1 and 2 conservatively assume uniformly distributed pressure, p = maximum pressure i.e $p_1 = p_2 = p$

Max Moment along cantilever face B or E

$M_1 = p\,(156)^2/2$ lb- in/in $= p(156)^2 \times 12/(2 \times 1000) = 146.02\,p$ kip-in/ft

Max. Moment due to Dead Load (D) = 146.02 × 15.07 = 2200.5 kip-in/ft = M_1

For dead weight moment, moment due to dead weight (5×150 psf) of foundation mat has to be subcontracted from M_1

Moment due to dead weight of foundation mat = (5×150/144)x$(156)^2$/2×(12/1000)
= 760.5 kip in/ft

Net Moment due to Dead Load (D) = 2200.5 - 760.5 = 1440.0 kip in/ft.

Max. Moment due to Live Load (L) = 146.02 × 2.89 = 422.0 kip in/ft = M_1

For live load moment, moment due to live load (250 psf) on basemat has to be subtracted from M_1

Moment due to live load on basemat = 250/144 $(156)^2$/2 × 12/100 = 253.5 kip in/ft

Net moment due to Live Load (L) = 422.0 - 253.5 = 168.5 Kip in/ft use 169.5 kip in/ft

Max. Moment due to Wind Load (W) = 146.02 × 0.99 = 144.6 kip in/ft

Max. Moment due to Tornado Missile (Wm) = 146.02 × 8.56 = 1250 kip in/ft

Max. Moment due to Tornado Wind (Ww) = 146.02 × 5.69 = 831 kip in/ft

Max. Moment due to Differential Pressure (Wp) = 146.02 × 5.91 = 863 kip in/ft

Max Moment due to SSE Seismic Load (Ess) = 146.02 × 14.83 = 2165 kip in/ft

Cantilever Moment along Transverse Face:

Refer to Figure 8A.1-23 for dimensional data and pressure designation.

Max. Moment along Cantilever face K = $p(118)^2/2 \times 12/1000$

$M_2 = 83.54\,p$ Kip in/ft < $146.02\,p$ Kip in/ft. does not govern.

Max. Moment along Cantilever face H for Case 3:

$$M_3 = \{p_3(370)^2/2 + (p_5 - p_3)\,(370)^2/(2 \times 3)\} \times 12/1000$$
$$= 821.4\,p_3 + 273.8\,p_5 - 273.8\,p_3$$
$$= 547.6\,p_3 + 273.8\,p_5$$

$$p_5 = p_3 + [(p_4 - p_3)/867] \times 370 = 0.5732\,p_3 + 0.4268\,p_4$$

$$\therefore\ M_3 = 547.6\,p_3 + 273.8(0.5732\,p_3 + 0.4268\,p_4) = 704.55\,p_3 + 116.85\,p_4$$

Max. moment along cantilever face H for Case 4:

$$M_4 = \{p_8\,(370)^2/2 + [(p_7 - p_8)/2] \times 370^2) \times 2/3\}12/1000 = 273.8\,p_8 + 547.6\,p_7$$

$$p_8 = [p_7/(0.5 \times 867)] \times (0.5 \times 867 - 370) = 0.1465\,p_7$$

$$\therefore\ M_4 = 0.1465 \times 273.8\,p_7 + 547.6\,p_7 = 587.707\,p_7$$

Max. Moment due to Dead Load (D) = $M_3 = 704.55\,p_3 + 116.85\,p_4$
$= 704.55 \times 5.89 + 116.85 \times 15.07 = 5910.7$ Kip in/ft

Max. Net Moment due to Dead Load (D) = $5910.7 - 5 \times 150 \times (370/12)^2 \times 1/2 \times 12/1000$
= 1,633 Kip in/ft

Max Moment due to Live Load (L) = $M_3 = 704.55\,p_3 + 116.85\,p_4$
$= 704.55 \times 1.82 + 116.85 \times 2.89 = 1620.0$ Kip in/ft

Max Net Moment due to Live Load (L) = $1620.0 - 250 \times (370/12)^2 \times 1/2 \times 12/1000$
= 194 kip in/ft

Max. Moment due to Wind Load (W) = $M_4 = 587.7\,p_7 = 587.7 \times 0.50 = 294$ kip in/ft

Max. Moment due to Tornado Missile (Wm) = $M_4 = 587.7\,p_7 = 587.7 \times 5.85$
= 3438 kip in/ft

Max. Moment due to Tornado Wind (Ww) = $M_4 = 587.7\,p_7 = 587.7 \times 2.87 = 1687$ kip in/ft

Max. Moment due to Tornado Differential Pressure (Wp) = $M_4 = 587.7\,p_7 = 587.7 \times 2.98$
= 1751 kip in/ft

Max. Moment due to SSE Seismic Load (Ess) = $M_4 = 587.7\,p_7 = 587.7 \times (7.18^2 + 1.92^2)^{0.5}$
= 4368 kip in/ft

(Based on engineering judgement, use foundation pressure due to seismic loads in horizontal x and vertical z directions only)

Moment for Internal Slab:

Considering two way slab action:

From Case 9a, Table 26, "Formulas for Stress and Strain", R.J. Roark, W.C. Young, Fifth edition:

$a = 307$; $b = 208$; $a/b = 1.476$

$$\text{Max } \beta 1 = 0.307 + \frac{(0.539 - 0.307)}{(1.5 - 1.0)}(1.476 - 1.0) = 0.5278$$

$$\text{Max Bending Stress} = 0.5278\ \frac{pb^2}{t^2}$$

$$\text{Section Modulus} = S = 1 \times t^2/6$$

$$\text{Max. Moment, } M_5 = \frac{0.5278 pb^2}{t^2} \times \frac{t^2}{6} = 0.5278 \times p \times 208^2/6 = 3806.1\,p \text{ lb in/in}$$

$$M_5 = 45.67\,p \text{ kip in/ft}$$

Considering one way slab action:

$$\text{Max. Moment} = \frac{pl^2}{12} = \frac{p(208^2)}{12} \text{ lb in/in} = \frac{p(208^2)}{12} \times \frac{12}{1000}$$
$$= 43.26\,p \text{ kip in/ft} \approx 45.67\,p \text{ kip in/ft}$$

Conservatively assume uniform pressure distribution, $p = p_1 = p_2 = p_4$.

Max. Moment due to Dead Load (D) = $M_5 = 45.67\,p = 45.67 \times 15.07 = 688.3$ Kip in/ft

Max. Net Moment due to Dead Load (D) = $688.3 - 45.67 \times 5 \times 150/144 = 450$ Kip in/ft

Max. Moment due to Live Load (L) = $M_5 = 45.67\,p = 45.67 \times 2.89 = 132.0$ Kip in/ft

Max. Net Moment due to Live Load (L) = $132.0 - 45.67 \times 250/144 = 53.0$ Kip in/ft

Max. Moment due to Wind Load (W) = $45.67 \times 0.99 = 45$ Kip in/ft

Max. Moment due to Tornado Missile (Wm) = $45.67 \times 8.56 = 391$ Kip in/ft

Max Moment due to Tornado Wind (Ww) = $45.67 \times 5.69 = 260$ Kip in/ft

Max Moment due to Tornado Differential Pressure (Wp) = 45.67×5.91 = 270 Kip in/ft

Max Moment due to SSE Seismic Load (Ess) = 45.67×14.83 = 677 Kip in/ft

5. Sear Stresses in the Basemat

Shear Stresses Along Longitudinal Face:

From Figure 8A.1-23, Cases 1 and 2, conservatively assume uniform pressure distribution.

Max Shear at Distance d = 60 – 3 = 57" from longitudinal faces B or E = $V = (156 - 57)\,p$
$= 99\,p$ lbs/in.

Shear Area = $A_v = 1 \times 57 = 57$ in²/in.

Max Shear Stress = $\sigma = V/57 = (99\,p)/57 = 1.737\,p$ lbs/in².

Total Shear Stress due to Dead Weight (D) = 1.737×15.07 = 26.2 psi

Net Shear Stress due to Dead Load (D) = 26.2 – 1.737×5×150/144 = 7.2 psi

Total Shear Stress due to Live Load (L) = 1.737×2.89 = 5.0 psi

Net Shear Stress due to Live Load (L) = 5.0 – 1.737×5×250/144 = 2.0 psi

Total Shear Stress due to Wind Load (W) = 1.737×0.99 = 1.7 psi

Total Shear Stress due to Tornado Missile Load (Wm) = 1.737×8.56 = 14.9 psi

Total Shear Stress due to Tornado Wind Load (Ww) = 1.737×5.69 = 9.9 psi

Total Shear Stress due to Differential Pressure (Wp) = 1.737×5.91 = 1.03 psi

Total Shear Stress due to SSE Seismic Load (Ess) = 1.737×14.83 = 25.8 psi

Shear Stresses along Transverse Face:

From Figure 8A.1-23 Case 3,

Max. Shear at distance d = 57" from transverse face H

Total Shear $V = (p_3 + p_6)(370\text{-}57)/2 = 156.5(p_3 + p_6)$

$p_6 = p_3 + [(p_4 - p_3)/867](370\text{-}57) = 0.639\,p_3 + 0.361\,p_4$

$V = 156.5(p_3 + 0.639\,p_3 + 0.361\,p_4) = 256.5\,p_3 + 56.5\,p_4$ lb/in

Shear Area = $A_V = 1 \times 57 = 57$ in^2/in

$\therefore$ Max. Shear Stress = $V/57 = (256.5\, p_3 + 56.5\, p_4)/57 = 4.5\, p_3 + 0.991\, p_4$

For Case 4,

Total Shear at distance $d = 57$" from transverse face H

$V = (p_9 + p_7)(370 - 57)/2$

$p_9 = [p_7/(0.5 \times 867)] \times (0.5 \times 867 - 370 + 57) = 0.278\, p_7$

$V = (0.278\, p_7 + p_7)(370 - 57)/2 = 200.00\, p_7$

$\therefore$ Max. Shear Stress = $V/57 = (200.00\, p_7)/57 = 3.509\, p_7$

Max. Shear Stress due to Dead Weight (D) = (4.5)(5.89) + (0.991)(15.07) = 41.4 psi

Net Shear Stress due to Dead Weight (D) = $41.4 - \dfrac{5 \times 150(370 - 57)}{144 \times 57} = 12.8$ psi

Max. Shear Stress due to Live Load (L) = $4.5 \times 1.82 + 0.99 \times 2.89 = 11.1$ psi

Net Shear Stress due to Live Load (L) = $11.1 - \dfrac{250(370 - 57)}{144 \times 57} = 1.5$ psi

Max. Shear Stress due to Wind Load (W) = $3.509\, p_7 = 3.509 \times 0.5 = 1.8$ psi

Max. Shear Stress due to Tornado Missile (Wm) = $3.509\, p_7 = 3.509 \times 5.85 = 20.5$ psi

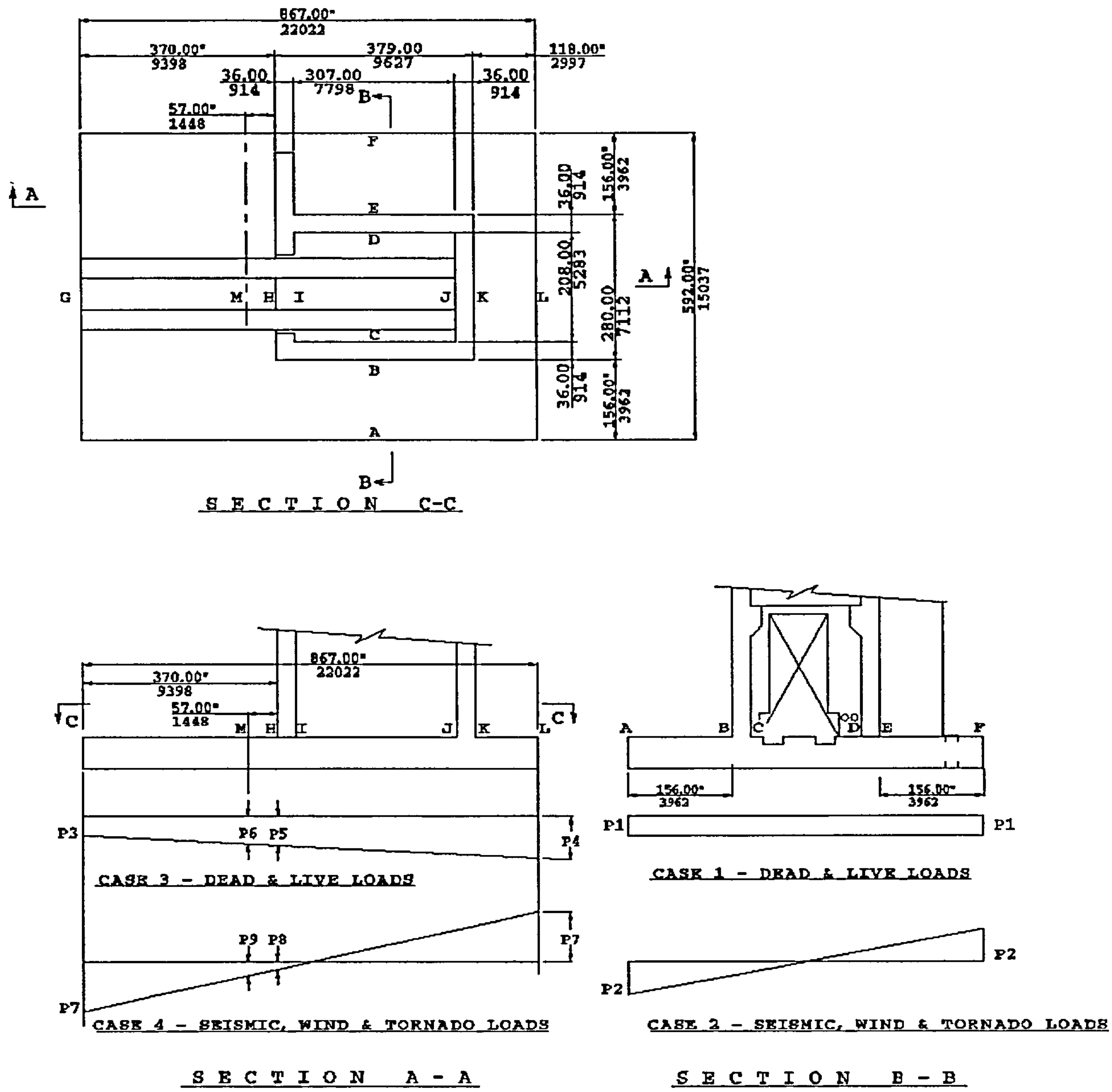
Max. Shear Stress due to Tornado Wind (Ww) = $3.509\, p_7 = 3.509 \times 2.87 = 10.1$ psi

Max. Shear Stress due to Tornado Differential Pressure (Wp) = $3.509\, p_7 = 3.509 \times 2.98$
= 10.5 psi

Max. Shear Stress due to SSE Seismic Load (Ess) = $3.509\, p_7 = 3.509 \times (7.18^2 + 1.92^2)^{0.5}$
= 26.1 psi

Results of the Basemat Evaluation are summarized in Tables 8.2-2 and 8.2-3. The maximum computed stresses and moments for all applied loads and load combinations are tabulated and compared to allowable values.

Figure 8A.1-23
Foundation Pressure Distribution



8A.1.6.2 Protective Cover Structural Analysis

The protective cover is designed to withstand the following accident loads:

- Tornado winds,
- Tornado generated missiles, and
- Design basis earthquake.

A. Tornado Winds

The 360 mph tornado wind load analysis is based on the methods used to evaluate the 150 mph normal wind load. The force, moment and stress results from the 150 mph wind load analysis are multiplied by a factor of $(360/150)^2 = 5.76$ to acquire the results for the 360 tornado wind loads. These results are listed in Tables 8.2-4a and Table 8.2-4b.

A differential pressure of 3 psi generated by the tornado winds is also assumed to act on the protective cover. A structural analysis for a 3 psi differential pressure is performed using the same classical engineering methods used to evaluate the normal wind loads. These results are also listed in Tables 8.2-4a and Table 8.2-4b.

B. Tornado Generated Missiles

The protective cover is analyzed to verify its adequacy for local barrier impingement of a DBT missile. Detail analysis of the protective cover has been performed and presented in Section 3.2.1.4. Based on the analysis shown on Section 3.2.1.4, there is a adequate protection against local design basis tornado missile impact damage. Local bending and distortion to protective cover is acceptable, since the DTS will not be operated during a tornado watch or warning.

C. Design Basis Earthquake

The maximum calculated seismic accelerations for the roof plate are 0.77g horizontally and 0.4g vertically. With the protective cover plate bolted to the support beam, the beam stresses due to the resulting 0.4g vertical acceleration are calculated by factoring the normal operating condition load analysis results reported in Section 8A.1.5.3. The stress results for the various bolts listed in Tables 8.2-4a and 8.2-4b are not calculated here, but are calculated using the same methods described below.

1.5 inch Roof Plate

Maximum stress = $0.4 \times 654.9 = 262.0$ psi.

Maximum deflection = $0.4 \times 0.0062 = 0.0025$ in.

Protective Cover Support Beams (W12×120)

Maximum bending stress = $0.4 \times 3.53 = 1.41$ ksi.

Maximum vertical deflection = $0.4 \times 0.246 = 0.098$ in.

1.5 inch Steel Wall

Horizontal seismic load = $0.29 \times 1.5 \times 1.77 = 0.335$ psi.

Maximum bending stress = $(0.335/0.3825) \times 723 = 633.1$ psi.

Maximum horizontal deflection = $(0.335/0.3825) \times 0.009 = 0.008$ in.

Total weight of protective cover steel (Table 8A.1-3) = 123,024 lb.

Total horizontal seismic load = $123{,}024 \times 0.77 = 94{,}728$ lb.

Minimum shear area of wall plate = $1.5 \times 2 \times 270 = 810$ in.^{2}

Maximum shear stress = $94{,}728/810 = 116.9$ psi.

W12×120 Columns

Maximum axial stress = $0.4 \times 0.38 = 0.15$ ksi.

Tributary horizontal seismic load = $0.77 \times (0.29 \times 1.5 \times 125 \times 12 + 120) = 594.8$ lb./ft.

Maximum bending stress = $(594.8/573.8) \times 1.22 = 1.26$ ksi.

Maximum horizontal deformation = $(594.8/573.8) \times 0.008 = 0.0083$ in.

8A.1.6.3 Roof Plate Structural Analysis

The maximum calculated seismic accelerations for the roof plate are 0.77g horizontally and 0.4g vertically. With the roof plate bolted to the support beam, the beam stresses due to the resulting 0.4g vertical acceleration are calculated by factoring the normal operating condition load analysis results reported in Section 8A.1.5.4. The stress results for the various bolts listed in Tables 8.2-4a and 8.2-4b are not calculated here, but are calculated using the same methods described below.

7 inch Plate

Maximum bending stress = 0.4×206.2 = 82.5 psi.

Roof Plate Support Beams (W14×550)

Maximum bending stress = 0.4×2.15 = 0.86 ksi.

8A.1.6.4 Mezzanine Plate Structural Analysis

The maximum calculated seismic accelerations for the mezzanine plate are 0.5g horizontally and 0.7g vertically. With the mezzanine plate bolted to the support beam, the beam stresses due to the resulting 0.7g vertical acceleration are calculated by factoring the normal operating condition load analysis results reported in Section 8A.1.5.5. The stress results for the various bolts listed in Tables 8.2-4a and 8.2-4b are not calculated here, but are calculated using the same methods described below.

1 ½ inch Plate

Maximum bending stress = 0.7×3,476.2 = 2,433.3 psi.

Mezzanine Floor Support Beams (W12×120)

Maximum bending stress = 0.7×7.06 = 4.94 ksi.

8A.1.6.5 Sliding Door Structural Analysis

The design of the sliding door is based on shielding requirements. Analyses are also performed to evaluate the effects of tornado wind, tornado missiles and seismic loads.

A. Tornado Wind Load

The sliding door design is evaluated for the effects of tornado wind loads in accordance with the design criteria indicated in Section 3.0. The maximum stresses induced in the sliding door by DBT wind pressure loads are very conservatively calculated using the correlation presented in Roark, page 228, Case 48 (Reference 8A.1.8- 2). A conservative wind pressure load, 419 lbs/ft^2, (0.02 Ma) is applied as a uniform load over the entire surface. (Note: for a design basis 360 mph wind, the design pressure at an elevation of 40 ft. = $(360/150)^2 \times 60.09$ = 346 psf.)

$$S = \frac{\beta w\, b^2}{t^2}$$

where $w = 419\ \text{lbs/ft}^2 = 2.91\ \text{lbs/in}^2$
$a = 240.$
$b = 109.5$
$a/b = 2.19$
$\beta = 0.792$

$$S = 0.792 \times \frac{2.91 \times 109.5^2}{7.0^2} = 564\ \text{psi}$$

$$S_{allow} = 1.6 \times 0.6 \times 36000 = 34{,}560\ \text{psi}$$

The analysis results show a maximum stress of 564 psi (3.9 Ma) which is less than the allowable stress of 34,560 psi (238.3 Ma). Since the resulting sliding door stress is a small fraction of the code allowable, DBT wind loads are not considered further.

B. Tornado Missiles

The thickness of the sliding door is 7" (bottom) and 9" (top) (178 mm and 229 mm, respectively). The walls are designed to provide adequate radiation shielding and easily meet the minimum acceptable barrier thickness requirements for local damage against tornado generated missiles, specified in Section 3.0. Detail analysis of the sliding door has been performed and presented in Section 3.2.1.4. Based on the analysis shown on Section 3.2.1.4, tornado missile impacts on the sliding door cause only superficial damage. The sliding door thickness is far greater than the minimum required thickness. Local damage to the outer surfaces of the sliding door will not compromise their confinement capability.

The maximum stress induced in the sliding door by the automobile impact load is calculated using the correlation presented in Roark, page 226, Case 38. The impact pressure, 196 psi (1.35 MPa) is applied as a uniform load over the impact area, 4029.4 in^2 (2.6 m^2). This results in a total applied force of 789.7 Kips, which is well above the 704 Kip maximum tornado missile load computed in Chapter 3.0. Substituting the sliding door physical dimensions and the pressure load into the correction, the maximum calculated stress is 8,704 psi (60 MPa) which is less than the allowable stress of 21,600 psi (148.9 MPa).

C. Seismic Evaluation

The maximum calculated seismic accelerations for the sliding door are 0.7g vertically, 0.37g longitudinally, and 0.5g laterally. With the sliding door hanging on the support rails, the door stresses due to the resulting 0.7g vertical acceleration are calculated by factoring the dead load analysis results reported in Section 8A.1.5.6. Table 8A.1-29 summarized the combined stresses.

Table 8A.1-29

Sliding Door and Major Components Stress Analysis Results Summary (Dead Loads, Handling Loads, and seismic Load)

Comp.	Calculated Stress			Allowable Stress		
	Tension	Bending	Shear	Tension	Bending	Shear
Door	180			34,560		
Door Wheel	144,500 (Load)			160,000 (Capacity)		
Wheel Axle		30,784	15,332		110,400	64,400
Axle Bracket	13,547		10,838	96,000		56,000
Bracket Bearing		36,125 (Bearing)			45,000 (Bearing)	
Support Bracket	1,348	10,246	1,246	96,000	96,000	56,000
Door Rail		7,857			110,400	

For the stress evaluation of the sliding door due to seismic acceleration in the lateral direction, the resulting equivalent static acceleration of 0.5g is assumed to be resisted by four (4) - 2"(508 mm) dia. pin. The local bearing stresses of the sliding door at the support pin locations are calculated to be;

$F = \text{Lateral force} = 85{,}000 \times 0.5 = 42{,}500 \text{ lbs}$

$A_b = \text{Bearing area} = 3.1416 \times 2/2 \times 7 \times 4 = 87.96 \text{ in}^2$

$A_s = \text{shear area} = 3.1414\ (1)^2 \times 2 \times 4 = 25 \text{ in}^2$

$S_{\text{bearing stress}} = 42{,}500/87.96 = 483 \text{ psi} < 1.6 \times 0.6 \times 42000 = 40{,}320 \text{ psi}$

$S_{\text{shear stress}} = 42{,}500/25 = 1{,}700 \text{ psi} < 1.4 \times 0.4 \times 42000 = 23{,}520 \text{ psi}$

Since the resulting sliding door stress is a small fraction of the Code allowable, SSE load is not considered further.

8A.1.7 Exhaust Fan and PLC Enclosure Evaluation

Schematic sketch of the steel structure that houses the exhaust fans and the programmable logic controllers (PLC) is shown in Figure 8A.1-24. This structure is classified as 'Not Important to Safety'. However, since damage to the PLC or the exhaust fans could result in operational difficulties and high costs due to repairs and operating times, the building has been analyzed to withstand the normal and accident loads. Structural design calculations have been performed using the same load combinations and structural acceptance criteria that were used in the design of the DTS steel structure. The detailed structural calculations for the fans and PLC enclosure are given below:

A. 1 1/2" Steel Plate Roof

From R.J. Roark & W.C. Young, "Formulas for Stress and Strain", Fifth edition, Table 26, Case 1 and Figure 3.6-1,

$a = 144$ in $\quad b = 96$ in $\quad a/b = 144/96 = 1.5 \quad$ t = 1.5 in

Interpolating the values of β and α for a/b = 1.5;

$$\beta = 0.4530 + \frac{(0.5172 - 0.4530)}{(1.6 - 1.4)}(1.5 - 1.4) = 0.4851$$

$$\alpha = 0.0770 + \frac{(0.0906 - 0.0770)}{(1.6 - 1.4)}(1.5 - 1.4) = 0.0838$$

Dead Load (D):

q = Areal Density
q = Density x (a x bx t) / (a x b)
= 0.29 (lb./in^3) $\times$ 1.5 (in) = 0.435 psi = q

Max. Bending Stress = $\beta q b^2/t^2$
= 0.4851 x 0.435 $\times$ (96)^2/ (1.5)^2 = 864.3 psi = 0.86 ksi

Max. Deflection = $\alpha q b^4/Et^3$

$$= \frac{(0.435)(0.0838)(96^4)}{(29 \times 10^6)(1.5^3)} = 0.0316 \text{ in.}$$

Live Load (L):

Snow Load = 100 psf = 0.6944 psi

Max. Bending stress = (0.6944/0.435) $\times$ 864.3 = 1379.8 psi = 1.38 ksi
Max. Deflection = (0.6944/0.435) $\times$ 0.0316 = 0.0505 in.

Wind load (W):

Max. wind velocity = 150 mph

From ANSI/ASCE 7-95,
Effective wind pressure, $w = q_h \, G \, C_p$

The wind coefficient values are taken from RAI#1, 3-6:
$q_h = q_z = 59.62$ psf @ 20' height
$G \, C_p = (0.85)\,(-1.3) = -1.105$

$w = 59.62 \times -1.105 = -65.88$ psf $= -0.458$ psi

Max. Bending Stress = (-0.458/0.435) $\times$ 864.3 = -910.0 psi = -0.91 ksi
Max. Deflection = (-0.458/0.435) $\times$ 0.0316 = -0.0333 in.

Wind Internal Negative Pressure
$= q_h \, G \, C_{pi} = 59.62 \times 0.18 = 10.73$ psf $= 0.0745$ psi

Max. Bending Stress = (0.0745/0.435) $\times$ 864.3 = 148.0 psi =0.15 ksi
Max. Deflection = (0.0745/0.435) $\times$ 0.0316 = 0.0054 in.

Tornado Wind (Ww):

Tornado Wind Velocity = 360 mph (Section 3.2 DTS TSAR)
Since wind pressure is proportional to velocity squared,

Max. Bending Stress = $(360/150)^2 \times (-910.0) = -5241.8$ psi = -5.24 ksi
Max. Deflection = $(360/150)^2 \times (-0.0333) = -0.1918$ in

Due to internal Negative Pressure,

Max. Bending Stress = $(360/150)^2 \times 148.0 = 852.5$ psi = 0.85 ksi
Max. Deflection = $(360/150)^2 \times 0.0054 = 0.0311$ in.

Tornado Wind Differential Pressure (Wp):

$w_p = -3$ psi (Section 3.2 of DTS TSAR)

Max. Bending Stress = (-3.0/0.435) $\times$ 864.3 = -5960.9 psi = -5.96 ksi
Max. Deflection = (-3.0/0.435) $\times$ 0.0316 = -0.2182 in

SSE Seismic Load (Ess):

Front DTS TSAR Table 8A.1-22

Vertical Seismic Acceleration at Mezzanine Floor Level = 0.7g
Horizontal Seismic Acceleration at mezzanine Floor Level = 0.5 g

Max. Stress in. the plate = (0.7/1.0) × 864.3 = 605.0 psi = 0.61 ksi
Max. Deflection = (0.7/1.0) × 0.0316 = 0.0221 in.

B. Roof Short Beams (W 12×120)

For Short Beam span l = 8.0 ft. = 96 in.

Dead Load (D):

Ref: ACI 318-63, "Building Code Requirements for Reinforced Concrete", pg. 131,

Tributary Load W on central beam = (0.435)(96/3)(2) + 120/12 = 37.84 lb/in.
= 37.84 × 12 = 454.1 lb/ft

Max. Moment $M = wl^2/8 =$ (454.1/8) $(8)^2$ = 3632.6 lb-ft. = 43.59 kip-in

$$\text{Max. Deflection} = \frac{5wl^4}{384EI} = \frac{(5)(454.1)(8\times12)^4}{(12)(384)(29\times10^6)I} = 1.443/I$$

For W12 × 120; $S_x = 163\ \text{in}^3$; $I_x = 1070\ \text{in}^4$

Max. Bending Stress = 43.59/163 = 0.267 ksi
Max. Deflection = 1.443/1070 = 0.0013 in.
Max. Reaction Load = 454.1× 8/2 = 1816 lb.

Live Load (L):

w_L = 0.6944 × 96 × 2/3 = 44.44 lb./in

Max. Bending Stress = 44.44/37.84 × 0.267 = 0.314 ksi
Max. Deflection = 44.44/37.84 × 0.0013 = 0.0016 in.
Max. Reaction Load = 44.44 × 96/2 = 2133 lb.

Wind load (W):

w = -0.458 × (96/3) × 2 = -29.3 lb/in

Max. Bending Stress = (-29.3/37.84) × 0.267 = -0.207 ksi use -0.21 ksi
Max. Deflection = (-29.3/37.84) × 0.0013 = -0.0010 in.

Wind Internal Negative Pressure:

$w = 0.0745 \times 96 \times 2/3 = 4.768$ lb/in

Max. Bending Stress = (4.768/37.84) × 0.267 = 0.034 ksi
Max. Deflection = (4.768/37.84) × 0.0013 = 0.0002 in.

Tornado Wind (Ww)

Since wind pressure is proportional to velocity squared,

Max. Bending Stress = $(360/150)^2 \times -0.207 = -1.192$ ksi
Max. Deflection = $(360/150)^2 \times -0.001 = -0.0058$ in.

Wind Negative Internal Pressure:

Max. Bending Stress = $(360/150)^2 \times 0.034 = 0.196$ ksi
Max. Deflection = $(360/150)^2 \times 0.0002 = 0.0012$ in.

Tornado Wind Differential Pressure (Wp):

$w_p = -3.0 \times 96 \times 2/3 = -192.0$ lb/in

Max. Bending Stress = -192/37.84 × 0.267 = -1.35 ksi
Max. Deflection = - 192/37.84 × 0.0013 = -0.007 in.

SSE Seismic Load (Ess):

Max. Vertical Acceleration = 0.7g (DTS TSAR Table 8A.1-22)

Max. Bending Stress = 0.7 × Dead Wt. Stress = 0.7 × 0.267 = 0.19 ksi
Max. Deflection = 0.7 × 0.0013 = 0.0009 in.

C. Long Edge Beams (W12 X 120)

Span = 12 ft.

Dead Load (D):

Ref: ACI 318-63, "Building Code Requirements for Reinforced Concrete", pg. 131,

Effective Distributed load for Edge Beam, $w' = \frac{ws}{3}(3 - m^2)\frac{1}{2}$

$m = A/B = 8/12 = 0.667$ $s = 8.0$

$\therefore w' = 0.435 \times 144 \times (8/3)(3 - 0.667^2)(1/2) = 213.4$ lb/ft

Weight of edge beam = 120 lb/ft

Max. Moment $M = (213.4 + 120)\ 12^2/8 = 333.44 \times (12^2/8) = 6001.9$ ft lb. = 72.02 kip- in.

Max. Bending Stress = 72.02/163.0 = 0.442 ksi

Max. Deflection = $\dfrac{(5)(333.44)(12\times12)^4}{(12)(384)(29\times10^6)I} = 5.36/I = 5.36/1070 = 0.0050$ in.

Live load (L):

$w_L = (0.6944/0.435) \times 213.4 = 340.66$ lb/ft

Max. Bending Stress = (340.66/333.44) $\times$ 0.442 = 0.45 ksi
Max. Deflection = (340.66/333.44) $\times$ 0.005 = 0.0051 in.

Wind load (Ww):

$w_w = (-0.458/0.435) \times 213.4 = -224.7$ lb/ft

Max. Bending Stress = (-224.7/333.44) $\times$ 0.442 = -0.298 ksi
Max. Deflection = (-224.7/333.44) $\times$ 0.005 = -0.0034 in.

Wind Negative Internal Pressure:

$w_w = (0.0745/0.435) \times 213.4 = 36.55$ lb/ft

Max. Bending Stress = (36.55/333.44) $\times$ 0.442 = 0.05 ksi
Max. Deflection = (36.55/333.44) $\times$ 0.005 = 0.0005 in.

SSE Seismic Load (Ess):

Max. Bending Stress = 0.7 $\times$ 0.442 = 0.31 ksi
Max. Deflection =0.7 x 0.005 =0.0035 in.

Tornado Wind (Ww):

Max. Bending Stress = $(360/150)^2 \times (-0.298) = -1.72$ ksi
Max. Deflection = $(360/150)^2 \times (-0.0034) = -0.0194$ in.

Tornado Wind Internal Negative Pressure:

Max. Bending Stress = $(360/150)^2 \times (0.05) = 0.29$ ksi
Max. Deflection = $(360/150)^2 \times 0.0005 = .00029$ in.

Tornado Differential Pressure (Wp):

$w_p = (-3.0/0.435) \times 213.4 = -1471.7$ lb/ft

Max. Bending Stress = $(-1471.7/333.44) \times 0.442 = -1.95$ ksi
Max. Deflection = $(-1471.7/333.44) \times 0.0050 = -0.0221$ in.

D. 1 1/2" Steel Wall Plate Panel

Wind Load (W):

Design Pressure $p = q_z\, G\, C_p \pm q_h\, G\, C_{pi} = 59.62 \times 0.85 \times 0.8 \pm 59.62 \times 0.18$
(Refer to RAI#1, 3-6 for wind pressure coefficients)

Design Pressure $p = 40.54 \pm 10.73 = 51.27$ psf. = 0.356 psi

From R.J. Roark & W.C. Young, "Formulas for Stress and Strain", Fifth Edition, Table 26, Case 1,

For a 12 ft $\times$ 12 ft panel, $a = 144$ in, $b = 144$ in, and $a/b = 1.0$

$\beta = 0.2874$; $\alpha = 0.0444$

Max. Bending Stress = $0.2874 \times 0.356\,(144/1.5)^2 = 942.9$ psi = 0.94 ksi

$$\text{Max. Deflection} = \frac{(0.0444)(0.356)(144^4)}{(29\times10^6)(1.5^3)} = 0.069 \text{ in.}$$

Tornado Wind Load (Ww):

Max. Bending Stress = $(360/150)^2 \times 942.9 = 5431.3$ psi = 5.43 ksi
Max. Deflection = $(360/150)^2 \times 0.069 = 0.40$ in.

Tornado Differential Pressure (Wp):

Max. Bending Stress = $(3.0/0.356) \times 942.9 = 7946.0$ psi = 7.95 ksi
Max. Deflection = $(3.0/0.356) \times 0.069 = 0.585$ in.

SSE Seismic Load (Ess):

Horizontal Seismic Coefficient = 0.5g (Table 8A.1-22)

Horizontal Seismic load = 0.5 x (0.29 × × 1.5) = 0.2175 psi

Max. Bending Stress = (0.2175/0.356) × 942.9 = 576.1 psi = 0.58 ksi
Max. Deflection = (0.2175/0.356) × 0.069 = 0.0424 in.

E. Columns (W 12x120)

Column Height = h = 12 ft. = 144 in.

Dead Load (D):

Critical Column is one supporting middle & edge beam on the outside long wall.

Max. Vertical Reaction at Top of the Column = (454.1)(8/2) + (333.44)(21.5/2)
= 5400.9 lb.

Max. Reaction load at Bottom of the Column
= 5400.9 + (0.29)(1.5)(12)(21.5/2)(144) + (12)(120)
= 14921.4 lb = 14.92 Kip.

Cross Sectional Area A = 35.3 in^2

Max. Axial Stress = 14.92/35.3 = 0.42 ksi

Live Load (L):

Max. Reaction load from Ceiling Beams ≈ (44.44)(8)(12/2) + (340.66)(21.5/2)
= 5795.2 lb = 5.80 kip.
Max. Axial Stress = (5.80/35.3) = 0.16 ksi

Wind Load (W):

Tributary Wind Load = (0.356)(21.5)(12/12) = 45.92 lb./in. = 45.92×12 = 551.1 lb/ft.

Max. Bending Moment = $\dfrac{wl^2}{8}$ = $551.1(12)^2/8$ = 9919.6 lb-ft = 119.04 kip-in

Max. Deflection $= \dfrac{5wl^4}{384EI} = \dfrac{(5)(551.1)(12\times 12)^4}{(12)(384)(29\times 10^6)I} = 8.87/I$

For W12×120; I_x = 1070 in.^{4}; S_x = 163 in^3

Max. Bending Stress = 119.04/163 = 0.73 ksi
Max. Deflection = 8.87/1070 = 0.008 in.

Tornado Wind Load (Ww):

Max. Bending Stress = $(360/150)^2 \times 0.73$ = 4.21 ksi
Max. Horizontal Deflection = $(360/150)^2 \times 0.008$ = 0.048 in.

Tornado Differential Pressure (Wp):

Tributary Load = (3.0)(21.5)(12/2) = 387.0 lb./in. = 4644 lb/ft.

Max. Bending Stress = (4644/551.1) × 0.73 = 6.15 ksi
Max. Horizontal Deflection = (4644/551.1) × 0.008 = 0.070 in.

SSE Seismic Load (Ess):

Horizontal Seismic Coefficient = 0.5g (DTS TSAR Table 8A.1-22)

Tributary Horizontal Seismic Load = 0.5{0.29×1.5×21.5×(12/2) + (120/12)}×12
= 369.7 lb/ft

Max. Bending Stress = (369.7/551.1) × 0.73 = 0.53 ksi
Max. Horizontal Deflection = (369.7/551.1) × 0.008 = 0.006 in.

Table 8A.1-30
HVAC FAN/PLC ENCLOSURE
Summary of Stresses in Steel Structures Due to Various Loads

Critical Component	Location	Type of Stress	Maximum Stresses (ksi)				
			Dead Load (D)	Live Load (L)	Wind Load (W)	Thermal Load (Ta)	SSE Seismic Load (Ess)
Roof Panel (1.5" PL.)	Mid-span of Panel	Bending	0.86	1.38	0.15/-0.91	NA	0.61
Roof Support Beams (W12x120)	Mid-span of Beam	Bending	0.27	0.31	0.03/-0.21	NA	0.19
Long Edge Beams (W12x120)	Mid-span of Beam	Bending	0.44	0.45	0.05/-0.30	NA	0.31
Wall Panel (1.5" PL.)	Mid-span of Panel	Bending	-	-	0.94	NA	0.58
Support Columns (W12x120)	Mid-span of Column	Axial Bending	0.42	0.16	0.73	NA	0.53

Note: plus (+) and minus (-) signs signify pressures acting toward and away from the surfaces respectively.

Table 8A.1-30
HVAC FAN/PLC ENCLOSURE
Summary of Stresses in Steel Structures Due to Various Loads (continued)

Critical Component	Location	Type of Stress	Total Tornado Load (W_t)					
			Tornado Wind (Ww)	Tornado Differential Pressure (Wp)	Tornado Missile* (Wm)	Ww + 0.5 Wp	Ww + Wm	Ww + 0.5 Wp + Wm
Roof Panel (1.5" PL.)	Mid-span of Panel	Bending	-5.24	-5.96	-	-8.22	-5.24	-8.22
Roof Support Short Beams (W12x120)	Mid-span of Beam	Bending	-1.19	-1.35	-	-1.87	-1.19	-1.87
Long Edge Beams (W12x120)	Mid-span of Beam	Bending	-1.72	-1.95	-	-2.70	-1.72	-2.70
Wall Panel (1.5" PL.)	Mid-span of Panel	Bending	5.43	7.95	-	9.41	5.43	9.41
Support Columns (W12x120)	Mid-span of Column	Axial / Bending	4.21	6.15	-	7.29	4.21	7.29

* Maximum depth of penetration for tornado generated missiles in. horizontal and vertical directions are 0.64" and 0.40" respectively. These depths of penetration are less than the available thickness of 1.5" of the cover plate, confirming no pucture. External kinetic energy will be absorbed in. the elastic/plastic deformation for the missile and target structure. Maximum stresses in. the vicinity of the impact will exceed the yield stress value. However due to the unibody type of construction, there will not be any collapse of the fan and PLC enclosure structure.

Table 8A.1-31
HVAC FAN/PLC ENCLOSURE
Summary of Stresses in Steel Structures Due to Various Load Combinations

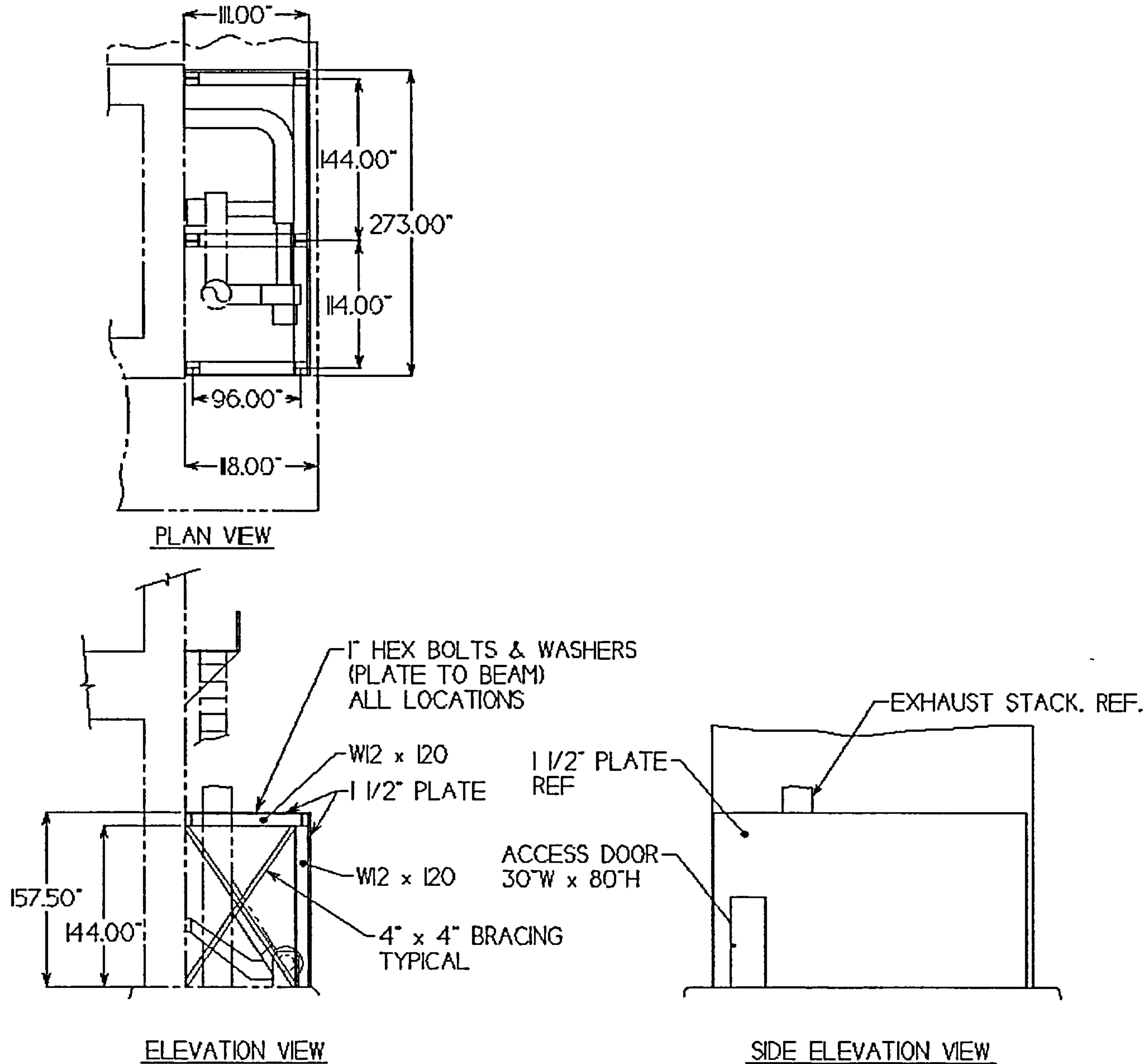
Critical Component	Location	Type of Stress	Maximum Stresses (ksi)						
			D+L	D+L+W	Allowable Stress 'S'*	D + L + Wt	D + L +T_a+ Ess	D+L+Ta	Allowable Stress MIN('1.6S', Fy**)
Roof Panel (1.5"PL.)	Mid-span of Panel	Bending	2.24	2.39	27.00	-5.98	2.85	2.24	36.00
Factor of safety			12.05	11.30	-	6.02	12.63	16.07	-
Roof Support Beams (W12x120)	Mid-span of Beam	Bending	0.58	0.61	24.00	-1.29	0.77	0.58	36.00
Factor of safety			41.38	39.34	-	27.91	46.75	62.07	-
Long edge beams (W12x120)	Mid-span of Beam	Bending	0.89	0.94	24.00	-1.81	1.20	0.89	36.00
Factor of safety			26.97	25.53	-	19.89	30.00	40.45	-
Wall Panel (1.5"PL.)	Mid-span of Panel	Bending	-	0.94	27.00	9.41	0.58	-	36.00
Factor of safety			-	28.72	-	3.83	62.07	-	-
Support Columns (W12x120)	Mid-span of Column	Bending	0.58	1.31	24.00	7.87	1.11	0.58	36.00
Factor of safety			41.38	18.32	-	4.57	32.43	62.07	-

1.0S is allowable stress based on AISC Manual of Steel Construction, ninth edition.
** Fy is yield stress of steel.

D = Dead Load W_t = Total Tornado Load W = Wind Load L = Live Load

E_{ss} = Seismic Load T_a = Thermal Load S = Allowable Stress

Figure 8A.1-24
Exhaust Fan & PLC Enclosure



8A.1.8 References

8A.1.8-1 American Concrete Institute ACI 349-1R: Code Requirements for Nuclear Safety Related Concrete Structures - Thermal effects.

8A.1.8-2 Formulas for Stress and Strain by R. Roark, Fourth Edition.

8A.1.8-3 ANSYS Engineering Analysis Systems User's Manual Volume 1 and 2, Revision 4.4A.

8A.1.8-4 NRC Regulatory Guide 1.76, "Design Basis Tornado for Nuclear Power Plants," April 1974.

8A.1.8-5 Guidelines for the Design and Assessment of Concrete Structures Subjected to Impact, UKAEA, SRDR439, Issue 3, May 1990.

8A.1.8-6 U.S. Nuclear Regulatory Commission, Regulatory Guide 1.60. Design Response Spectra for Seismic Design of Nuclear Power Plants, 1973.

8A.1.8-7 U.S. Nuclear Regulatory Commission, Regulatory Guide 1.61. Damping Values for Seismic Design of Nuclear Power Plants, 1973.

8A.1.8-8 J.R. Mcdonald, K.C. Mehta, and J.E. Minor, "Design Guidelines for Wind Resistant Structures," Institute for Disaster Research and Department of Civil Engineering, Texas Tech University, Lubbock, Texas, June 1975.

8A.1.8-9 "Design of Structures for Missile Impact," Bechtel Topical Report, BC-TOP-9A (8.51).

8A.1.8-10 Formulas for Stress and Strain by R. Roark, Fifth Edition.

Appendix 8A.2 Cask Transfer Subsystem Analysis

This appendix describes the analysis performed on the cask transfer subsystems. The Cask Transfer Subsystem accepts vertical casks at the entrance to the Preparation Area and moves it laterally in the X direction to the Lower Access Area, aligns it with the Cask Mating Subsystem and supports it during transfer of the spent fuel assembly.

The following calculations form a part of this appendix:

1. Analysis of the Locking Pins
2. Analysis of the Transmission Cradles
3. Analysis of the anti-derailing devices
4. Analysis of the guidance rollers and wheels.

The Cask Transfer Subsystem is designed to receive 157 shipments of PWR fuel or 133 shipments of BWR fuel per year (or approximately 53 shipments of PWR fuel or 44 shipments of BWR fuel per 100 day operating period).

The operating period is defined as the calendar year less the annual maintenance periods. The standard operating period is 300 days/year (24 hours/working day), with an average cycle of a 100 day operating period followed by a 21 - 22 day maintenance period.

The production period is defined as the operating period less the shutdown periods due to routine maintenance or due to equipment failure and the corresponding repair time.

The cask transfer subsystem is designed to operate at temperatures between 40°F to 130°F, or at temperatures between 20°F to 200°F for short periods of time.

8A2.1 Source Cask Transfer Subsystem General Description

The transfer of the source cask is performed by a motor driven trolley on rails. This trolley is designed to be loaded with the source cask. Centering guides ensure that the cask is properly positioned on the trolley.

The source cask is held onto the trolley by means of its lower trunnions. Single-piece devices, all-bolted on the trolley plate and above the trunnions, are removable and cask specific. The bolted plates weigh a maximum of 60 pounds (27 kg) and are manually removed.

The trolley structure and the cask holding system prevent the cask from falling due to any design event.

The cask is elevated 15.7 inches (400 mm) above the base of the trolley, to allow proper alignment with the source cask mating subsystem.

When the trolley is stopped in a specific position in the Preparation area or in the Lower Access Area, it is locked at its front by means of a vertical pin actuated by a jack, which penetrates into the concrete base mat of the DTS. The locking pin prevents the trolley from accidental forward and backward movement along the rails, and also prevents the trolley from moving due to a seismic event.

The general characteristics of the source cask transfer trolley are presented in Table 8A.2-1.

Table 8A.2-1
Source Cask Transfer Trolley Characteristics

	U.S. Units	Metric Units
Overall Dimensions	10.2 ft x 8.5 ft x 4.4 ft	3.1 m x 2.6 m x 1.3 m
Runway Length	49 ft	15 m
Span	9.1 ft	2.8 m
Wheelbase	6.6 ft	2 m
Maximum Design Load	30 tons	27.2 mtons
Material	Main components are painted carbon steel. Wheels are carbon steel. The beams and plates are A36. The bolts are A193-B7.	
Coating	Coating meets requirements of Category A - Service Level 1 coating as defined in ASME-NOG-1.	

The trolley moves along the X-axis through the use of an synchronous motor/brake with a manual brake disengagement. Two of the four trolley wheels (one on each side) are driven. The trolley has two speeds 0.7 ft/min (0.2 m/min) and 10 ft/min (3 m/min). The trolley has 2 braking systems: a service brake and an emergency brake. The emergency brake is used as a parking brake.

Trolley guidance is made by two sets of lateral rollers on one of the two runway rails. An anti-taking off device is implemented on both rails. See Figure 8A.2-1. Source cask guidance, during loading, is made by four centering guides.

A summary of the calculated dimensions, based on seismic and static loads is presented in Table 8A.2-2.

Figure 8A.2-1

Trolley Guidance and Anti-Taking Off Device

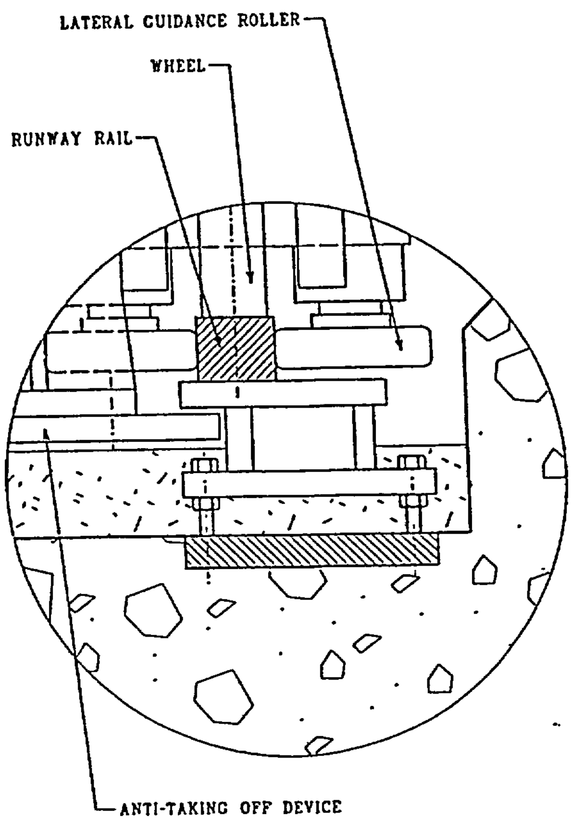


Table 8A.2-2

Calculated Dimensions of Source Cask Trolley

Part	Load	Calculated Size
Bolts of Cradle	Seismic	6 bolts M30 (1.2 in.)
Plate of anti-taking off device	Seismic	30 mm thick (1.2 in.)
Bolts of anti-taking off Device	Seismic	4 bolts M16 (0.6 in)
Diameter of the locking pin	Seismic	80 mm (3.2 in.)
Wheel diameter	Static	450 mm (17.7 in.)
Rail width minimum	Static	40 mm (1.6 in.)
Guidance roller	Static	50.8 mm (2.00 in.)
Rail height minimum	Static	31.75 mm (1.25 in.)

8A.2.2 Receiving Cask Transfer Subsystem General Description

The transfer of the receiving cask is performed by a motor driven trolley on rails. This trolley is designed to be loaded with the receiving cask. The structure of the trolley prevents the cask from tipping under all design events. Centering guides ensure that the cask is properly positioned on the trolley.

The receiving cask is held onto the trolley by means of its lower trunnions. Single-piece devices, all-bolted on the trolley plate and above the trunnions, are removable and cask specific. The bolted plates weigh a maximum of 60 pounds (27 kg) and are manually removed.

When the trolley is stopped in a specific position in the Preparation area or in the Lower Access Area, it is locked at its front by means of a vertical pin actuated by a jack, which penetrates into the concrete base mat of the DTS. The locking pin prevents the trolley from accidental forward and backward movement along the rails, and also prevents the trolley from moving due to a seismic event.

The general characteristics of the receiving cask transfer trolley are presented in Table 8A.2-3.

Table 8A.2-3

Receiving Cask Transfer Trolley Characteristics

	U.S. Units	Metric Units
Overall Dimensions	11.2 ft x 10.2 ft x 3.4 ft	3.4 m x 3.1 m x 1.0 m
Runway Length	49 ft	15 m
Span	9.2 ft	2.8 m
Wheelbase	8.9 ft	2.7 m
Maximum Design Load	125 tons	113.4 mtons
Material	Main components are painted carbon steel. Wheels are carbon steel. The beams and plates are A36. The bolts are A193-B7.	
Coating	Coating meets requirements of Category A - Service Level 1 coating as defined in ASME-NOG-1.	

The trolley moves along the X-axis through the use of an synchronous motor/brake with a manual brake disengagement. Two of the four trolley wheels (one on each side) are driven. The trolley has two speeds 0.7 ft/min (0.2 m/min) and 10 ft/min (3 m/min). The trolley has 2 braking systems: a service brake and an emergency brake. The emergency brake is used as a parking brake.

Trolley guidance is made by two sets of lateral rollers on one of the two runway rails. An anti-taking off device is implemented on both rails. See Figure 8A.2-1. Receiving cask guidance, during loading, is made by four centering guides.

A summary of the calculated dimensions, based on seismic and static loads is presented in Table 8A.2-4.

Table 8A.2-4

Calculated Dimensions of Receiving Cask Trolley

Part	**Load**	**Calculated Size**
Bolts of Cradle	Seismic	6 bolts M30 (1.2 in.)
Plate of anti-taking off device	Seismic	40 mm thick (1.6 in.)
Bolts of anti-taking off Device	Seismic	4 bolts M24 (1 in)
Diameter of the locking pin	Seismic	120 mm (4.8 in.)
Wheel diameter	Static	700 mm (27.6 in.)
Rail width minimum	Static	100 mm (3.9 in.)
Guidance roller	Static	88.9 mm (3.50 in.)
Rail height minimum	Static	50.8 mm (2.00 in.)

8A.2.3 Runway Rails - General Description

The same runway rails are used by the source and receiving cask transfer subsystems. The runway rail length is 49 feet (15 m). At the end of each runway, there is a bumper guard. The rail tolerances are shown in Table 8A.2-5.

At the attachment position (beneath the cask mating subsystems), the overall tolerance shall be, in the vertical direction (for rails + trolley on rails + cask on trolley) ± 5 mm/m (± 0.005 in/in).

The runway rails are shown in Figure 8A.2-2.

Table 8A.2-5

Rail Tolerances

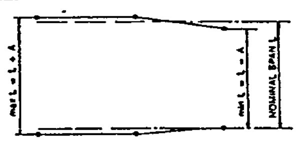
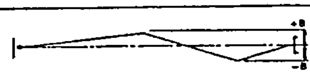
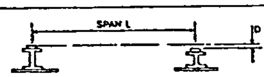
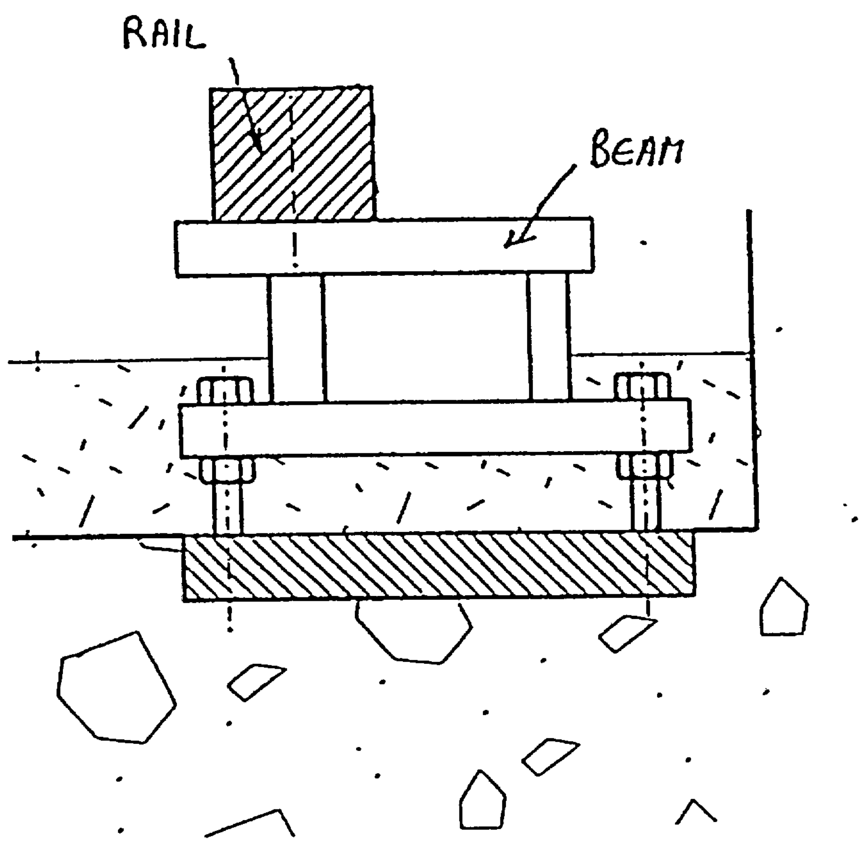
ITEM	FIGURE	OVERALL TOLERANCE	MAXIMUM RATE OF CHANGE
SPAN		L ≤ 50' A = 3/16"	1/4" in 20'-0"
Straightness		B = 3/8"	1/4" in 20'-0"
Elevation		C = 3/8"	1/4" in 20'-0"
Rail-to-rail elevation		L ≤ 50' D = ± 3/16"	1/4" in 20'-0"

Figure 8A.2-2

Runway Rails



8A.2.4 Source Cask Trolley Calculations

8A.2.4.1 Assumptions

It is assumed that the trolley is protected from environmental loads such as wind and snow by the DTS or the Preparation Area Roof. When the trolley is in the cask loading area, the cask is fully sealed and locked. It is assumed that damage due to the source cask during loading on to the trolley has been evaluated as part of the licensing process of the source cask.

The transfer trolley for the source (or receiving) cask will be subjected to maximum stresses during a seismic event. The stresses in the trolley will be the same regardless of its location on the rails. The only other accident that could occur would be for the source and receiving cask transfer trolleys to bump into each other. It is assumed that the bumpers will be designed such that the forces produced during the accident event are less than those developed from a seismic event. During fuel transfer, the trolleys will be pinned in their respective positions, and will not bump into each other during normal operation or a seismic event. Analyses will be conducted on a site-specific bases to ensure that the source (or receiving) cask will not tip over due to a tornado missile impact while in the Preparation Area.

The trunnion hold downs and the anti-derailing devices are calculated with the seismic load and the live load. The guiding roller wheels are calculated with the live load with motion.

The material properties used in the analysis are taken from NOG-1, Tables NOG 4211-1 and 4221-1 and are presented in Table 8A.2-6 below.

The trolley and casks are assumed to be rigid. The length of the source cask is 4,826 mm (190 inches). The outside diameter of the source cask is 1,028 mm (40.5 inches).

Table 8A.2-6

Properties of Materials

Material	Yield Strength	Ultimate Strength
A36	36 ksi (248 MPa)	58 ksi (399 MPa)
A193-B7	75 ksi (517 MPa)	100 ksi (689 MPa)

8A.2.4.2 Design Criteria

The design criteria are taken from ASME NOG-4300 and are repeated below. The nomenclature of NOG-4120 is used.

For beams subjected to axial tension and bending:

$$\sigma / \sigma_a + \sigma_{bx} / \sigma_{abx} + \sigma_{by} / \sigma_{aby} \leq 1.0 \qquad (1)$$

where $\sigma_a = \sigma_{abx} = \sigma_{aby} = 0.9$ Sy (for the seismic case)

σ_a is the axial stress
σ_{abx} and σ_{aby} are the stresses due to the bending moment.

The maximum allowable shear stress under seismic load is 0.5 σ_y.

An additional safety factor of 1.2 is used to take into account imprecision of the data.

For the beams, which are made from A36 steel:

The tensile stress allowable is:

$$F/A + M_{bx}x/ I_x + M_{by}y/ I_y < 0.9\ \sigma_y/1.2 = 186 \text{ MPa} = 27.0 \text{ ksi} \qquad (2)$$

The shear stress allowable is $0.5\sigma_y /1.2 = 103$ MPa = 15 ksi.

where A is the cross sectional area.
F is the axial force
M_{bx} and M_{by} are bending moments about the x and y axes
I_x and I_y are the moduli of inertia

The allowable stresses in the bolts are taken from NOG-4513. For seismic loading, the maximum allowable tensile load is 0.5 σ_u, and the maximum allowable shear stress is 0.26 σ_u. An additional safety factor of 1.2 is used to take into account imprecision of the data.

Therefore, the allowable tensile stress is 0.5 x 100 ksi / 1.2 = 287 MPa =41.6 ksi. The allowable shear stress is 149 MPa = 21.6 ksi.

The transfer trolleys shall be designed such that deflections do not impair proper operation of machinery as required by ASME NOG-4340.

8A.2.4.3 Seismic Loading

The source cask and source cask trolley are assumed to be rigid. Since the trolley and source cask are not perfectly rigid, an addition factor of 1.5 is used.

The trolley is analyzed for a horizontal g loading of 0.25 g x 1.5 = 0.375 g and a vertical g loading of 0.17 g x 1.5 = 0.255 g.

The trolley's response to each of the three components of seismic input are combined by taking the square root of the sum of the squares (SRSS) per NOG-4153.10:

$SRSS = \sqrt{(Sx^2 + Sy^2 + Sz^2)}$

The seismic analysis was performed for two load combinations: seismic loading + static loading and seismic loading - static loading. The static load is the live load of the cask and the trolley due to gravity.

The acceleration due to gravity is $g = 9.81 \text{ m/sec}^2$.

8A.2.4.4 Operational Loading

The following operational loads are taken into account:

- The live load of the cask and trolley under gravity (Used for calculating the stresses on the wheels)
- The transverse horizontal derailing load (24% of the live load of the cask plus the trolley dead load).This load is used to size the guiding rollers.

An isometric sketch of the cask on the trolley is shown in Figure 8A.2-3. A top view of the cask on the trolley is shown in Figure 8A.2-4. The bumpers are welded to the beams of the trolley.

8A.2.4.5 Evaluation of Bolts

The trunnion cradles are shown in Figure 8A.2-5. Six M30 (1.2 in. diameter) bolts are evaluated using the maximum vertical reaction in the + Z direction on the cradle. This reaction is obtained by subtracting the static load from the seismic load.

X direction seismic loading

The seismic load in the X direction is applied at the center of gravity of the cask, G. The load is reacted at the point C (Figure 8A.2-3) which is the center of the compression zone. The seismic loading in the X direction is reacted by the trolley base at C and by the cradle supports. The force in the x direction is the seismic acceleration in the x direction times the mass of the cask.

$F_x = ma_x$

The distance to the center of the compression zone is $L_1 = D/3$ where D is the diameter of the cask. OG is the distance between the center of gravity and the base of the cask. Assuming that the cask weight is approximately equally distributed along its length, OG = L/2.

The reaction forces in the z direction due to the x axis seismic acceleration at the trunnion locations are equal due to symmetry :

$R_{BZX} = R_{AZX}$

Figure 8A.2-3

Isometric Sketch of Cask on Trolley

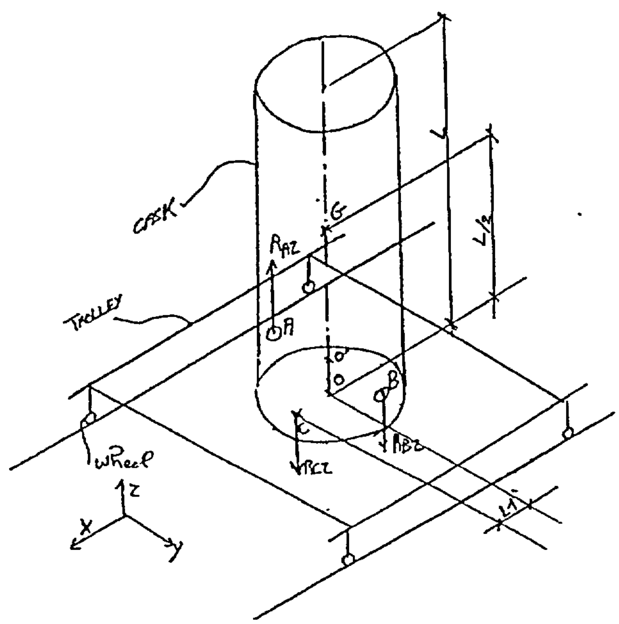


Figure 8A.2-4

Top View of Cask on Trolley

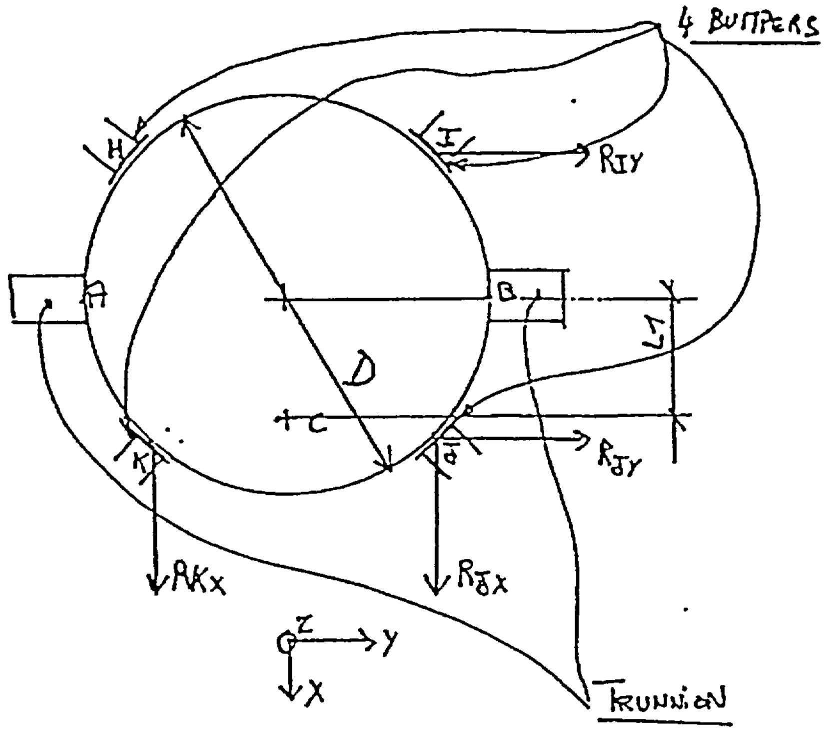
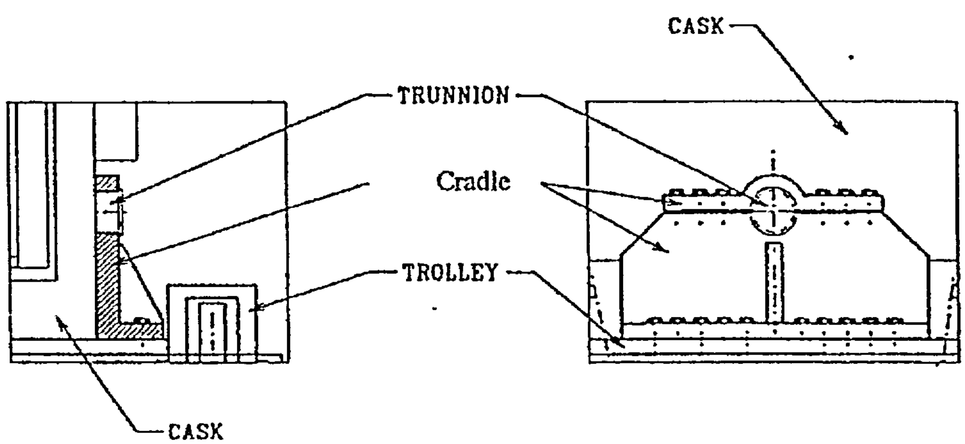


FIGURE 8A2-5

CASK TRANSFER SUBSYSTEM TRUNNION CRADLES



Then, summing the equations in the z direction:

$$R_{BZX} + R_{AZX} + R_{CZX} = 0$$

Therefore,

$$R_{AZX} = R_{BZX} = -R_{CZX}/2$$

Summing the moments about C to 0:

$$F_x \text{ x } OG - (R_{AZX} + R_{BZX})L_1 = 0 \text{ or}$$

$$R_{AZX} = R_{BZX} = F_x \text{ x } OG/2L_1 = ma_xL/2/(2D/3) = ma_x(3L/4D)$$

This reaction force is taken by the cradle.

The x axis reaction is equally taken by the two bumpers. Figure 8A.2-4)

$$R_{KX} = R_{JX} = ma_x/2$$

<u>Y direction seismic loading</u>

The seismic load in the Y direction is applied at the center of gravity of the cask, G. The load is reacted at the point B (Figure 8A.2-3) which is the center of the compression zone and by one cradle at location A.

The force in the y direction is the seismic acceleration in the y direction times the mass of the cask.

$$F_y = ma_y$$

OG is the distance between the center of gravity and the base of the cask. Assuming that the cask weight is approximately equally distributed along its length, OG = L/2.

AB is the distance between the two cradles. AB is approximately equal to D.

The static equations are:

$$F_yOG - R_{AZY}AB = 0$$

Therefore,

$$R_{AZY} = -R_{BZY} = F_yOG/AB = ma_yL/2D \text{ (cradle reaction force)}$$

Summing the forces in the Y direction:

$R_{IY} = R_{JY} = ma_y/2$ (bumper reaction forces)

Z direction Seismic Loading

The Z direction seismic loading is equally divided between the two cradles:

$F_z = ma_z$

$R_{AZZ} = R_{BZZ} = ma_z/2$ (cradle reaction loads)

Static Z direction loads

The weight of the cask is equally divided by the two cradles.

$P_z = -Mg$

The vertical reactions at the two cradle locations are R_{AZS} and R_{BZS}.

$R_{AZS} = R_{BZS} = -Mg/2$ (at the cradles)

Load Combination

Combining the results:

The maximum +Z reaction on the cradle is: (Seismic - Static)

$R_{AZ} = \sqrt{(R_{AZX}{}^2 + R_{AZY}{}^2 + R_{AZZ}{}^2)} + R_{AZS}$

The maximum X and Y reaction on the bumper is at J, where:

$R_{JX} = ma_x/2$

and $R_{JY} = ma_y/2$

Solving numerically:

$R_{AZX} = ma_x(3L/4D) = 388,577$ N

$R_{AZY} = ma_yL/2D = 259,052$ N

$R_{AZZ} = ma_z/2 = 37{,}524$ N

$R_{AZS} = -Mg/2 = -147{,}150$ N

$R_{JX} = ma_x/2 = 110{,}363$ N

$R_{JY} = ma_y/2 = 110{,}363$ N

Therefore, the maximum reaction force in the Z direction on the cradle is:

$$R_{AZ} = \sqrt{(R_{AZX}^2 + R_{AZY}^2 + R_{AZZ}^2)} + R_{AZS} = 321{,}366 \text{ N} = 72{,}293 \text{ lbf}$$

Six M30 bolts attach each cradle. The vertical force R_{AZ} is the tension in the bolts.

The cross sectional area for an M30 bolt is $A_B = 561$ mm^2

The tensile stress in the bolt is:

$$\sigma = R_{AZ}/6A_B = 96 \text{ MPa} = 13.9 \text{ ksi} \leq 287 \text{ MPa} = 41.6 \text{ ksi}$$

The safety factor in the bolts is SF = 287/96 = 3. Therefore the stresses are acceptable.

8A.2.4.6 Evaluation of Anti-Taking Off Device and Locking Pin

The Anti-taking off device is shown in Figure 8A.2-1. The anti-taking off device prevents the trolley from tipping during a seismic event. This section calculates the stresses on the anti-taking off device. Figures 8A.2-6 through 8A.2-8 are used to perform this analysis.

The anti-taking off device is sized to withstand the maximum vertical force in the Z-direction. This force is obtained by combining the static and seismic loads. The reaction forces for each load step (x-direction seismic load, y-direction seismic load, z-direction seismic load and static load) are calculated and then combined to determine the maximum reaction force.

X direction Seismic Load

The seismic load on the trolley (M_t) is applied at G' shown in Figure 8A.2-8. The seismic load on the cask (M) is applied at G shown in Figure 8A.2-8. The seismic load is reacted by compression on the wheel at locations A & D and tension in the anti taking-off devices at locations B and C.

The vertical reaction forces due to the x direction seismic load at the anti-taking off devices are defined as R_{BZX} and R_{CZX}. From symmetry, $R_{BZX} = R_{CZX}$.

O'G is the vertical distance between the center of gravity of the cask and the base of the anti-taking off device.

Figure 8A.2-6

Cask on Trolley

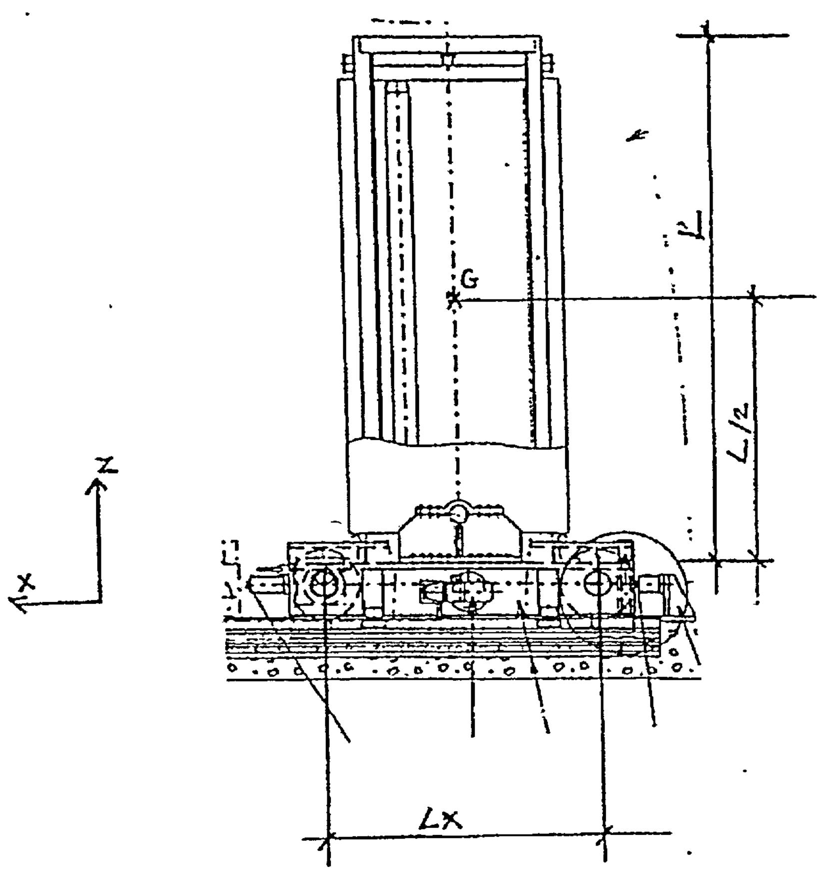


Figure 8A.2-7

Rails of Trolley

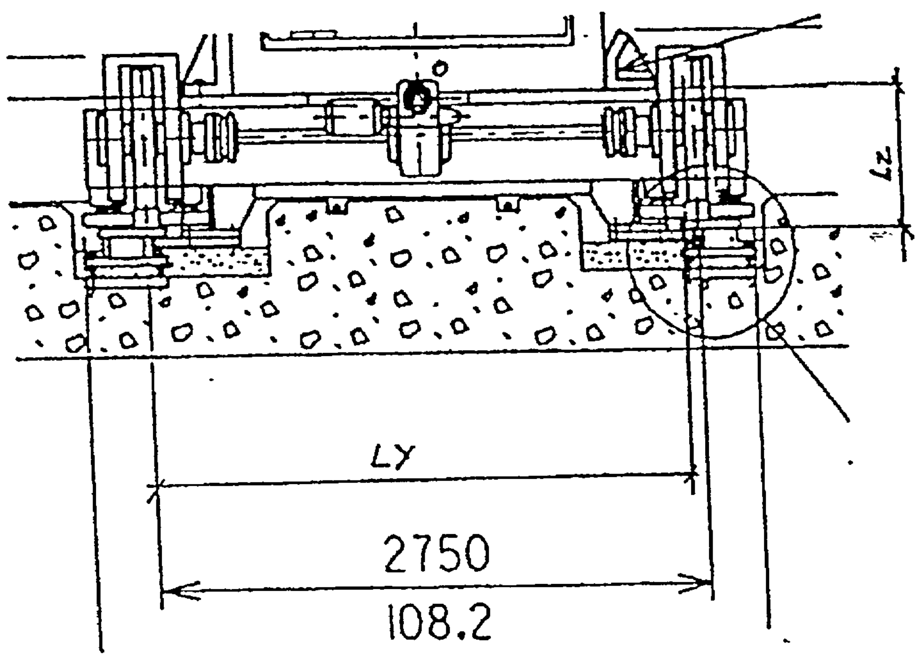
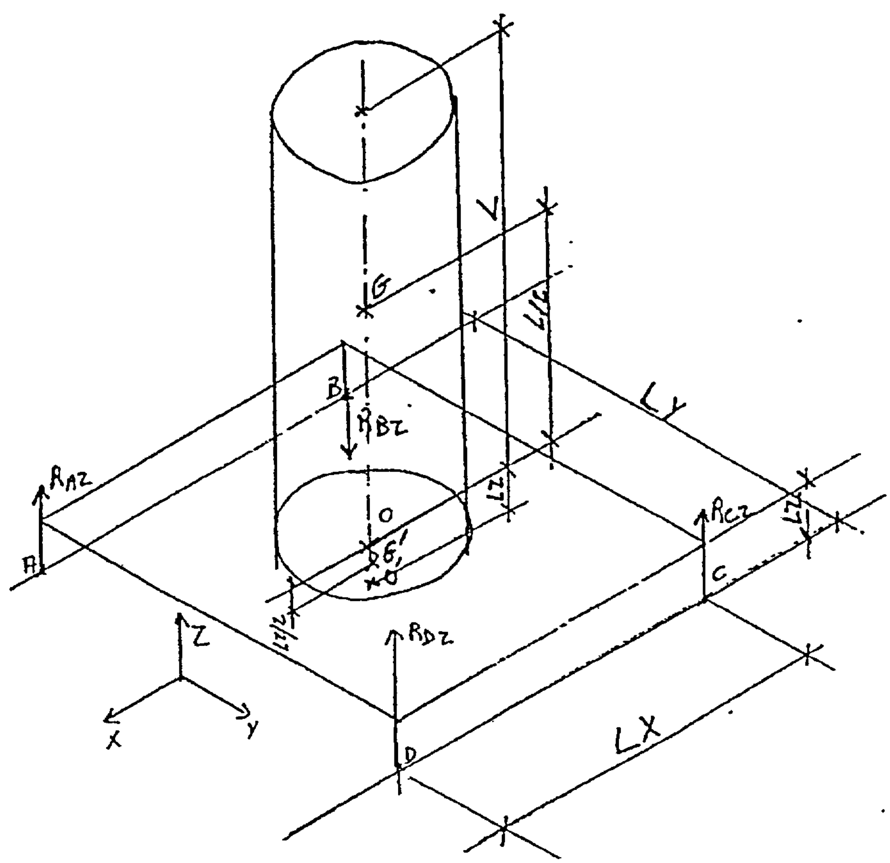


Figure 8A.2-8

Cask on Trolley
Showing Distances between Reaction Forces



From Figure 8A.2-8,

$$O'G = L/2 + L_Z.$$

O'G' is the vertical distance between the center of gravity of the trolley and the base of the anti-taking off device.

$$O'G' = L_Z/2.$$

Summing the moments to zero about AD:

$$M_t a_x O'G' + M a_x O'G - (R_{BZX} + R_{CZX})L_x = 0$$

Therefore,

$$R_{BZX} = (M_t a_x O'G' + M a_x O'G)/2L_x \text{ or}$$

$$R_{BZX} = (M_t a_x L_Z/2 + M a_x (L/2 + L_Z))/2L_x$$

The locking pin takes the force in the x direction:

$$R_X = (M + M_t)a_x$$

<u>Y direction Seismic Loading</u>

The seismic load on the trolley (M_t) is applied at G' shown in Figure 8A.2-8. The seismic load on the cask (M) is applied at G shown in Figure 8A.2-8. The seismic load is reacted by compression on the wheel at locations C & D and tension in the anti taking-off devices at locations A and B.

The vertical reaction forces due to the y direction seismic load at the anti-taking off devices are defined as RB_{AZY} and R_{BZY}. From symmetry, $R_{AZY} = R_{BZY}$.

O'G is the vertical distance between the center of gravity of the cask and the base of the anti-taking off device.

From Figure 8A.2-8,

$$O'G = L/2 + L_Z.$$

O'G' is the vertical distance between the center of gravity of the trolley and the base of the anti-taking off device.

$$O'G' = L_Z/2.$$

Summing the moments to zero about AD:

$$M_t a_y O'G' + M a_y O'G - (R_{AZY} + R_{BZY})L_y = 0$$

Therefore,

$$R_{BZY} = (M_t a_y O'G' + M a_y O'G)/2L_y \text{ or}$$

$$R_{BZY} = (M_t a_y L_Z/2 + M a_x (L/2 + L_Z))/2L_y$$

The y direction force is taken by the superior plate under the rail on the two anti-taking off devices at A and B.

$$R_{AY} = R_{BY} = (M + M_t)a_y/2$$

Z Direction Seismic Loading

The z direction seismic loads are taken equally by the four anti-taking off devices:

$$R_{BZZ} = (M + M_t)a_z/4$$

Static Z Direction Loading

The static compression load is taken equally by the four wheels. The dead load is:

$$P_Z = (M + M_t)g$$

The vertical compression at point B is:

$$R_{BZS} = -(M + M_t)g/4$$

Load Combination

Combining the results:

The maximum vertical force on the anti-taking off device is the combination of the static + seismic load:

$$R_{BZ} = \sqrt{(R_{BZX}^2 + R_{BZY}^2 + R_{BZZ}^2)} - R_{BZS}$$

The maximum force on the locking pin is:

$$R_X = (M + M_t)a_x$$

The maximum force in the Y direction on the anti-taking off device is:

$R_{BY} = (M + M_t)a_y / 2$

Solving numerically:

given $M = 30$ tons
$L = 4{,}826$ mm
$M_t = 15$ tons
$L_x = 2000$ mm
$L_Y = 2550$ mm
$L_z = 600$ mm
$a_x = 0.375$ g
$a_y = 0.375$ g

Then $R_{BZX} = 87{,}269$ N
$R_{BZY} = 68{,}446$ N
$R_{BZZ} = 28{,}142$ N
$R_{BZS} = -110{,}363$ N

The forces taken by the anti-taking off device are:

$R_{BZ} = 4{,}061 \text{ N} = 912 \text{ lbf}$
$R_{BY} = 82{,}772 \text{ N} = 18{,}694 \text{ lbf}$

The force on the locking pin is:

$R_x = 165{,}544 \text{ N} = 37{,}216 \text{ lbf}$

The dimensions of the Anti-Taking Off Device are shown in Figure 8A.2-9. The anti-taking off device is made from A36 carbon steel, with A193 B7 bolts.

The maximum bending moment in the plate is at bolt position 2:

$M_{max} = R_{BZ}\, a = (4061 \text{ N})(50\text{mm}) = 203 \text{ mN}$

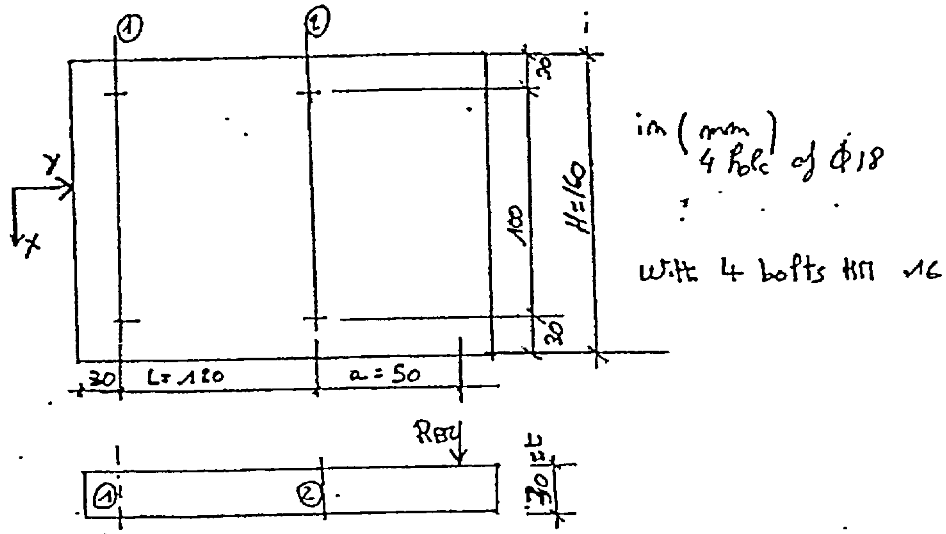
The maximum tension in a bolt occurs at position 2:

F = tension in bolt
A_b = bolt cross sectional area = 157 mm^2

Then $2FL - R_{BZ}(L + a) = 0$

Figure 8A.2-9

Source Cask Anti-Taking Off Device



Solving for F:

$$F = R_{BZ}\,(L + a)/2L = 2880\ N$$

The tensile stress in the bolt is:

$$\sigma = F/A_b = 2880/157 = 19\ MPa \leq\ 287\ MPa$$

The safety factor is:

$$SF = 287/19 = 15$$

The section modulus of the plate is:

$$S = (H\text{-}2{*}18)t^3/6t = 18{,}600\ mm^3$$

where t = the thickness of the plate = 30 mm and H = 160 mm.

The bending stress in the plate is:

$$\sigma = M_{max}/\ S = \ 11\ MPa = 1.6\ ksi \leq 186\ MPa = 27\ ksi$$

The safety factor is SF = 186/11 = 17.

The force R_{BY} is taken directly by the trolley.

The locking pin is shown in Figure 8A.2-10. The pin diameter is D = 2L = 80 mm.
The shear stress in the pin is $\tau = R_x / A = 33$ MPa ≤ 103 MPa = 15 ksi where

$R_x = 165{,}544\ N = 37{,}216\ lbf$

and $A = \pi D^2/4 = 5{,}026\ mm^2$

The safety factor on the locking pin is SF = 103/33 = 3.1.

8A.2.4.7 Evaluation of Wheels

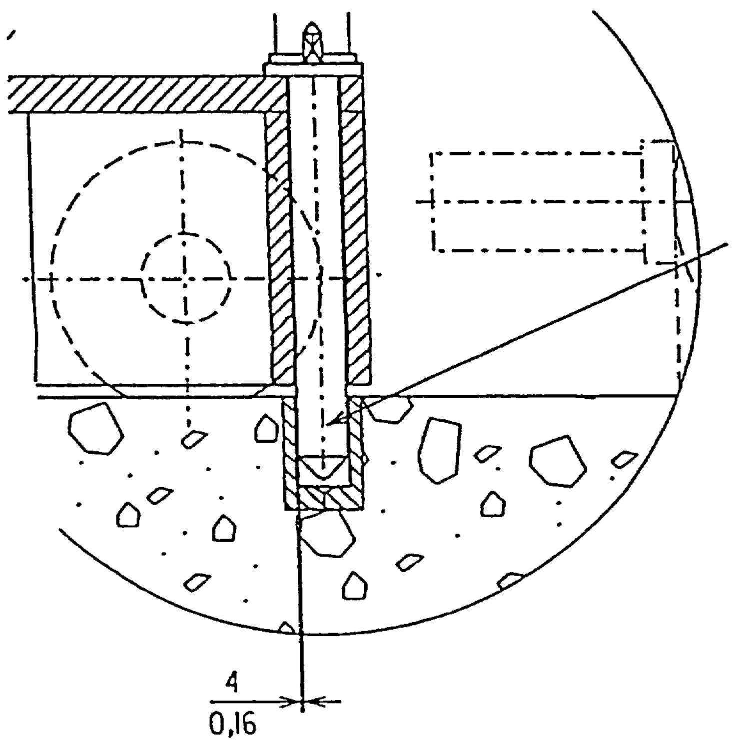
The wheels are sized based on static loads. The trolley wheels are 450 mm in diameter (17.7 in. The effective width of the rail head is 40 mm (1.6 in). The allowable wheel load is taken from NOG 5452.3:

$$P_a = KbD\ (lbs)$$

where $K = 1300\ (BHN/260)^{0\ 333} = 1393$

Figure 8A.2-10

Locking Pin



The load is equally distributed between the four wheels. Therefore the load on each wheel is:

$$F_z = (M + M_t)g/4 = 110{,}363 \text{ N} = 24{,}811 \text{ lbf}$$

The allowable load is $P_a = KbD = 1393(1.6)(17.7) = 39{,}449$ lbf. Therefore the loads on the wheels are acceptable. The safety factor is SF = 39449/24811 = 1.5.

8A.2.4.8 Evaluation of the Guidance Rollers

The purpose of the guidance rollers is to prevent the trolley from derailing in the event that it begins to misalign transversely with the rails during normal operation. There are two sets of guidance rollers with each set adjacent to each trolley wheel on one rail. Conservatively (since the wheels are sliding and rolling simultaneously), the transverse load (F_y) required to maintain the trolley on track can be assumed to be equal to the dead load of the trolley plus the loaded cask multiplied by the coefficient of sliding friction (f_z).

The transverse load on each guidance roller is:

$$F_y = (M + M_t)g\, f_z /2 = 52{,}974 \text{ N} = 11{,}910 \text{ lbf}$$

Where: M = Mass of the rated load = 30,000 kg
M_t = Mass of the cask trolley = 15,000 kg
$g = 9.81$ m/sec^2
f_z = 0.24 for a steel wheel on a steel rail
(Reference 8-17, Mark's Standard Handbook for Mechanical Engineers)

Therefore, utilize a McGill (or equivalent) CFH-2-B stud type cam follower with a roller diameter of 2.00 in., roller width of 1.25 in. and a maximum static rated capacity of 21,140 lbf. The safety factor is SF = 21,140/11,910 = 1.78

Since the transverse derailing load of 52,974 N is less than the seismic transverse derailing load $R_{BY}/2$ of 41,386 N, the trolley anti-taking off device is structurally adequate as the normal operation anti-derailing device per the analysis of Section 8A.24.6. Hence the guidance rollers need not be classified as SSCs important to safety.

8A.2.4.9 Summary of Stresses - Source Cask Transfer Trolley

The sizes of the structural components of the source cask trolley, together with the calculated stresses and allowable stresses are presented in Table 8A.2-7.

Table 8A.2-7

Summary Source Cask Transfer Trolley Stresses

Part	Loading	Allowable stress or value	Calculated Stress or Value	Size	Safety Factor
Bolts of Cradle	Seismic	287 MPa	96 MPa	6 bolts M30 (1.2 in dia.)	3
Plate of anti-taking off device	Seismic	186 MPa	11 MPa	30 mm thick (1.2 in)	17
Bolts of anti-taking off Device	Seismic	287 MPa	19 MPa	4 bolts M16 (0.6 in)	15
Diameter of the locking pin	Seismic	103 MPa	33 MPa	D = 80 mm (3.2 in)	3.1
Wheel diameter	Static	39,449 lbf	24,811 lbf	D = 450 mm (17.7 in)	1.5
Rail width minimum	Static	39,449 lbf	24,811 lbf	b = 40 mm (1.6 in)	1.5
Guidance roller	Static	21,140 lbf	11,910 lbf	D = 50.8 mm (2.00 in)	1.78
Rail height minimum	Static	21,140 lbf	11,910 lbf	h= 31.75 mm (1.25 in)	1.78

8A.2.5 Receiving Cask Trolley Calculations

8A.2.5.1 Assumptions

It is assumed that the trolley is protected from environmental loads such as wind and snow by the DTS or the Preparation Area Roof. When the trolley is in the cask loading area, the cask is fully sealed and locked. It is assumed that potential damage to the receiving cask during loading on to the trolley has been evaluated as part of the licensing process of the source cask.

The transfer trolley for the source (or receiving) cask will be subjected to maximum stresses during a seismic event. The stresses in the trolley will be the same regardless of its location on the rails. The only other accident that could occur would be for the source and receiving cask transfer trolleys to bump into each other. It is assumed that the bumpers will be designed such that the forces produced during the accident event are less than those developed from a seismic event. During fuel transfer, the trolleys will be pinned in their respective positions, and will not bump into each other during normal operation or a seismic event. Analyses will be conducted on a site specific bases to ensure that the source (or receiving) cask will not tip over due to a tornado missile impact while in the Preparation Area.

The trunnion hold downs and the anti-derailing devices are calculated with the seismic load and the live load. The guiding roller wheels are calculated with the live load with motion.

The material properties used in the analysis are taken from NOG-1, Tables NOG 4211-1 and 4221-1 and are presented in Table 8A.2-6.

The trolley and casks are assumed to be rigid. The length of the receiving cask is 5,290 mm (208.3 inches). The outside diameter of the receiving cask is 1,855 mm (73.0 inches).

8A.2.5.2 Design Criteria

The design criteria are taken from ASME NOG-4300 and presented in Section 8A.2.4.

8A.2.5.3 Seismic Loading

The receiving cask and receiving cask trolley are assumed to be rigid. Since the trolley and source cask are not perfectly rigid, an addition factor of 1.5 is used.

The trolley is analyzed for a horizontal g loading of 0.25 g x 1.5 = 0.375 g and a vertical g loading of 0.17 g x 1.5 = 0.255 g.

The trolley's response to each of the three components of seismic input are combined by taking the square root of the sum of the squares (SRSS) per NOG-4153.10:

$$SRSS = \sqrt{(Sx^2 + Sy^2 + Sz^2)}$$

The seismic analysis was performed for two load combinations: seismic loading + static loading and seismic loading - static loading. The static load is the live load of the cask and the trolley due to gravity.

The acceleration due to gravity is $g = 9.81$ m/sec^2.

8A.2.5.4 Operational Loading

The following operational loads are taken into account:

- The live load of the cask and trolley under gravity (Used for calculating the stresses on the wheels)

- The transverse horizontal derailing load (24% of the live load of the cask plus the trolley dead load). This load is used to size the guidance rollers.

An isometric sketch of the cask on the trolley is shown in Figure 8A.2-3. A top view of the cask on the trolley is shown in Figure 8A.2-4. The bumpers are welded to the beams of the trolley.

8A.2.5.5 Evaluation of Bolts

Six M30 (1.2 in. diameter) bolts are evaluated using the maximum vertical reaction in the + Z direction on the cradle. This reaction is obtained by subtracting the static load from the seismic load.

X direction seismic loading

The seismic load in the X direction is applied at the center of gravity of the cask, G. The load is reacted at the point C (Figure 8A.2-3) which is the center of the compression zone.

The seismic loading in the X direction is reacted by the trolley base at C and by the cradle supports.

The force in the x direction is the seismic acceleration in the x direction times the mass of the cask.

$$F_x = ma_x$$

The distance to the center of the compression zone is $L_1 = D/3$ where D is the diameter of the cask.

OG is the distance between the center of gravity and the base of the cask. Assuming that the cask weight is approximately equally distributed along its length, OG = L/2.

The reaction forces in the z direction due to the x axis seismic acceleration at the trunnion locations are equal due to symmetry :

$$R_{BZX} = R_{AZX}$$

Then, summing the equations in the z direction:

$$R_{BZX} + R_{AZX} + R_{CZX} = 0$$

Therefore,

$$R_{AZX} = R_{BZX} = - R_{CZX} /2$$

Summing the moments about C to 0:

$$F_x \text{ x } OG - (R_{AZX} + R_{BZX})L_1 = 0 \text{ or}$$

$$R_{AZX} = R_{BZX} = F_x \text{ x } OG/2L_1 = ma_xL/2/(2D/3) = ma_x(3L/4D)$$

This reaction force is taken by the cradle.

The x axis reaction is equally taken by the two bumpers. Figure 8A.2-4)

$$R_{KX} = R_{JX} = ma_x/2$$

Y direction seismic loading

The seismic load in the Y direction is applied at the center of gravity of the cask, G. The load is reacted at the point B (Figure 8A.2-3) which is the center of the compression zone and by one cradle at location A.

The force in the y direction is the seismic acceleration in the y direction times the mass of the cask.

$$F_y = ma_y$$

OG is the distance between the center of gravity and the base of the cask. Assuming that the cask weight is approximately equally distributed along its length, OG = L/2.

AB is the distance between the two cradles. AB is approximately equal to D.

The static equations are:

$$F_yOG - R_{AZY}AB = 0$$

Therefore,

$$R_{AZY} = - R_{BZY} = F_yOG/AB = ma_yL/2D \text{ (cradle reaction force)}$$

Summing the forces in the Y direction:

$$R_{IY} = R_{JY} = ma_y /2 \text{ (bumper reaction forces)}$$

Z direction Seismic Loading

The Z direction seismic loading is equally divided between the two cradles:

$F_z = ma_z$

$R_{AZZ} = R_{BZZ} = ma_z /2$ (cradle reaction loads)

Static Z direction loads

The weight of the cask is equally divided by the two cradles.

$P_z = -Mg$

The vertical reactions at the two cradle locations are R_{AZS} and R_{BZS}.

$R_{AZS} = R_{BZS} = -Mg/2$ (at the cradles)

Combining the results:

The maximum +Z reaction on the cradle is: (Seismic - Static)

$R_{AZ} = \sqrt{(R_{AZX}^2 + R_{AZY}^2 + R_{AZZ}^2)} + R_{AZS}$

The maximum X and Y reaction on the bumper is at J, where:

$R_{JX} = ma_x /2$

and $R_{JY} = ma_y /2$

Solving numerically:

$R_{AZX} = ma_x(3L/4D) = 983{,}520$ N

$R_{AZY} = ma_yL/2D = 655{,}682$ N

$R_{AZZ} = ma_z /2 = 156{,}348$ N

$R_{AZS} = -Mg/2 = -613{,}125$ N

$R_{JX} = ma_x /2 = 229{,}922$ N

$R_{JY} = ma_y /2 = 229{,}922$ N

Therefore, the maximum reaction force in the Z direction on the cradle is:

$$R_{AZ} = \sqrt{(R_{AZX}{}^2 + R_{AZY}{}^2 + R_{AZZ}{}^2)} + R_{AZS} = 579{,}215 \text{ N} = 130{,}300 \text{ lbf}$$

Six M30 bolts attach each cradle. The vertical force R_{AZ} is the tension in the bolts.

The cross sectional area for an M30 bolt is $A_B = 561 \text{ mm}^2$

The tensile stress in the bolt is:

$$\sigma = R_{AZ}/6A_B = 172 \text{ MPa} = 25 \text{ ksi} \leq 287 \text{ MPa} = 41.6 \text{ ksi}$$

The safety factor in the bolts is SF = 287/172 = 1.6. Therefore the stresses in the bolts are acceptable.

8A.2.5.6 Evaluation of Anti-Taking Off Device and Locking Pin

The Anti-taking off device is shown in Figure 8A.2-1. The anti-taking off device prevents the trolley from tipping during a seismic event. This section calculates the stresses on the anti-taking off device. Figures 8A.2-6 through 8A.2-8 are used to perform this analysis.

The anti-taking off device is sized to withstand the maximum vertical force in the Z-direction. This force is obtained by combining the static and seismic loads. The reaction forces for each load step (x-direction seismic load, y-direction seismic load, z-direction seismic load and static load) are calculated and then combined to determine the maximum reaction force.

X direction Seismic Load

The seismic load on the trolley (M_t) is applied at G' shown in Figure 8A.2-8. The seismic load on the cask (M) is applied at G shown in Figure 8A.2-8. The seismic load is reacted by compression on the wheel at locations A & D and tension in the anti taking-off devices at locations B and C.

The vertical reaction forces due to the x direction seismic load at the anti-taking off devices are defined as R_{BZX} and R_{CZX}. From symmetry, $R_{BZX} = R_{CZX}$.

O'G is the vertical distance between the center of gravity of the cask and the base of the anti-taking off device.

From Figure 8A.2-8,

$$O'G = L/2 + L_Z.$$

O'G' is the vertical distance between the center of gravity of the trolley and the base of the anti-taking off device.

$O'G' = L_Z/2.$

Summing the moments to zero about AD:

$$M_t a_x O'G' + M a_x O'G - (R_{BZX} + R_{CZX})L_x = 0$$

Therefore,

$$R_{BZX} = (M_t a_x O'G' + M a_x O'G)/2L_x \text{ or}$$

$$R_{BZX} = (M_t a_x L_Z/2 + M a_x(L/2 + L_Z))/2L_x$$

The locking pin takes the force in the x direction:

$$R_X = (M + M_t)a_x$$

Y direction Seismic Loading

The seismic load on the trolley (M_t) is applied at G' shown in Figure 8A.2-8. The seismic load on the cask (M) is applied at G shown in Figure 8A.2-8. The seismic load is reacted by compression on the wheel at locations C & D and tension in the anti taking-off devices at locations A and B.

The vertical reaction forces due to the y direction seismic load at the anti-taking off devices are defined as RB_{AZY} and R_{BZY}. From symmetry, $R_{AZY} = R_{BZY}$.

O'G is the vertical distance between the center of gravity of the cask and the base of the anti-taking off device.

From Figure 8A.2-8,

$O'G = L/2 + L_Z.$

O'G' is the vertical distance between the center of gravity of the trolley and the base of the anti-taking off device.

$O'G' = L_Z/2.$

Summing the moments to zero about AD:

$$M_t a_y O'G' + M a_y O'G - (R_{AZY} + R_{BZY})L_y = 0$$

Therefore,

$$R_{BZY} = (M_t a_y O'G' + M a_y O'G)/2L_y \text{ or}$$

$$R_{BZY} = (M_t a_y L_Z /2 + M a_x (L/2 + L_Z)) / 2L_y$$

The y direction force is taken by the superior plate under the rail on the two anti-taking off devices at A and B.

$$R_{AY} = R_{BY} = (M + M_t) a_y / 2$$

Z Direction Seismic Loading

The z direction seismic loads are taken equally by the four anti-taking off devices:

$$R_{BZZ} = (M + M_t) a_z / 4$$

Static Z Direction Loading

The static compression load is taken equally by the four wheels. The dead load is:

$$P_Z = (M + M_t) g$$

The vertical compression at point B is:

$$R_{BZS} = - (M + M_t) g/4$$

Load Combination

Combining the results:

The maximum vertical force on the anti-taking off device is the combination of the static + seismic load:

$$R_{BZ} = \sqrt{(R_{BZX}^2 + R_{BZY}^2 + R_{BZZ}^2)} - R_{BZS}$$

The maximum force on the locking pin is:

$$R_X = (M + M_t) a_x$$

The maximum force in the Y direction on the anti-taking off device is:

$$R_{BY} = (M + M_t) a_y / 2$$

Solving numerically:

given $M = 125$ tons
$L = 5{,}290$ mm

$M_t = 20$ tons
$L_x = 2770$ mm
$L_Y = 2550$ mm
$L_z = 700$ mm
$a_x = 0.375$ g
$a_y = 0.375$ g

Then $R_{BZX} = 289{,}617$ N
$R_{BZY} = 306{,}654$ N
$R_{BZZ} = 90{,}683$ N
$R_{BZS} = -355{,}612$ N

The forces taken by the anti-taking off device are:

$R_{BZ} = 78{,}525$ N = 17,047 lbf
$R_{BY} = 266{,}710$ N = 59,960 lbf

The force on the locking pin is:

$R_x = 533{,}420$ N = 119,920 lbf

The dimensions of the Anti-Taking Off Device are shown in Figure 8A.2-11. The anti-taking off device is made from A36 carbon steel, with A193 B7 bolts.

The maximum bending moment in the plate is at bolt position 2:

$M_{max} = R_{BZ}\, a = (78{,}525 \text{ N})(50\text{mm}) = 3{,}975$ mN

The maximum tension in a bolt occurs at position 2:

F = tension in bolt
A_b = bolt cross sectional area = 353 mm^2

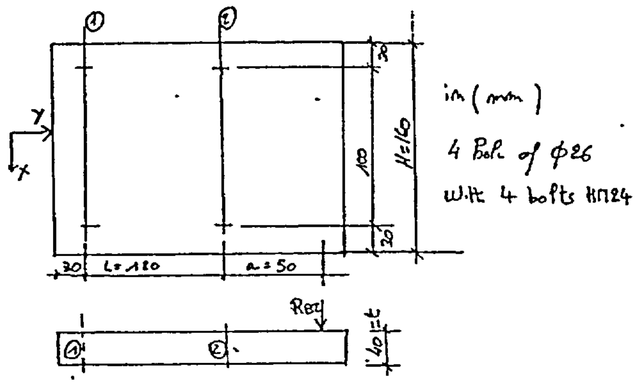
Then $2FL - R_{BZ} (L + a) = 0$

Solving for F:

$F = R_{BZ} (L + a)/2L =$ 53,800 N per bolt

Figure 8A.2-11

Receiving Cask Anti-Taking Off Device



The tensile stress in the bolt is:

$\sigma = F/A_b = 53{,}800/353 = 153$ MPa $\leq$ 287 MPa
The safety factor is:

SF = 287/153 = 1.8

The section modulus of the plate is:

$S = (H\text{-}2{*}26)t^3/6t = 28{,}800\ \text{mm}^3$

where t = the thickness of the plate = 40 mm and H = 160 mm.

The bending stress in the plate is:

$\sigma = M_{max}/\ S =$ 138 MPa = 20 ksi $\leq$ 186 MPa = 27 ksi

The safety factor is SF = 186/138 = 1.3.

The force R_{BY} is taken directly by the trolley.

The locking pin is shown in Figure 8A.2-10. The pin diameter is D = 2L = 120 mm. The shear stress in the pin is $\tau = R_x / A = 47$ MPa $\leq$ 103 MPa = 15 ksi where

$R_x = 533{,}420$ N = 119,919 lbf

and $A = \pi D^2/4 = 11{,}310\ \text{mm}^2$

The safety factor on the locking pin is SF = 103/47 = 2.2.

8A.2.5.7 Evaluation of Wheels

The wheels are sized based on static loads. The trolley wheels are 700 mm in diameter (27.6 in). The effective width of the rail head is 100 mm (3.9 in). The allowable wheel load is taken from NOG 5452.3:

$P_a = KbD$ (lbs)

where $K = 1300\ (BHN/260)^{0.333} = 1393$

The load is equally distributed between the four wheels. Therefore the load on each wheel is:

$F_z = (M + M_t)g/4 = 355{,}612$ N = 79,946 lbf

The allowable load is $P_a = KbD = 1393(3.9)(27.6) =$ 151,141 lbf. Therefore the loads on the wheels are acceptable. The safety factor is SF =151141/79946 = 1.9.

8A.2.5.8 Evaluation of the Guidance Rollers

The purpose of the guidance rollers is to prevent the trolley from de-railing in the event that it begins to mis-align transversely with the steel during normal operation. There are two sets of guidance rollers with each set adjacent to each trolley wheel on one rail. Conservatively (since the wheels are sliding and rolling simultaneously), the transverse load (F_y) required to maintain the trolley on track can be assumed to be equal to the dead load of the trolley plus the loaded cask multiplied by the coefficient of sliding friction (f_z).

The transverse load on each guidance roller is:

$$F_y = (M + M_t)g\ f_z /2 = 170{,}694 \text{ N} = 38{,}372 \text{ lbf}$$

Where: M = Mass of the rated load = 125,000 kg
M_t = Mass of the cask trolley = 20,000 kg
$g = 9.81$ m/sec^2
f_z = 0.24 for a steel wheel on a steel rail
(Reference 8-17, Mark's Standard Handbook for Mechanical Engineers)

Therefore, utilize a McGill (or equivalent) CFH-3 1/2-B stud type cam follower with a roller diameter of 3.50 in., roller width of 2.00 in. and a maximum static rated capacity of 63,250 lbf. The safety factor is SF = 63,250/38,372 = 1.65

Since the transverse derailing load of 170,694 N is less than the seismic transverse derailing load $R_{BY}/2$ of 133,355 N, the trolley anti-taking off device is structurally adequate as the normal operation anti-derailing device per the analysis of Section 8A.2.5.6. Hence the guidance rollers need not be classified as SSCs important to safety.

8A.2.5.9 Summary of Stresses - Receiving Cask Transfer Trolley

The sizes of the structural components of the source cask trolley, together with the calculated stresses and allowable stresses are presented in Table 8A.2-8.

Table 8A.2-8

Summary Receiving Cask Transfer Trolley Stresses

Part	Loading	Allowable stress or value	Calculated Stress or Value	Size	Safety Factor
Bolts of Cradle	Seismic	287 MPa	172 MPa	6 bolts M30 (1.2 in dia.)	1.6
Plate of anti-taking off device	Seismic	186 MPa	138 MPa	40 mm thick (1.6 in)	1.3
Bolts of anti-taking off Device	Seismic	287 MPa	153 MPa	4 bolts M24 (1 in)	1.8
Diameter of the locking pin	Seismic	103 MPa	47 MPa	D = 120 mm (4.8 in)	2.2
Wheel diameter	Static	151,141 lbf	79,946 lbf	D = 700 mm (27.6 in)	1.9
Rail width minimum	Static	151,141 lbf	79,946 lbf	b = 100 mm (3.9 in)	1.9
Guidance roller	Static	63,250 lbf	38,372 lbf	D = 88.9 mm (3.50 in)	1.65
Rail height minimum	Static	63,250 lbf	38,372 lbf	h = 50.8 mm (2.00 in)	1.65