APPENDIX 3.AT - STRESS ANALYSIS OF THE HI-TRAC WATER JACKET FOR THE ALTERNATE CONSTRUCTION

3.AT.1 Introduction: This calculation determines the stress level in the HI-TRAC water jacket and loaded water jacket welds under the combined effects of internal pressure caused by heating, by hydrostatic effects, and by dynamic effects during lifting and transport. This calculation supports a proposed fabrication enhancement

3.AT.2 Methodology: Formulas from theory of elastic plates are used to calculate maximum stress in the outer enclosure panels and in the top and bottom closure panels. References to tables and figures are references to HI-STORM TSAR, HI-951312.

3.AT.3 References:

[3.1] Roark's Formulas for Stress and Strain, 6th Edition, McGraw-Hill, 1989.

[3.2] Mathcad 2000, Mathsoft, 2000.

[3.3] Strength of Materials Part II, S.P. Timoshenko, McGraw-Hill, 3rd Edition, 1956.

3.AT.4 125 Ton HI-TRAC

3.AT.4.1 Input Data: All dimensions taken from Holtec drawings for HI-TRAC

Thickness of enclosure shell panels	$t_v := .375 \cdot in$	minimum value
Thickness of bottom closure	$t_p := 1.0 \cdot in$	
Bottom closure outer diameter	$OD := 91.972 \cdot in + t_v$	
Bottom flange inner diameter	$ID := 81.25 \cdot in$	
Number of Ribs	$N_r := 12$	
Radial Rib thickness	$t_r := 1.25 \cdot in$	

The allowable strength is (SA-516,Gr.70, Table 3.1.10 of the HI-STORM TSAR), For membrane stress, the allowable strength value is:

$S_a := 17500 \cdot psi$ For a bending stress evaluation, this value is increased 50%.

The ultimate strength of the base material (used to evaluate the welds) is

$S_u := 70000 \cdot psi$ (Table 3.3.2)

3.AT.4.2 Calculations

3.AT.4.2.1 Design pressure

For the purpose of this calculation, the design pressure must be first calculated by adding the saturation pressure at the peak jacket temperature (Table 2.2.1 of the HI-STORM TSAR) with the hydrodynamic pressure of the water.

The saturation pressure is conservatively set at:

$p_{sat} := 70 \cdot psi$

If the water density and the height of the water jacket are:

$\gamma_{water} := 62.4 \cdot \frac{lbf}{ft^3}$ $h_{jacket} := 168.75 \cdot in$ (Holtec BM-1880)

The hydrostatic pressure at the bottom of the water jacket is:

$p_{hs} := \gamma_{water} \cdot h_{jacket}$ $p_{hs} = 6.09\,psi$

The dynamic load factor for vertical transport of HI-TRAC is 0.15, so the pressure needs to be amplified by 1.15. The hydrodynamic pressure at the base of the water jacket is then calculated as:

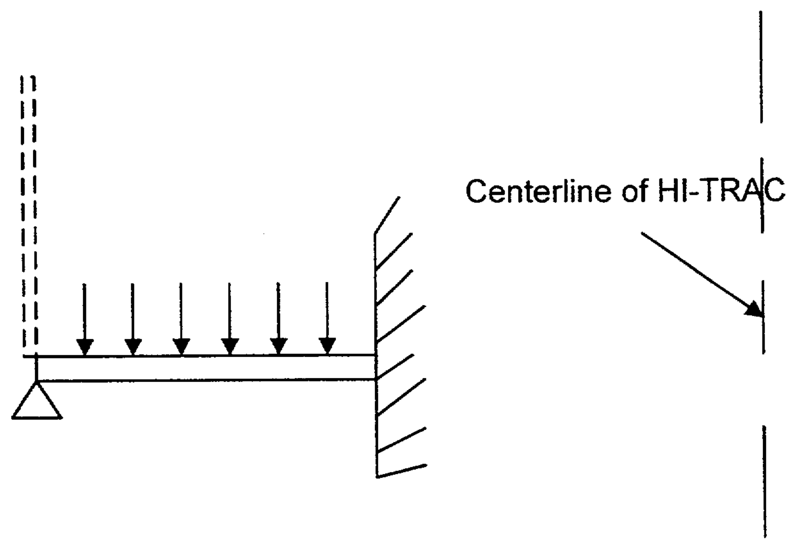
$p_{hd} := 1.15 \cdot p_{hs}$ $p_{hd} = 7.01\,psi$

The design pressure is:

$q := p_{sat} + p_{hd}$ $q = 77.01\,psi$

3.AT.4.2.2 Bottom annular flange:

The flange is considered as an annular plate clamped at the inside diameter, and conservatively assumed pinned at the outside diameter. That is, there is no welded connection, capable of transmitting a moment, assumed to exist at the connection of the bottom flange with the enclosure shell panels of the outer enclosure shell panels to the annulus. Intermediate vertical support from the radially oriented ribs is conservatively neglected. The results are obtained from Section 23, case 4 of Figure 72 in [3.3].



Insert a graph for stress factor "k"

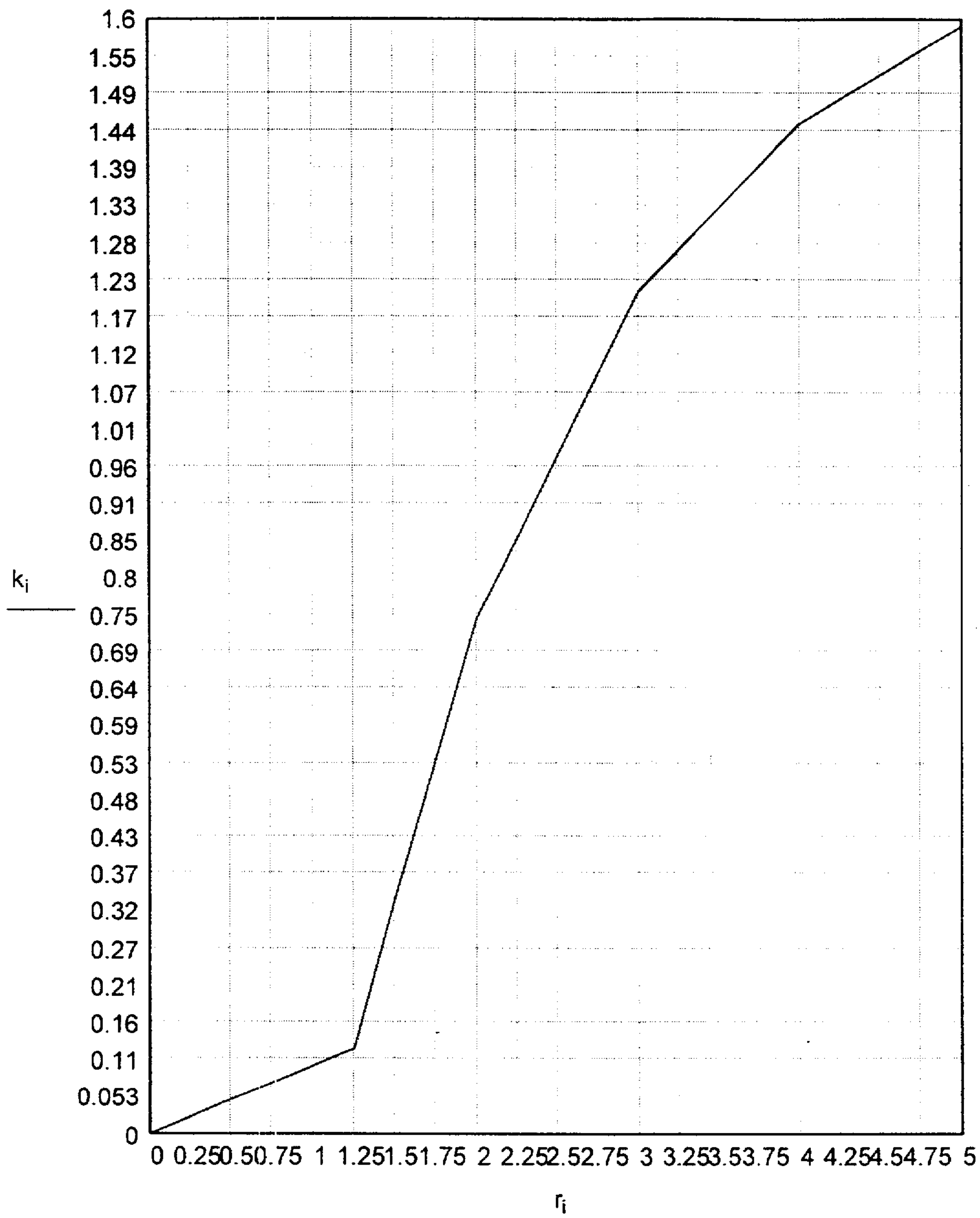
$i := 1 .. 7$

$r_i :=$

0
1.25
1.5
2.
3.
4.
5.

$k_i :=$

0
.122
.336
.74
1.21
1.45
1.59



$radius\_ratio := \frac{OD}{ID}$ $\qquad$ $radius\_ratio = 1.137$

From graph $\qquad$ $k := 0.114$

The maximum bending stress in the annular flange is obtained from [3.3] as:

$$\sigma_{plate} := k \cdot q \cdot \left(\frac{OD}{2 \cdot t_p}\right)^2 \qquad \sigma_{plate} = 1.872 \times 10^4 \, psi \qquad < 1.5S_a$$

The safety factor in the annular flange is. $\frac{1.5 \cdot S_a}{\sigma_{plate}} = 1.403$

This is a conservative result as it neglects the effect of partial clamping action at the connection with the outer enclosure panels. The result for the bottom flange bounds the result for the top flange since the applied pressure is less.

3.AT.4.2.3 Outer Enclosure Panels

These curved panels are treated as 1 inch deep (in the vertical direction) arch sections. Since the end of each panel strip, around the periphery, is welded to a radially oriented support, the boundary condition at the connection, because of symmetry around the periphery, is a clamped condition.

The stress is a mean stress

$$\sigma_{mean} := q \cdot \frac{\left(\frac{OD}{2}\right)}{t_v} \qquad \sigma_{mean} = 9.482 \times 10^3 \, psi$$

plus a bending stress that is computed from a solution in [3.1]. The increment in stress due to the radial rib restraint is (see Attachment 1 at end of section)

$$\sigma_{bending} := 5315 \cdot psi$$

$$\sigma_{total} := \sigma_{mean} + \sigma_{bending} \qquad \sigma_{total} = 1.48 \times 10^4 \, psi$$

$$SF_{max} := \frac{1.5 \cdot S_a}{\sigma_{total}} \qquad SF_{max} = 1.774$$

$$SF_{mean} := \frac{S_a}{\sigma_{mean}} \qquad SF_{mean} = 1.846$$

3.AT.4.2.4 Weld Stress and Panel Direct Stress

Enclosure outer panel fillet weld

$t_{weld} := .25 \cdot in$ conservatively assume a fillet weld

The stress in the vertical outer enclosure panel vertical weld is computed by using the results from the arch solution. Specifically,

$$V_R := 45.83 \cdot \frac{lbf}{in}$$

$$\tau_{weld} := \frac{V_R}{.707 \cdot t_{weld}} \qquad \tau_{weld} = 259.292\,psi$$

Using the allowable weld stress from ASME, Section III, Subsection NF

$$SF := \frac{.3 \cdot S_u}{\tau_{weld}} \qquad SF = 80.99$$

Top and Bottom Flange weld to outer shell

This is a double fillet weld. The assumed load is the maximum bending moment developed in the bottom flange.

$\sigma_{plate} = 1.872 \times 10^4 psi$

$$M_{flange} := \frac{\sigma_{plate} \cdot t_p^2}{6} \qquad M_{flange} = 3.119 \times 10^3 in \cdot \frac{lbf}{in}$$

The fillet weld size at this location is $t_{fweld} := 0.375 \cdot in$

The moment capacity of the weld is the effective force through the throat of the weld on each surface of the flange multiplied by the distance between the centroids of each of the welds (flange thickness + 2/3 weld leg size).

Therefore, the force at the weld throat is

$$F_{throat} := \frac{M_{flange} \cdot 1 \cdot in}{(t_p + .667 \cdot t_{fweld})} \qquad F_{throat} = 2.495 \times 10^3 \, lbf$$

Therefore the shear stress in the throat of each weld in

$$\tau_{fweld} := \frac{F_{throat}}{0.7071 \cdot t_{fweld} \cdot 1 \cdot in} \qquad \tau_{fweld} = 9.41 \times 10^3 \, psi$$

Therefore, the safety factor for this weld is

$$SF_{fweld} := \frac{.3 \cdot S_u}{\tau_{fweld}} \qquad SF_{fweld} = 2.232$$

The primary membrane stress developed in the radially oriented portion of of the outer enclosure panels is computed as follows for the 1" strip width:

$$Load := 2 \cdot V_R \qquad Load = 91.66 \frac{lbf}{in}$$

The panel radially oriented direct stress is

$$\sigma_{direct} := \frac{Load}{t_v} \qquad \sigma_{direct} = 244.426 \, psi$$

The safety factor is

$$SF_{radial} := \frac{S_a}{\sigma_{direct}} \qquad SF_{radial} = 71.596$$

3.AT.4.2.5 Conclusion

The design is acceptable for a maximum water jacket pressure of

$q = 77.008 \, psi$

A 10% over pressure will be acceptable for a hydrotest for leaks.

$$p_{test} := 1.1 \cdot q \qquad p_{test} = 84.709 \, psi$$

3.AT.5 100 Ton HI-TRAC

3.AT.5.1 Input Data: All dimensions taken from Holtec drawing 2145 for HI-TRAC

Thickness of outer enclosure panel	$t_v := .25 \cdot in$	conservative minimum
Thickness of bottom flange	$t_p := 1.0 \cdot in$	
Bottom panel outer diameter	$OD := 88.0 \cdot in + t_v$	
Bottom panel inner diameter	$ID := 78.0 \cdot in$	
Number of Ribs	$N_r := 10$	
Radial Rib thickness	$t_r := 1.25 \cdot in$	

3.AT.5.2 Calculations

3.AT.5.2.1 Design pressure

For the purpose of this calculation, the design pressure must be first calculated by adding the saturation pressure at the peak jacket temperature (Table 2.2.1 of the HI-STORM TSAR) with the hydrodynamic pressure of the water.

The saturation pressure is conservatively set at:

$$p_{sat} = 70\,psi$$

If the water density and the height of the water jacket are:

$\gamma_{water} := 62.4 \cdot \frac{lbf}{ft^3}$ $\quad h_{jacket} := 168.75 \cdot in$ $\quad$ (Holtec drawing no. 2145)

The hydrostatic pressure at the bottom of the water jacket is:

$p_{hs} := \gamma_{water} \cdot h_{jacket}$ $\quad p_{hs} = 6.09\,psi$

Incorporating the dynamic amplification assumed during the lifting operation,

$p_{hd} := 1.15 \cdot p_{hs}$ $\quad p_{hd} = 7.01\,psi$

The design pressure is, therefore:

$q := p_{sat} + p_{hd}$ $\quad q = 77.01\,psi$

3.AT.5.2.2 Bottom annular flange:

The flange is considered to be clamped at the inside diameter, and pinned at the outside diameter. That is, there is no welded connection of the side plates to the annulus. The results are obtained from Section 23, case 4 of Figure 72 in [3.3].

$$\text{radius\_ratio} := \frac{OD}{ID} \qquad \text{radius\_ratio} = 1.131 \qquad k := 0.112$$

The maximum stress in the annular flange is

$$\sigma_{plate} := k \cdot q \cdot \left(\frac{OD}{2 \cdot t_p}\right)^2 \qquad \sigma_{plate} = 1.679 \times 10^4 \text{psi} \qquad < 1.5S_a$$

The calculated safety factor is. $\dfrac{1.5 \cdot S_a}{\sigma_{plate}} = 1.563$

3.AT.5.2.3 Outer Enclosure Panels

These curved panels are treated as 1 inch deep (in the vertical direction) arch sections. Since the end of each panel strip, around the periphery, is welded to a radially oriented support, the boundary condition at the connection, because of symmetry around the periphery, is a clamped condition.

The stress is a mean stress

$$\sigma_{mean} := q \cdot \frac{\left(\frac{OD}{2}\right)}{t_v} \qquad \sigma_{mean} = 1.359 \times 10^4 \text{psi}$$

plus a bending stress that is computed from a solution in [3.1]. The increment in stress due to the radial rib restraint is (see Attachment 1 at end of section)

$$\sigma_{bending} := 5754 \cdot \text{psi}$$

$$\sigma_{total} := \sigma_{mean} + \sigma_{bending} \qquad \sigma_{total} = 1.935 \times 10^4 \text{psi}$$

$$SF_{max} := \frac{1.5 \cdot S_a}{\sigma_{total}} \qquad SF_{max} = 1.357$$

$$SF_{mean} := \frac{S_a}{\sigma_{mean}} \qquad SF_{mean} = 1.288$$

3.AT.5.2.4 Weld Stress and Panel Direct Stress

Enclosure outer panel fillet weld

$t_{weld} := .125 \cdot in$ conservatively assume a fillet weld.

The stress in the vertical outer enclosure panel vertical weld is computed by using the results from the arch solution. Specifically,

$$V_R := 12.738 \cdot \frac{lbf}{in}$$

$$\tau_{weld} := \frac{V_R}{.707 \cdot t_{weld}} \qquad \tau_{weld} = 144.136\,psi$$

Using the allowable weld stress from ASME, Section III, Subsection NF

$$SF := \frac{.3 \cdot S_u}{\tau_{weld}} \qquad SF = 145.696$$

Top and Bottom Flange weld to outer shell

This is a double fillet weld. The assumed load is the maximum bending moment developed in the bottom flange.

$\sigma_{plate} = 1.679 \times 10^4\,psi$

$$M_{flange} := \frac{\sigma_{plate} \cdot t_p^{\,2}}{6} \qquad M_{flange} = 2.799 \times 10^3\,in \cdot \frac{lbf}{in}$$

The fillet weld size at this location is $t_{fweld} := 0.25 \cdot in$

The moment capacity of the weld is the effective force through the throat of the weld on each surface of the flange multiplied by the distance between the centroids of each of the welds (flange thickness + 2/3 weld leg size).

Therefore, the force at the weld throat is $F_{throat} := \dfrac{M_{flange} \cdot 1 \cdot in}{\left(t_p + .667 \cdot t_{fweld}\right)}$

$F_{throat} = 2.399 \times 10^3\, lbf$

Therefore the shear stress in the throat of each weld in

$\tau_{fweld} := \dfrac{F_{throat}}{0.7071 \cdot t_{fweld} \cdot 1 \cdot in}$ $\tau_{fweld} = 1.357 \times 10^4\, psi$

Therefore, the safety factor for this weld is $SF_{fweld} := \dfrac{.3 \cdot S_u}{\tau_{fweld}}$

$SF_{fweld} = 1.548$

The primary membrane stress developed in the radially oriented portion of of the outer enclosure panels is computed as follows for the 1" strip width:

$Load := 2 \cdot V_R$ $Load = 25.476 \dfrac{lbf}{in}$

The panel radially oriented direct stress is

$\sigma_{direct} := \dfrac{Load}{t_v}$ $\sigma_{direct} = 101.904\, psi$

The safety factor is $SF_{radial} := \dfrac{S_a}{\sigma_{direct}}$ $SF_{radial} = 171.731$

3.AT.5.2.5 Conclusion

The design is acceptable for a maximum water jacket pressure of $q = 77.008\, psi$

Based on the result that the minimum safety factor is 1.168, a 10% over pressure will be acceptable for a hydrotest for leaks.

$p_{test} := 1.1 \cdot q$ $p_{test} = 84.709\, psi$

ATTACHMENT 1 to Appendix

BENDING STRESS CALCULATION -125 TON CASK

Cases 5-14 Loading Terms

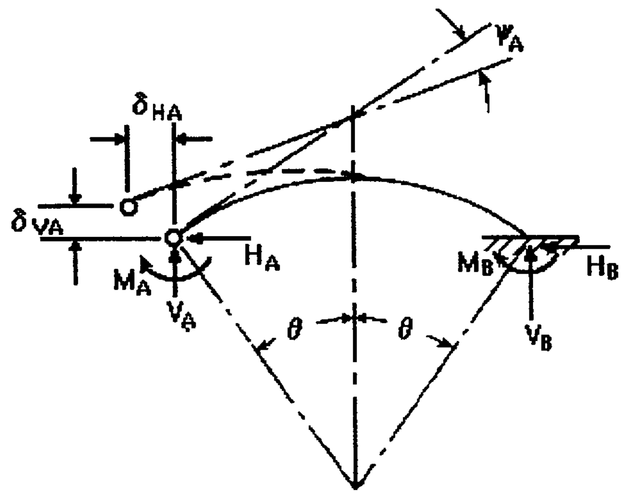
Partial Uniformly Distributed Radial Loading

This file contains the general formulas for the reaction moment, horizontal end reaction, vertical end reaction, horizontal deflection, vertical deflection and angular rotation for a circular arch with a partial uniformly distributed radial loading. Because the constants and loading terms necessary to calculate these formulas remain the same under certain conditions, the following 10 restraint conditions with the above load have been included in this file:

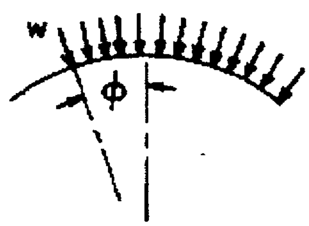
- **Case 5:** Left end fixed, right end fixed

The following sketches should be referred to for definitions of dimensions and loadings:

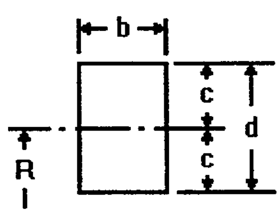
Circular arch



Partial uniformly distributed radial loading



Solid rectangular section



Enter dimensions

Radius of curvature measured to centroid of section: $R := \frac{91.972}{2} \cdot in + 0.375 \cdot \frac{in}{2}$

Height of rectangular section: $d := 0.375 \cdot in$

Width of rectangular section: $b := 1 \cdot in$

Conditions

If R/d ≥ 8, then the beam is thin.

$\frac{R}{d} = 123.129$ $\quad thin := if\left(\frac{R}{d} \geq 8, 1, 0\right)$ $\quad thick := if\left(\frac{R}{d} < 8, 1, 0\right)$

Constants

Half-height: $c := \frac{d}{2}$ $\qquad c = 0.188\,in$

$\int_{area} \frac{dA}{r}$ is equal to $b \cdot \ln\left(\frac{\frac{R}{c} + 1}{\frac{R}{c} - 1}\right) = 8.122 \times 10^{-3}\,in$

Shape constant for rectangle: $F := \frac{6}{5}$
See article 7.10 on page 201 in Roark.

For all of the cross sections shown in this table this centroidal axis perpendicular to the plane of bending is a principal axis of the cross section.

Moment of inertia of section about centroidal axis perpendicular to the plane of bending:

$I_c := \frac{b \cdot d^3}{12}$ $\qquad I_c = 4.395 \times 10^{-3}\,in^4$

Area:

$A := b \cdot d$ $\qquad A = 0.375\,in^2$

Distance from centroidal axis to neutral axis measured toward center of curvature:

$$h := c\cdot\left(\frac{R}{c} - \frac{2}{\ln\left(\frac{\frac{R}{c}+1}{\frac{R}{c}-1}\right)}\right)\cdot thick + \frac{I_c}{R\cdot A}\cdot thin \qquad h = 2.538\times 10^{-4}\,in$$

$$k_i := \left(\frac{1}{\frac{3\cdot h}{c}}\right)\cdot\left(\frac{1-\frac{h}{c}}{\frac{R}{c}-1}\right) \qquad k_i = 1.003$$

$$k_o := \left(\frac{1}{\frac{3\cdot h}{c}}\right)\cdot\left(\frac{1+\frac{h}{c}}{\frac{R}{c}+1}\right) \qquad k_o = 0.997$$

Once this is done, enter the computed values below.

Principal centroidal moment of inertia: $I_c := 0.004395\cdot in^4$

Area of cross section: $A := 0.375\cdot in^2$

Shape constant: $F := \frac{6}{5}$

Distance between axes: $h := .002538 \cdot in$

Enter dimensions, properties and loading of arch

Radius of curvature: $R = 46.173\,in$

Half-span of the beam: $\theta := 15 \cdot deg$

Height of cross section: $d := .375 \cdot in$

Modulus of elasticity: $E := 29 \cdot 10^6 \cdot \frac{lbf}{in^2}$

Poisson's ratio: $\nu := 0.3$

Load: $w := 77.01 \cdot \frac{lbf}{in}$ (1" strip)

Angle from vertical to load: $\phi := 15 \cdot deg$

Constants

These constants are used in the formulas to calculate the reaction moment, horizontal and vertical end reactions, horizontal and vertical deflections and angular rotation at A:

$$G := \frac{E}{2 \cdot (1 + \nu)} \qquad thin := if\left(\frac{R}{d} \geq 8, 1, 0\right) \qquad thick := if\left(\frac{R}{d} < 8, 1, 0\right)$$

$$\alpha := thin \cdot \left(\frac{I_c}{A \cdot R^2}\right) + thick \cdot \left(\frac{h}{R}\right)$$

$$\beta := thin \cdot \left(\frac{F \cdot E \cdot I_c}{G \cdot A \cdot R^2}\right) + thick \cdot \left[\frac{2 \cdot F \cdot (1 + \nu) \cdot h}{R}\right]$$

$$k_1 := 1 - \alpha + \beta \qquad k_2 := 1 - \alpha$$

$s := \sin(\theta)$ $\qquad$ $n := \sin(\phi)$

$c := \cos(\theta)$ $\qquad$ $e := \cos(\phi)$

$$B_{HH} := 2\cdot\theta\cdot c^2 + k_1\cdot(\theta - s\cdot c) - k_2\cdot 2\cdot s\cdot c$$

$$B_{HV} := -2\cdot\theta\cdot s\cdot c + k_2\cdot 2\cdot s^2 \qquad B_{VH} := B_{HV}$$

$$B_{HM} := -2\cdot\theta\cdot c + k_2\cdot 2\cdot s \qquad B_{MH} := B_{HM}$$

$$B_{VV} := 2\cdot\theta\cdot s^2 + k_1\cdot(\theta + s\cdot c) - k_2\cdot 2\cdot s\cdot c$$

$$B_{VM} := 2\cdot\theta\cdot s \qquad B_{MV} := B_{VM}$$

$$B_{MM} := 2\cdot\theta$$

Loading terms

$$LF_H := w\cdot R\cdot\left[\frac{k_1}{2}\cdot\left(s\cdot c\cdot e + c^2\cdot n - \theta\cdot e - \phi\cdot e\right) + k_2\cdot(s + n - \theta\cdot c - \phi\cdot c)\right]$$

$$LF_V := w\cdot R\cdot\left[\begin{array}{l}\frac{k_1}{2}\cdot\left(\theta\cdot n + \phi\cdot n + s\cdot c\cdot n + s^2\cdot e\right) \ldots \\ + k_2\cdot\left(\theta\cdot s + \phi\cdot s - 2\cdot s\cdot c\cdot n + 2\cdot c^2\cdot e - c - e\right)\end{array}\right]$$

$$LF_M := w\cdot R\cdot\left[k_2\cdot(\theta + \phi - s\cdot e - c\cdot n)\right]$$

Formulas for horizontal and vertical deflections, reaction moment, horizontal and vertical end reactions and angular rotation at the left edge

Case 5j **Left end fixed, right end fixed**

Horizontal deflection: $\delta_{HA} := 0 \cdot in$

Vertical deflection: $\delta_{VA} := 0 \cdot in$

Angular rotation: $\psi_A := 0 \cdot deg$

Because the above equal zero, the following three equations are solved simultaneously using Mathcad's solve block for H_A, V_A and M_A:

Enter guess values:

$H_A := 1000 \cdot lbf$ $V_A := 1000 \cdot lbf$ $M_A := 100 \cdot lbf \cdot ft$

Define the three linear expressions:

Given

$$B_{HH} \cdot H_A + B_{HV} \cdot V_A + \frac{B_{HM} \cdot M_A}{R} = LF_H$$

$$B_{VH} \cdot H_A + B_{VV} \cdot V_A + \frac{B_{VM} \cdot M_A}{R} = LF_V$$

$$B_{MH} \cdot H_A + B_{MV} \cdot V_A + \frac{B_{MM} \cdot M_A}{R} = LF_M$$

Solve for H_A, V_A and M_A:

$$\begin{pmatrix} H_A \\ V_A \\ M_A \end{pmatrix} := \text{Find}(H_A, V_A, M_A)$$

Horizontal end reaction: $H_A = -3.258 \times 10^3 \, \text{lbf}$

Vertical end reaction: $V_A = 920.314 \, \text{lbf}$

Reaction moment: $M_A = -15.531 \, \text{lbf} \cdot \text{ft}$

$$\sigma_{bending} := M_A \cdot \frac{t_v \cdot k_i}{2 \cdot I_c} \qquad \sigma_{bending} = -5.315 \times 10^3 \, \text{psi}$$

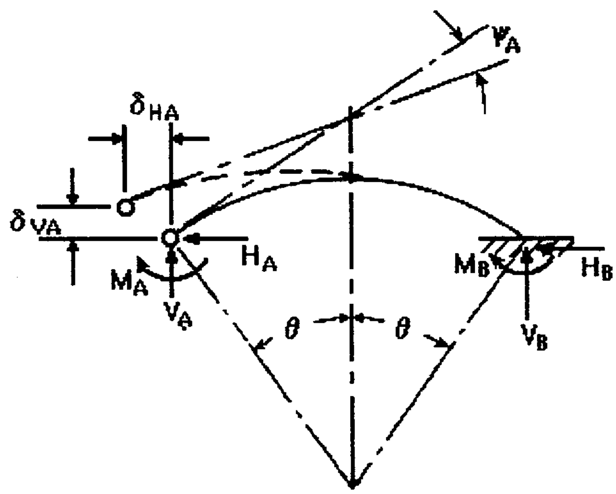
Radial Reaction

$$V_R := V_A \cdot \cos(\theta) + H_A \cdot \sin(\theta) \qquad V_R = 45.83 \, \text{lbf}$$

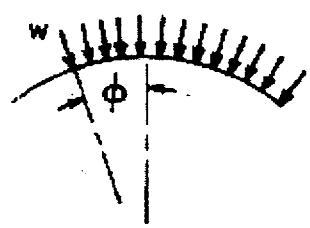
ATTACHMENT 2 to Appendix

BENDING STRESS CALCULATION -100 TON CASK

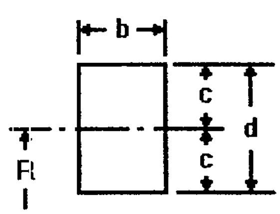
Circular arch



Partial uniformly distributed radial loading



Solid rectangular section



Enter dimensions

Radius of curvature measured to centroid of section: $R := \frac{88.25}{2} \cdot in + 0.25 \cdot \frac{in}{2}$

Height of rectangular section: $d := 0.25 \cdot in$

Width of rectangular section: $b := 1 \cdot in$

Conditions

$\frac{R}{d} = 177$ $\quad$ $thin := if\left(\frac{R}{d} \geq 8, 1, 0\right)$ $\quad$ $thick := if\left(\frac{R}{d} < 8, 1, 0\right)$

Constants

Half-height: $c := \frac{d}{2}$ $\quad$ $c = 0.125\,in$

$\int_{area} \frac{dA}{r}$ is equal to $b \cdot \ln\left(\frac{\frac{R}{c}+1}{\frac{R}{c}-1}\right) = 5.65 \times 10^{-3}\,in$

Shape constant for rectangle: $F := \frac{6}{5}$
See article 7.10 on page 201 in Roark.

For all of the cross sections shown in this table this centroidal axis perpendicular to the plane of bending is a principal axis of the cross section.

Moment of inertia of section about centroidal axis perpendicular to the plane of bending:

$I_c := \frac{b \cdot d^3}{12}$ $\quad$ $I_c = 1.302 \times 10^{-3}\,in^4$

Area: $A := b \cdot d$ $\quad$ $A = 0.25\,in^2$

Distance from centroidal axis to neutral axis measured toward center of curvature:

$$h := c \cdot \left(\frac{R}{c} - \frac{2}{\ln\left(\frac{\frac{R}{c}+1}{\frac{R}{c}-1} \right)} \right) \cdot thick + \frac{I_c}{R \cdot A} \cdot thin \qquad h = 1.177 \times 10^{-4} in$$

$$k_i := \left(\frac{1}{\frac{3 \cdot h}{c}} \right) \cdot \left(\frac{1 - \frac{h}{c}}{\frac{R}{c} - 1} \right) \qquad k_i = 1.002$$

$$k_o := \left(\frac{1}{\frac{3 \cdot h}{c}} \right) \cdot \left(\frac{1 + \frac{h}{c}}{\frac{R}{c} + 1} \right) \qquad k_o = 0.998$$

Principal centroidal moment of inertia: $I_c := 0.001302 \cdot in^4$

Area of cross section: $A := 0.25 \cdot in^2$

Shape constant: $F := \frac{6}{5}$

Distance between axes: $h := .0001177 \cdot in$

Enter dimensions, properties and loading of arch

Radius of curvature: $R = 44.25 in$

Half-span of the beam: $\theta := 18 \cdot deg$

Height of cross section: $d = 0.25 in$

Modulus of elasticity: $E := 29 \cdot 10^6 \cdot \frac{lbf}{in^2}$

Poisson's ratio: $\nu := 0.3$

Load: $w := 77.01 \cdot \frac{lbf}{in}$ (1" strip)

Angle from vertical to load: $\phi := 18 \cdot deg$

Constants

These constants are used in the formulas to calculate the reaction moment, horizontal and vertical end reactions, horizontal and vertical deflections and angular rotation at A:

$$G := \frac{E}{2 \cdot (1 + \nu)} \qquad thin := if\left(\frac{R}{d} \ge 8, 1, 0\right) \qquad thick := if\left(\frac{R}{d} < 8, 1, 0\right)$$

$$\alpha := thin \cdot \left(\frac{I_c}{A \cdot R^2}\right) + thick \cdot \left(\frac{h}{R}\right)$$

$$\beta := thin \cdot \left(\frac{F \cdot E \cdot I_c}{G \cdot A \cdot R^2}\right) + thick \cdot \left[\frac{2 \cdot F \cdot (1 + \nu) \cdot h}{R}\right]$$

$$k_1 := 1 - \alpha + \beta \qquad k_2 := 1 - \alpha$$

$$s := \sin(\theta) \qquad n := \sin(\phi)$$

$c := \cos(\theta) \qquad e := \cos(\phi)$

$$B_{HH} := 2\cdot\theta\cdot c^2 + k_1\cdot(\theta - s\cdot c) - k_2\cdot 2\cdot s\cdot c$$

$$B_{HV} := -2\cdot\theta\cdot s\cdot c + k_2\cdot 2\cdot s^2 \qquad B_{VH} := B_{HV}$$

$$B_{HM} := -2\cdot\theta\cdot c + k_2\cdot 2\cdot s \qquad B_{MH} := B_{HM}$$

$$B_{VV} := 2\cdot\theta\cdot s^2 + k_1\cdot(\theta + s\cdot c) - k_2\cdot 2\cdot s\cdot c$$

$$B_{VM} := 2\cdot\theta\cdot s \qquad B_{MV} := B_{VM}$$

$$B_{MM} := 2\cdot\theta$$

Loading terms

$$LF_H := w\cdot R\cdot\left[\frac{k_1}{2}\cdot\left(s\cdot c\cdot e + c^2\cdot n - \theta\cdot e - \phi\cdot e\right) + k_2\cdot(s + n - \theta\cdot c - \phi\cdot c)\right]$$

$$LF_V := w\cdot R\cdot\left[\begin{array}{l}\frac{k_1}{2}\cdot\left(\theta\cdot n + \phi\cdot n + s\cdot c\cdot n + s^2\cdot e\right)\ \ldots \\ + k_2\cdot\left(\theta\cdot s + \phi\cdot s - 2\cdot s\cdot c\cdot n + 2\cdot c^2\cdot e - c - e\right)\end{array}\right]$$

$$LF_M := w\cdot R\cdot\left[k_2\cdot(\theta + \phi - s\cdot e - c\cdot n)\right]$$

RESULTS

Horizontal deflection: $\delta_{HA} := 0\cdot in$

Vertical deflection: $\delta_{VA} := 0\cdot in$

Angular rotation: $\psi_A := 0\cdot deg$

Enter guess values:

$H_A := 1000 \cdot lbf \qquad V_A := 1000 \cdot lbf \qquad M_A := 100 \cdot lbf \cdot ft$

Define the three linear expressions:

Given

$$B_{HH} \cdot H_A + B_{HV} \cdot V_A + \frac{B_{HM} \cdot M_A}{R} = LF_H$$

$$B_{VH} \cdot H_A + B_{VV} \cdot V_A + \frac{B_{VM} \cdot M_A}{R} = LF_V$$

$$B_{MH} \cdot H_A + B_{MV} \cdot V_A + \frac{B_{MM} \cdot M_A}{R} = LF_M$$

Solve for H_A, V_A and M_A:

$$\begin{pmatrix} H_A \\ V_A \\ M_A \end{pmatrix} := \text{Find}(H_A, V_A, M_A)$$

Horizontal end reaction: $H_A = -3.2 \times 10^3 \, lbf$

Vertical end reaction: $V_A = 1.053 \times 10^3 \, lbf$

Reaction moment: $M_A = -4.985 \, lbf \cdot ft$

$\sigma_{bending} := M_A \cdot \frac{t_v \cdot k_i}{2 \cdot I_c} \qquad \sigma_{bending} = -5.754 \times 10^3 \, psi$

Radial Reaction

$V_R := V_A \cdot \cos(\theta) + H_A \cdot \sin(\theta) \qquad V_R = 12.738 \, lbf$

APPENDIX AU

LARGE TORNADO MISSILE IMPACT ANLYSIS FOR HI-TRAC 100 AND HI-TRAC 125 TRANSFER CASK SYSTEMS

1.0 Purpose

In this appendix, the structural integrity of the 100-ton and 125-ton HI-TRAC transfer cask systems with an alternate water jacket design is evaluated under the postulated impact of a large tornado missile. The alternate design of the water jacket significantly reduces the decontamination effort. The new design also changes the thickness of the water jacket shell and the thickness of the ribs. This work supports a fabrication enhancement for the HI-TRAC.

The pressure retaining capacity of the revised water jacket design is evaluated in Appendix AT in this supplement. The drop and tipover events are bounded by the results presented in the TSAR since the new design is "more flexible" and will absorb more energy during the event. Penetrant missile impacts are examined in the TSAR and are evaluated without credit for the water jacket structure. Therefore, to structurally qualify the alternate water jacket design, we need only demonstrate that after a missile strike, the MPC is retrievable from the HI-TRAC. Impacts. Therefore, to qualify the new design, we re-examine the large tornado missile strike and demonstrate that there are no permanent deformations of the inner shell of the HI-TRAC transfer cask.

2 Model

Two finite-element models are developed to perform the large tornado missile impact analysis for the HI-TRAC 100 and HI-TRAC 125 cask systems, respectively. The models are constructed based on the applicable drawings [1]. Table 1 details the geometry of the 100-ton and 125-ton HI-TRAC cask systems used in the impact simulations. Each of the finite-elements models consists of 15 independent parts representing the structural components of the HI-TRAC System. These modeling parts represent the transfer lid plates, the bottom flange, the interior and exterior shell, the lead shielding, the top flange, the top lid, the lower and upper trunnions, the radial channels and outer closure plates of the water jacket, the MPC (steel plates and the basket fuel zone), and the water. The water is added in the model because the alternate water jacket design has fewer radial ribs than before to transfer impact load to the outer shell of the HI-TRAC system. Therefore, it is not conservative to neglect the water in the analysis. Gaps between the MPC and the transfer cask inner shell and lids are included in the model. Using symmetry, only half of the structure is modeled. The HI-TRAC model is restrained at the ends to equilibrate any applied missile impact force. Figures 1 through 3 show the finite-element model for the 100-ton HI-TRAC transfer cask system; the model for the 125-ton HI-TRAC system is similar.

The HI-TRAC transfer cask system is made from a number of materials. Most of the structural components of the HI-TRAC system are represented by elasto-plastic materials: (*MAT_PIECEWISE_LINEAR_PLASTICITY) in the finite element model. The water inside the water jacket is characterized by a specific elastic material model for fluid: (*MAT_ELASTIC_FLUID). The MPC and the contained fuel are modeled in two parts that represent the lid and baseplate, and the fuel area. An elastic material is used for both parts. The MPC model is identical to that used in the handling accident simulations for the HI-STAR and HI-STORM cask systems already reviewed and approved in the TSAR..

The large tornado missile impact is simulated by a total input force-time relationship applied at nodes encompassing an interface area on the water jacket. The total force is apportioned to the nodes lying within and on the boundary of the interface area. The force-time relation is obtained from a NRC approved topical report [3]. The interface contact area, appropriate to the large missile, is obtained from [4]. The force-time relation (during the rise to a maximum value), is given by the expression [3, Equation. D-6]:

$$F(t) = 0.625 V_s W_m \sin(20t)$$

$$V_s = 184.6 \text{ ft./sec.}$$

$$W_m = 3960 \text{ lb.}$$

The time "t" in the formula is in "seconds".

Figure 4 shows the interface force-time data imposed on the HI-TRAC water jacket. The interface area was assumed approximately mid-way along the length of the cask and is equivalent to a 20 sq. ft. load patch.

3.0 Results from Analysis

Figures 5 and 6 show the Von Mises stress distribution in the inner shell and in the water jacket for the 100-ton HI-TRAC transfer cask at the instant when the stresses reach the maximum values. For the 125-ton HI-TRAC transfer cask, the corresponding results are shown in Figures 7 and 8, respectively. Table 2 summarizes results from these figures as well as the strain data from the two simulations. No permanent deformation is developed in the inner shell due to the impact in either simulation.

4.0 COMPUTER CODES AND FILES

The computer code utilized in this analysis is LS-DYNA [2] validated under Holtec's QA system. LS-DYNA has an extensive finite-element and material description library and can account for various time-dependent contact conditions that normally arise between the various structural components during the impact analysis.

The input and the output files associated with the analysis are stored on Holtec's server disk and tape archived as required by Holtec's QA procedures under the following address:

F:\PROJECTS\5014\JZ\HI-TRAC\MISSLE\REV11

All LS-DYNA simulations were performed under the Windows NT environment.

5.0 CONCLUSIONS

The finite-element impact analysis of the 100-ton and 125-ton HI-TRAC transfer cask systems with the alternate water jacket structure leads to the following conclusions:

The maximum stress developed in the HI-TRAC transfer cask is below Level D allowable values during the impact by a large tornado missile.

No permanent deformation is generated due to the impact by a large tornado missile.

Therefore, the alternate water jacket construction meets all postulated requirements in the TSAR.

6.0 REFERENCES

[1] Holtec Drawings for the Alternate water jacket construction, (proposed) August, 2000.

[2] LS-DYNA3D, Version 950, Livermore Software Technology Corporation, September 1999.

[3] Design of Structures for Missile Impact, BC-TOP-9A, Revision 2, Bechtel Power Corporation Topical Report, September, 1974.

[4] Missiles Generated by Natural Phenomena, NUREG-0800, SRP 3.5.1.4.

Table 1 Key Cask Input Data

ITEM	HI-TRAC -125	HI-TRAC – 100
Total HI-TRAC Weight	152,636 lb.	109,214 lb.
Lead Weight	79,109 lb.	49,810 lb.
Overall Length of the Transfer Cask	207.875 inches	204.125 inches
Length x Width of Transfer Lid	128 in. x 93 in.	128 in. x 89 in.
Outside Diameter of the Radial Channels	94.625 inches	91.0 inches
Inner Shell Diameter	68.75 inches	68.75 inches
Outer Radius of Top Lid	40.625 inches	39.0 inches
Longitudinal Distance Between Point on Transfer Lid and Point on Top Lid where Vertical Displacements are measured (inch)	192.25 inches	191.60 inches
MPC Weight (including fuel)	88,857 lb.	88,857 lb.
MPC Height	190.5 inches	190.5 inches
MPC Diameter	68.375 inches	68.375 inches
MPC Bottom Plate Thickness	2.5 inches	2.5 inches
MPC Top Plate Thickness	9.5 inches	9.5 inches

Table 2 Large Tornado Missile Impact Analysis Results

ITEM	CALCULATED VALUE –125 TON	CALCULATED VALUE – 100 TON	ALLOWABLE VALUE
Maximum Stress in Water Jacket (ksi)	33.697	33.383	58.7
Maximum Stress in Inner Shell (ksi)	18.669	15.6	58.7
Maximum Plastic Strain in Water Jacket	0.0	0.0	-
Maximum Plastic Strain in Inner Shell	0.0	0.0	-

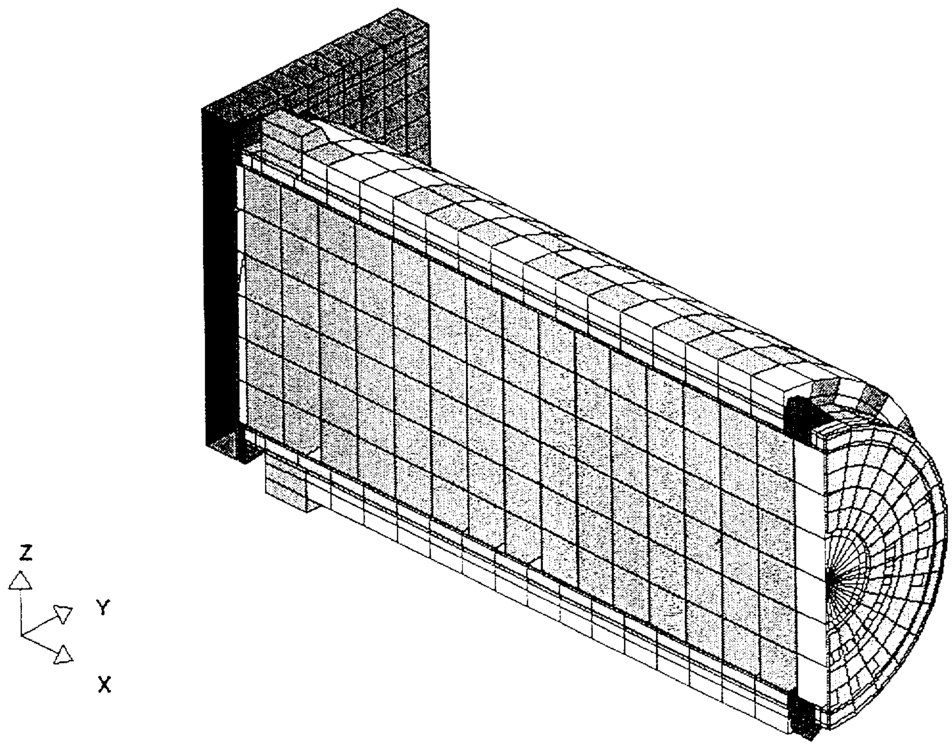


Figure 1 HI-TRAC 100 Finite Element Model

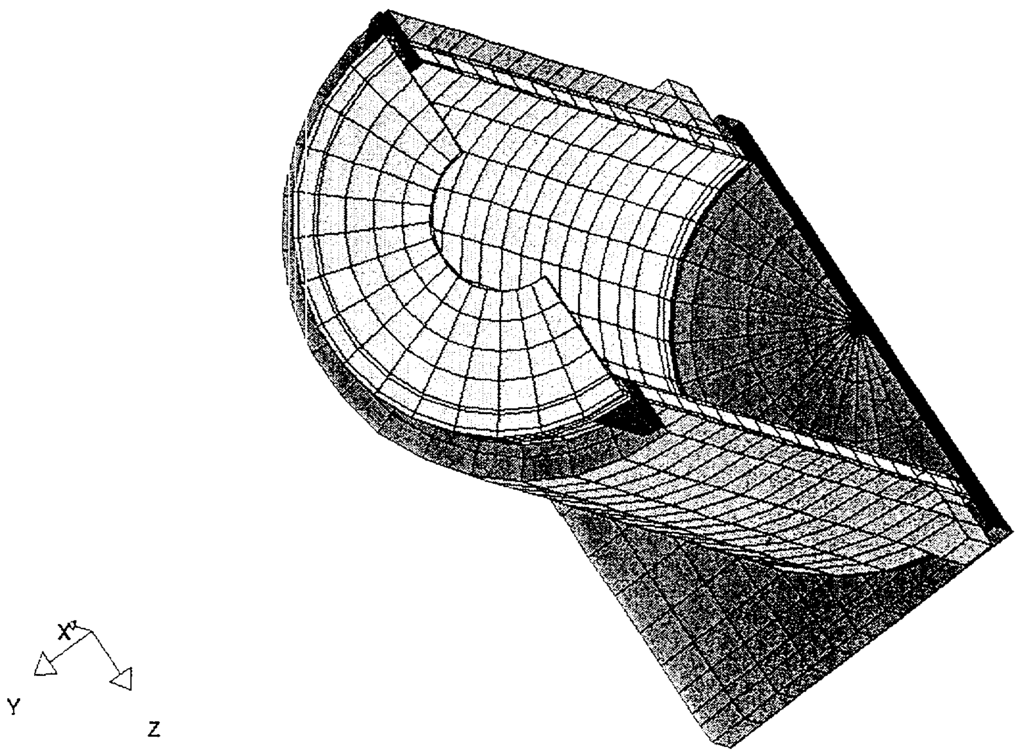


Figure 2 HI-TRAC 100 Finite Element Model (Water and Fuel not Shown)

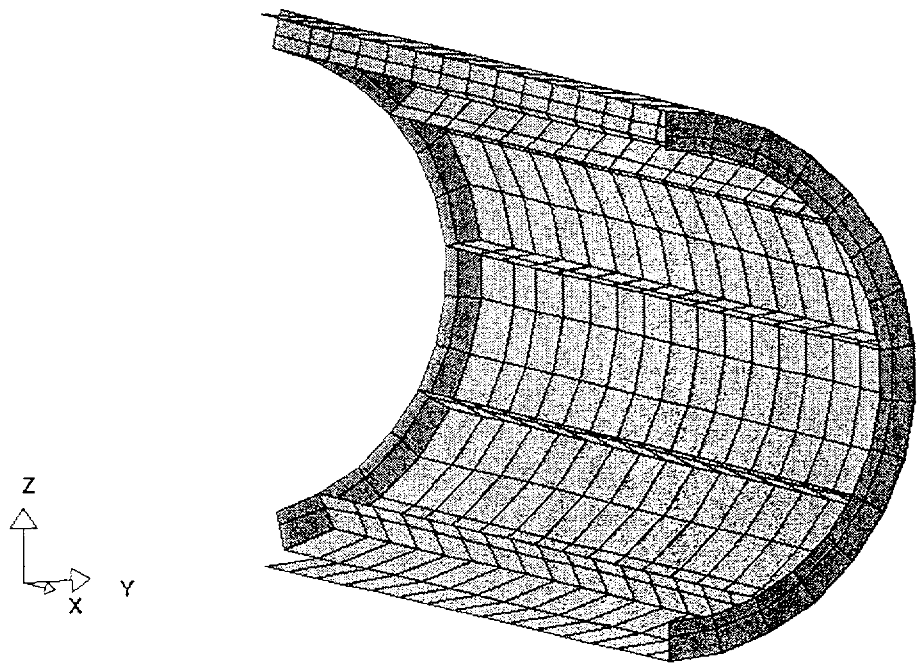


Figure 3 HI-TRAC 100 Finite Element Model – Water Jacket

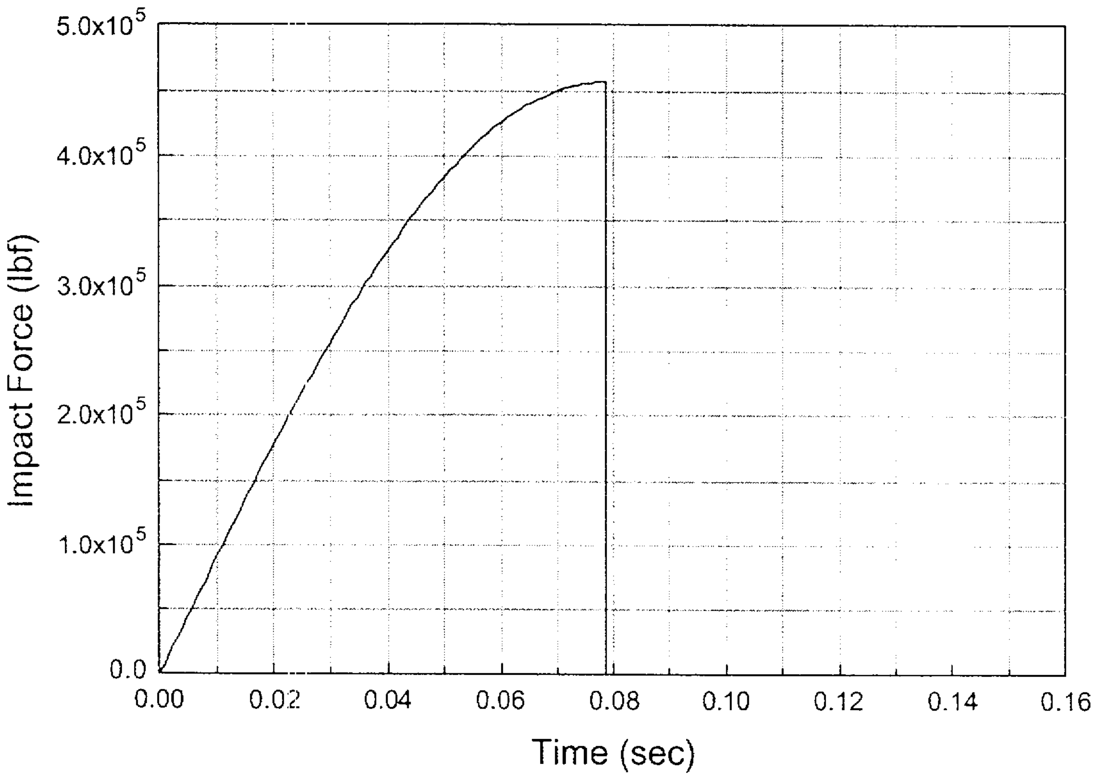


Figure 4 Total Force Applied to the Cask Due to a Large Tornado Missile Impact

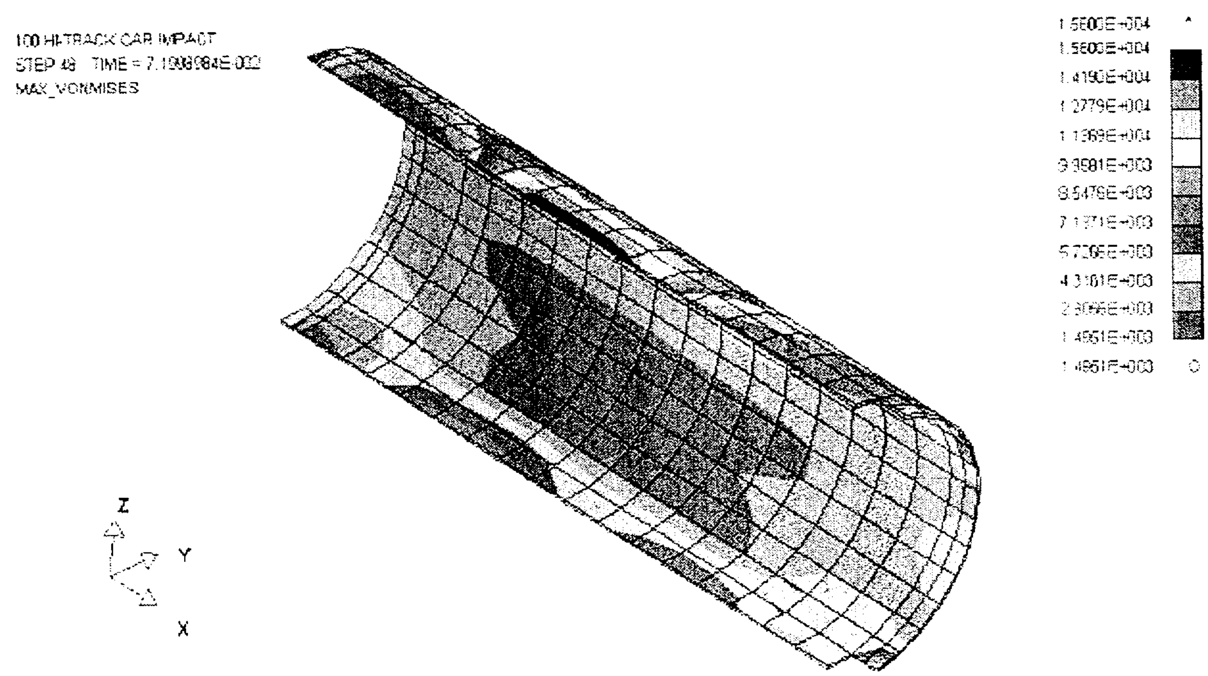


Figure 5 Maximum Von Mises Stress of the Inner Shell of HI-TRAC 100

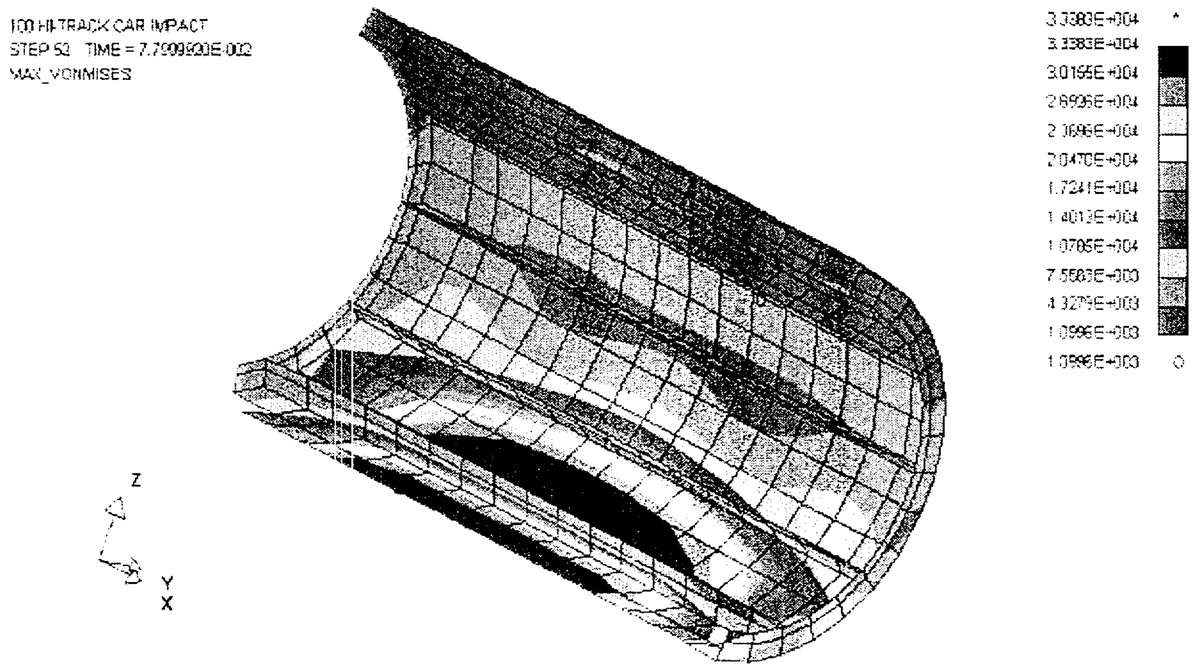


Figure 6 Maximum Von Mises Stress of the Water Jacket of HI-TRAC 100

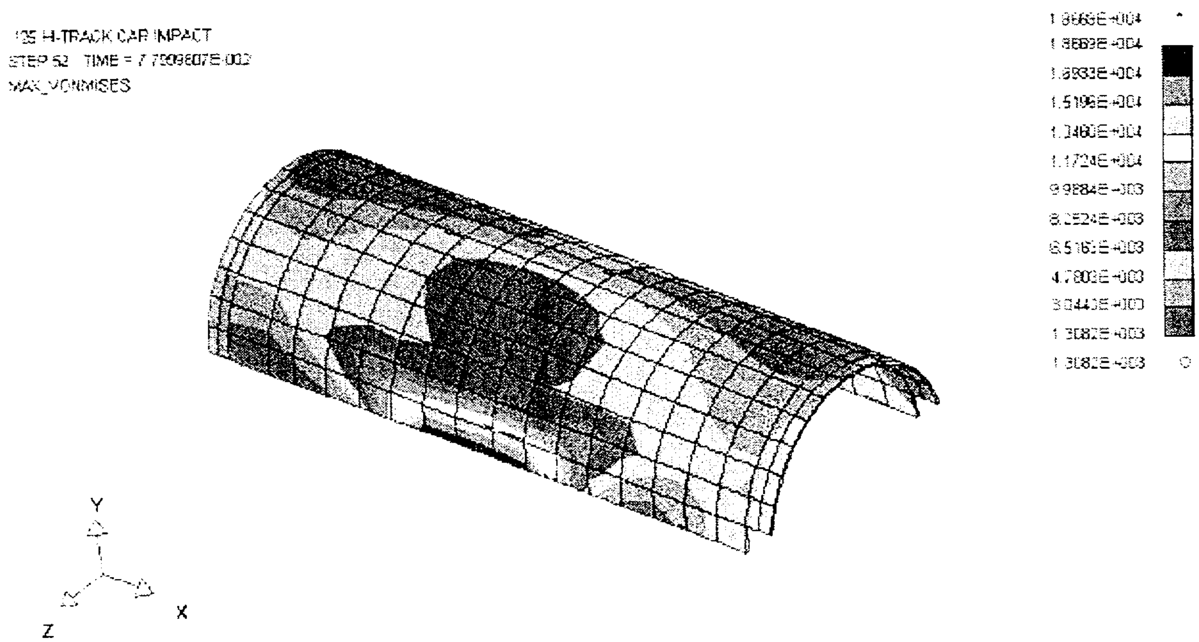


Figure 7 Maximum Von Mises Stress of the Inner Shell of HI-TRAC 125

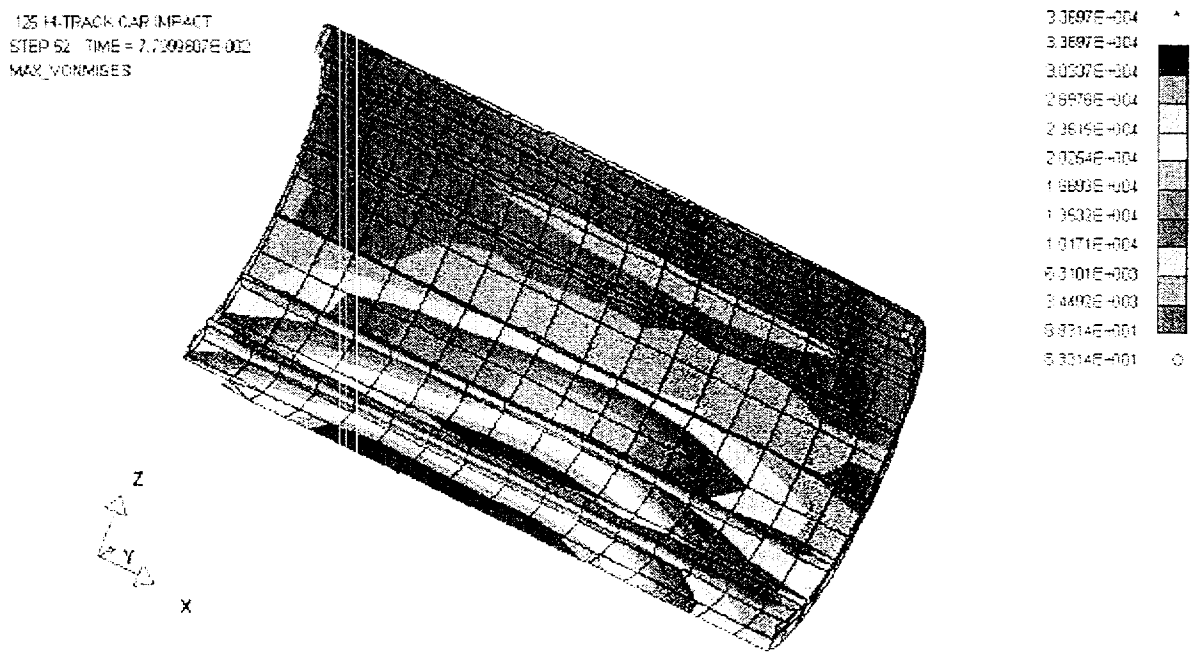


Figure 8 Maximum Von Mises Stress of the Water Jacket of HI-TRAC 125

SUPPLEMENT # 2 COVER PAGE

Hi2002481

Total of 6 pages including this cover sheet

CALCULATION SHEET

Project No. 5014 | Report No. Support of Justification for HI-STORM 100S tipover | Other | Page 2 | Rev.

Prepared By [signature] 8/12/00 Date | Reviewed By | Date

To compute the ratio, following the description, we note that

$$\frac{\ddot{\theta}_{100S}}{\ddot{\theta}_{100}} = \frac{\sqrt{\frac{k L_{100S}^3}{3 I_{A100S}}} \cdot \dot{\theta}_{o100S}}{\sqrt{\frac{k L_{100}^3}{3 I_{A100}}} \cdot \dot{\theta}_{o100}}$$

$$= \sqrt{\frac{L_{100S}^3}{L_{100}^3} \frac{I_{A100}}{I_{A100S}}} \; \frac{\dot{\theta}_{o100S}}{\dot{\theta}_{o100}}$$

But $\frac{\dot{\theta}_{100S}}{\dot{\theta}_{100}} = \left(\frac{I_{A100}}{I_{A100S}}\right)^{1/2}$ (p. 9 of Calc #2 HI-2002481

Therefore, ratio of accelerations is

$$\boxed{A_{RATIO} = \left(\frac{L_{100S}}{L_{100}}\right)^{1.5} \frac{I_{A100}}{I_{A100S}}}$$

From drawing 3608, 1495: $L_{100S} = 221''$, $L_{100} = 231.25$

$$\left(\frac{L_{100S}}{L_{100}}\right)^{1.5} = .934$$

HI-2002481

Project No. 5014 Report No. Other

Prepared By [signature] 8/12/00 Date Reviewed By Date

From Calc # 2 of 2002481, for HI-STORM 100

$$I_A = 2.233 \times 10^7 \quad (p.9)$$

For HI-STORM 100S, use same Mathcad Calc + simply change height dimension. Since concrete is now on top and is a large cylinder, account for approx 50% by adding height to 221"

From Mathcad

$$I_{A_S} = 2.205 \times 10^7$$

$$\therefore \cancel{\frac{I_{A100S}}{I}} \quad \frac{I_{A100}}{I_{A100S}} = \frac{2.233}{2.205} = 1.013$$

$$A_{ratio} = .934 \times 1.013 = .9459$$

The following pages have the modified analysis using the calc #2 method

Compute the Cask Moment of inertia for HI-STORM 100S. Consider the mass distribution as uniform over the length. The mass moment of inertia is computed as a hollow cylinder having an effective mass equal to the total mass of the loaded HI-STORM 100. All formulas used are derived in the Appendix 3.A of the FSAR. Here, we use the formulas without further explaination.

For the HI-STORM 100S, we change the dimensions but not the weight.

$W_{total} := 356521 \cdot lbf$ — We same representative weight for HI-STORM 100S as for HI-STORM 100.

Note that the calculation of angular speed at impact will be independent of the weight. This is because, the angular speed is proportional to the ratio of weight to mass moment of inertia. Since the mass moment of inertia is linearly proportional to the weight, the effect dissapears. Therefore, although a value for weight must be input in order to evaluate the individual quantities, the result for angular velocity at impact are independent of the weight input into this calculation.

$h := 118.5 \cdot in$ est. $\quad L := 226.5 \cdot in$

For HI-STORM 100S, include some of top concrete in moment of inertia calc. Add 5.5 " to total height

$d := 132.5 \cdot in \quad d_i := 72.5 \cdot in$

$$r := \sqrt{\frac{d^2}{4} + (h)^2} \qquad r = 135.762\,in$$

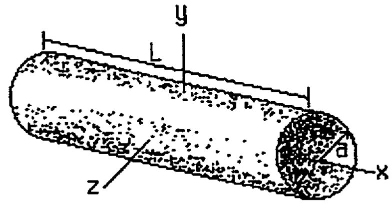
The effective mass density is computed below

$$Volume := \frac{\pi}{4} \cdot \left(d^2 - d_i^2\right) \cdot (2 \cdot h) \qquad Volume = 2.29 \times 10^6\, in^3$$

$$\rho_{eff} := \frac{W_{total}}{Volume \cdot g} \qquad \rho_{eff} = 4.033 \times 10^{-4}\, lbf \cdot \frac{sec^2}{in^4}$$

We compute the mass moment of inertia by subtracting the results obtained for two solid cylinders.

Determine the mass moment of inertia, *I*, of a circular cylinder about the *x*-, *y*-, and *z*-axes, as shown, given the length, *L*, and radius, *a*, of the cylinder, and its mass.



Using the outside dimension

$L = 226.5\,\mathrm{in}$ $\qquad a := \frac{d}{2}$

$mass := \rho_{eff} \cdot \pi \cdot a^2 \cdot L$

$I_{yo} := \frac{1}{12} \cdot mass \cdot \left(3 \cdot a^2 + L^2\right)$ Moment about y

$I_{zo} := I_{yo}$ Moment about z

$I_{yo} = 6.767 \times 10^6\,\mathrm{lbf \cdot in \cdot sec^2}$ $\qquad I_{zo} = 6.767 \times 10^6\,\mathrm{lbf \cdot in \cdot sec^2}$

Using the inside dimension

$L = 226.5\,\mathrm{in}$ $\qquad a := \frac{d_i}{2}$ $\qquad mass := \rho_{eff} \cdot \pi \cdot a^2 \cdot L$

$I_{yi} := \frac{1}{12} \cdot mass \cdot \left(3 \cdot a^2 + L^2\right)$ Moment about y

$I_{zi} := I_{yi}$ Moment about z

$I_{yi} = 1.736 \times 10^6\,\mathrm{lbf \cdot in \cdot sec^2}$ $\qquad I_{zi} = 1.736 \times 10^6\,\mathrm{lbf \cdot in \cdot sec^2}$

The moment of inertia of the hollow cylinder about its centroid is

$I_z := I_{zo} - I_{zi}$ $\qquad I_z = 5.031 \times 10^6\,\mathrm{lbf \cdot in \cdot sec^2}$

The moment of inertia about the lower pivot point A (Fig. 3.A.16 in TSAR) is

$$I_A := I_z + \frac{W_{total}}{g} \cdot r^2 \qquad I_A = 2.205 \times 10^7\, lbf \cdot in \cdot sec^2$$

Find the angle θ per equation 3.A.12

$$\theta := acos\left(\frac{d}{2 \cdot r}\right) \qquad \theta = 60.792\, deg$$

$$\cos(\theta) = 0.488$$

Then the initial angular velocity for the tipover calculation is

$$\omega := \sqrt{\frac{2 \cdot W_{total} \cdot r}{I_A} \cdot \left(1 - \frac{d}{2 \cdot r}\right)} \qquad \omega = 1.499\, s^{-1}$$

SUPPLEMENT 3 CALCULATIONS 1 AND 2

CALCULATION #1

ECO 1024-20, Rev 0, Items 1 and 2

Evaluate Weld size change on inner and outer shell

$t_{w1} := .375 \cdot in$ or $t_{w2} := 2 \cdot 0.25 \cdot in$

tw1 governs $t_w := t_{w1}$ $t_w = 0.375\,in$

For computations where weight of concrete enters as a load, to bound considerations of high density concrete that is used in some units,assume for direct load calculation purposes only

$$\gamma_c := 200 \cdot \frac{lbf}{ft^3}$$

Evaluate hydrostatic pressure at base of unit during pour of concrete (while still wet)

$h := 209.75 \cdot in - 2 \cdot in$ dwg. 3443, sheet 4 rev1 2" deducted for baseplate

$p_{hs} := \gamma_c \cdot h$ $p_{hs} = 24.045\,psi$

Compute hoop stress in outer shell during pour

$D_{os} := 132.5 \cdot in$ dwg. 3443, sheets 2 or 5, Rev 1.

Unit Hoop force is $T_c := p_{hs} \cdot \frac{D_{os}}{2}$ $T_c = 1.593 \times 10^3 \frac{lbf}{in}$

Evaluate longitudinal weld hoop stress

$\tau_{weld} := \frac{T_c}{t_w}$ $\tau_{weld} = 4.248 \times 10^3\,psi$

Allowable weld stress per Table NF-3324.5(a)-1 of Subsection NF, Section III of ASME

$\sigma_u := 70000 \cdot psi$ ultimate strength

$\tau_a := 0.3 \cdot \sigma_u$ $\tau_a = 2.1 \times 10^4\,psi$

$SF_{pour} := \frac{\tau_a}{\tau_{weld}}$ $SF_{pour} = 4.944$ No issue!

CALCULATION #2 ECO-1024-20 Items, 3, 4, 5, and 8

Here we evaluate the inlet vent top plate as a beam strip supported by the vent vertical plate and by the radial rib. We use results from Roark's Handbook, 6th Edition (electronic version).

Input data comes from Appendix 3.DS in this calculation package Rev. 2 update. Dimensions come from Dwgs. 3443

We do the analysis using the bounding heavier concrete and check against the Reg. Guide 3.61 requirement.

$W := 405000 \cdot lbf$ per Appendix 3.DS in this Supplement to the Calc. Package

The load applied at one vent is $Load := \frac{3 \cdot W}{4} \cdot 1.15$ where we have multiplied the actual lifted load, including the amplifier, by 3 and divided by the number of support vents under a bottom lift.

$Load = 3.493 \times 10^{5}\, lbf$

The length of the loaded vent plate can be ascertained from Dwg. 3443 and the BOM

Outer shell OD $D_o := 132.5 \cdot in$ outer shell thickness $t_o := 0.75 \cdot in$

Inner shell ID $D_i := 73.5 \cdot in$ inner shell thickness $t_i := 1.25 \cdot in$

Then the lenght of the loaded region is:

$w := 0.5 \cdot \left[D_o - 2 \cdot t_o - \left(D_i + 2 \cdot t_i\right)\right]$ $w = 27.5\, in$

The length between the two vertical vent plates and the radial rib is:

$L := 16.5 \cdot in$ Dwg. 3443, sheet 5

The load per unit length is

$q := \frac{Load}{L}$ $q = 2.117 \times 10^{4} \frac{lbf}{in}$

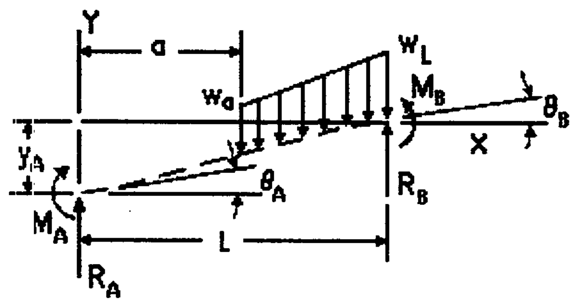
The thickness of the plate is $t_p := 2 \cdot in$

Assuming a beam deflection, the moment of inertia of the complete width is $I := \frac{w \cdot t_p^{3}}{12}$ $I = 18.333\, in^{4}$

Table 3 Shear, Moment, Slope and Deflection Formulas for Elastic Straight Beams

Case 2c Partial Distributed Load; Left End Simply Supported, Right End Fixed

Partial distributed load



Left end simply supported, right end fixed

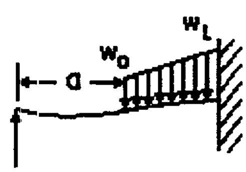


Table 1

Area moment of inertia:	$I \equiv 18.333 \cdot in^4$	
Length of beam:	$L \equiv 16.5 \cdot \frac{in}{2}$	
Distance from left edge to load:	$a \equiv 0 \cdot ft$	
Modulus of elasticity:	$E \equiv 28.0 \cdot 10^6 \cdot \frac{lbf}{in^2}$	Table 3.3.2, FSAR
Unit load at a:	$w_a \equiv 21170 \cdot \frac{lbf}{in}$	
Unit load at L:	$w_L \equiv 21170 \cdot \frac{lbf}{in}$	

Boundary values

The following specify the reaction forces (R), moments (M), slopes (θ) and deflections (y) at the left and right ends of the beam (denoted as A and B, respectively).

At the left end of the beam (simply supported):

$$R_A := \frac{w_a}{8 \cdot L^3} \cdot (L - a)^3 \cdot (3 \cdot L + a) + \frac{w_L - w_a}{40 \cdot L^3} \cdot (L - a)^3 \cdot (4 \cdot L + a)$$

$$R_A = 6.549 \times 10^4 \, \text{lbf}$$

$$M_A := 0 \cdot \text{lbf} \cdot \text{in}$$

$$\theta_A := \frac{-w_a}{48 \cdot E \cdot I \cdot L} \cdot (L - a)^3 \cdot (L + 3 \cdot a) - \frac{w_L - w_a}{240 \cdot E \cdot I \cdot L} \cdot (L - a)^3 \cdot (2 \cdot L + 3 \cdot a)$$

$$\theta_A = -0.028 \, \text{deg}$$

$$y_A := 0 \cdot \text{in}$$

At the right end of the beam (fixed):

$$R_B := \frac{w_a + w_L}{2} \cdot (L - a) - R_A \qquad R_B = 1.092 \times 10^5 \, \text{lbf}$$

$$M_B := \left[R_A \cdot L - \frac{w_a}{2} \cdot (L - a)^2 \right] - \frac{w_L - w_a}{6} \cdot (L - a)^2$$

$$M_B = -1.801 \times 10^5 \, \text{lbf} \cdot \text{in}$$

$$\theta_B := 0 \cdot \text{deg}$$

$$y_B := 0 \cdot \text{in}$$

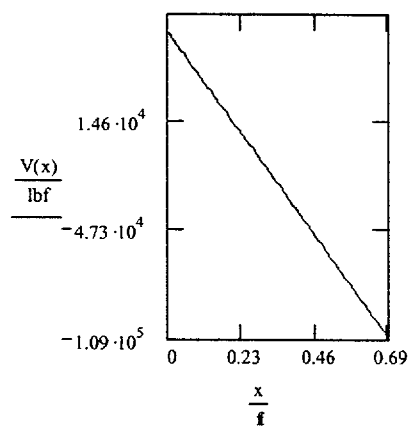
General formulas and graphs for transverse shear, bending moment, slope and deflection as a function of x

$x := 0 \cdot L, .01 \cdot L .. L$ x ranges from 0 to L, the length of the beam.

$x_l := .5 \cdot ft$ Define a point along the length of the beam.

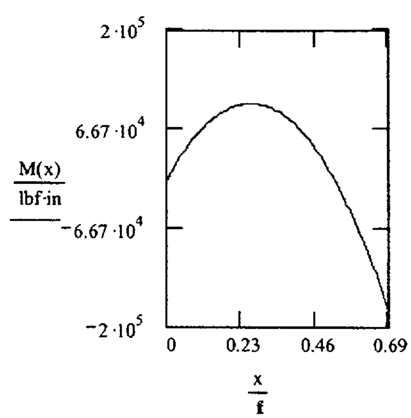
Transverse shear $V(x) := R_A - w_a \cdot (x - a) \cdot (x > a) - \frac{w_L - w_a}{2 \cdot (L - a)} \cdot (x - a)^2 \cdot (x > a)$

$V(x_l) = -6.153 \times 10^4 \, lbf$



Bending moment $M(x) := M_A + R_A \cdot x - \frac{w_a}{2} \cdot (x - a)^2 \cdot (x > a) - \frac{w_L - w_a}{6 \cdot (L - a)} \cdot (x - a)^3 \cdot (x > a)$

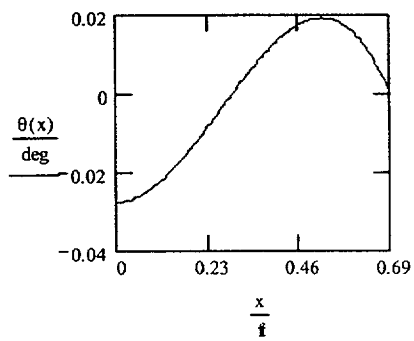
$M(x_l) = 1.191 \times 10^4 \, lbf \cdot in$



Slope

$$\theta(x) := \theta_A + \frac{M_A \cdot x}{E \cdot I} + \frac{R_A \cdot x^2}{2 \cdot E \cdot I} - \frac{w_a}{6 \cdot E \cdot I} \cdot (x-a)^3 \cdot (x>a) - \frac{w_L - w_a}{24 \cdot E \cdot I \cdot (L-a)} \cdot (x-a)^4 \cdot (x>a)$$

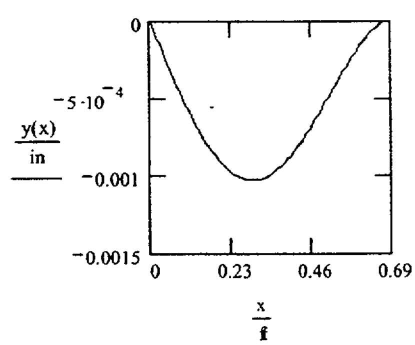
$$\theta(x_1) = 0.019 \, \text{deg}$$



Deflection

$$y(x) := y_A + \theta_A \cdot x + \frac{M_A \cdot x^2}{2 \cdot E \cdot I} + \frac{R_A \cdot x^3}{6 \cdot E \cdot I} - \frac{w_a}{24 \cdot E \cdot I} \cdot (x-a)^4 \cdot (x>a) \ldots$$
$$+ -\frac{w_L - w_a}{120 \cdot E \cdot I \cdot (L-a)} \cdot (x-a)^5 \cdot (x>a)$$

$$y(x_1) = -5.285 \times 10^{-4} \, \text{in}$$



Selected maximum values of moments and deformations

Note: The signs in this section correspond to direction.

The subscripts **maxpos/neg** refer to the maximum magnitude of the most positive or negative value for the given parameters.

If a = 0 and $w_L = w_a$ (uniform load on entire span), then

At x = 3L/8,

$$M_{maxpos} := \frac{9 \cdot w_a \cdot L^2}{128} \qquad M_{maxpos} = 1.013 \times 10^5 \, lbf \cdot in$$

At x = L,

$$M_{maxneg} := \frac{-w_a \cdot L^2}{8} \qquad M_{maxneg} = -1.801 \times 10^5 \, lbf \cdot in$$

At x = 0,

$$\theta_{maxneg} := \frac{-w_a \cdot L^3}{48 \cdot E \cdot I} \qquad \theta_{maxneg} = -0.028 \, deg$$

At x = 0.4512 L,

$$y_{maxneg} := -0.0054 \cdot \frac{w_a \cdot L^4}{E \cdot I} \qquad y_{maxneg} = -1.032 \times 10^{-3} \, in$$

Bending stress in top vent plate

$$\sigma := \frac{M_{maxneg} \cdot t_p}{2 \cdot I} \qquad \sigma = -9.824 \times 10^3 \, psi$$

$\sigma_y := 33150 \cdot psi$ for SA516-70

$$SF_{pb} := \frac{\sigma_y}{-\sigma} \qquad SF_{pb} = 3.374$$

Weld stress (vent top plate-to-vent side plate and vent side plate-to-overpack base plate)

Assume for qualification purposes, a one sided fillet weld

$t_w := 0.375 \cdot in$ $\quad w = 27.5 \, in$

Weld throat area is $\quad A_{weld} := .7071 \cdot t_w \cdot w \quad A_{weld} = 7.292 \, in^2$

$R_A = 6.549 \times 10^4 \, lbf$

$$\tau_{weld} := \frac{R_A}{A_{weld}} \qquad \tau_{weld} = 8.982 \times 10^3 \, psi$$

$$SF_{weldA} := \frac{.6 \cdot \sigma_y}{\tau_{weld}} \qquad SF_{weldA} = 2.214$$

Note that this is an evaluation for 3W

For weld B, the rib takes the entire load in compression $\quad t_{rib} := 0.75 \cdot in \quad$ Dwg 3443, BOM

Rib compressive stress

$$\sigma_{rib} := \frac{2 \cdot R_B}{w \cdot t_{rib}} \qquad \sigma_{rib} = 1.058 \times 10^4 \, psi$$

$$SF_{rib} := \frac{\sigma_y}{\sigma_{rib}} \qquad SF_{rib} = 3.132$$

Calculation 3, labelled Appendix 3.DS in this supplement to the calculation package, computes the safety factor for the rib-to-inner and outer shell welds.

The above computations qualify the vent plate, the vent plate structural welds, and the radial rib for the bottom lift. Reg. Guide 3.61 requirements are met.

SUPPLEMENT 3 - CALCULATION 3

CALCULATION #3 HI-STORM TIPOVER - 100S LID ANALYSIS

Supports ECO 1024-20 and RAI Responses. This replaces Appendices 3.AO and 3.AP in Proposed Rev. 1 of the HI-STORM FSAR.

3.1 Introduction

The fully loaded HI-STORM 100S, with the top lid in place, hypothetically tips over onto the ISFSI pad generating a resultant deceleration load that is bounded by 45 G's at the top of the fuel basket and 49 G's at the top of the storage overpack lid, per Appendix 3.A. In this appendix, the necessary stress analyses are performed to insure that the concrete shielding maintains its position after a non-mechanistic tipover event. Of particular interest is the concrete shield on the outside of the lid of the HI-STORM 100S. It is required that the shielding remain in place subsequent to any accident condition of storage. Appendix 3.K addresses the top lid of the longer HI-STORM 100 that has a different lid configuration. The G levels from the tipover of the HI-STORM 100S are those from Appendix 3.A reduced by the factor 0.946 based on a calculation in the supplement associated with Rev. 1 of this report.

3.2 Methodology

Strength of materials formulations are used to estimate weld stress and shell stresses in the enclosing metal shells surrounding the concrete shielding. The following acceptance criteria are used:

1. For the component parts, other than the studs, the shear ring, and the overpack top plate (acting to resist the shear during the non-mechanistic tipover), Level D stress limits from Appendix F of the ASME Code Section III are used to evaluate safety factors. Use of these limits ensures no gross shape changes occur within the lid.

2. The lid studs in tension, the shear ring, the overpack top plate acting to resist the movement of the lid shear ring, and the welds between the lid shell ring and the lid shear ring and between the overpack top plate and the overpack all serve to maitain the position of the lid relative to the overpack. Safety factors for these items are based on loss of function (this being defined as reaching ultimate strength in either tension or shear).

3.3 Input Data - HI-STORM 100S (from Dwg. 3443 and BOM.)

3.3.1 Geometry

Lid bolt diameter $d_{bolt} := 3.25 \cdot in$ Number of bolts $NB := 4$ Bolt circle $D_{bolt} := 103 \cdot in$

dwg. 3443, sheet 3

Lid top plate thickness $t_{lid} := 4 \cdot in$ Lid top plate diameter $d_{lid} := 126 \cdot in$

Note that the top lid is really two 2" thick plates, one slightly smaller than the diameter given above. The analysis is conservative using the larger diameter. The two plates are welded together around their periphery.

Shield block shell thickness $t_{block} := 0.5 \cdot in$ BOM, Dwg. 3443, sheet 2 or 7, item 26

Shield block height $L_{shieldblock} := 10.0 \cdot in$ Dwg. 3443, BOM

Shield Block outer shell OD $d_{ob} := 86 \cdot in$ Dwg. 3443, sheet 7

Shield Block Cover Plate Thickness $t_{ring} := 0.25 \cdot in$ Dwg. 3443 BOM

Lid Fillet weld size $t_{weld} := 0.25 \cdot in$ Dwg. 3443, sheet 7

Lid shear ring plate thickness and shell top plate thickness $t_{lidbottom} := 0.75 \cdot in$

Lid shear ring (item 31) od and id $D_{srod} := 108 \cdot in$ $D_{srid} := 75 \cdot in$ Dwg. 3443, sheet 2, BOM

Shear ring weld (item 18 to item 31) $t_{srweld} := 0.375 \cdot in$ (dwg. 3443, sheet 7)

Outer Lid shell thickness $t_{outer} := 0.75 \cdot in$

Inner Lid shell thickness $t_{inner} := 1.25 \cdot in$

(Items 19 and 20) Dwg. 3443, BOM

Overpack top plate OD, ID, and weld size $od_{tp} := 132 \cdot in$ $id_{tp} := 109 \cdot in$ $t_{tpweld} := 0.25 \cdot in$

Inner and Outer Shell weld size $t_{sweld} := 0.25 \cdot in$ Assume fillet welds for analysis

Outer Lid shell OD $D_{OD} := 125 \cdot in$

Inner Lid shell ID $d_{ID} := 72.5 \cdot in$ Item 19, sheet 7

Note that the outer plate and inner lid shell thicknesses are identical to the outer and inner shell thicknesses of the HI-STORM barrel.

lid shell length $L_{shell} := 6 \cdot in$

3.3.2 Weight Densities

Concrete $\gamma_c := 150 \cdot \frac{lbf}{ft^3}$ Steel $\gamma_s := 0.283 \cdot \frac{lbf}{in^3}$

3.4 Analyses

3.4.1 Shear Ring and Overpack Top Plate Stress Analysis

Assume the lid is decelerated uniformly by

$G := 48.5 \cdot .946$ Design basis deceleration per Table 3.A.4 of Appendix 3.A (accounts for reduction in input since HI-STORM 100S is shorter, so impact velocity is less (0.946 reduction in input angular velocity per FSAR)

Note that the load path is developed in the following manner:

The studs do not participate in shear resistance by virtue of the large clearance holes that promote easy assembly. The studs serve only to resist the tendency for the lid to rotate off the shell subsequent to a hypothetical tipover event; therefore, all shear loads from the vertical inertia force of the lid are transferred to the inner annular ring of the lid (Item 31 on dwg. 3443,sheet 7). The in-plane load in this annular shear ring is directly transferred to the shell top plate (item 11, dwg 3443, sheet 5), which is an annular ring welded to the overpack outer shell.

We first compute the total deceleration load transferred to item 31 based on the total decelerated weight and demonstrate that the shear ring, the overpack top plate (item 11, annular ring), and the weld to the overpack outer shell (item 12 on dwg. 3443) have sufficient capacity to meet NF Level D stress levels.

Weight of lid(s)

$$W_{lid} := \gamma_s \cdot t_{lid} \cdot \pi \cdot \frac{d_{lid}^{\,2}}{4} \qquad W_{lid} = 1.411 \times 10^4\,\mathrm{lbf}$$

Weight of shield block top plate

$$W_{top} := \gamma_s \cdot t_{ring} \cdot \pi \cdot \frac{d_{ob}^{\,2}}{4} \qquad W_{top} = 410.973\,\mathrm{lbf}$$

Weight of shield block shell

$$W_{shell} := \gamma_s \cdot t_{block} \cdot L_{shieldblock} \cdot \pi \cdot \left(d_{ob}\right) \qquad W_{shell} = 382.3\,\mathrm{lbf}$$

Weight of Shield Block Concrete

$W_{shield} := \gamma_c \cdot 33 \cdot \mathrm{ft}^3$ Refer to BOM on sheet 1 of drawings for cu ft of concrete $W_{shield} = 4.95 \times 10^3\,\mathrm{lbf}$

The total weight of the assemblage calculated so far is

$W_{total} := W_{lid} + W_{top} + W_{shell} + W_{shield}$

$W_{total} = 1.986 \times 10^4 \, lbf$

The remaining weight is associated with the inner and outer shells, the duct plates, the concrete surrounding the ducts, and the lid shell ring (item 18 on BOM) plate. For the total weight of the lid, we use the bounding weight from the dwgs.

For subsequent calculations where the total weight is required, use the bounding weight from Table 3.2.1 for the HI-STORM 100S lid.

$W_{lid} := 25500 \cdot lbf$ $\qquad W_{lid} = 2.55 \times 10^4 \, lbf$

Of this total weight, the shear ring, item 31 comprises approximately

$t_{lidbottom} = 0.75 \, in$ $\qquad D_{srod} = 9 \, ft$ $\qquad D_{srid} = 6.25 \, ft$

$W_{I31} := \gamma_s \cdot \frac{\pi}{4} \cdot \left(D_{srod}^{\,2} - D_{srid}^{\,2}\right) \cdot t_{lidbottom}$ $\qquad W_{I31} = 1.007 \times 10^3 \, lbf$

Assume for the present, that the total load has been transferred to item 31 and we now evaluate the capacity of item 11 and the weld to the overpack outer shell to resist this in-plane load.

$F_t := W_{lid} \cdot G$ $\qquad F_t = 1.17 \times 10^6 \, lbf$

From Table 3.3.2, the ultimate strength of the steel material (@ 350 degrees F) is

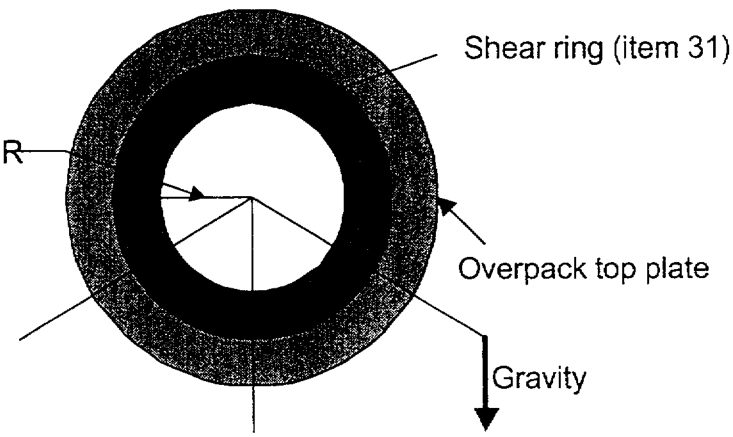
$S_u := 70000 \cdot psi$

The allowable shear strength, under failure conditions, is taken as 60% of the ultimate strength.

$\tau_{allowable} := .6 \cdot S_u$ $\qquad \tau_{allowable} = 4.2 \times 10^4 \, psi$

The allowable bearing strength is taken as 90% of the ultimate strength at failure.

The lid shear ring will bear on the inner diameter area of the overpack top plate. We conservatively assume a total bearing angle of 120 degrees for the computation of average bearing pressure (radially oriented). The bearing angle value is consistent with the common practice associated with a saddle support configuration in a heat exchanger.



$R := \frac{D_{srod}}{2}$ $R = 54\,in$ $Lc := 2 \cdot R \cdot \sin(60 \cdot deg)$ $Lc = 93.531\,in$

$t_{lidbottom} = 0.75\,in$

The radial bearing stress is $\sigma_{bearing} := \frac{F_t}{Lc \cdot t_{lidbottom}}$ $\sigma_{bearing} = 1.668 \times 10^4\,psi$

The safety factor is: $SF_{bear} := \frac{.9 \cdot S_u}{\sigma_{bearing}}$ $SF_{bear} = 3.777$

The load will be transferred to at least 180 degrees of weld to the overpack outer shell(item 11 to item 12 peripheral weld), less one inlet duct length, plus two lengths of groove weld connecting two halves of the top plate (assuming the worst tipover orientation).

$t_{groove} := 0.375in$ $L_{groove} := od_{tp} - id_{tp}$ $L_{groove} = 1.917\,ft$ $L_{duct} := 25 \cdot in$

$A_{weld} := .7071 \cdot t_{tpweld} \cdot \frac{\pi}{2} \cdot od_{tp} + t_{groove} \cdot L_{groove} - .7071 \cdot t_{tpweld} \cdot L_{duct}$ $A_{weld} = 40.859\,in^2$

$\tau_{weld} := \frac{F_t}{A_{weld}}$ $\tau_{weld} = 2.863 \times 10^4\,psi$ $SF_{shear} := \frac{.6 \cdot S_u}{\tau_{weld}}$ $SF_{shear} = 1.467$

Therefore the outer top plate weld to the overpack shell will resist the shear. Note that for most orientations of the drop more than 180 degrees of weld will resist the load. We next examine how the inertia loads are transferred to the shear ring.

Note that the lateral inertia load is transferred to the shear ring through a weld between item 31(the shear ring) and item 18 (the lid shell ring). The weld area available to resist shear is:

$t_{srweld} = 0.375\,in$ $\qquad A_{sr} := .7071 \cdot t_{srweld} \cdot \pi \cdot D_{srid}$

The amplified load through th s weld is

$F_{t1} := (W_{lid} - W_{I31}) \cdot G \qquad F_{t1} = 1.124 \times 10^{6}\,lbf$

$\tau_{srweld} := \dfrac{F_{t1}}{A_{sr}} \qquad \tau_{srweld} = 1.799 \times 10^{4}\,psi \qquad SF_{srshear} := \dfrac{.6 \cdot S_u}{\tau_{srweld}} \qquad SF_{srshear} = 2.335$

3.4.2 Inner and Outer Lid Shell Analysis (items 19 and 20 on dwgs 3443)

The total load to be transferred is $\qquad F_{t1} = 1.124 \times 10^{6}\,lbf$

The total lid shell base metal area (from both inner and outer shell) available to resist this load is

$Area := \pi \cdot (D_{OD} - t_{outer}) \cdot t_{outer} + \pi \cdot (d_{ID} + t_{inner}) \cdot t_{inner} - 4 \cdot 25 \cdot in \cdot (t_{outer} + t_{inner})$

$Area = 382.373\,in^{2}$

The shear stress in the base metal of the two shells is

$\tau_{base} := \dfrac{F_{t1}}{Area} \qquad \tau_{base} = 2.939 \times 10^{3}\,psi$ Minimal stress!

The weld metal area to transfer the load to the shell is $\qquad t_{sweld} = 0.25\,in$

$Area_{weld} := \pi \cdot (D_{OD}) \cdot 0.7071 t_{sweld} + \pi \cdot (d_{ID}) \cdot 0.7071 t_{sweld} - 2 \cdot (4 \cdot L_{duct}) \cdot .7071 \cdot t_{sweld} \qquad Area_{weld} = 74.328\,in^{2}$

The shear stress in the weld group is

$\tau_{weld2} := \dfrac{F_{t1}}{Area_{weld}} \qquad \tau_{weld2} = 1.512 \times 10^{4}\,psi$

Therefore, the safety factor for this weld, under the postulated accident, is (for the actual lid components above the shear ring, we conservatively use 42% of the ultimate as the allowable weld stress instead of a failure weld stress of 60% of the ultimate in tension).

$\tau_{allowable} := 0.42 \cdot S_u \qquad SF_2 := \dfrac{\tau_{allowable}}{\tau_{weld2}} \qquad SF_2 = 1.945$

We conclude that the amplified load can be transferred from the inner and outer shells to item 18 (the lid shell ring) without exceeding weld stress limits. This calculation bounds the result for the lid shell-to-lid weld.

3.4.3 Shield Block Shell (Item 26)-to-Lid Top Plate Weld

The weld is an all around fillet weld of thickness $t_{weld} = 0.25\,in$ $d_{ob} = 86\,in$

$$Area_{weld} := \pi \cdot (d_{ob} + .667 \cdot t_{weld}) \cdot (0.7071 \cdot t_{weld}) \qquad Area_{weld} = 47.853\,in^2$$

The load to be resisted by this weld is the weight of the shield block, the shield block shell, and the shield block top plate.

$$W_{lw} := (W_{top} + W_{shell} + W_{shield}) \qquad W_{lw} = 5.743 \times 10^3\,lbf$$

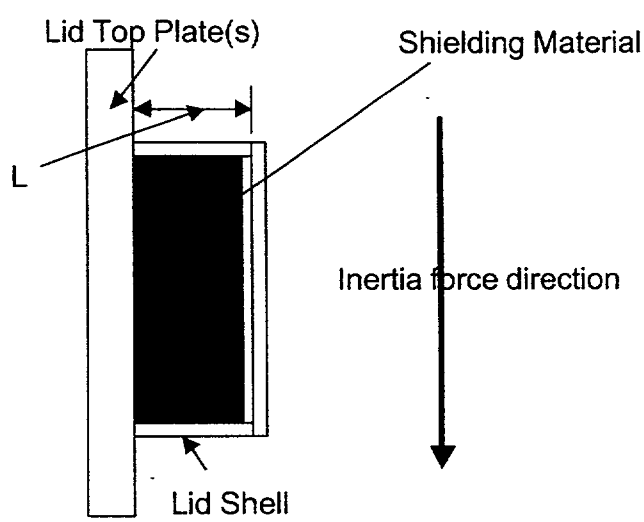
The shear stress in the weld is $\tau_{weld} := \dfrac{W_{lw} \cdot G}{Area_{weld}}$ $\tau_{weld} = 5.507 \times 10^3\,psi$

$$SF_3 := \frac{\tau_{allowable}}{\tau_{weld}} \qquad SF_3 = 5.339$$

3.5 Shield Block Shell Stress Evaluation

3.5.1 Consideration of the shield block shell as a short beam cantilevered from the lid top plate and subject to the amplified weight of the shielding material plus its own amplified weight.

We consider the following sketch that shows a "side" view of the lid top plate, the shield block top plate and the shield shell:



The following analysis computes the "axial" stress in the shield shell due to bending as a short beam.

$L := L_{shieldblock}$ $L = 10\,in$

$t := t_{block}$ $t = 0.5\,in$ $t_{weld} = 0.25\,in$

$d := d_{ob}$ $d = 86\,in$

The amplified load applied to the "beam" is

$Load := \left(W_{top} + W_{shell} + W_{shield}\right)\cdot G$ $Load = 2.635 \times 10^{5}\,lbf$

The area moment of inertia of the weld metal is (base calculation on weld throat thickness)

$I := \frac{\pi}{64}\cdot\left[\left(d + 2.0\cdot.7071\cdot t_{weld}\right)^{4} - (d)^{4}\right]$ $I = 4.443 \times 10^{4}\,in^{4}$

The stress induced by the bending moment is

$\sigma_{bending} := \frac{Load\cdot(0.5\cdot L)\cdot d}{2\cdot I}$ $\sigma_{bending} = 1.275 \times 10^{3}\,psi$

Accounting for bending and shear stress in the weld, the safety factor on the weld needs to be reevaluated.

$SF_4 := \frac{\tau_{allowable}}{\sqrt{\tau_{weld}^{2} + \sigma_{bending}^{2}}}$ $SF_4 = 5.201$ $\sqrt{\tau_{weld}^{2} + \sigma_{bending}^{2}} = 5.652 \times 10^{3}\,psi$

3.5.2 Consideration of circumferential stress in the shield shell

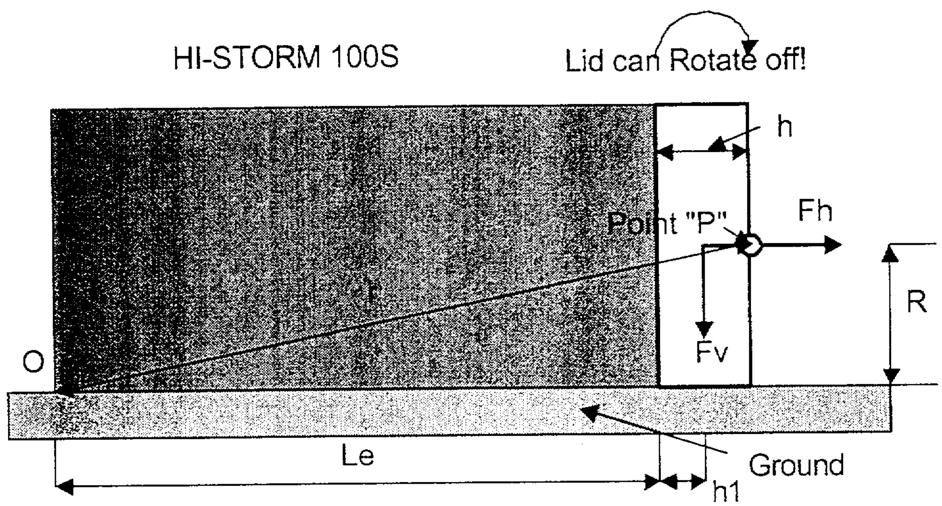
The shield shell is prevented from departing from a circular shape by the top and bottom plates. The effect of these end restraints is felt through an axial distance equal to the so called "bending boundary layer". The bending boundary layer extends along the shell axis approximately a distance equal to $2(td/2)^{1/2}$.

$L_{bl} := 2\cdot\sqrt{\frac{d}{2}\cdot t}$ $L_{bl} = 9.274\,in$

Since the bending boundary layer extends from each end a distance equal to the shell length, it is concluded that the shell does not experience any peripheral stresses due to ring type deformation modes.

3.6 Structural Integrity of Lid Bolts Under Tension During Non-Mechanistic Tipover

The preceding calculation is premised on the assumption that the lid does not exhibit a rigid body rotation after the impact that would cause gross seperation from the body of the overpack. The following figure illustrates the concern:



For purposes of calculation, the following dimensions are applicable:

$Le := 210.5 \cdot in$ dwg 3443, sheet 3 $R := \frac{133.875 \cdot in}{2}$

$h := 231.25 \cdot in - Le$ $h = 20.75\,in$ $h1 := 8.5 \cdot in$ assumed

The drop calculations described in Appendix 3.A of the HI-STORM FSAR report peak vertical deceleration at Point P for the HI-STORM 100, which bounds the result for the HI-STORM 100S. In this calculation, we have used

$G_v := G$ $G_v = 45.881$

The graphical results leading to the tables in Appendix 3A are reported elsewhere in this supplement. Of interest to the computation to follow is the vertical velocity at point "P" at the instant of peak vertical acceleration. We find from examining the graphical data that and correcting for the shorter geometry

$$V_v := -280 \cdot \frac{in}{sec} \cdot .946$$

$r := \sqrt{(Le + h)^2 + R^2}$ $\qquad r = 240.743\,\text{in}$

The angular velocity at peak vertical deceleration is: $\omega := \dfrac{V_v}{(Le + h)}$ $\qquad \omega = -1.145 \dfrac{1}{\text{sec}}$

We can find a result for the tangential deceleration at point "P" from the following equations:

Define Si=sin, Co=cos

$Si := \dfrac{R}{r}$ $\qquad Si = 0.278$ $\qquad Co := \dfrac{(Le + h)}{r}$ $\qquad Co = 0.961$

The acceleration at point P can be broken into tangent and normal components and into horizontal and vertical coefficients. At the instant of peak vertical deceleration, the following two equations relate the tangential and normal to the vertical and horizontal components.

$G_v := A_t \cdot Co + A_n \cdot Si$ $\qquad G_h := A_t \cdot Si - A_n \cdot Co$

where

$G_v = 45.881$ $\qquad A_n := \dfrac{\omega^2 \cdot r}{g}$ $\qquad A_n = 0.818$

Then, from the first of the two equations above, we can determine

$A_t := \dfrac{1}{Co} \cdot (G_v - A_n \cdot Si)$ $\qquad A_t = 47.528$

and therefore $\qquad G_h := A_t \cdot Si - A_n \cdot Co$ $\qquad G_h = 12.429$

The forces tending to rotate the cask from the lid are: $\qquad W_{lid} = 2.55 \times 10^4\,\text{lbf}$

$F_h := W_{lid} \cdot G_h$ $\qquad F_h = 3.169 \times 10^5\,\text{lbf}$

$F_v := W_{lid} \cdot G_v$ $\qquad F_v = 1.17 \times 10^6\,\text{lbf}$

These forces are resisted solely by stretching the four lid bolts.

To resist the net horizontal force, a direct force develops at each stud location

$F_{direct} := \dfrac{F_h}{4}$ $\qquad F_{direct} = 7.923 \times 10^4\,\text{lbf}$

To resist the moment, we assume "edging" of the lid about its bottom corner point. The maximum force, "F", in the bolt furthest from the rotation axis, is computed as:

$$M := F\cdot(.5\cdot D_{bolt} + R) + 2\cdot F\cdot\frac{R^2}{R + \frac{D_{bolt}}{2}} + F\cdot\frac{(R - .5\cdot D_{bolt})^2}{(R + .5\cdot D_{bolt})}$$

If a single bolt is furthest from the rotation point

$$M := 2\cdot F\cdot(R + .5\cdot D_{bolt}\cdot 0.7071) + 2\cdot F\cdot\frac{(R - .5\cdot D_{bolt}\cdot 0.7071)^2}{(R + .5\cdot D_{bolt}\cdot 0.7071)}$$

If the bolt orientation is 45 degrees from the first case

The overtuning moment is:

$M_h := F_h\cdot R$ $\quad M_h = 1.768\times 10^6\ \mathrm{lbf\cdot ft}$

$M_v := F_v\cdot h1$ $\quad M_v = 8.287\times 10^5\ \mathrm{lbf\cdot ft}$ $\quad M := M_h + M_v$

Compute $\quad x1 := R + .5\cdot D_{bolt}$ $\quad x1 = 118.438\ \mathrm{in}$

$x2 := \frac{R^2}{x1}$ $\quad x2 = 37.831\ \mathrm{in}$

$x3 := \frac{(R - .5\cdot D_{bolt})^2}{x1}$ $\quad x3 = 2.012\ \mathrm{in}$

Therefore from the first orientation, F=F1

$F_1 := \frac{M}{(x1 + 2\cdot x2 + x3)}$ $\quad F_1 = 1.589\times 10^5\ \mathrm{lbf}$

Compute $\quad x1 := R + .5\cdot D_{bolt}\cdot .7071$ $\quad x1 = 103.353\ \mathrm{in}$

$x2 := \frac{(R - .5\cdot D_{bolt}\cdot .7071)^2}{x1}$ $\quad x2 = 9.014\ \mathrm{in}$

Therefore from the second orientation, F=F2

$F_2 := \frac{M}{2(x1 + x2)}$ $\quad F_2 = 1.387\times 10^5\ \mathrm{lbf}$

Therefore, the maximum stud load is $\quad F_{max} := F_1 + F_{direct}$ $\quad F_{max} = 2.381\times 10^5\ \mathrm{lbf}$

The stud capacity at failure is based on the ultimate strength @200 deg. F $S_u := 107130 \cdot psi$

Table 3.3.4 of FSAR

The stress area of the stud is $A_{stress} := 7.1 \cdot in^2$ Machinery's Handbook

$$SF_{rotation} := \frac{S_u \cdot A_{stress}}{F_{max}} \qquad SF_{rotation} = 3.194$$

To ensure that the stud carries no shear, the holes in the lid have been enlarged. and a 3/8" thick washer has been emplaced between the nut and the lid surface to transfer the load.

$t_{washer} := 0.375 \cdot in$ The minimum shear circle of the washer is set as the flat of the nut

Dwg 3443, sheet 8 $d_{nut} := 4.875 \cdot in$

The area available for shear is $A_{washer} := \pi \cdot d_{nut} \cdot t_{washer}$ $A_{washer} = 5.743\, in^2$

Based on washer failure in shear at 60% of the material ultimate strength, the required washer ultimate strength is:

$$S_{uwasher} := \frac{F_{max}}{.6 \cdot A_{washer}} \qquad S_{uwasher} = 6.91 \times 10^4 psi$$

A washer from Alloy X or SA 516-Gr. 70 is acceptable.

3.7 Conclusions

Stress in the shield block shell remains below Level A values.

All welds connecting the shield block shells and the shield shell to the lid have stress levels below the Level D limit for welds from ASME Section III, Subsection NF. Therefore, the shield materials remain in place.

The shear ring and the overpack top plate, acting in concert, maintain the lid in lateral position relative to the overpack. The studs do not experience any shear. Based on stud failure at ultimate strength in tension and washer failure at ultimate strength in shear, we also conclude that the lid will not rotate away from the overpack.

It is concluded that the HI-STORM 100S lid will remain in place after a hypothetical tipover event and continue to provide the necessary radiation shielding.

SUPPLEMENT 3 - CALCULATION 4

4.0 INTRODUCTION

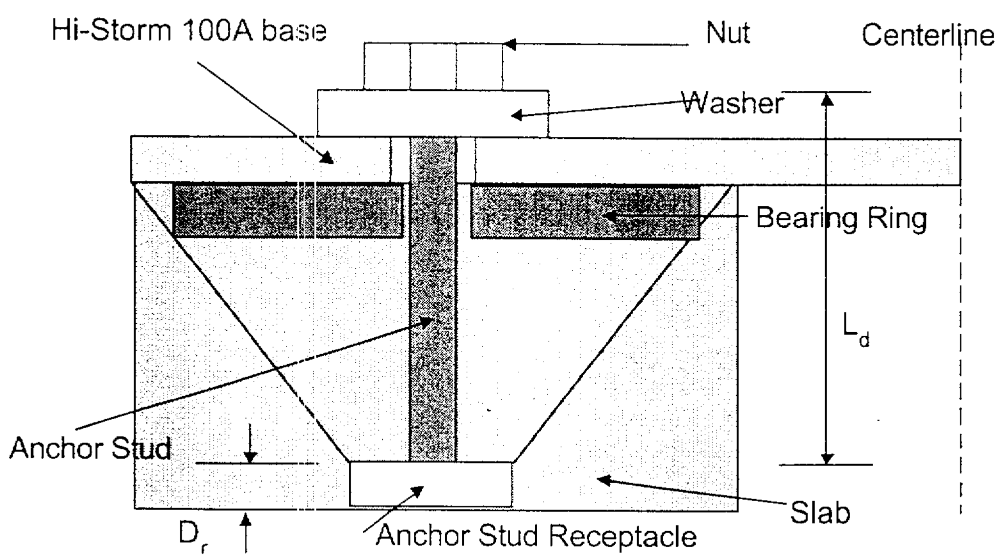
This Calculation, written in MATHCAD 2000, contains necessary supporting calculations for the typical anchorage described in LAR-1014-1 (Response to RAI#1 question). The intent of this calculation is to demonstrate that a practical anchorage can be constructed. In what follows, References to FSAR mean LAR-1014-1 unless otherwise noted. The analysis herein is based on standard concrete cone construction with reinforcement as necessary to mitigate concrete cracking. Reinforcement, as necessary, is assumed in place as outlined in Appendix B of ACI-349-97, Section B.4.4 and associated commentary on B.4 (see Figs. B-4 and B-5 in the commentary section).

The calculations herein focus only on the anchor stud and the anchor stud receptacle. Only the receptacle is designed in accordance with ACI; the anchor stud itself meets the limits of ASME Code NF and Appendix D and has been qualified in Chapter 3 of the FSAR, (LAR-1014-1).

4.1 EMBEDMENT LENGTH

(per ACI 349-97, Appendix B)

The following sketch is used for determination of minimum embedment length of anchor bolts:



A more detailed figure is presented in the FSAR,

4.1.1 Input Data

$D_{bc} := 139.5 \cdot in$ Bolt circle diameter per Dwg. 3187 of FSAR

$f_c := 5000 \cdot psi$ Greater than Minimum concrete strength per FSAR Table 2.0.4 .

$t_s := 2 \cdot in$ $t_{wash} := 1 \cdot in$ HI-STORM 100SA baseplate, washer thickness

$\sigma_u := 125000 \cdot psi$ Maximum ultimate strength of stud from Table 2.0.4 of FSAR

$S_u := 70000 \cdot psi$ Maximum ultimate strength of anchor receptacle

$\sigma_y := 38000 \cdot psi$ Minimum Yield strength of anchor receptacle

4.1.2 Stud Capacity

The anchor stud is not part of the embedment, but is part of the cask; therefore, the steel embedment structure is the anchor receptacle plus any steelworks necessary to hold it in place. The fact that the receptacle may be buried in the concrete at depth Ld does not alter the code jurisdiction. Therefore, in establishing requirements on the embedment and the concrete, the strength of the anchor stud need not be considered except to the extent that the receptacle should have essentially the same load capacity.

Anchor stud nominal diameter $d_b := 2 \cdot in$

Nominal area of anchor stud $A_b := \frac{\pi}{4} \cdot d_b^2$ $A_b = 3.142\, in^2$

Stress Area of anchor stud $A_{stress} := 2.5 \cdot in^2$ Machinery's Handbook, 23rd Edition, Table 3a, p.1484.

Under seismic loading, the anchor stud capacity is $CapStud := 0.7 \cdot \sigma_u \cdot A_{stress}$

$CapStud = 2.188 \times 10^5\, lbf$

Under the same condition, the capacity of the receptacle is governed by the yield strength in accord with B.6.5.1 of Appendix B of ACI 349-97. If we seek an exact match of safety factor = Capacity/Demand, then we need to set the loaded area of the anchor stud receptacle as:

$Area := \frac{CapStud}{0.9 \cdot \sigma_y}$ $Area = 0.044\, ft^2$ $\frac{Area}{A_{stress}} = 2.558$

4.1.3 Capacity Control

We now check to see whether the steel strength controls in accord with Subsection B.5.1

Assume the following parameters for a typical design

$L_d := 42 \cdot in$ $D_r := 8 \cdot in$ $Head\_diam := 6 \cdot in$

Note that the depth of the anchor receptacle is chosen so that the anchor stud is flush with the ISFSI pad surface.

$\mathbf{R} := \mathbf{L_d} - \mathbf{t_s} - \mathbf{t_{wash}} + \mathbf{D_r} + .5 \cdot \mathbf{Head\_diam}$ $\quad \mathbf{R} = 50\,\mathbf{in}$ Note R < 69" so full wedge can be developed around overpack periphery.

$\phi_c := 0.85$ Number of Studs $\mathbf{NS} := 28$

Spacing between studs (around perimeter of stud circle of cask (average))

$$s := \pi \cdot \frac{\mathbf{D_{bc}}}{\mathbf{NS}} \qquad s = 15.652\,\mathbf{in} \qquad f_c = 5 \times 10^3\,\mathbf{psi}$$

Concrete support is provided by a 360 degree wedge projecting beyond the cask and under the cask (if the ISFSI slab is deep enough per interpretation of Figure B.4.2 in Appendix B of ACI-349-97)

In order that this be so, then the pad depth should be at least

$$H_{pad} := \frac{D_{bc} + 2 \cdot (L_d - t_s - t_{wash} + D_r)}{2} \qquad H_{pad} = 9.729\,\mathrm{ft} \qquad H_{pad} = 116.75\,\mathbf{in}$$

If this is so, then the projected area of concrete per stud is

$$A_{net} := 2 \cdot (L_d - t_s - t_{wash} + D_r) \cdot s - \mathbf{Head\_diam} \cdot s \qquad A_{net} = 9.565\,\mathrm{ft}^2$$

Following Subsection B.4.2, the tensile strength of concrete is

$$\sigma_{concrete} := 4 \cdot \phi_c \cdot \sqrt{f_c \cdot \mathbf{psi}}$$

and the concrete capacity is $P_{concrete} := \sigma_{concrete} \cdot A_{net}$ $\quad P_{concrete} = 3.311 \times 10^5\,\mathbf{lbf}$

Note that this is in excess of the anchor stud capacity so that actual safety factors for the concrete are greater than those for the anchor stud and/or the stud receptacle. That is,

$$SF_1 := \frac{P_{concrete}}{\mathbf{CapStud}} \qquad \frac{P_{concrete}}{\mathbf{CapStud}} = 1.514$$

However, to meet the intent of Subsection B.5.1.1, we evaluate

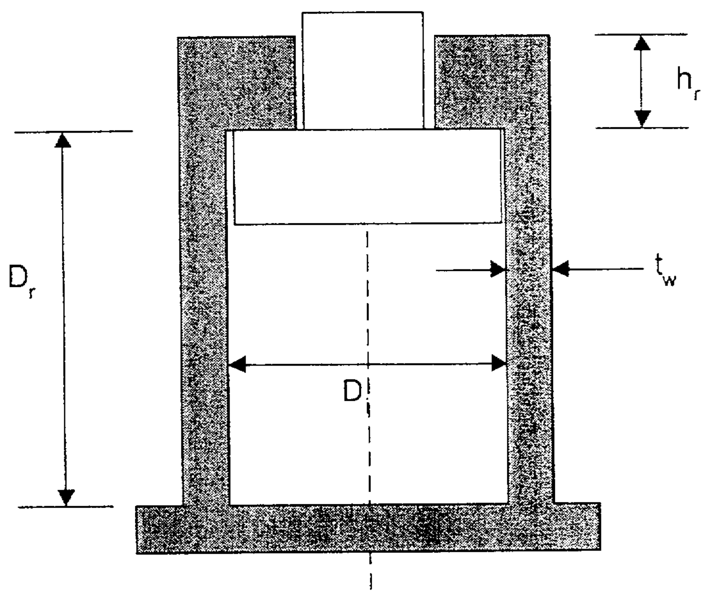
$$P_{steel} := S_u \cdot \mathbf{Area} \qquad P_{steel} = 4.477 \times 10^5\,\mathbf{lbf} \qquad \frac{P_{concrete}}{P_{steel}} = 0.74 \quad < 1.0$$

We conclude that the requirements of B.5.1.1 are not met and that additional reinforcement to inhibit cracking of the concrete is mandated to comply with Subsection B.4.4. Once this is recognized, we may trade off concrete depth for reinforcement or increase the embedment depth (and therefore the slab depth) to permit a design without any additional reinforcement to inhibit concrete cracking.

The remainder of this calculation presents the detailed strength analysis of the anchor receptacle.

4.2 Anchor Receptacle

Consider the following figure showing the anchor receptacle concept:



4.2.1 Input Data

$\mathbf{D_r} = 8\,\mathbf{in}$ $\quad$ $\mathbf{Area} = 6.396\,\mathbf{in}^2$ $\quad$ $\mathbf{d_b} = 2\,\mathbf{in}$

$\mathbf{D_i} := 1.75 \cdot \mathbf{d_b}$ $\quad$ $\mathbf{D_i} = 3.5\,\mathbf{in}$

$$\mathbf{D_o} := \mathbf{D_i} \cdot \left(1 + \frac{4 \cdot \mathbf{Area}}{\pi \cdot \mathbf{D_i}^2}\right)^{\frac{1}{2}} \qquad \mathbf{D_o} = 4.516\,\mathbf{in}$$

Therefore, $\quad \mathbf{t_w} := \frac{1}{2} \cdot (\mathbf{D_o} - \mathbf{D_i}) \qquad \mathbf{t_w} = 0.508\,\mathbf{in}$

Use $\quad \mathbf{t_w} := 0.625 \cdot \mathbf{in}$

4.2.2 Calculations

Bearing Area of receptacle/nut interface

$$\mathbf{Area_{bearing}} := \frac{\pi}{4}\cdot\left(\mathbf{D_i}^2 - \mathbf{d_b}^2\right) \qquad \mathbf{Area_{bearing}} = 6.48\,\mathbf{in}^2$$

$$\mathbf{A_{ring}} := \frac{\pi}{4}\cdot\left[\left(\mathbf{D_i} + \mathbf{t_w}\right)^2 - \mathbf{d_b}^2\right]$$

This area is computed based on the mean diameter of the cylindrical barrel.

$$\sigma_y = 3.8\times 10^4\,\mathbf{psi}$$

$$\mathbf{Bearing\_Capacity} := 0.9\cdot\sigma_y\cdot\mathbf{Area_{bearing}} \qquad \mathbf{Bearing\_Capacity} = 2.216\times 10^5\,\mathbf{lbf}$$

This should be greater than the preload

$$\mathbf{Preload} := 150000\cdot\mathbf{lbf}$$

$$\mathbf{SF_2} := \frac{\mathbf{Bearing\_Capacity}}{\mathbf{Preload}} \qquad \mathbf{SF_2} = 1.477$$

$$\mathbf{SF_3} := \frac{\mathbf{Bearing\_Capacity}}{\mathbf{CapStud}} \qquad \mathbf{SF_3} = 1.013$$

It is not apparent that bearing capacity of the receptacle must be evaluated under the seismic load condition; nevertheless, we demonstrate the acceptability of the design by comparing to the capacity of the stud.

Per Subsection B.6 of Appendix B of ACI-349-97, the allowable shear strength is

$$\tau_a := 0.55\cdot\sigma_y \qquad \tau_a = 2.09\times 10^4\,\mathbf{psi}$$

Therefore the minimum required depth to transfer the anchor stud capacity is:

$$\mathbf{h_r} := \frac{\mathbf{CapStud}}{\pi\cdot\mathbf{D_i}\cdot\tau_a} \qquad \mathbf{h_r} = 0.952\,\mathbf{in}$$

Since the head of the anchor stud is made of a high strength material, it is stiffer than the annular ring at the top of the anchor receptacle. Therefore, the resultant load at the interface shifts toward the inner diameter of the receptacle. Nevertheless, for conservatism, we consider the load as uniformly distributed and compute the hoop stress state by considering an annular plate of thickness h_r that is free at the inner radius and pinned at the outer (mean) radius.

The Solution is given in Roark's Handbook, 6th Edition and is presented below:

$\mathbf{Pressure} := \dfrac{\mathbf{CapStud}}{\mathbf{A_{ring}}}$ $\mathbf{Pressure} = 2.14 \times 10^{4}\ \mathbf{psi}$ Consistent with ring dimensions

Table 24 Formulas for shear, moment and deflection of flat circular plates of constant thickness

Cases 2a - 2d Annular Plate With Uniformly Distributed Pressure q Over the Portion from r_0 to a; Outer Edge Simply Supported

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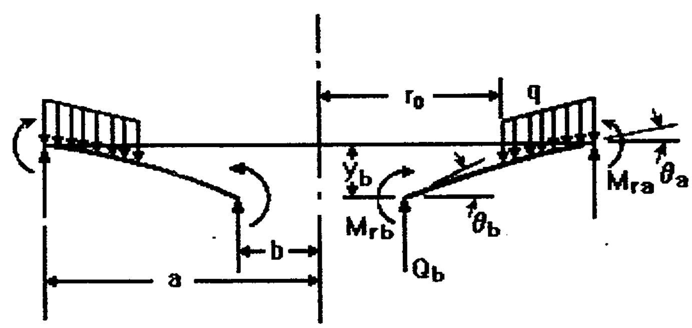
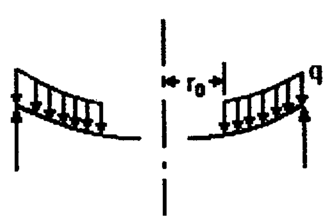
This file corresponds to Cases 2a - 2d in *Roark's Formulas for Stress and Strain.*

Annular plate with a uniformly distributed pressure q over the portion from r_o to a



Outer edge simply supported, inner edge free



Enter dimensions, properties and loading

Choose t > than minimum shear depth

Plate dimensions:

thickness: $t \equiv 2.5 \cdot in$

outer radius: $a \equiv 1.75 \cdot in + \frac{0.625}{2} \cdot in$

inner radius: $b \equiv 1 \cdot in$

Applied pressure: $q \equiv 21400 \cdot psi$

Modulus of elasticity: $E \equiv 29 \cdot 10^6 \cdot \frac{lbf}{in^2}$

Poisson's ratio: $v \equiv 0.3$

Radial location of applied load: $r_o \equiv 1 \cdot in$

Constants

Shear modulus: $G \equiv \frac{E}{2 \cdot (1 + v)}$

D is a plate constant used in determining boundary values; it is also used in the general equations for deflection, slope, moment and shear. K_{sb} and K_{sro} are tangential shear constants used in determining the deflection due to shear:

$$D \equiv \frac{E \cdot t^3}{12 \cdot (1 - v^2)} \qquad D = 4.149 \times 10^7 \, lbf \cdot in$$

$$K_{sro} \equiv -0.30 \cdot \left[1 - \left(\frac{r_o}{a} \right)^2 \cdot \left(1 + 2 \cdot \ln\left(\frac{a}{r_o} \right) \right) \right]$$

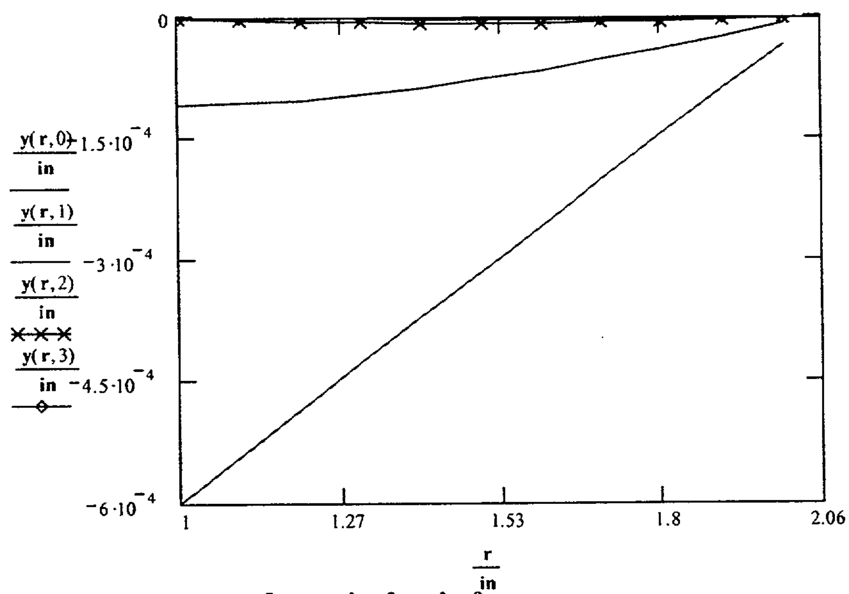
$$K_{sb} \equiv K_{sro}$$

General formulas and graphs for deflection, slope, moment, shear and stress as a function of r

Define r, the range of the radius and i, the vector index:

$\mathbf{r \equiv b, 1.1 \cdot b .. a}$ $\qquad\qquad$ $\mathbf{i \equiv 0..3}$

Deflection $\quad y(r,i) := y_{b_i} + \theta_{b_i} \cdot r \cdot F_1(r) + M_{rb_i} \cdot \frac{r^2}{D} \cdot F_2(r) + Q_{b_i} \cdot \frac{r^3}{D} \cdot F_3(r) - q \cdot \frac{r^4}{D} \cdot G_{11}(r)$



Inner edge free: i = 0
Inner edge guided: i = 1
Inner edge simply supported: i = 2
Inner edge fixed: i = 3

The following values are listed in order of inner edge:

- **free (i = 0)**
- **guided (i = 1)**
- **simply supported (i = 2)**
- **fixed (i = 3)**

Deflection at points b and a (inner and outer radius) due to bending:

Deflection at points b and r_o due to shear:

$$\frac{y_b}{in} = \begin{pmatrix} -5.993 \times 10^{-4} \\ -1.072 \times 10^{-4} \\ 0 \\ 0 \end{pmatrix}$$

$$\frac{y_a}{in} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \qquad \frac{y_{sb}}{in} = \begin{pmatrix} -4.158 \times 10^{-4} \\ -4.158 \times 10^{-4} \\ 0 \\ 0 \end{pmatrix} \qquad \frac{y_{sro}}{in} = \begin{pmatrix} -9.775 \times 10^{-5} \\ -9.775 \times 10^{-5} \\ 0 \\ 0 \end{pmatrix}$$

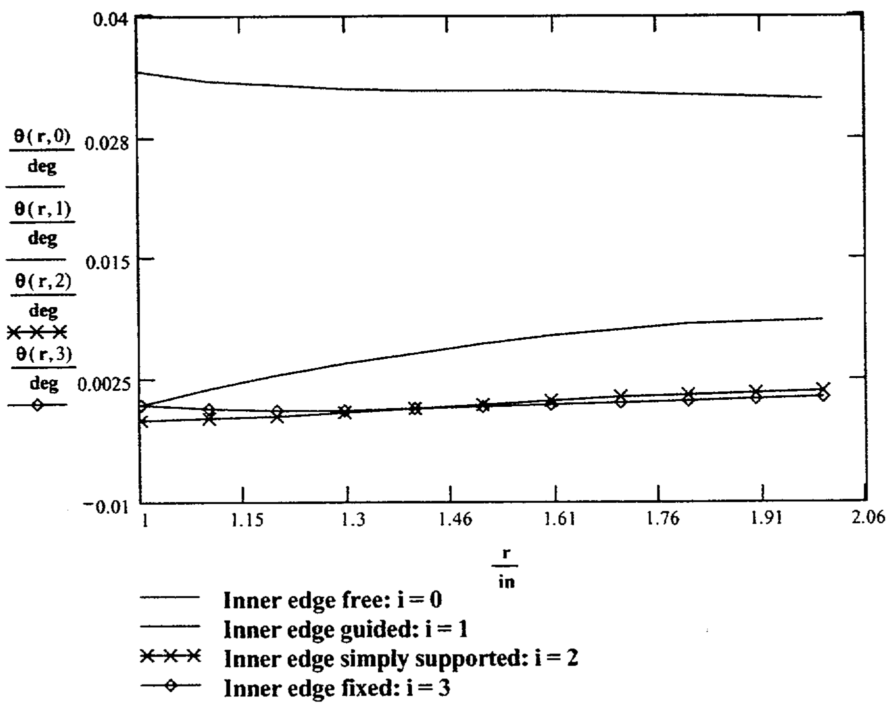
Maximum deflection (magnitude):

$$Y_{(r-b)\frac{100}{in},\,i} := y(r,i) \qquad A_{Y_i} := \max\left(Y^{\langle i \rangle}\right) \qquad B_{Y_i} := \min\left(Y^{\langle i \rangle}\right)$$

$$y_{max_i} := \left(A_{Y_i} > -B_{Y_i}\right) \cdot A_{Y_i} + \left(A_{Y_i} \le -B_{Y_i}\right) \cdot B_{Y_i}$$

$$\frac{y_{max}}{in} = \begin{pmatrix} -5.993 \times 10^{-4} \\ -1.072 \times 10^{-4} \\ -8.302 \times 10^{-6} \\ -3.771 \times 10^{-6} \end{pmatrix}$$

Slope $$\theta(r,i) := \theta_{b_i} \cdot F_4(r) + M_{rb_i} \cdot \frac{r}{D} \cdot F_5(r) + Q_{b_i} \cdot \frac{r^2}{D} \cdot F_6(r) - q \cdot \frac{r^3}{D} \cdot G_{14}(r)$$



The following values are listed in order of inner edge:

- **free (i = 0)**
- **guided (i = 1)**
- **simply supported (i = 2)**
- **fixed (i = 3)**

Slope at points b and a (inner and outer radius):

$$\frac{\theta_b}{deg} = \begin{pmatrix} 0.034 \\ 0 \\ -1.573 \times 10^{-3} \\ 0 \end{pmatrix} \qquad \frac{\theta_a}{deg} = \begin{pmatrix} 0.031 \\ 8.53 \times 10^{-3} \\ 1.337 \times 10^{-3} \\ 7.288 \times 10^{-4} \end{pmatrix}$$

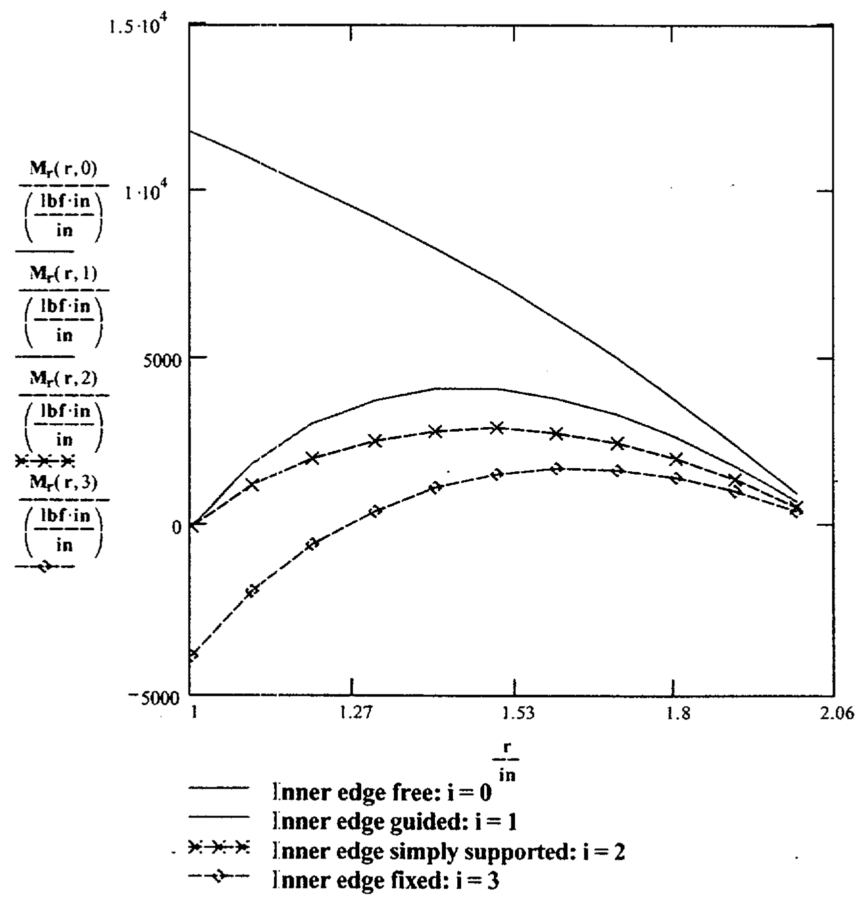
Maximum slope (magnitude):

$S_{(r-b)\frac{100}{in},\,i} := \theta(r,i)$ $\qquad As_i := \max\left(S^{\langle i\rangle}\right)$ $\qquad Bs_i := \min\left(S^{\langle i\rangle}\right)$

$$\theta_{max_i} := \left(As_i > -Bs_i\right)\cdot As_i + \left(As_i \le -Bs_i\right)\cdot Bs_i$$

$$\frac{\theta_{max}}{deg} = \begin{pmatrix} 0.034 \\ 8.567\times 10^{-3} \\ -1.573\times 10^{-3} \\ 7.158\times 10^{-4} \end{pmatrix}$$

Radial moment $\qquad M_r(r,i) := \theta_{b_i}\cdot\frac{D}{r}\cdot F_7(r) + M_{rb_i}\cdot F_8(r) + Q_{b_i}\cdot r\cdot F_9(r) - q\cdot r^2\cdot G_{17}(r)$



The following values are listed in order of inner edge:

- **free (i = 0)**
- **guided (i = 1)**
- **simply supported (i = 2)**
- **fixed (i = 3)**

Moment at points b and a (inner and outer radius):

$$\frac{M_{rb}}{\left(\frac{lbf \cdot in}{in}\right)} = \begin{pmatrix} 0 \\ 1.176 \times 10^{4} \\ 0 \\ -3.839 \times 10^{3} \end{pmatrix}$$

$$\frac{M_{ra}}{\left(\frac{lbf \cdot in}{in}\right)} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix}$$

Maximum radial moment (magnitude):

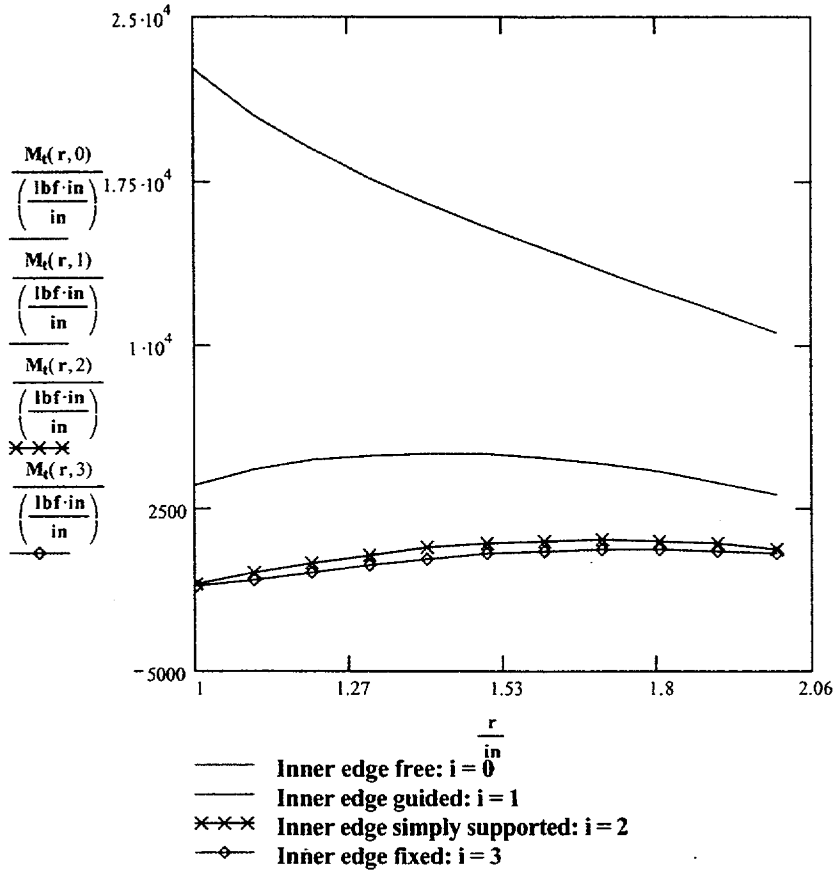
$$Mr_{(r-b)\frac{100}{in}, i} := M_r(r, i) \qquad A_{mr_i} := \max\left(Mr^{\langle i \rangle}\right) \qquad B_{mr_i} := \min\left(Mr^{\langle i \rangle}\right)$$

$$Mr_{max_i} := \left(A_{mr_i} > -B_{mr_i}\right) \cdot A_{mr_i} + \left(A_{mr_i} \le -B_{mr_i}\right) \cdot B_{mr_i}$$

$$\frac{Mr_{max}}{\left(\frac{lbf \cdot in}{in}\right)} = \begin{pmatrix} 4.077 \times 10^{3} \\ 1.176 \times 10^{4} \\ 2.912 \times 10^{3} \\ -3.839 \times 10^{3} \end{pmatrix}$$

Transverse moment

$$M_t(r,i) := \frac{\theta(r,i)\cdot D\cdot\left(1-\nu^2\right)}{r} + \nu\cdot M_r(r,i)$$



The following values are listed in order of inner edge:

- **free (i = 0)**
- **guided (i = 1)**
- **simply supported (i = 2)**
- **fixed (i = 3)**

Transverse moment at points b and a (inner and outer radius) due to bending:

$\dfrac{M_t(b,i)}{\left(\dfrac{lbf \cdot in}{in}\right)} =$	$\dfrac{M_t(a,i)}{\left(\dfrac{lbf \cdot in}{in}\right)} =$
$2.251 \cdot 10^4$	$9.953 \cdot 10^3$
$3.528 \cdot 10^3$	$2.726 \cdot 10^3$
$-1.037 \cdot 10^3$	427.139
$-1.152 \cdot 10^3$	232.864

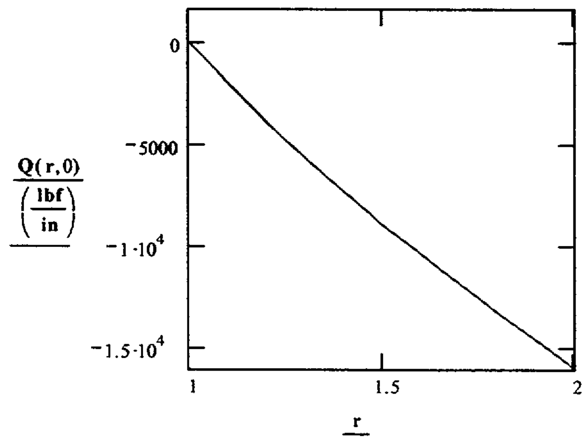
Maximum tangential moment (magnitude):

$$Mt_{(r-b)\frac{100}{in},\,i} := M_t(r,i) \qquad A_{mt_i} := \max\left(Mt^{\langle i \rangle}\right) \qquad B_{mt_i} := \min\left(Mt^{\langle i \rangle}\right)$$

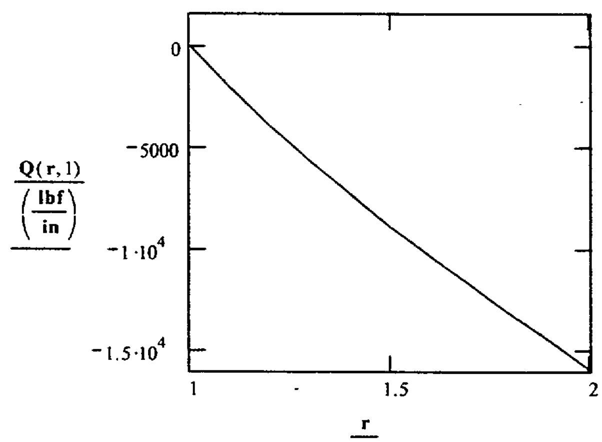
$$Mt_{max_i} := \left(A_{mt_i} > -B_{mt_i}\right) \cdot A_{mt_i} + \left(A_{mt_i} \le -B_{mt_i}\right) \cdot B_{mt_i}$$

$$\frac{Mt_{max}}{\frac{lbf \cdot in}{in}} = \begin{pmatrix} 2.251 \times 10^4 \\ 4.986 \times 10^3 \\ -1.037 \times 10^3 \\ -1.152 \times 10^3 \end{pmatrix}$$

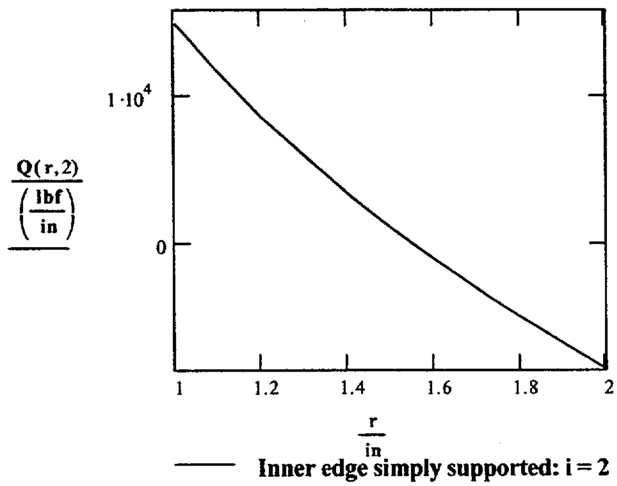
Shear $Q(r,i) := Q_{b_i} \cdot \frac{b}{r} - \frac{q}{2 \cdot r} \cdot \left(r^2 - r_o^2\right) \cdot \left(r > r_o\right)$

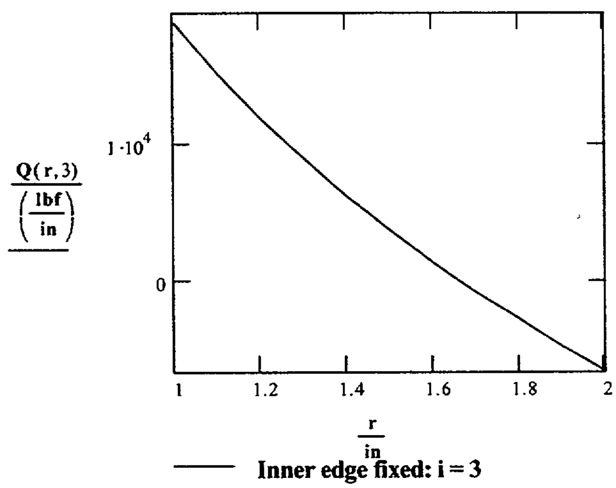


Inner edge free: $i = 0$



Inner edge guided: $i = 1$


Q(r,2)
lbf
in
1·10^4
0
1
1.2
1.4
1.6
1.8
2
r
in
Inner edge simply supported: i = 2


Q(r,3)
lbf
in
1·10^4
0
1
1.2
1.4
1.6
1.8
2
r
in
Inner edge fixed: i = 3

The following values are listed in order of inner edge:

- **free (i = 0)**
- **guided (i = 1)**
- **simply supported (i = 2)**
- **fixed (i = 3)**

Shear at points b and a (inner and outer radius):

$$\frac{Q_b}{\frac{lbf}{in}} = \begin{pmatrix} 0 \\ 0 \\ 1.49 \times 10^4 \\ 1.89 \times 10^4 \end{pmatrix} \qquad \frac{Q_a}{\frac{lbf}{in}} = \begin{pmatrix} -1.688 \times 10^4 \\ -1.688 \times 10^4 \\ -9.655 \times 10^3 \\ -7.718 \times 10^3 \end{pmatrix}$$

Maximum shear (magnitude):

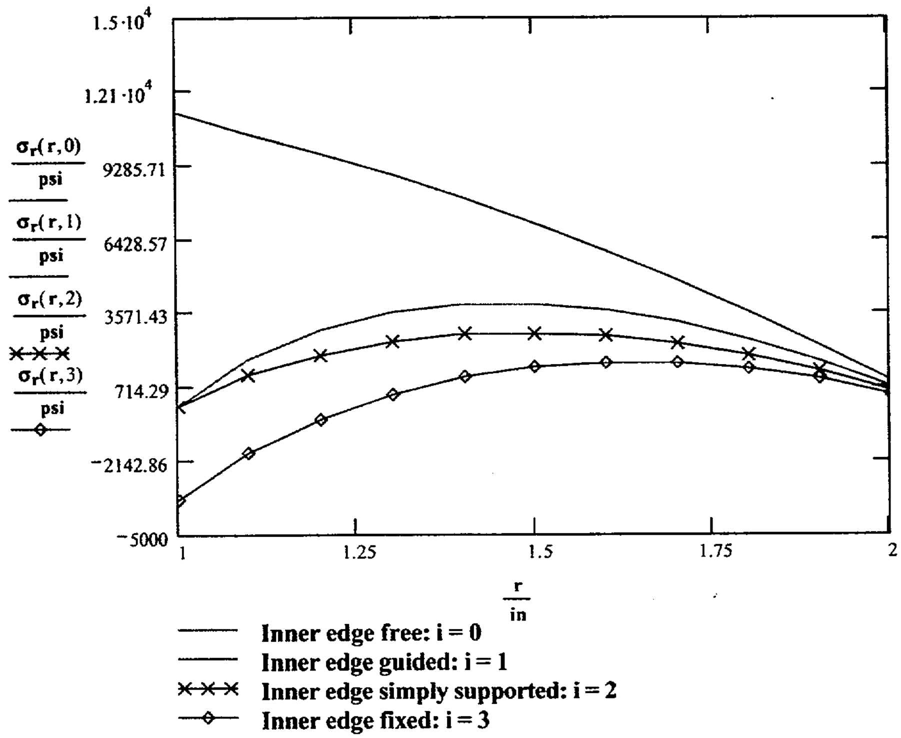
$$Q_{(r-b)\frac{100}{in},\, i} := Q(r, i) \qquad A_{Q_i} := \max\left(Q^{\langle i \rangle}\right) \qquad B_{Q_i} := \min\left(Q^{\langle i \rangle}\right)$$

$$Q_{max_i} := \left(A_{Q_i} > -B_{Q_i}\right) \cdot A_{Q_i} + \left(A_{Q_i} \le -B_{Q_i}\right) \cdot B_{Q_i}$$

$$\frac{Q_{max}}{\frac{lbf}{in}} = \begin{pmatrix} -1.605 \times 10^4 \\ -1.605 \times 10^4 \\ 1.49 \times 10^4 \\ 1.89 \times 10^4 \end{pmatrix}$$

Radial bending stress

$$\sigma_r(r,i) := \frac{6 \cdot M_r(r,i)}{t^2}$$



The following values are listed in order of inner edge:

- **free (i = 0)**
- **guided (i = 1)**
- **simply supported (i = 2)**
- **fixed (i = 3)**

Radial bending stress at points b and a (inner and outer radius):

$\frac{\sigma_r(b,i)}{psi} =$

0
$1.129\cdot10^{4}$
0
$-3.686\cdot10^{3}$

$\frac{\sigma_r(a,i)}{psi} =$

0
0
0
$-1.737\cdot10^{-12}$

Maximum radial bending stress (magnitude):

$$\sigma r_{(r-b)\frac{100}{in},\, i} := \sigma_r(r,i)$$

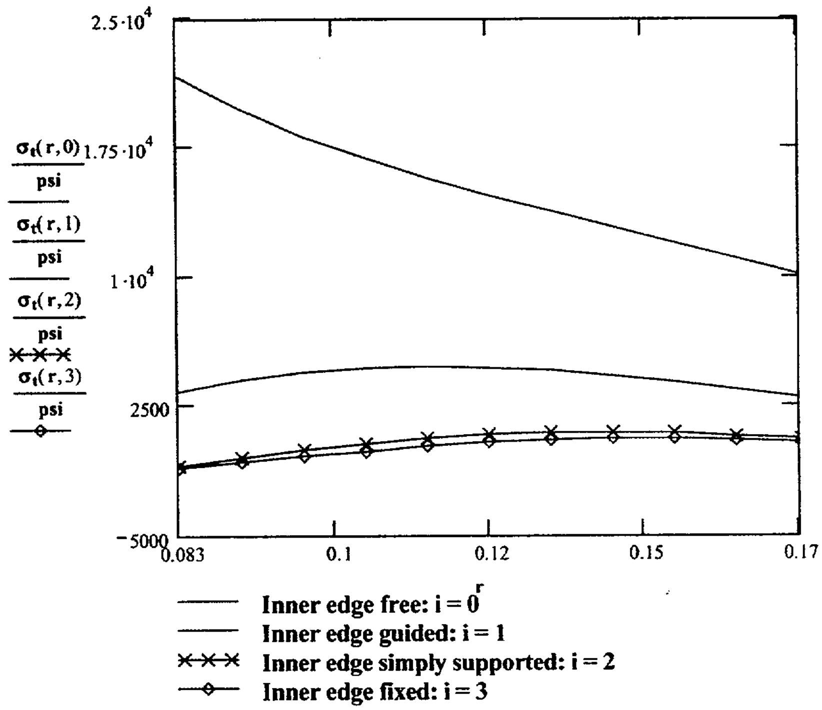
$$Ar_i := \max\left(\sigma r^{\langle i\rangle}\right) \qquad Br_i := \min\left(\sigma r^{\langle i\rangle}\right)$$

$$\sigma r_{max_i} := \left(Ar_i > -Br_i\right)\cdot Ar_i + \left(Ar_i \le -Br_i\right)\cdot Br_i$$

$$\frac{\sigma r_{max}}{psi} = \begin{pmatrix} 3.914\times10^{3} \\ 1.129\times10^{4} \\ 2.795\times10^{3} \\ -3.686\times10^{3} \end{pmatrix}$$

Transverse bending stress

$$\sigma_t(r, i) := \frac{6 \cdot M_t(r, i)}{t^2}$$



The following values are listed in order of inner edge:

- **free (i = 0)**
- **guided (i = 1)**
- **simply supported (i = 2)**
- **fixed (i = 3)**

Tangential bending stress at points b and a (inner and outer radius):

$\frac{\sigma_t(b,i)}{psi} =$	$\frac{\sigma_t(a,i)}{psi} =$
$2.161 \cdot 10^4$	$9.555 \cdot 10^3$
$3.387 \cdot 10^3$	$2.617 \cdot 10^3$
-995.107	410.053
$-1.106 \cdot 10^3$	223.549

$$SF_4 := \frac{.9 \cdot \sigma_y}{\sigma_t(b,0)}$$

$$SF_4 = 1.582$$

Maximum radial bending stress (magnitude):

$$\sigma t_{(r-b)\frac{100}{in},i} := \sigma_t(r,i) \qquad At_i := \max\left(\sigma t^{\langle i \rangle}\right) \qquad Bt_i := \min\left(\sigma t^{\langle i \rangle}\right)$$

$$\sigma t_{max_i} := \left(At_i > -Bt_i\right) \cdot At_i + \left(At_i \le -Bt_i\right) \cdot Bt_i$$

$$\frac{\sigma t_{max}}{psi} = \begin{pmatrix} 2.161 \times 10^4 \\ 4.786 \times 10^3 \\ -995.107 \\ -1.106 \times 10^3 \end{pmatrix}$$

Review the maximum values for deflection, slope, moment, shear and stress

$$\frac{y_{max}}{in} = \begin{pmatrix} -5.993 \times 10^{-4} \\ -1.072 \times 10^{-4} \\ -8.302 \times 10^{-6} \\ -3.771 \times 10^{-6} \end{pmatrix} \qquad \frac{\theta_{max}}{deg} = \begin{pmatrix} 0.034 \\ 8.567 \times 10^{-3} \\ -1.573 \times 10^{-3} \\ 7.158 \times 10^{-4} \end{pmatrix}$$

$$\frac{Mr_{max}}{\frac{lbf \cdot in}{in}} = \begin{pmatrix} 4.077 \times 10^{3} \\ 1.176 \times 10^{4} \\ 2.912 \times 10^{3} \\ -3.839 \times 10^{3} \end{pmatrix} \qquad \frac{Mt_{max}}{\frac{lbf \cdot in}{in}} = \begin{pmatrix} 2.251 \times 10^{4} \\ 4.986 \times 10^{3} \\ -1.037 \times 10^{3} \\ -1.152 \times 10^{3} \end{pmatrix}$$

$$\frac{\sigma r_{max}}{psi} = \begin{pmatrix} 3.914 \times 10^{3} \\ 1.129 \times 10^{4} \\ 2.795 \times 10^{3} \\ -3.686 \times 10^{3} \end{pmatrix} \qquad \frac{\sigma t_{max}}{psi} = \begin{pmatrix} 2.161 \times 10^{4} \\ 4.786 \times 10^{3} \\ -995.107 \\ -1.106 \times 10^{3} \end{pmatrix}$$

$$\frac{Q_{max}}{\frac{lbf}{in}} = \begin{pmatrix} -1.605 \times 10^{4} \\ -1.605 \times 10^{4} \\ 1.49 \times 10^{4} \\ 1.89 \times 10^{4} \end{pmatrix}$$

Total deflection of plate (bending induced plus shear induced):

At b (inner radius):

$$y_{b.total} := \overrightarrow{y_b + y_{sb}} \qquad \frac{y_{b.total}}{in} = \begin{pmatrix} -1.015 \times 10^{-3} \\ -5.23 \times 10^{-4} \\ 0 \\ 0 \end{pmatrix}$$

At r_o (loading point):

$$y_{ro_i} := y(r_o, i)$$

$$y_{ro.total} := \overrightarrow{y_{ro} + y_{sro}} \qquad \frac{y_{ro.total}}{in} = \begin{pmatrix} -6.97 \times 10^{-4} \\ -2.049 \times 10^{-4} \\ 0 \\ 0 \end{pmatrix}$$

The remainder of the document displays the general plate functions and constants used in the equations above.

$$C_1 \equiv \frac{1+\nu}{2}\cdot\frac{b}{a}\cdot\ln\left(\frac{a}{b}\right) + \frac{1-\nu}{4}\cdot\left(\frac{a}{b}-\frac{b}{a}\right)$$

$$C_7 \equiv \frac{1}{2}\cdot\left(1-\nu^2\right)\cdot\left(\frac{a}{b}-\frac{b}{a}\right)$$

$$C_2 \equiv \frac{1}{4}\cdot\left[1-\left(\frac{b}{a}\right)^2\cdot\left(1+2\cdot\ln\left(\frac{a}{b}\right)\right)\right]$$

$$C_8 \equiv \frac{1}{2}\cdot\left[1+\nu+(1-\nu)\cdot\left(\frac{b}{a}\right)^2\right]$$

$$C_3 \equiv \frac{b}{4\cdot a}\cdot\left[\left[\left(\frac{b}{a}\right)^2+1\right]\cdot\ln\left(\frac{a}{b}\right)+\left(\frac{b}{a}\right)^2-1\right]$$

$$C_9 \equiv \frac{b}{a}\cdot\left[\frac{1+\nu}{2}\cdot\ln\left(\frac{a}{b}\right)+\left(\frac{1-\nu}{4}\right)\cdot\left[1-\left(\frac{b}{a}\right)^2\right]\right]$$

$$C_4 \equiv \frac{1}{2}\cdot\left[(1+\nu)\cdot\frac{b}{a}+(1-\nu)\cdot\frac{a}{b}\right]$$

$$L_{11} \equiv \frac{1}{64}\cdot\left[1+4\cdot\left(\frac{r_o}{a}\right)^2-5\cdot\left(\frac{r_o}{a}\right)^4-4\cdot\left(\frac{r_o}{a}\right)^2\cdot\left[2+\left(\frac{r_o}{a}\right)^2\right]\cdot\ln\left(\frac{a}{r_o}\right)\right]$$

$$C_5 \equiv \frac{1}{2}\cdot\left[1-\left(\frac{b}{a}\right)^2\right]$$

$$L_{14} \equiv \frac{1}{16}\cdot\left[1-\left(\frac{r_o}{a}\right)^4-4\cdot\left(\frac{r_o}{a}\right)^2\cdot\ln\left(\frac{a}{r_o}\right)\right]$$

$$C_6 \equiv \frac{b}{4\cdot a}\cdot\left[\left(\frac{b}{a}\right)^2-1+2\cdot\ln\left(\frac{a}{b}\right)\right]$$

$$L_{17} \equiv \frac{1}{4}\cdot\left[1-\left(\frac{1-\nu}{4}\right)\cdot\left[1-\left(\frac{r_o}{a}\right)^4\right]-\left(\frac{r_o}{a}\right)^2\cdot\left[1+(1+\nu)\cdot\ln\left(\frac{a}{r_o}\right)\right]\right]$$

Boundary values due to bending:

At the inner edge of the plate:

$$Q_b \equiv \begin{bmatrix} 0\cdot\frac{\mathrm{lbf}}{\mathrm{in}} \\ 0\cdot\frac{\mathrm{lbf}}{\mathrm{in}} \\ q\cdot a\cdot\left(\frac{C_1\cdot L_{17} - C_7\cdot L_{11}}{C_1\cdot C_9 - C_3\cdot C_7}\right) \\ q\cdot a\cdot\left(\frac{C_2\cdot L_{17} - C_8\cdot L_{11}}{C_2\cdot C_9 - C_3\cdot C_8}\right) \end{bmatrix} \qquad M_{rb} \equiv \begin{bmatrix} 0\cdot\frac{\mathrm{lbf}\cdot\mathrm{in}}{\mathrm{in}} \\ \frac{q\cdot a^2}{C_8}\cdot L_{17} \\ 0\cdot\frac{\mathrm{lbf}\cdot\mathrm{in}}{\mathrm{in}} \\ -q\cdot a^2\cdot\left(\frac{C_3\cdot L_{17} - C_9\cdot L_{11}}{C_2\cdot C_9 - C_3\cdot C_8}\right) \end{bmatrix}$$

$$y_b \equiv \begin{bmatrix} \frac{-q\cdot a^4}{D}\cdot\left(\frac{C_1\cdot L_{17}}{C_7} - L_{11}\right) \\ \frac{-q\cdot a^4}{D}\cdot\left(\frac{C_2\cdot L_{17}}{C_8} - L_{11}\right) \\ 0\cdot\mathrm{in} \\ 0\cdot\mathrm{in} \end{bmatrix} \qquad \theta_b \equiv \begin{bmatrix} \frac{q\cdot a^3}{D\cdot C_7}\cdot L_{17} \\ 0\cdot\mathrm{deg} \\ \frac{-q\cdot a^3}{D}\cdot\left(\frac{C_3\cdot L_{17} - C_9\cdot L_{11}}{C_1\cdot C_9 - C_3\cdot C_7}\right) \\ 0\cdot\mathrm{deg} \end{bmatrix}$$

At the outer edge of the plate:

$$y_a \equiv \begin{pmatrix} 0\cdot\mathrm{in} \\ 0\cdot\mathrm{in} \\ 0\cdot\mathrm{in} \\ 0\cdot\mathrm{in} \end{pmatrix} \qquad Q_a \equiv \begin{bmatrix} \frac{-q}{2\cdot a}\cdot\left(a^2 - r_o^2\right) \\ \frac{-q}{2\cdot a}\cdot\left(a^2 - r_o^2\right) \\ Q_{b_2}\cdot\frac{b}{a} - \frac{q}{2\cdot a}\cdot\left(a^2 - r_o^2\right) \\ Q_{b_3}\cdot\frac{b}{a} - \frac{q}{2\cdot a}\cdot\left(a^2 - r_o^2\right) \end{bmatrix}$$

$$\theta_a \equiv \begin{bmatrix} \frac{q \cdot a^3}{D} \cdot \left(\frac{C_4 \cdot L_{17}}{C_7} - L_{14} \right) \\ \frac{q \cdot a^3}{D} \cdot \left(\frac{C_5 \cdot L_{17}}{C_8} - L_{14} \right) \\ \theta_{b_2} \cdot C_4 + Q_{b_2} \cdot \frac{a^2}{D} \cdot C_6 - \frac{q \cdot a^3}{D} \cdot L_{14} \\ M_{rb_3} \cdot \frac{a}{D} \cdot C_5 + Q_{b_3} \cdot \frac{a^2}{D} \cdot C_6 - \frac{q \cdot a^3}{D} \cdot L_{14} \end{bmatrix} \qquad M_{ra} \equiv \begin{pmatrix} 0 \cdot \frac{lbf \cdot in}{in} \\ 0 \cdot \frac{lbf \cdot in}{in} \\ 0 \cdot \frac{lbf \cdot in}{in} \\ 0 \cdot \frac{lbf \cdot in}{in} \end{pmatrix}$$

Due to tangential shear stresses:

$$y_{sb} \equiv \begin{pmatrix} \frac{K_{sb} \cdot q \cdot a^2}{t \cdot G} \\ \frac{K_{sb} \cdot q \cdot a^2}{t \cdot G} \\ 0 \cdot in \\ 0 \cdot in \end{pmatrix} \qquad y_{sro} \equiv \begin{pmatrix} \frac{K_{sro} \cdot q \cdot r_o^2}{t \cdot G} \\ \frac{K_{sro} \cdot q \cdot r_o^2}{t \cdot G} \\ 0 \cdot in \\ 0 \cdot in \end{pmatrix}$$

$$F_1(r) \equiv \frac{1+\nu}{2} \cdot \frac{b}{r} \cdot \ln\left(\frac{r}{b}\right) + \frac{1-\nu}{4} \cdot \left(\frac{r}{b} - \frac{b}{r}\right) \qquad F_6(r) \equiv \frac{b}{4 \cdot r} \cdot \left[\left(\frac{b}{r}\right)^2 - 1 + 2 \cdot \ln\left(\frac{r}{b}\right)\right]$$

$$F_2(r) \equiv \frac{1}{4} \cdot \left[1 - \left(\frac{b}{r}\right)^2 \cdot \left(1 + 2 \cdot \ln\left(\frac{r}{b}\right)\right)\right] \qquad F_7(r) \equiv \frac{1}{2} \cdot \left(1 - \nu^2\right) \cdot \left(\frac{r}{b} - \frac{b}{r}\right)$$

$$F_3(r) \equiv \frac{b}{4 \cdot r} \cdot \left[\left[\left(\frac{b}{r}\right)^2 + 1\right] \cdot \ln\left(\frac{r}{b}\right) + \left(\frac{b}{r}\right)^2 - 1\right] \qquad F_8(r) \equiv \frac{1}{2} \cdot \left[1 + \nu + (1 - \nu) \cdot \left(\frac{b}{r}\right)^2\right]$$

$$F_4(r) \equiv \frac{1}{2} \cdot \left[(1 + \nu) \cdot \frac{b}{r} + (1 - \nu) \cdot \frac{r}{b}\right]$$

$$F_9(r) \equiv \frac{b}{r} \cdot \left[\frac{1+\nu}{2} \cdot \ln\left(\frac{r}{b}\right) + \frac{1-\nu}{4} \cdot \left[1 - \left(\frac{b}{r}\right)^2\right]\right]$$

$$F_5(r) \equiv \frac{1}{2} \cdot \left[1 - \left(\frac{b}{r}\right)^2\right]$$

$$G_{11}(r) \equiv \frac{1}{64} \cdot \left[1 + 4 \cdot \left(\frac{r_o}{r}\right)^2 - 5 \cdot \left(\frac{r_o}{r}\right)^4 - 4 \cdot \left(\frac{r_o}{r}\right)^2 \cdot \left[2 + \left(\frac{r_o}{r}\right)^2\right] \cdot \ln\left(\frac{r}{r_o}\right)\right] \cdot (r > r_o)$$

$$G_{14}(r) \equiv \frac{1}{16}\cdot\left[1-\left(\frac{r_o}{r}\right)^4-4\cdot\left(\frac{r_o}{r}\right)^2\cdot\ln\left(\frac{r}{r_o}\right)\right]\cdot(r>r_o)$$

$$G_{17}(r) \equiv \frac{1}{4}\cdot\left[1-\left(\frac{1-\nu}{4}\right)\cdot\left[1-\left(\frac{r_o}{r}\right)^4\right]-\left(\frac{r_o}{r}\right)^2\cdot\left[1+(1+\nu)\cdot\ln\left(\frac{r}{r_o}\right)\right]\right]\cdot(r>r_o)$$

The final calculation is an evaluation of the local bending strength of the receptacle. We have shown previously that the depth of the top of the embedment

$$t = 2.5\,\mathrm{in}$$

is governed by the requirement that the hoop stress remain below the limits of the ACI 349 Code for the steel. The safety factor has been computed as:

$$SF_4 = 1.582$$

The thickness of the cylindrical barrel of the anchor stud receptacle has been initially chosen to ensure that the axial load capacity, based on safety factor, is the same as the anchor stud, even though the anchor stud stress limits come from the ASME Code, Section III, Appendix F. The ACI Code did not envision any evaluation of secondary stress limits arising from structural discontinuities. Consistent with our assumption in the annular plate calculation above, since the annular plate comprising the receptacle load transfer interface has been defined to extend to the barrel centerline and assumed to be a pinned joint, we need not evaluate any local bending stress.

4.3 Conclusions

A typical embedment design has been evaluated in this calculation. To the extent practical, ACI 349-97 Appendix B criteria have been employed. The same(or greater) safety factors as the anchor stud have been assured by the geometry established. The additional reinforcing to mitigate tensile cracking of the concrete cone is not evaluated herein; the commentary in ACI-349R-97, concerning Appendix B, should be followed when placing the additional reinforcement.

With the geometry chosen, the computed embedment safety factors, based on anchor stud design limits from ASME Section III, Subsection NF and Appendix F, are

$SF_1 = 1.514$ Concrete tensile cracking (design)

$SF_2 = 1.477$ Embedment interface bearing (preload)

$SF_3 = 1.013$ Embedment interface bearing (Stud Capacity)

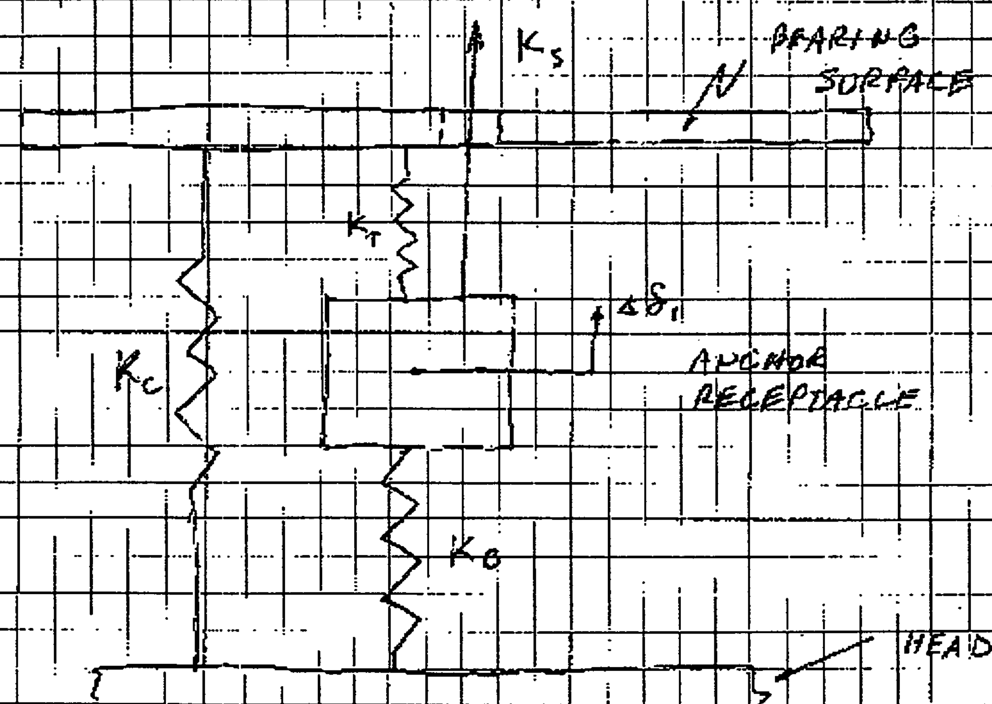
$SF_4 = 1.582$ Circumferential stress in bearing ring of embedment

The area of the receptacle cylinder has been chosen to provide a safety factor of exactly 1.0 when the anchor stud has reached its Level D limit per the ASME Code.

ANCHORED CASK PRELOAD - GENERAL ANALYSIS

SCOPE: Develop general formulation for anchored cask preload

CONFIGURATION



DEFINITIONS

K_S = spring constant of anchor stud

K_T = spring rate of upper receptacle

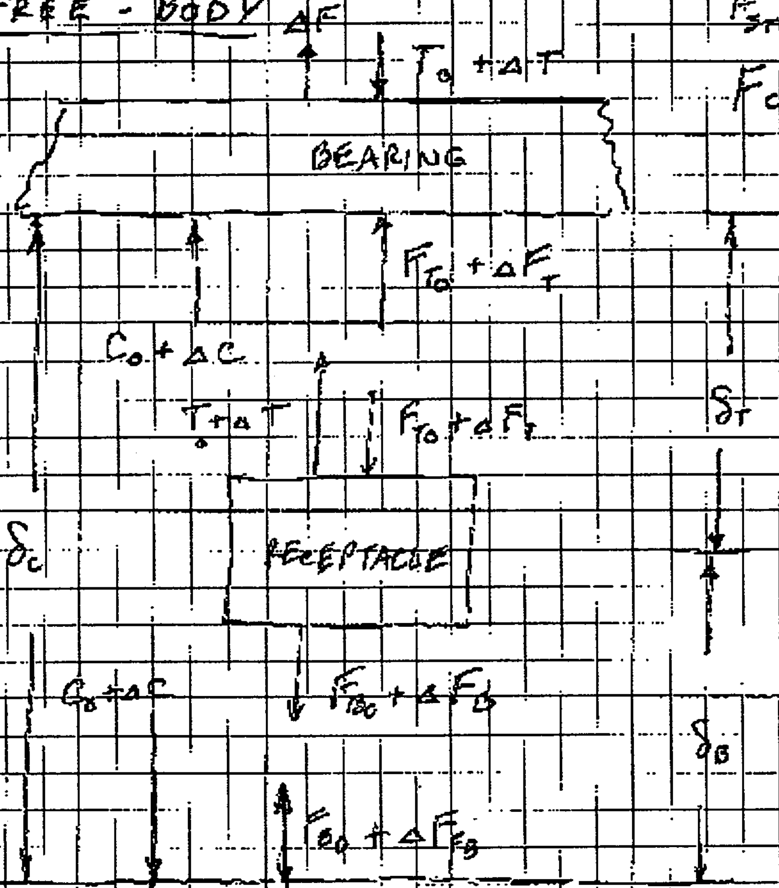
K_B = spring rate of lower receptacle

K_C = spring rate of concrete core

F_T, F_B, F_C, F_{STUD} = compression in upper receptacle; tension in lower receptacle; compression in concrete ; tension in stud

Note a "zero" subscript indicates preload

FREE - BODY



$F_{STUD} = T$

$F_C = C$

ΔF = APPLIED LOAD

1. PRELOAD CONDITION - APPLY T_o, $\Delta F = 0$

$$T_o = F_{To} + F_{Bo} \quad (1)$$

$$0 = C_o - F_{Bo} \quad (2)$$

$$\Delta\delta_c = \Delta\delta_T + \Delta\delta_B \quad (3)$$

From eq. 3, using spring constants

$$-\frac{C_o}{K_c} = -\frac{F_{TO}}{K_T} + \frac{F_{BO}}{K_B} \qquad (4)$$

Therefore, using eq (2) to eliminate C_o

$$\frac{F_{TO}}{K_T} = F_{BO}\left(\frac{1}{K_B} + \frac{1}{K_C}\right)$$

or

$$F_{BO} = \frac{K_B K_C}{K_B + K_C}\,\frac{F_{TO}}{K_T} \qquad (5)$$

Define

$$K_{BE} = \frac{K_B K_C}{K_B + K_C} \qquad (6)$$

so that

$$F_{BO} = \frac{K_{BE}}{K_T}\,F_{TO} \qquad (7)$$

Therefore, using (7) in (1) to solve for F_{TO} and then using (7), gives

$$F_{TO} = \frac{K_T}{K_T + K_{BE}}\,T_o \qquad (8)$$

$$F_{BO} = C_o = \frac{K_{BE}}{K_T + K_{BE}}\,T_o \qquad (9)$$

2. LOAD CONDITION $\Delta F \neq 0$

$$\Delta F - T_0 - \Delta T + C_0 + \Delta C + F_{T0} + \Delta F_T = 0$$

or

$$\Delta F - \Delta T + \Delta C + \Delta F_T = 0 \qquad (10)$$

(Free body of Bearing)

Similarly, a free body analysis of the receptacle gives

$$\Delta F_T + \Delta F_B - \Delta T = 0 \qquad (11)$$

The stretch of the stud and the compression of the upper receptacle are the same. Therefore,

$$\frac{\Delta T}{K_S} = -\frac{\Delta F_T}{K_T} \qquad (12)$$

Similarly, we can relate the compression of the concrete, the change in length of the lower receptacle and the upper receptacle. Therefore

$$-\frac{\Delta C}{K_C} = -\frac{\Delta F_T}{K_T} + \frac{\Delta F_B}{K_B} \qquad (13)$$

The four equation (10)-(13) enable a solution for ΔT, ΔC, ΔF_T, ΔF_B in terms of ΔF.

From (12)

$$\Delta T = -\frac{K_S}{K_T}\Delta F_T \qquad (14)$$

Then (11) gives

$$\Delta F_T + \Delta F_B + \frac{K_S}{K_T}\Delta F_T = 0 \text{ , or}$$

$$\Delta F_T = -\frac{K_T}{K_T + K_S}\Delta F_B \qquad (15)$$

Next, substituting (15) into (13) gives

$$-\frac{\Delta C}{K_C} = \frac{\Delta F_B}{K_T + K_S} + \frac{\Delta F_B}{K_B} = \frac{K_B + K_T + K_S}{K_B(K_T + K_S)}\Delta F_B$$

Finally, substituting into (10) gives

$$\Delta F + \frac{K_S}{K_T}\left(-\frac{K_T\,\Delta F_B}{K_T + K_S}\right) - \frac{K_C\,(K_B + K_T + K_S)}{K_B\,(K_T + K_S)}\Delta F_B - \frac{K_T}{K_T + K_S}\Delta F_B = 0$$

or

$$\frac{\Delta F}{\Delta F_B} = \left[\frac{K_T}{K_T + K_S} + \frac{K_S}{K_T + K_S} + \frac{K_C}{K_B}\frac{(K_B + K_T + K_S)}{K_T + K_S}\right]$$

$$= \frac{K_T K_B + K_S K_B + K_C K_B + K_C K_T + K_C K_S}{K_B(K_T + K_S)}$$

Define

$$\alpha = \frac{K_B K_T + K_B K_S}{K_T K_B + K_S K_B + K_C K_B + K_C K_T + K_C K_S}$$

$$\Delta F_B = \alpha\,\Delta F$$

SUMMARY

$$\Delta F_B = \alpha \Delta F \qquad \alpha = \frac{K_B(K_T + K_S)}{K_B(K_T + K_S + K_C) + K_C(K_T + K_S)} \tag{16}$$

$$\Delta F_T = -\frac{K_T}{K_T + K_B}\Delta F_B = -\beta \Delta F$$

$$\beta = \frac{K_B K_T}{K_B(K_T + K_S + K_C) + K_C(K_T + K_S)} \tag{17}$$

$$\Delta T = -\frac{K_S}{K_T}\Delta F_T = +\frac{K_S}{K_T}\beta \Delta F \tag{18}$$

Finally,

$$\Delta C = -\frac{K_C(K_B + K_T + K_S)}{K_B(K_T + K_S)}\Delta F_B = -\gamma \Delta F \tag{19}$$

$$\gamma = \frac{K_C(K_B + K_T + K_S)}{K_C(K_T + K_S + K_B) + K_B(K_T + K_S)}$$

SUPPLEMENT 3
CALCULATION 6

A REVISION OF APPENDIX 3.D IN FSAR THAT SPECIFICALLY ADDRESSES THE HI-STORM 100S VERTICAL HANDLING

FOR CLARITY IN PRESENTATION, APPENDIX 3.D HAS BEEN MODIFIED AND GIVEN THE NOTATION APPENDIX 3.DS IN THIS CALCULATION PACKAGE SUPPLEMENT

APPENDIX 3.DS: VERTICAL HANDLING OF HI-STORM 100S OVERPACK WITH HEAVIEST MPC and BOUNDING CONCRETE WEGHT

3.DS.1 Introduction

There are two vertical lifting scenarios for the HI-STORM 100 during the normal operation procedures at the ISFSI pad. The first scenario considers the vertical lifting of a fully loaded HI-STORM 100S with four synchronized hydraulic jacks, each positioned at each of the four inlet vents located at the bottom end. This operation allows the installation of air pads under the HI-STORM 100S baseplate. The second scenario considers the lifting of a fully loaded HI-STORM 100S vertically through the four lifting lugs located at the top end. The lifting device assemblage is constructed such that the lift forces at each lug are parallel to the longitudinal axis of the HI-STORM 100S during the operation. The state of stress induced on the cask components as a result of these operations is determined, analyzed, and the structural integrity evaluated. In the FSAR, HI-2002444, a finite element analysis was used to effect a qualification of the HI-STORM 100 overpack for lifting. The assertion was made in LAR 1014-1 that the results were bounding for the HI-STORM 100S by virtue of the lower weight and the fact that there was no alteration of the structural components. In this additional calculation supporting ECO 1024-20, Rev. 0, there are some structural changes necessitating a revision to the calculation. In particular, the load path during a lift from either above or below has been made more direct by revising the location of the anchor block and radial ribs to be in a direct line with the inlet ducts. Therefore, a structural integrity evaluation is easily performed using classical strength of materials. Also, to encompass a bounding weight scenario, the weight density of the concrete in the overpack and the lid is raised to 200 lb/cu.ft. Thus, the bounding weight for the various lifting computations is increased. In what follows, calculations are provided only for the components that are directly affected by the change in structure and/or the increase in concrete weight.

3.DS.2 Assumptions

a. Conservatively, the analysis takes credit only for the structural rigidity of the outer and inner shells and for the radial ribs of the HI-STORM 100S. No credit for the structural rigidity of the MPC pedestal shield is assumed. Hence, the weight of the radial concrete shielding, the MPC pedestal shield, and the MPC are respectively applied as load during the vertical lifting of HI-STORM 100S. Property values used are approximately equal to the final values set in the Tables in Chapter 3. Drawings 3443 (9 sheets) provide the reference for all dimensions.

b. The acceleration of gravity of 1.15g is considered in order to account for a 15% dynamic load factor due to lifting. The 15% increase, according to Reference 2, is considered in crane standards as appropriate for low speed lifting operations.

c. The bounding concrete density (200 lb./cu.ft.) is assumed to exist in the lid and in the body of the overpack. The pedestal shield has normal weight concrete. Therefore, the analyses for the structural behavior of the HI-STORM 100 overpack baseplate as a plate-like structure is unaffected by the heavier concrete and applies to the HI-STORM 100S.

3.DS.3 Analysis Methodology - Bottom Lift at the Inlet Vents

One-quarter of the load is applied to the top of the inlet vent and is directed to the radial plate and to the sided of the inlet duct. Part of the load is eventually distributed to the shells connected to the radial ribs by shear flow action. The analysis of the load path consists of the upper plate supported by the radial rib, the walls of the duct, and the two shells.

3.DS.4 Analysis Methodology - Top End Lift

3.DS.4.1 Model at Top near Lift Points

The anchor block and radial ribs are revisited using the same model as employed in Appendix 3.D of the FSAR

3.DS.4.2 Model at Bottom near baseplate

The baseplate need not be re-evaluated herein because none of the changes contemplated in the ECO affect the baseplate and its performance as a plate supporting a lateral load.

3.DS.5 Stress Evaluation

For all analyses, safety evaluation is based on the consideration of all components as Class 3 plate and shell support structures per the ASME Code Section III, Subsection NF.

The total lifted weight is conservatively assumed as W=315,000 lb. + 90,000 lb = 405,000 lb with the additional weight ascribed to the heavy concrete assumed in this analysis

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3.DS.6 Bolt and Anchor Block Thread Stress Analysis under Three Times Lifted Load

In this section, the threads of the bolt and the bolt anchor block are analyzed under three times the lifted load. The thread system is modeled as a cylindrical area of material under an axial load. The diameter of the cylinder area is the basic pitch diameter of the threads, and the length of the cylinder is the length of engagement of the threads. See Holtec HI-STORM 100 drawing numbers 3443 (sheet 8).

3.DS.6.1 Geometry

The basic pitch diameter of the threads is: $d_p = 3.0876"$ (Machinery's Handbook, 23th Edition, Table 3a, p.1484.)

The thread engagement length is: L = 6.5 in. (ECO item #6 and dwg. 3443, sheet 4)

The shear area of the cylinder that represents the threads: $A = 3.14159xL \times d_p$

The shear stress on this cylinder under three times the load is: 3W x 1.15/nA = 5,540 psi

where the total weight, *W*, and the number of lift points, *n*, are 405,000 pounds and 4, respectively, and the 1.15 represents the inertia amplification.

3.DS.6.2 Stress Evaluation

The yield strength of the anchor block material, SA-350, LF2, (ECO item #71) at 350 degrees F is taken as 31,400 psi per Table Y-1 of ASME Code, Section II, Part D. Note that the concrete temperature does not exceed 200 deg. F during this normal operation. Therefore, the anchor block and the radial ribs, which are buried in the concrete, will not have a higher temperature. Therefore, use of 350 deg. F for establishing strength limits is an additional conservatism. Assuming the yield strength in shear to be 60% of the yield strength in tension gives the thread shear stress safety factor under three times the lifted load as:

SF(thread shear - 3 x lifted load) = .6 x 31,400/5,540 = 3.40

The lifting stud material is SA193-B7 (ECO item #16). The yield strength of the stud material at 350 degrees F is 83,700 psi per Table Y-1 of the ASME Code, Section II, Part D.

The load per lift stud is P = 3W/4 x 1.15 = 349,313 lb.

The stud tensile stress area is (see Machinery's Handbook, 23rd Edition, p. 1484)

A = 7.10 sq. inch.

Therefore, the tensile stress in the stud under three times the lifted load is

Stress = P/A = 49,199 psi

The factor of safety on tensile stress in the lifting stud, based on three times the lifted load, is:

SF(stud tension – 3 x lifted load) = 83,700/49,199 = 1.70

Note: If we apply ANSI N14.6 lifting criteria to the stud and to the thread, it is clear that the thread meets limits of 6 on yield (this governs over a factor of 10 on ultimate strength) but that the stud tension does not meet such limits. From this we confirm that any lifting stud must have minimum yield strength at temperature in excess of 100 ksi. For any other condition, the limits of SA193-B7 will be used since that is the stud material when the system is not being lifted from above.

3.DS.7 Weld Evaluation

In this section, weld stress evaluations are performed for the weldments considered to be in the primary load path during lifting operations. The allowable stress for the welds is obtained from Reference [3].

3.DS.7.1 Anchor Block-to-Radial Rib (Lift from Top)

There are double sided fillet welds that attach the anchor block to the radial ribs (see drawing 3443, sheet 4 and ECO item #48). The following dimensions are used for analysis:

Total Length of weld = L = 24" + 6" (Continuous weld along sides and bottom - see drawing 3443 sheet 4)

Weld leg size = t = 0.5" Weld Leg Area = 2 x 30 x 0.5 = 30 sq. in. (double fillet)

Weld throat allowable shear stress = S_a = 0.3Su where S_u is the ultimate strength of the base metal (per [3]) = .3 x 70,000 psi (Table U-1 of ASME Sec. II, Part D gives the ultimate strength of the anchor block base material SA350-LF2, which is the same as the radial rib material SA516-70).

S_a = 21,000 psi

The following calculations provide a safety factor for the weld in accordance with the requirements of the ASME Code, Section III, Subsection NF for Class 3 plate and shell supports:

Allowable load per anchor block (2 welds) = S_a x 2 x 0.7071 x t x L = 445,473 lb.

Calculated Load (including 15% inertia amplification) = 405,000 lb x 1.15/4 = 116,438 lb.

SF(ASME Code) = 445,473 lb./116,438 lb. = 3.83

The following calculations provide a safety factor for the weld in accordance with the requirements of Regulatory Guide 3.61:

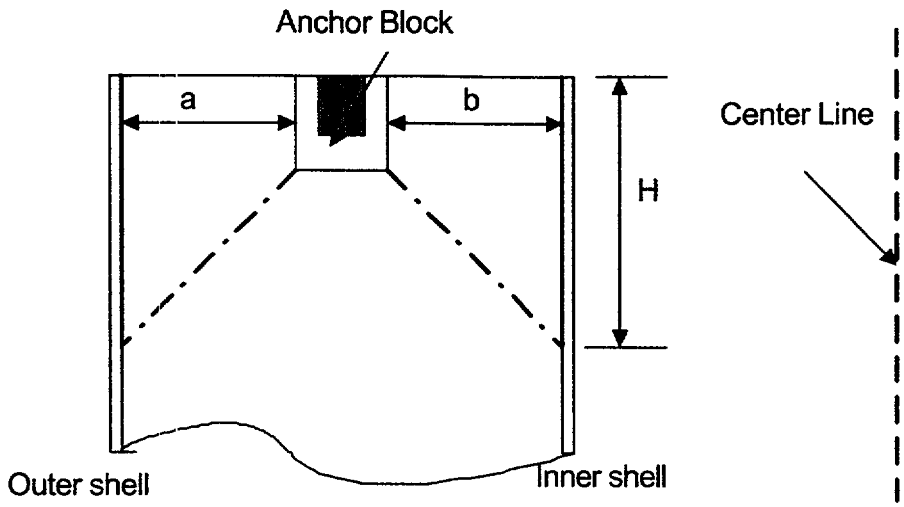
Allowable load per anchor block (2 welds) = 0.6 x 31,400 x 2 x.7071 x t x L = 399,653 lb.

Calculated Load (3 x weight) = 405,000 lb x 3/4 x 1.15 = 349,313 lb.

SF(Reg. Guide 3.61) = 399,653 lb./349,313 lb. = 1.144

3.DS.7.2 Radial Rib-to-Inner and Outer Shell (Lift from Top)

The load transferred to the radial ribs from the bolt anchor blocks is dispersed through the rib and also transferred to the inner and outer shell of the storage overpacks. A conservative estimate of the safety factors inherent in the vertical welds connecting the radial ribs to the inner and outer shells is obtained by assuming that the entire load is dispersed into the shells. The length of weld assumed to act in the load transfer is based on a dispersion angle of 45 degrees as shown in the sketch below:



From the geometry of the structure,

b = 11.0" Drawing 3443, sheet 4
a = 11.3125"

The depth of the effective weld to each shell is conservatively computed as the depth of the anchor block plus "b", or

H = 12" + 11" = 23"

The weld leg area available for load transfer is (double fillet to each of two shells):

Weld Area = 2 x (2 x 23" x 0.375") = 34.5 sq. in. (ECO item #47)

Since the effective area of the totality of weld assumed effective to transfer the load to the shells exceeds the weld area already shown to be acceptable at the anchor block-to-radial rib connection, we conclude that the anchor block-to-radial rib weld safety factors conservatively bound from below the safety factors for the radial rib to inner and outer shell welds in this load application.

3.DS.7.3 Baseplate-to-Inner Shell (Top Lift (bounds bottom lift))

The weld between the storage overpack baseplate and the storage overpack inner shell is an all-around double fillet weld (except at the duct locations (see drawing 1495, sheet 2)). To bound both the top and bottom lift, it is conservatively assumed that this weld supports a lifted load consisting of the weights of the loaded MPC, the pedestal shield concrete and steel, and the MPC baseplate (i.e., the structural action of the weld to the outer shell is conservatively neglected).

Therefore, the weld is subject to the following total load

116,067 lb. (MPC and pedestal shield) + 7967 lb. (baseplate) (from calculation package weight tables)

so that the applied load in the weld is conservatively assumed as:

Load = 124,034 lb

The weld is a fillet weld with total weld leg size "t" at mean diameter D =76" - 1.25", or

t = 0.75" (a 3/8" double fillet weld assumed at the mean diameter)
D = 74.75"

From Dwg. 3443, sheet 5 for the HI-STORM 100S storage overpack, the width of each inlet vent is

w = 16.5"

Therefore, the total linear length (around the periphery) of fillet weld available to transfer the load is

L = 3.14159 x D - 4 x w = 168.834"

Therefore, the weld throat area is

Area = 0.7071 x t x L = 89.54 sq. inches

The capacity of the weld per the ASME Code Section III Subsection NF is defined as Lc1

Lc1 = 21,000 psi x Area = 1,880,340 lb.

The capacity of the weld per Regulatory Guide 3.61 is defined as Lc2

Lc2 = .6 x 33,150 psi x Area = 1,780,951 lb.

Since 3 x lifted load bounds 1.15 x lifted load, it is clear that the Regulatory Guide 3.61 criteria produce the minimum safety factor. The calculated safety factor at this location is

SF = Lc2/(Load x 1.15) = 12.49

3.DS.7.4 Inlet Vent-to Baseplate Weld and Horizontal Plate of Inlet Vent(Bottom Lift)

Calculation #2 in Revision 2 of HI-2002481 contains this evaluation.

3.D.8 Stress Analysis of the Pedestal Shield

The pedestal shield concrete serves to support the loaded MPC and the pedestal platform during normal storage. The pedestal shield concrete is confined by the surrounding pedestal shell that serves, during the lifting operation, to resist radial expansion of the concrete cylinder due to the Poisson Ratio effect under the predominate axial compression of the concrete pedestal shield.

The compressive load capacity of the concrete making up the pedestal shield is the compression area x allowable compressive stress. From Table 3.3.5, the allowable compressive stress in the concrete is (the pedestal concrete is assumed to be normal weight concrete):

σ_c = 1535 psi

The concrete cylinder diameter (see ECO 1024-20, item 41, and item 14 in BOM, dwg 3443, sheet 2) is

D_c = 67.75"

Therefore, the load capacity per the ACI 318.1 concrete code (Reference [3.3.2] in Section 3.8 of this FSAR), defined as Lc4, is

Lc4 = σ_c x compression area of concrete cylinder = 1535 psi x 3605 sq. inch = 5,533,716 lb.

The applied load is conservatively assumed as the summed weight of the loaded MPC plus the pedestal platform plus the pedestal concrete shield.

W = 90,000 lb. (Table 3.2.1) + 5611 lb. (dwg. 3443, sheet 2 BOM) + 3600 lb. (24 cu ft x 150 #/cu.ft) = 99,211 lb.

Conservatively applying the Regulatory Guide 3.61 criteria to the concrete (interpret the allowable compressive stress as the "yield stress" for this evaluation) gives a safety factor

SF = Lc4/3W*1.15 = 16.167 (Note that the 1.15 accounts for inertia effects during the lift)

The pedestal shell is assumed to fully confine the concrete. Therefore, during compression of the concrete, a maximum lateral (radially oriented) pressure is applied to the pedestal shell due to the Poisson Ratio effect. This pressure varies linearly with concrete depth. Assuming the Poisson's Ratio of the concrete to be $\nu = 0.2$, the maximum pressure on the pedestal shell is

$p_{confine} = \nu/(1-\nu)$ x (3W x 1.15/compression area of concrete cylinder) = 0.25 x 94.95 psi
= 23.74 psi

Conservatively neglecting variations with depth of concrete, the hoop stress in the confining pedestal shell is obtained as follows:

t = pedestal shell thickness = 0.25"
R = pedestal shell mean radius = (0.5 x 68.375" - .5 x 0.25") = 34.0625"

Hoop Stress = $p_{confine}$ x R/t = 3,235 psi

This gives a safety factor based on the Regulatory Guide 3.61 criteria equal to

SF = 33,150 psi/Hoop Stress = 10.24

3.D.9 Conclusion

The design of the HI-STORM 100S is adequate for the bottom end lift through the inlet vents. The design of the HI-STORM 100S is also adequate for the top end lift through the lifting lugs. Safety factors are established based on requirements of the ASME Code Section III, Subsection NF for Class 3 plate and shell supports and also on the requirements of USNRC Regulatory Guide 3.61.

3.D.10 References

1. Not Used

2. Crane Manufacturer's Association of America (CMAA), Specification #70, 1988, Section 3.3.

3. ASME Code Section III, Subsection NF-3324.5, Table NF-3324.5(a)-1, 1995

SUPPLEMENT 3 - CALCULATION 7

This "calculation" contains figures and appendices that have been relocated to the Calculation Package per agreement with the NRC staff during review of Proposed Rev. 1 of the HI-STORM 100 FSAR. Also included are detailed stress outputs for the MPC24E,

Included in this calculation are the following:

Figures from Appendix 3.A that were removed from that appendix in the FSAR

Appendices 3.N-3.S (information on finite element modeling previously included in FSAR Rev. 0)

Appendix 3.T (Finite element analysis results previously included in FSAR for MPC 24,68, and 32(added in Prop. Rev.1))

Finite Element Results Supporting MPC-24E Structural Integrity Analysis

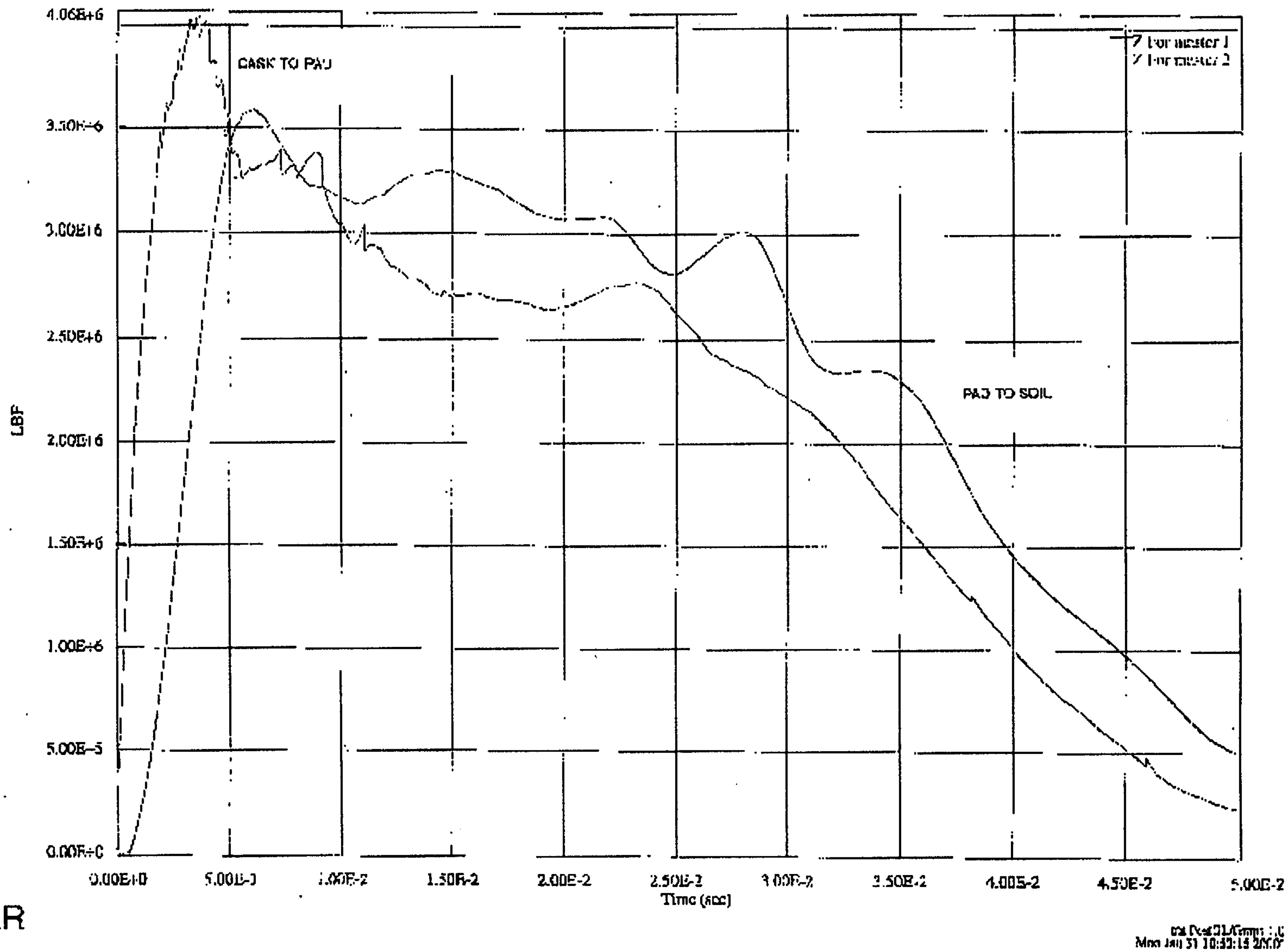


FIG. 3.A.19 Tipover Scenario: Impact Force Time Histories

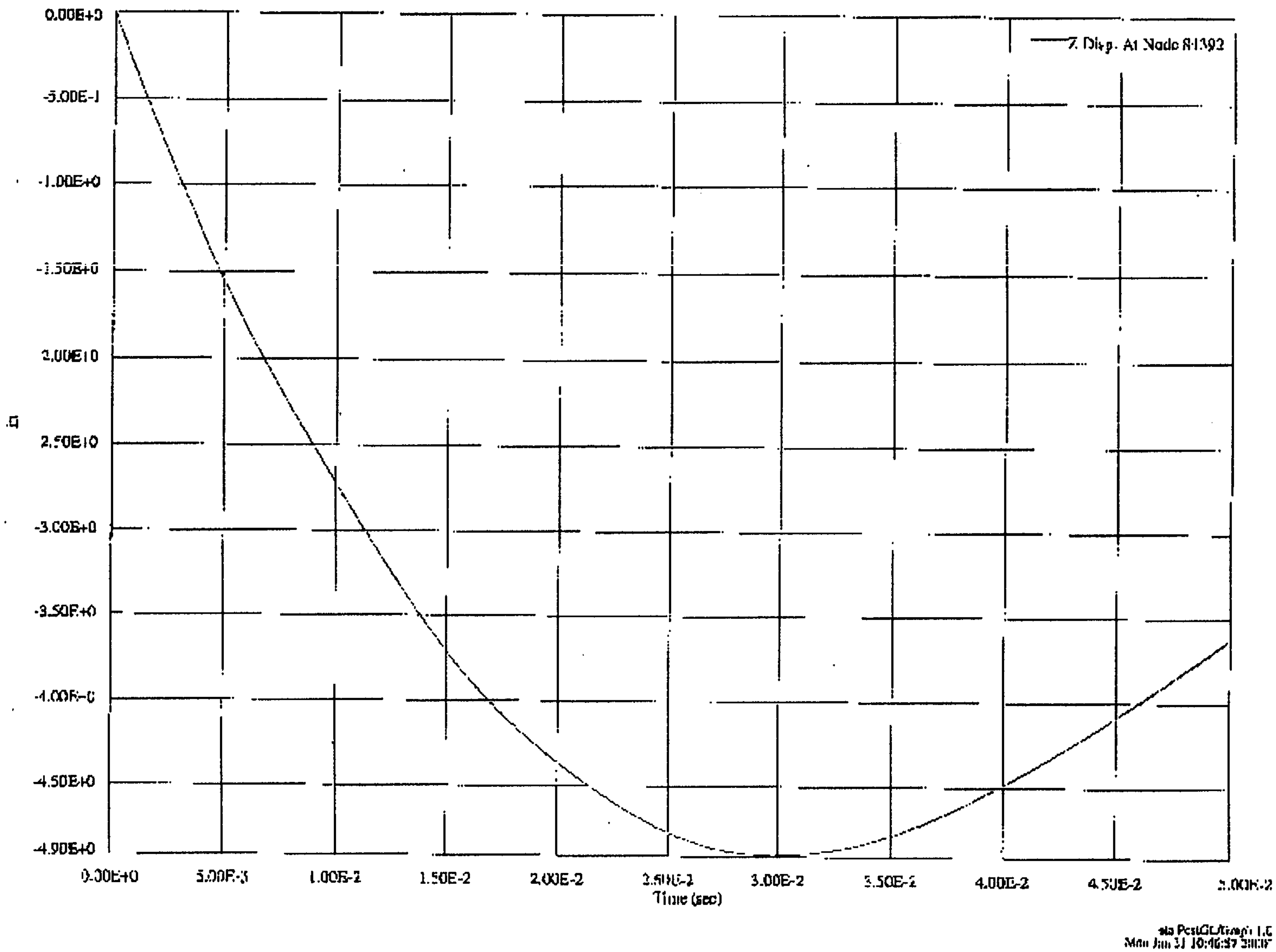


FIG. 3.A.20 Tipover Scenario: Channel A2 Displacement Time History

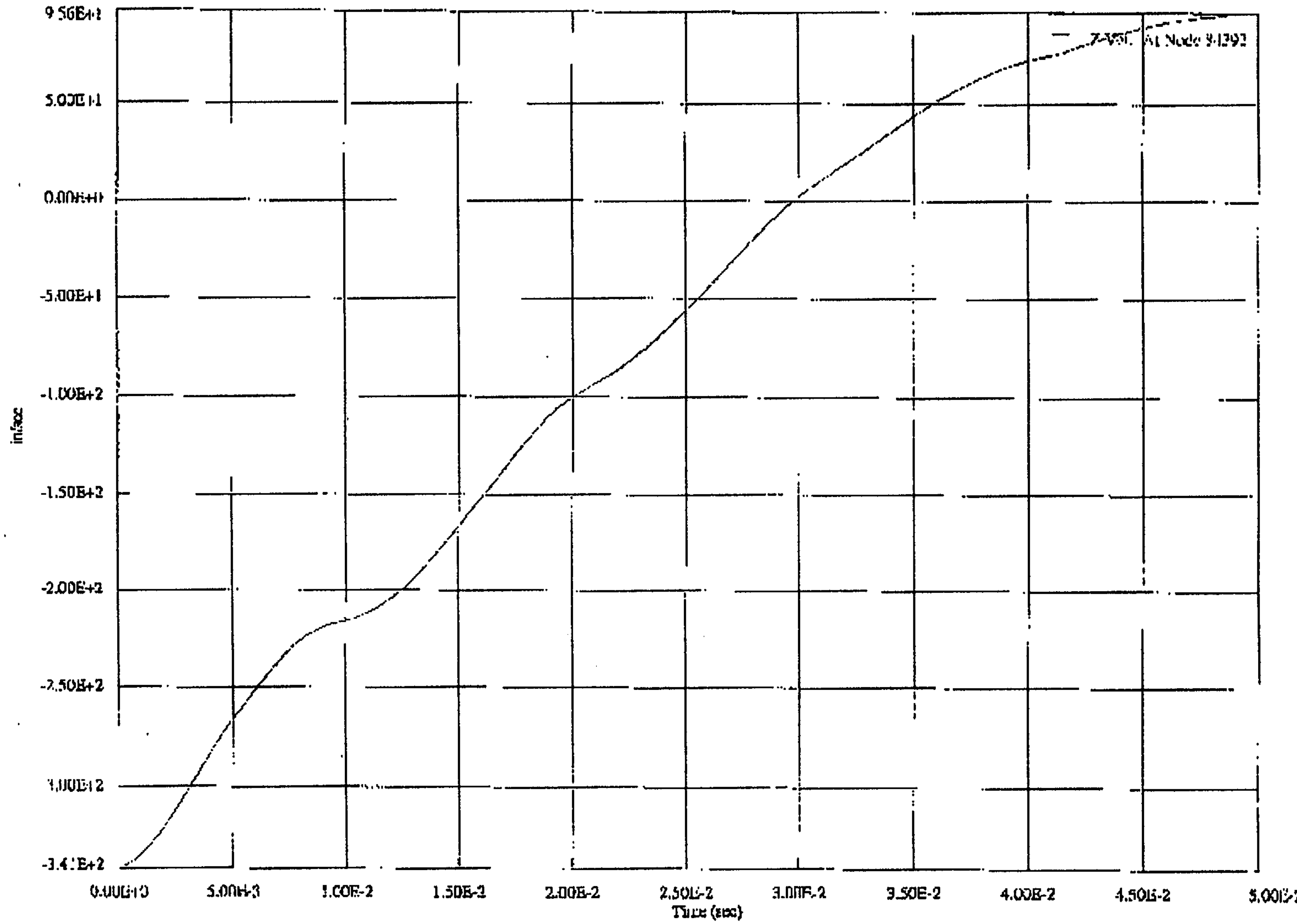


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FIG 3.A.21 Tipover Scenario: Channel A2 Velocity Time History

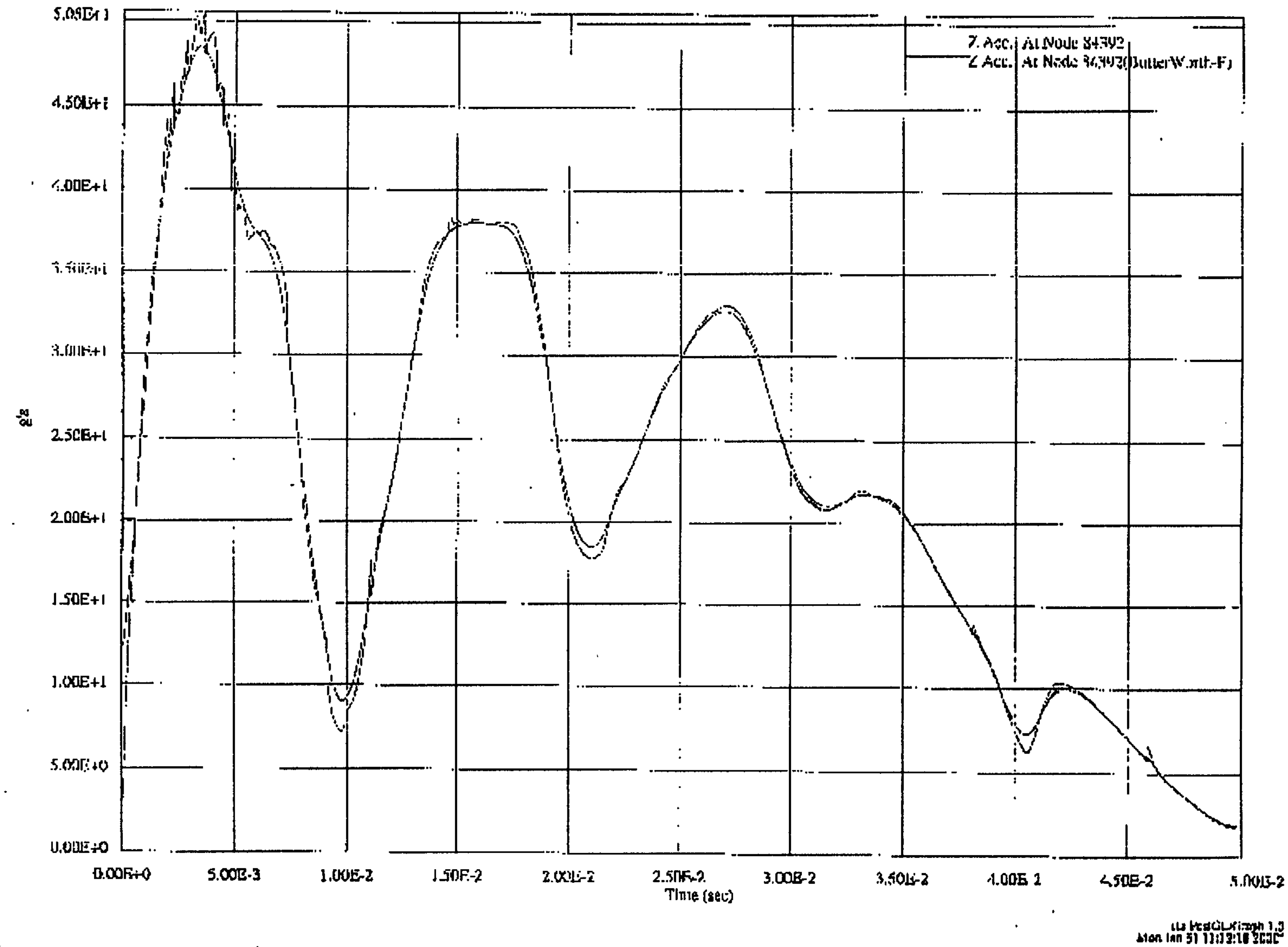

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FIG 3A.22 Tipover Scenario: Channel A2 Deceleration Time Histories

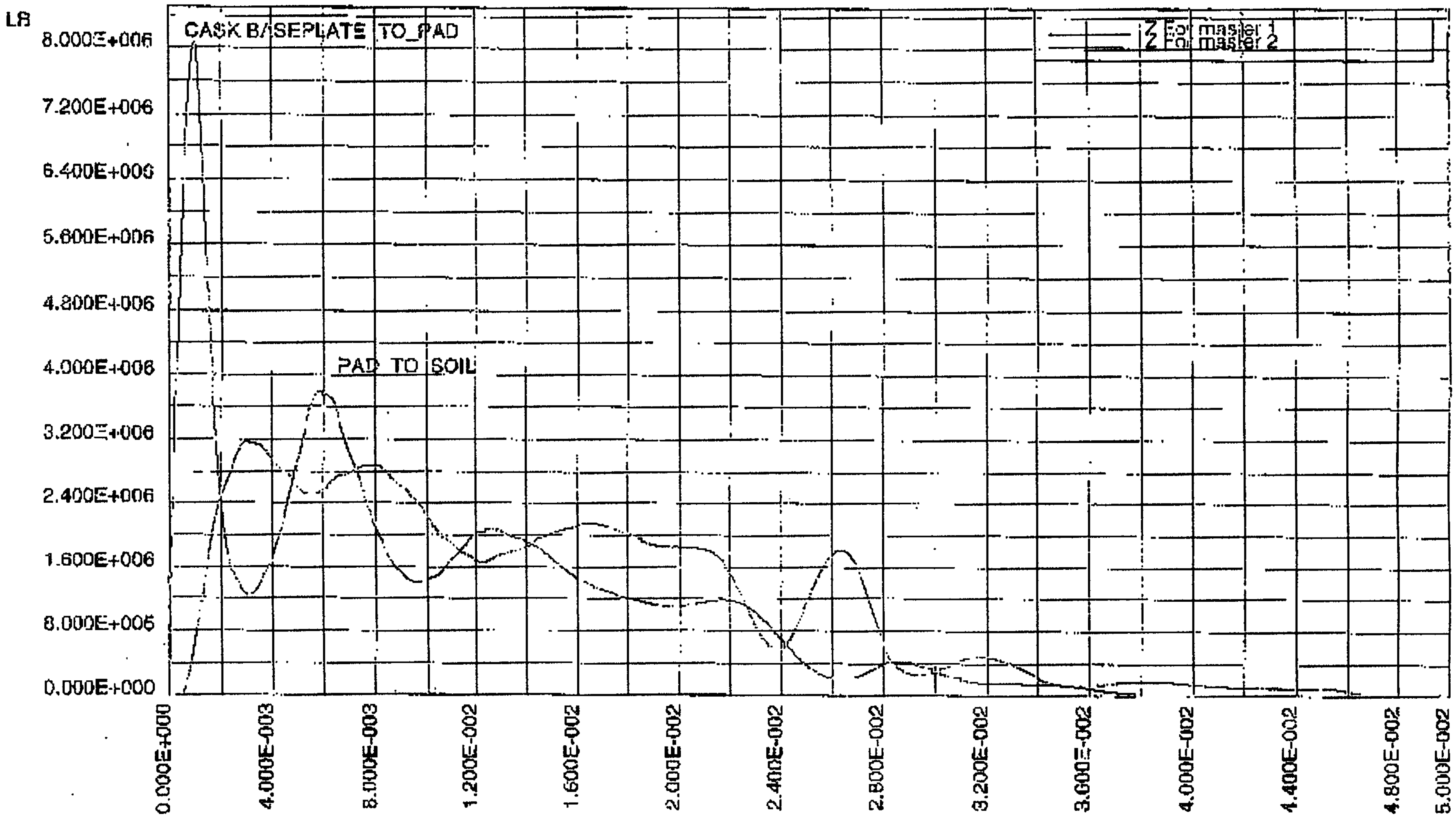

Fig. 3.A.23 End-Drop Scenario: Impact Force Time Histories

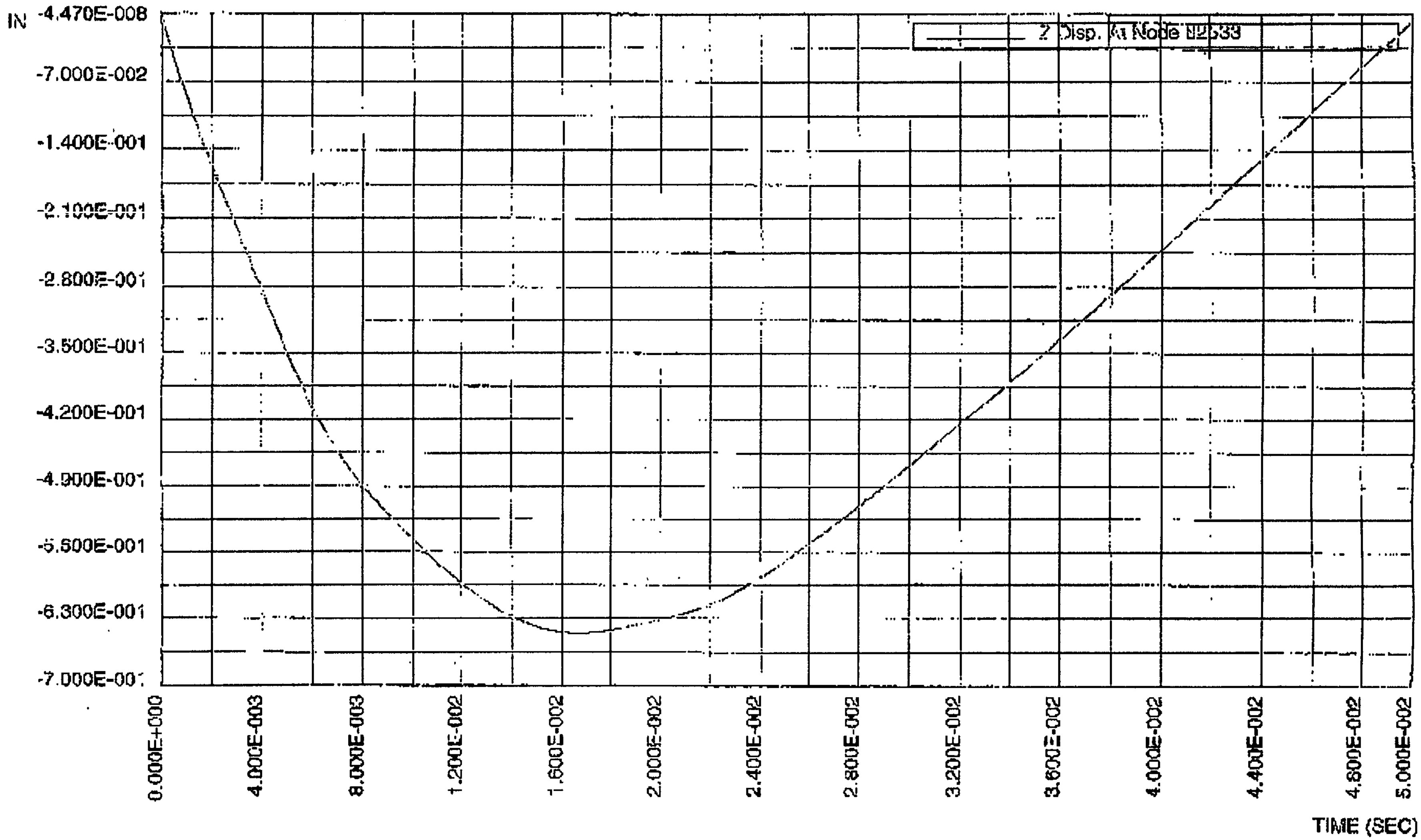

Fig. 3.A.24 End-Drop Scenario: Channel Al Displacement Time History

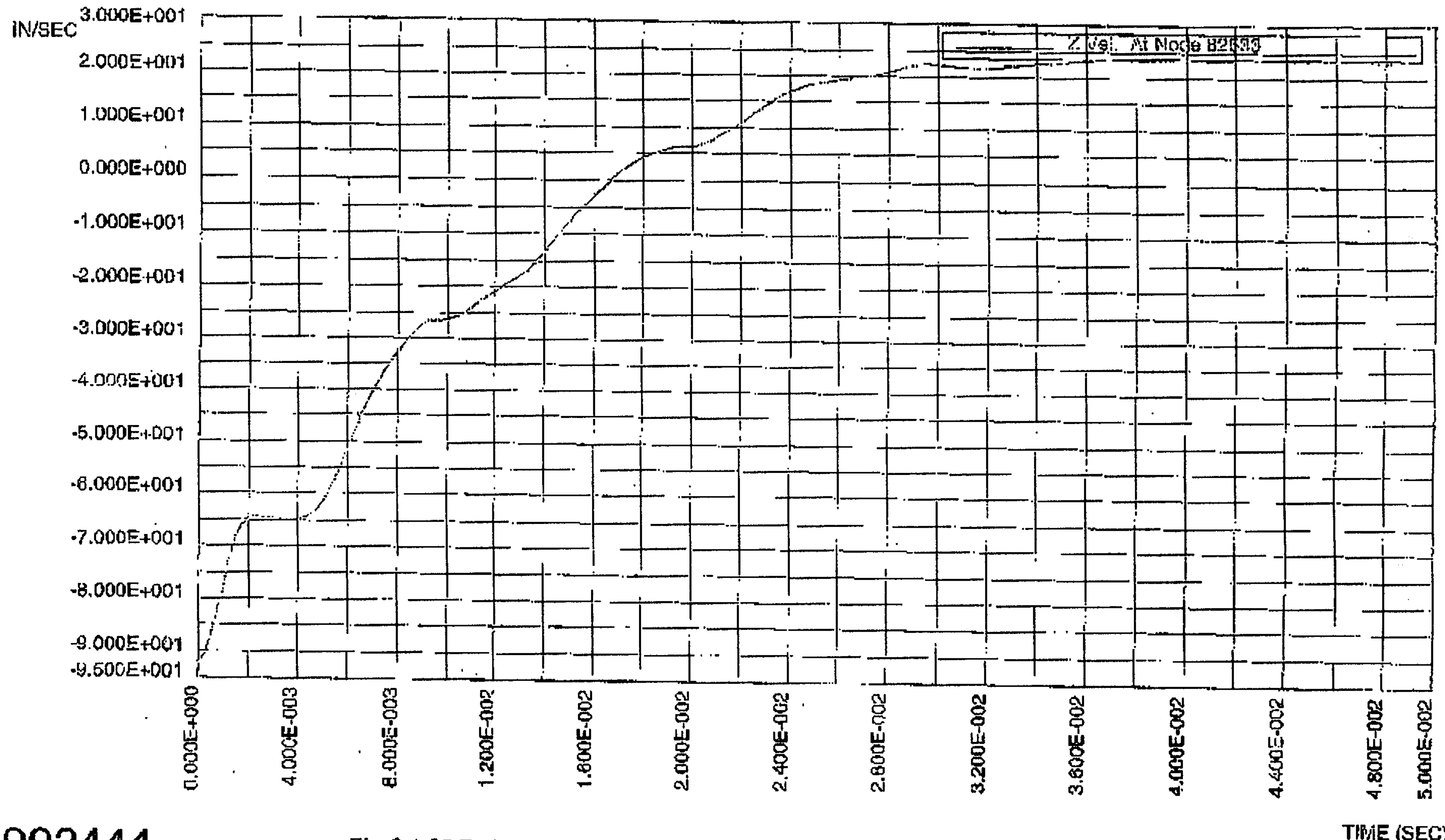

Fig. 3.A.25 End-Drop Scenario: Channel Al Velocity Time History

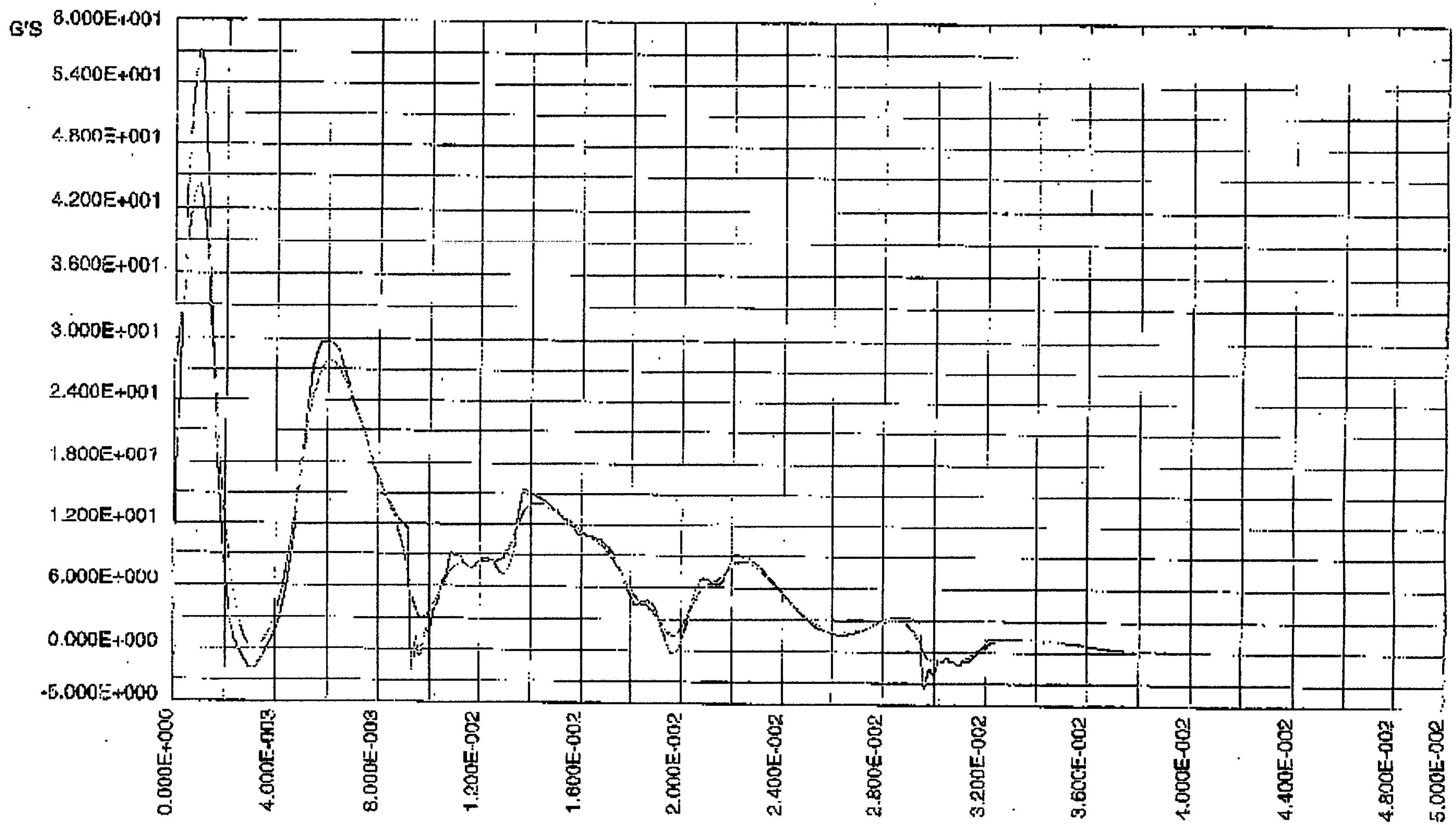

Fig. 3.A.26 End-Drop Scenario: Average Baseplate Top Plate Deceleration Time Histories

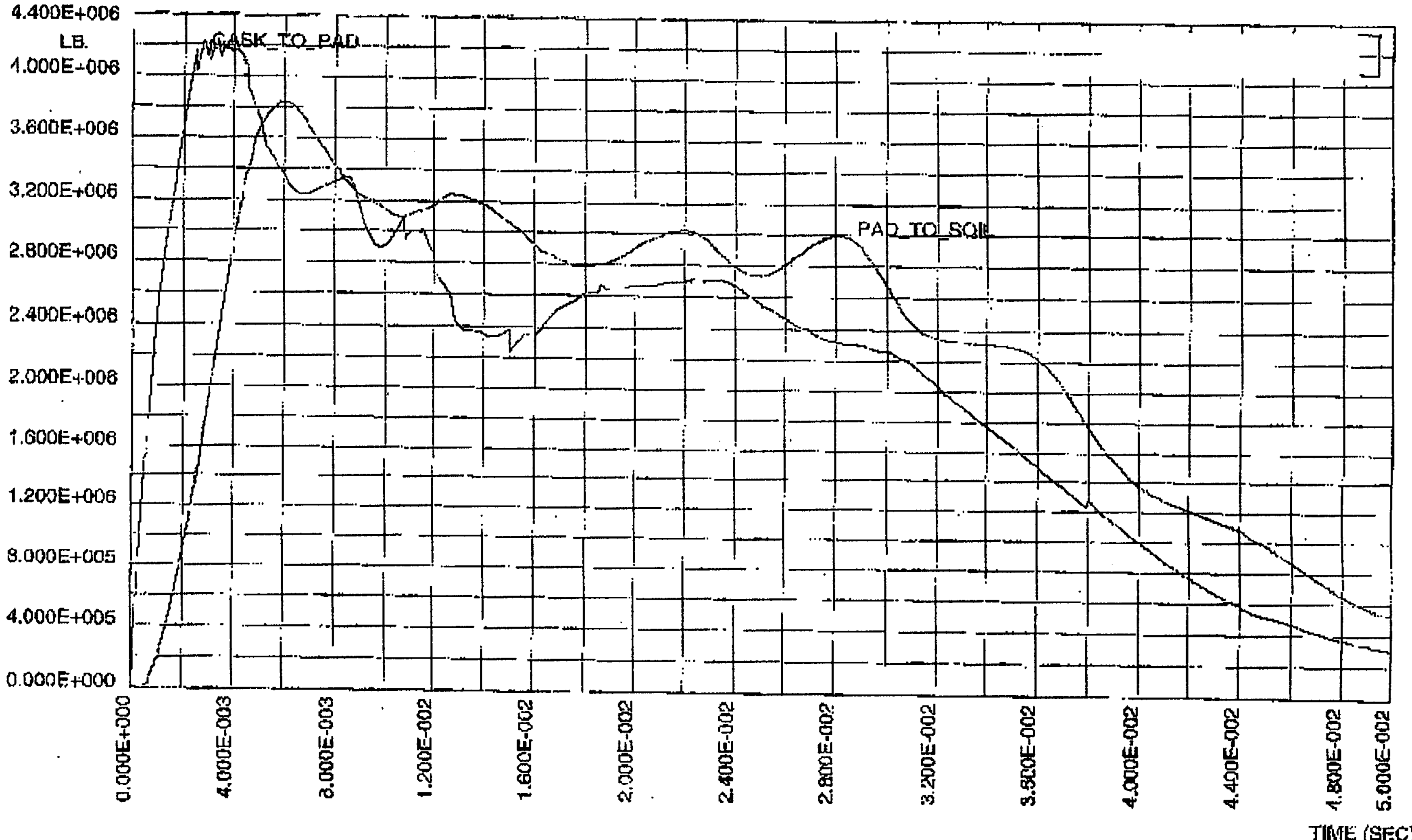

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FIGURE 3.A.27 TIPOVER (WITH INCREASED INITIAL CLEARANCE): IMPACT FORCE TIME HISTORIES

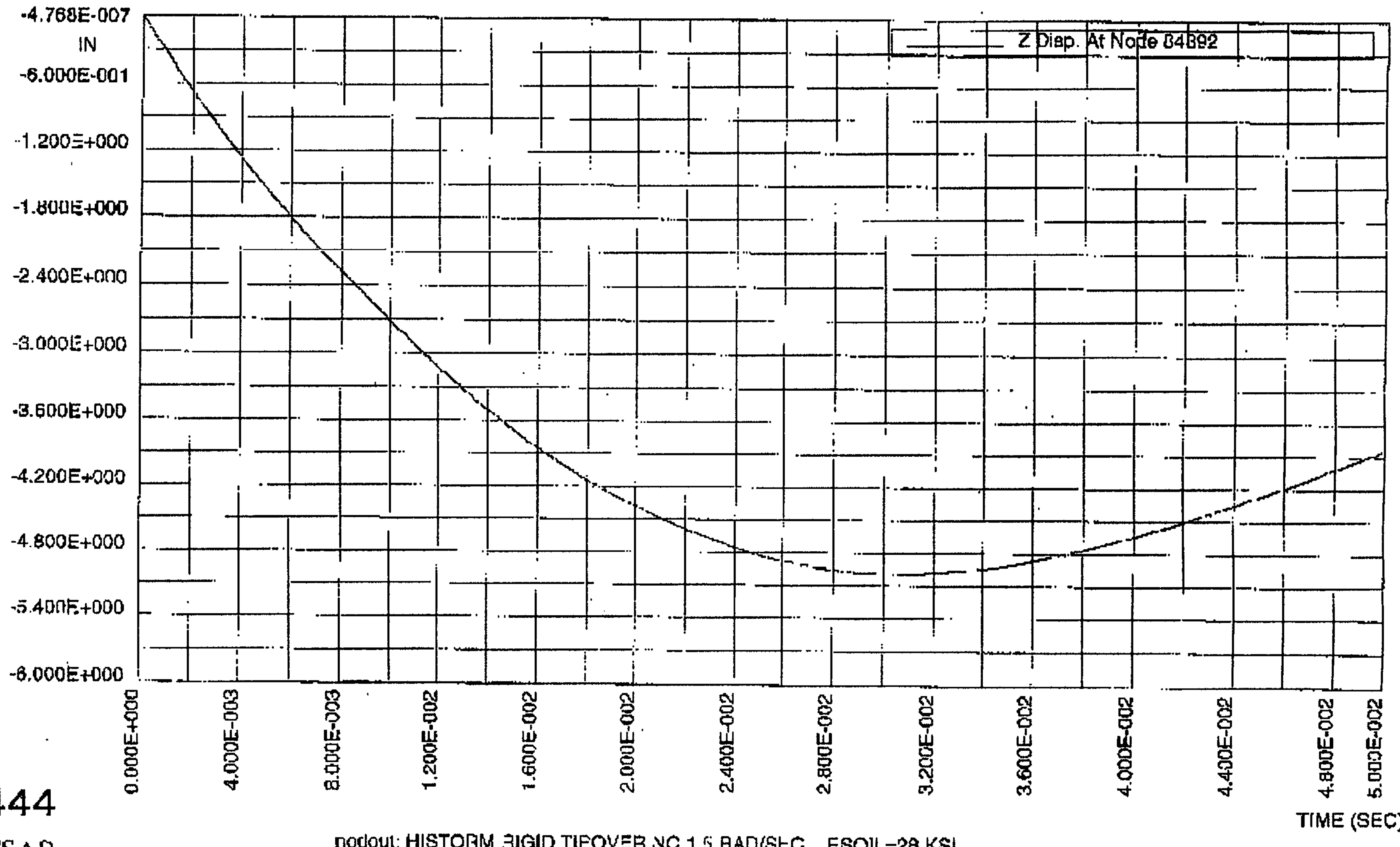

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FIGURE 3.A.28 TIPOVER (WITH INCREASED INITIAL CLEARANCE): CHANNEL A2 DISPLACEMENT TIME HISTORY

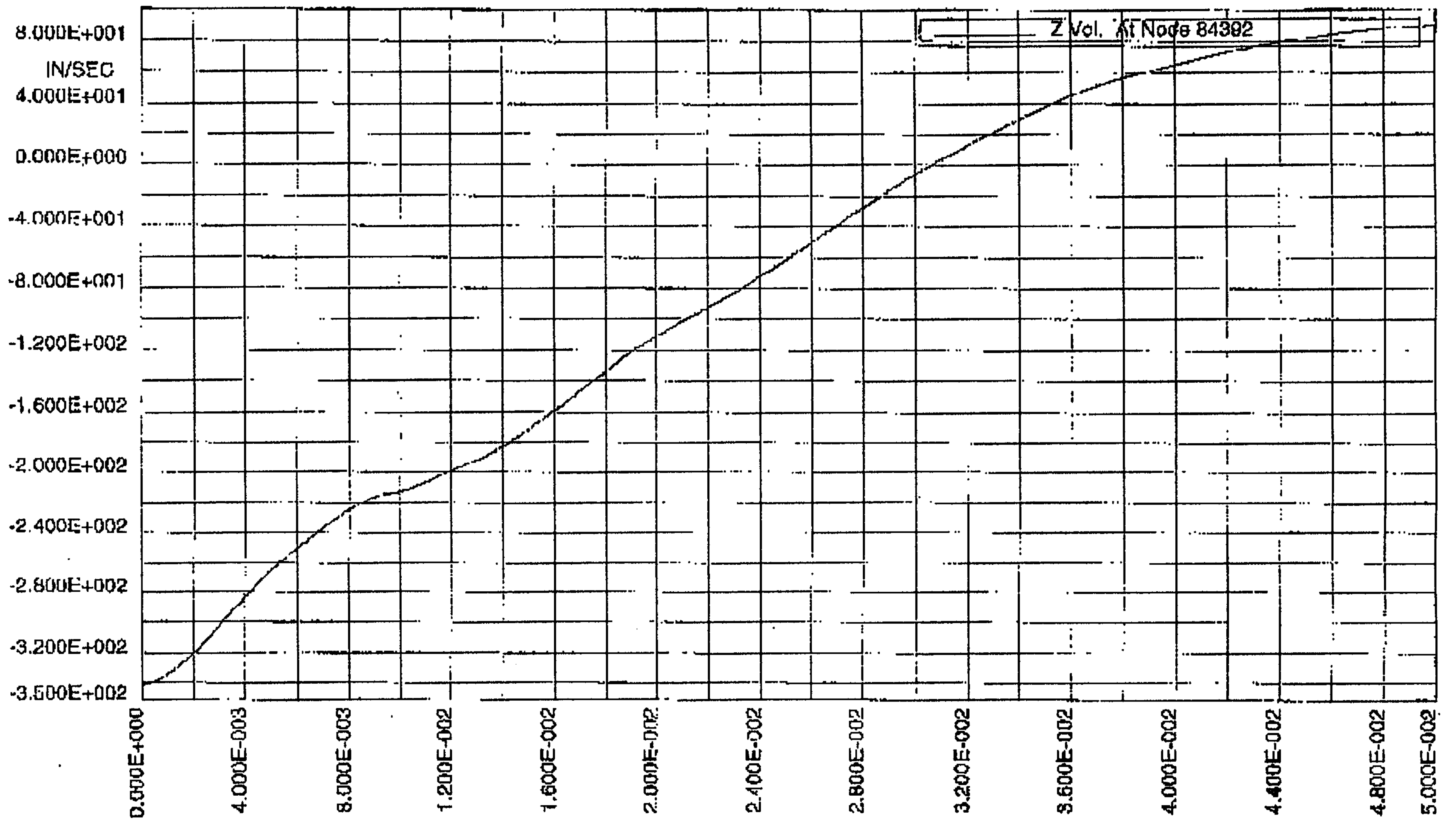

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FIGURE 3.A.29 TIPOVER (WITH INCREASED INITIAL CLEARANCE): CHANNEL A2 VELOCITY TIME HISTORY

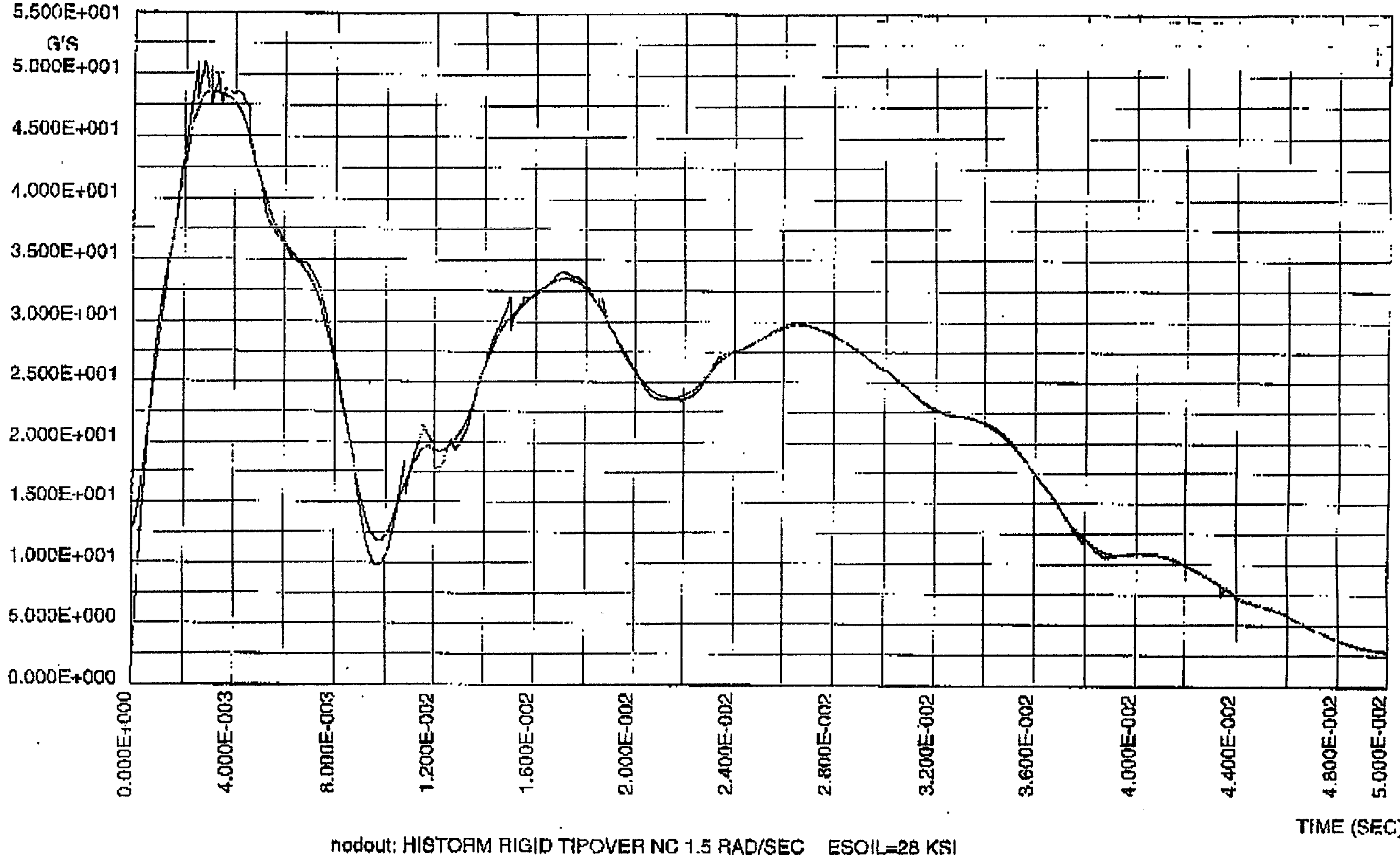
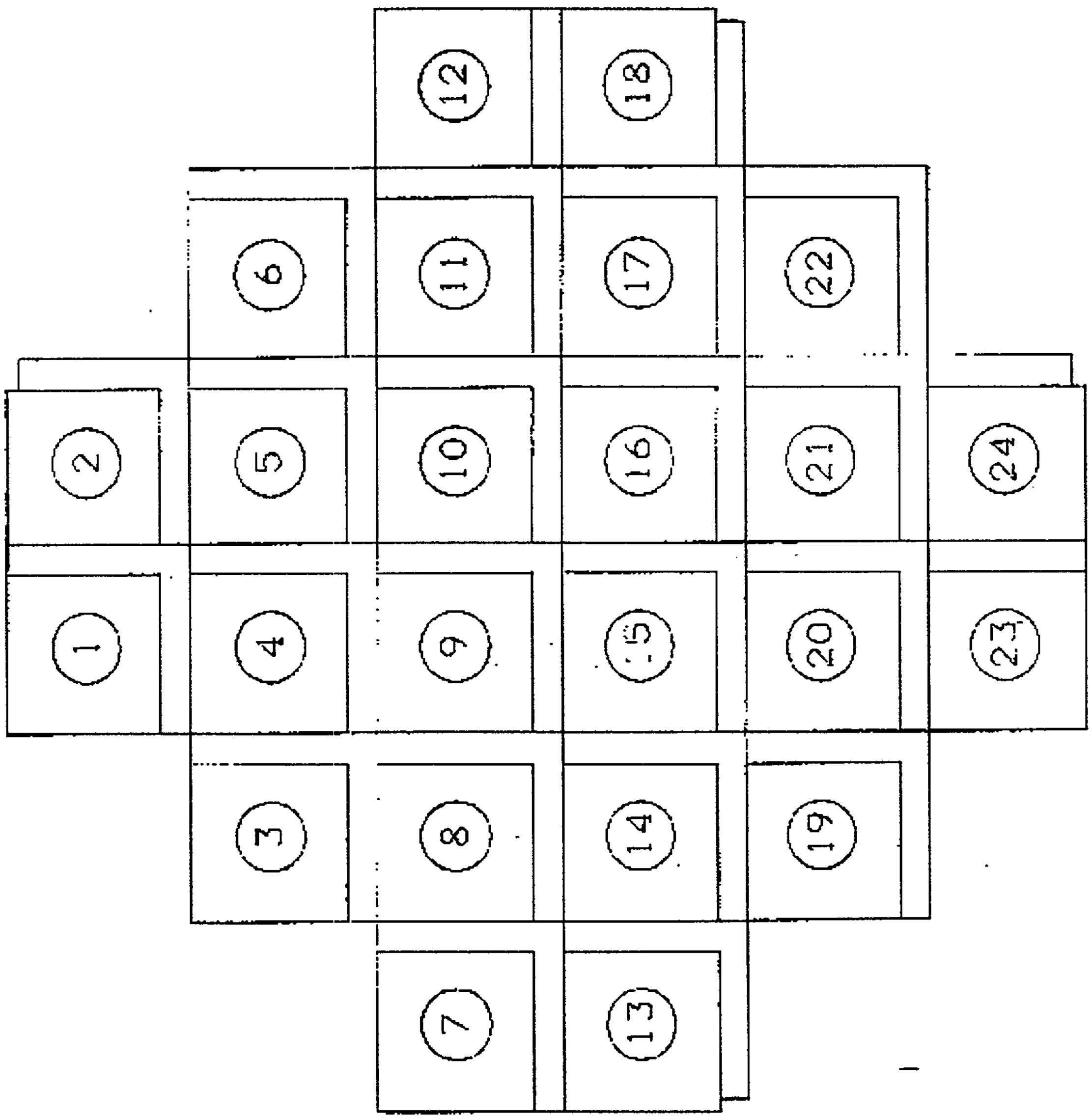


FIGURE 3.A.30 TIPOVER (WITH INCREASED INITIAL CLEARANCE): TOP LID PLATE DECELERATION TIME HISTORY

Appendix 3.N - Detailed Finite Element Listings for the MPC-24 Fuel Basket

Twenty-six (26) pages total including cover page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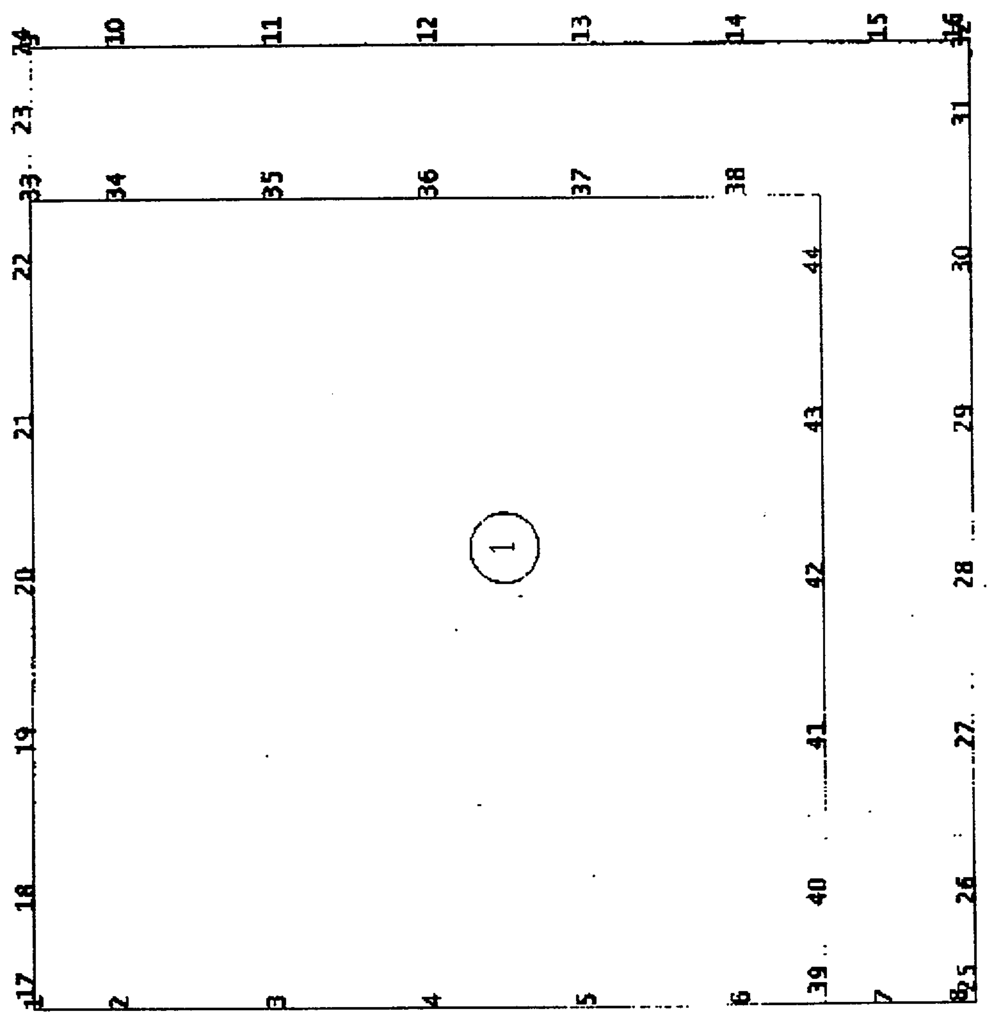

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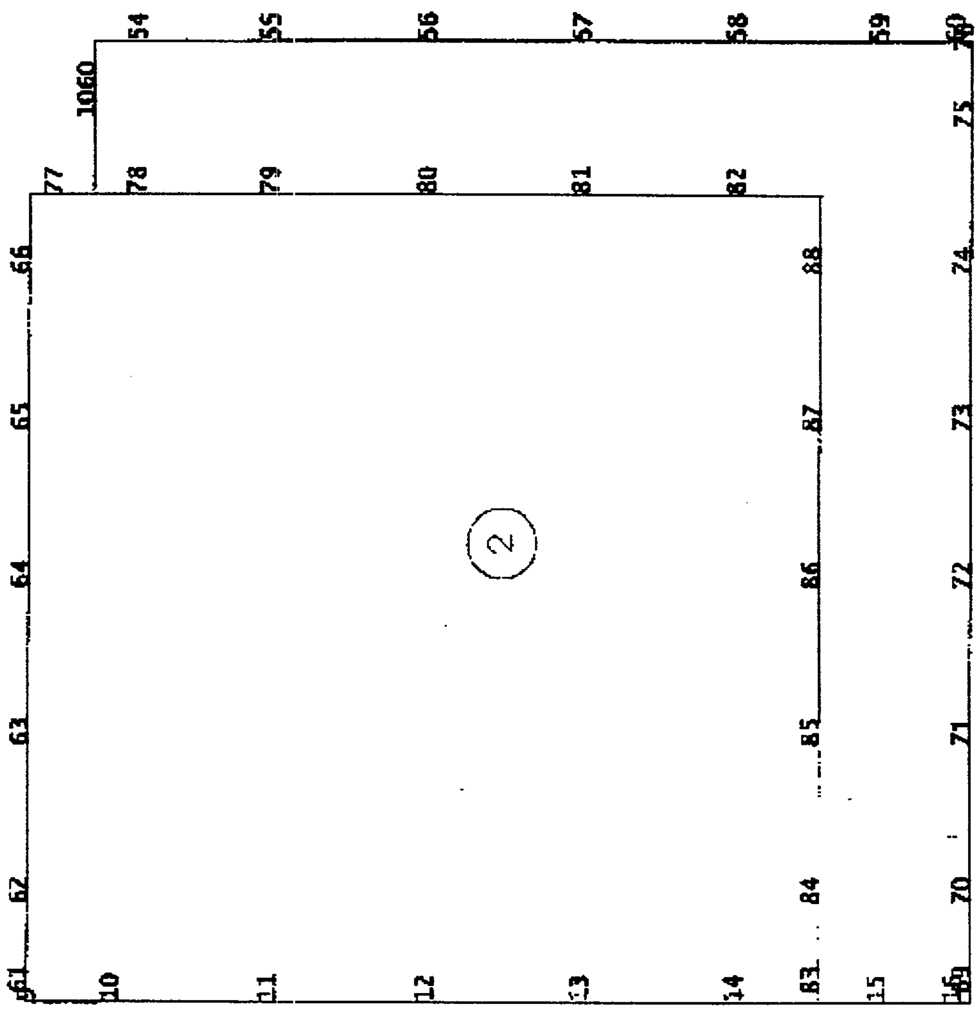
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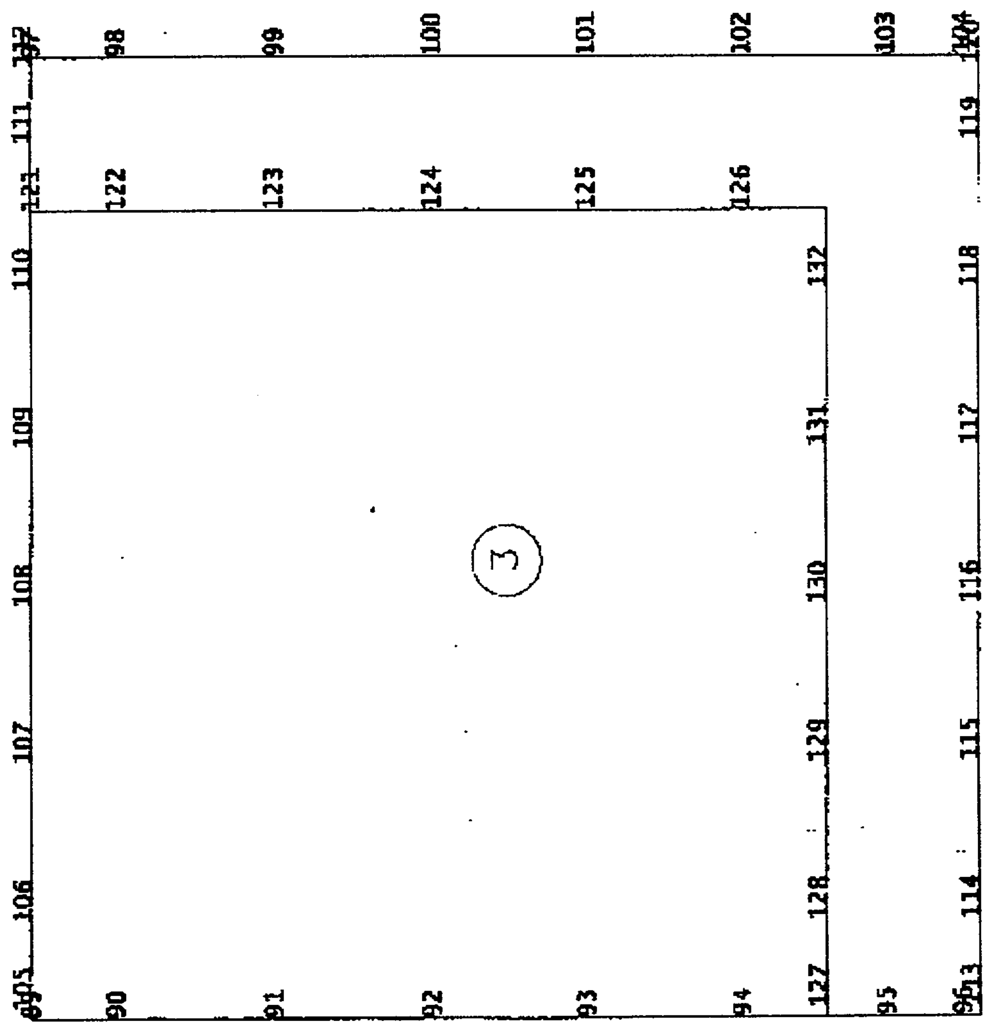
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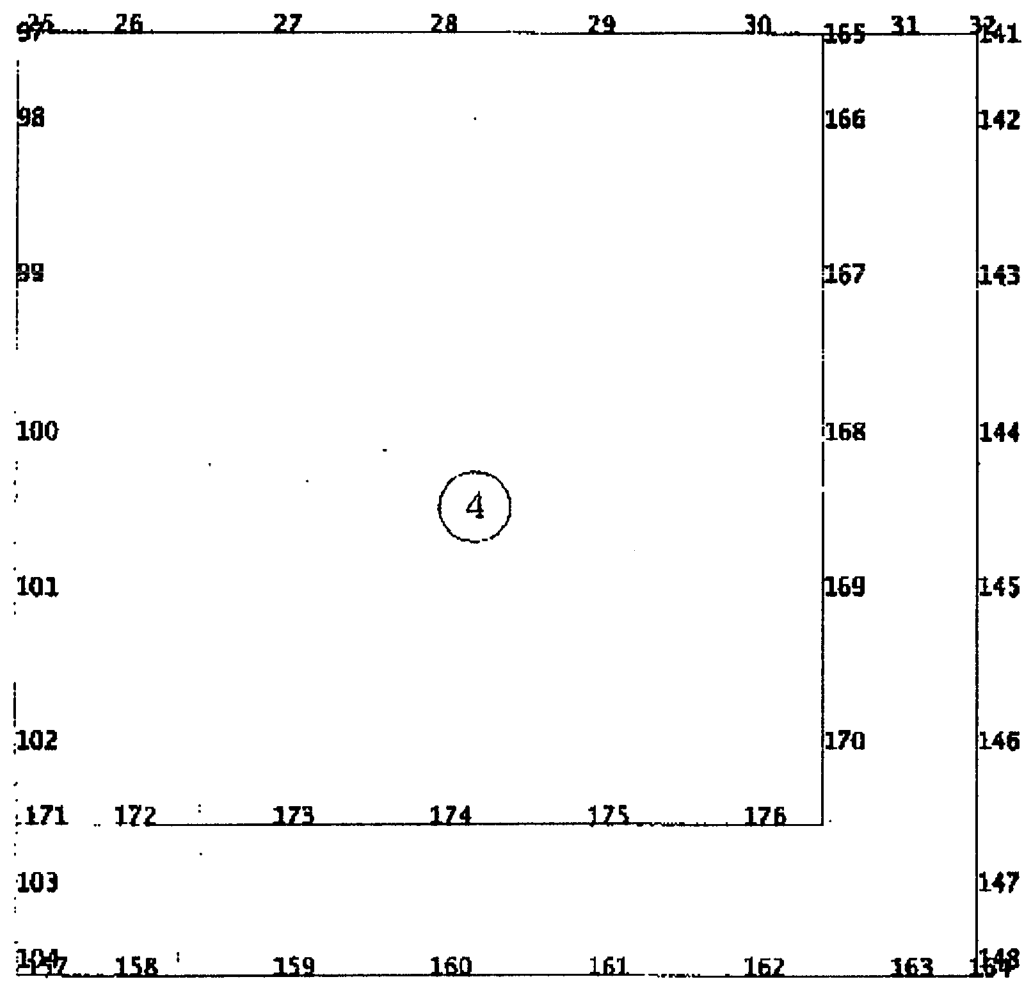
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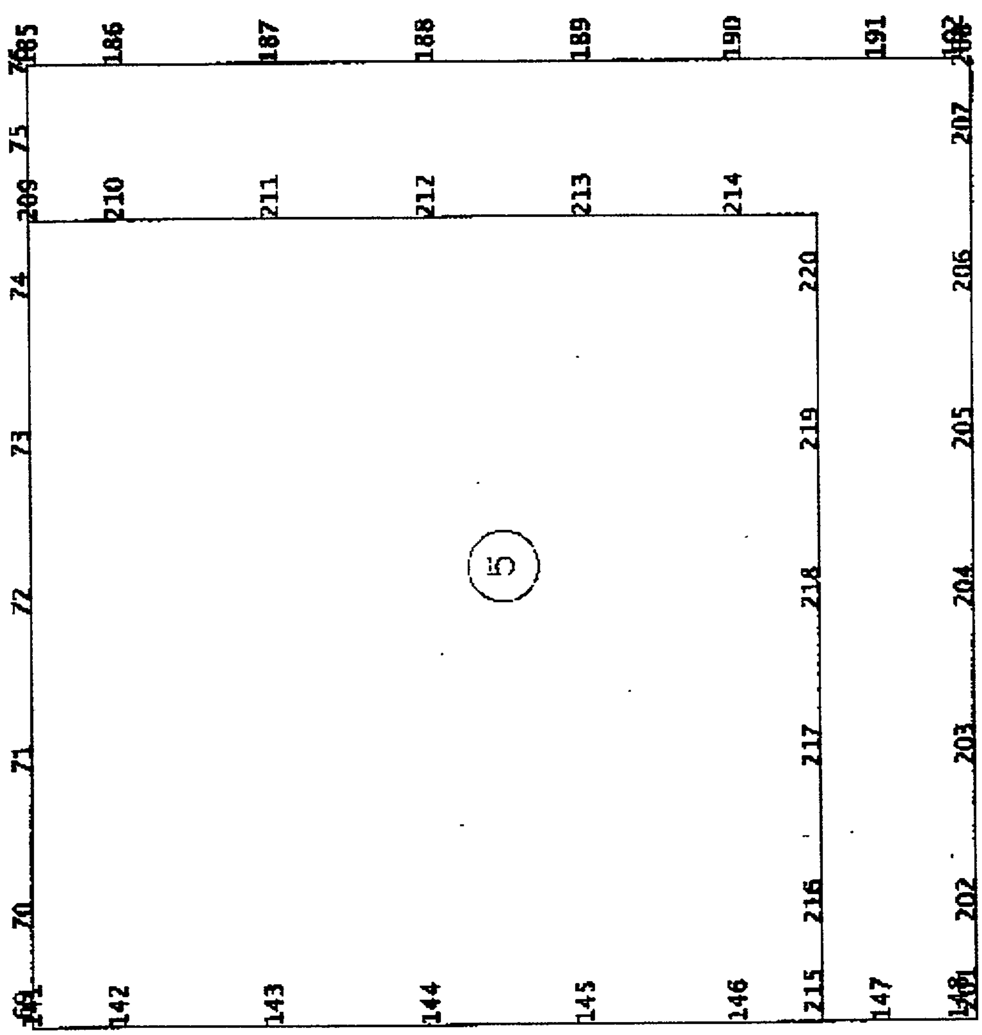
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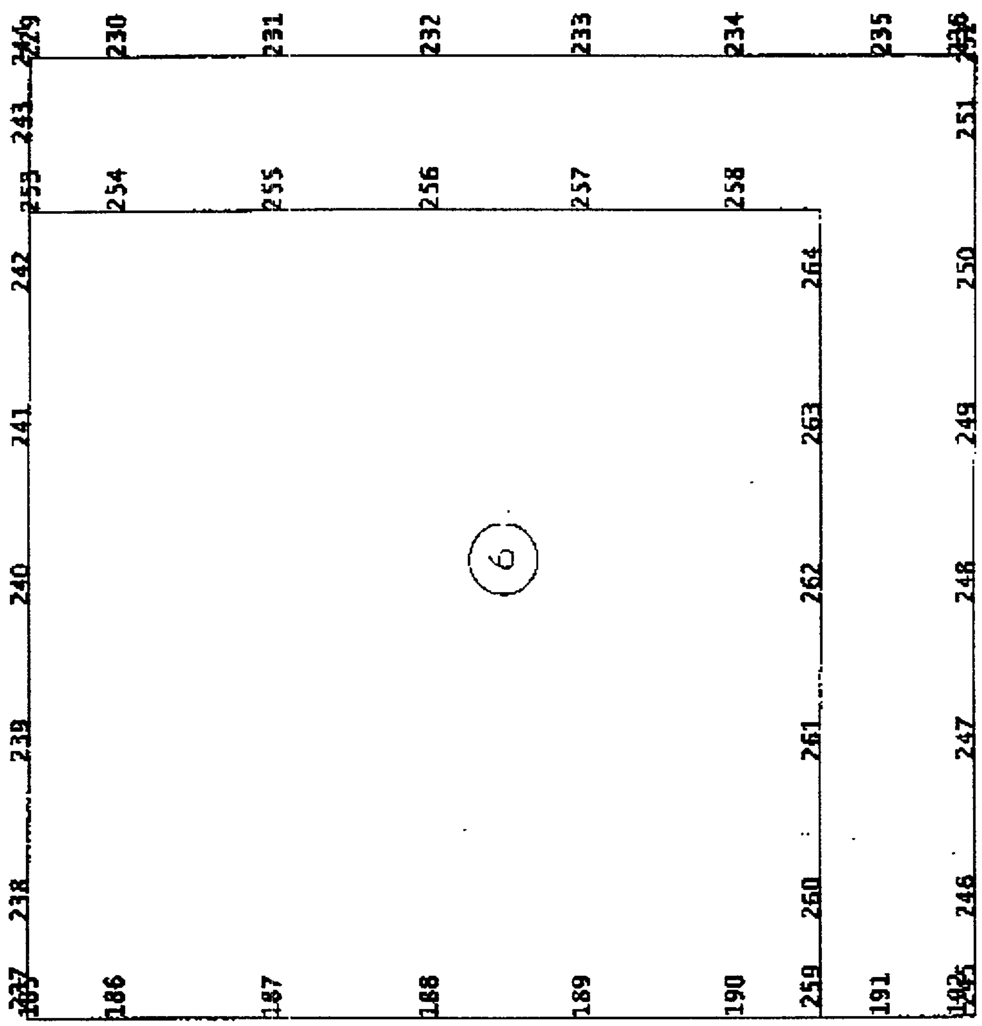
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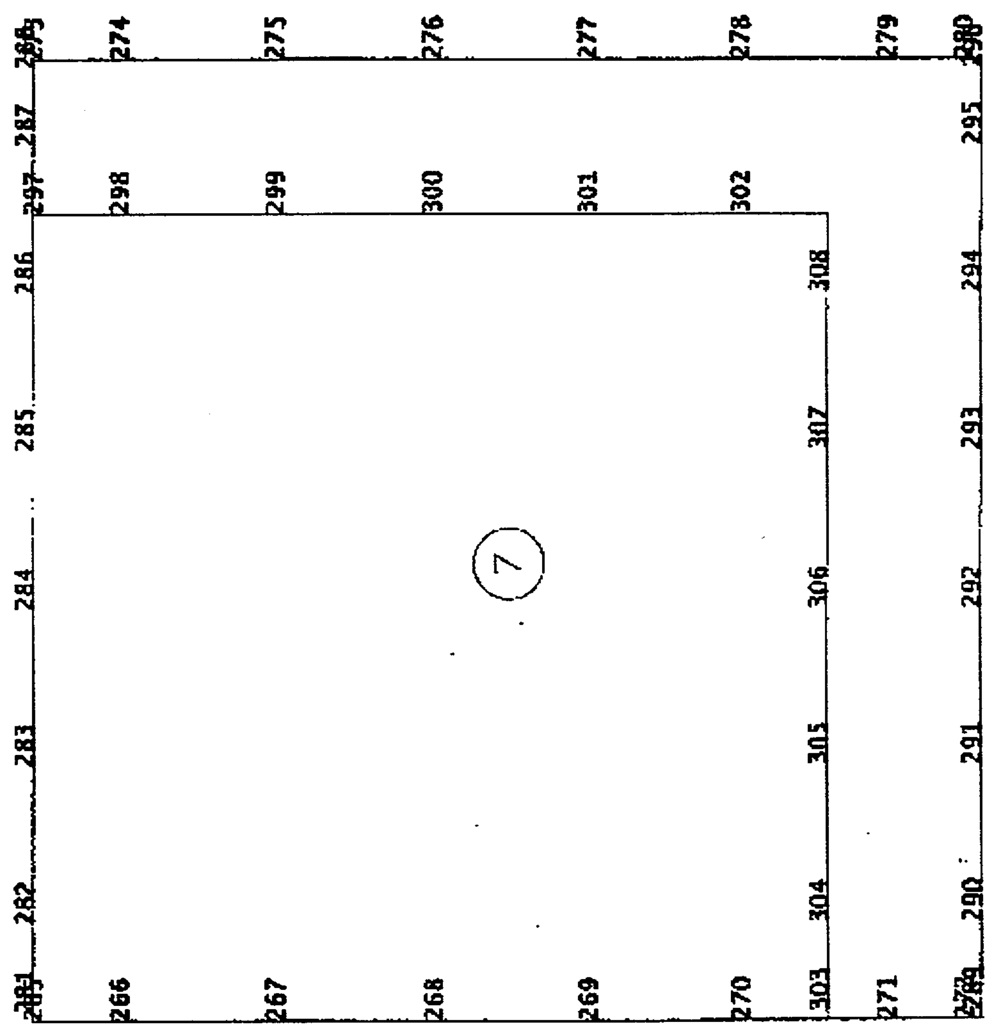
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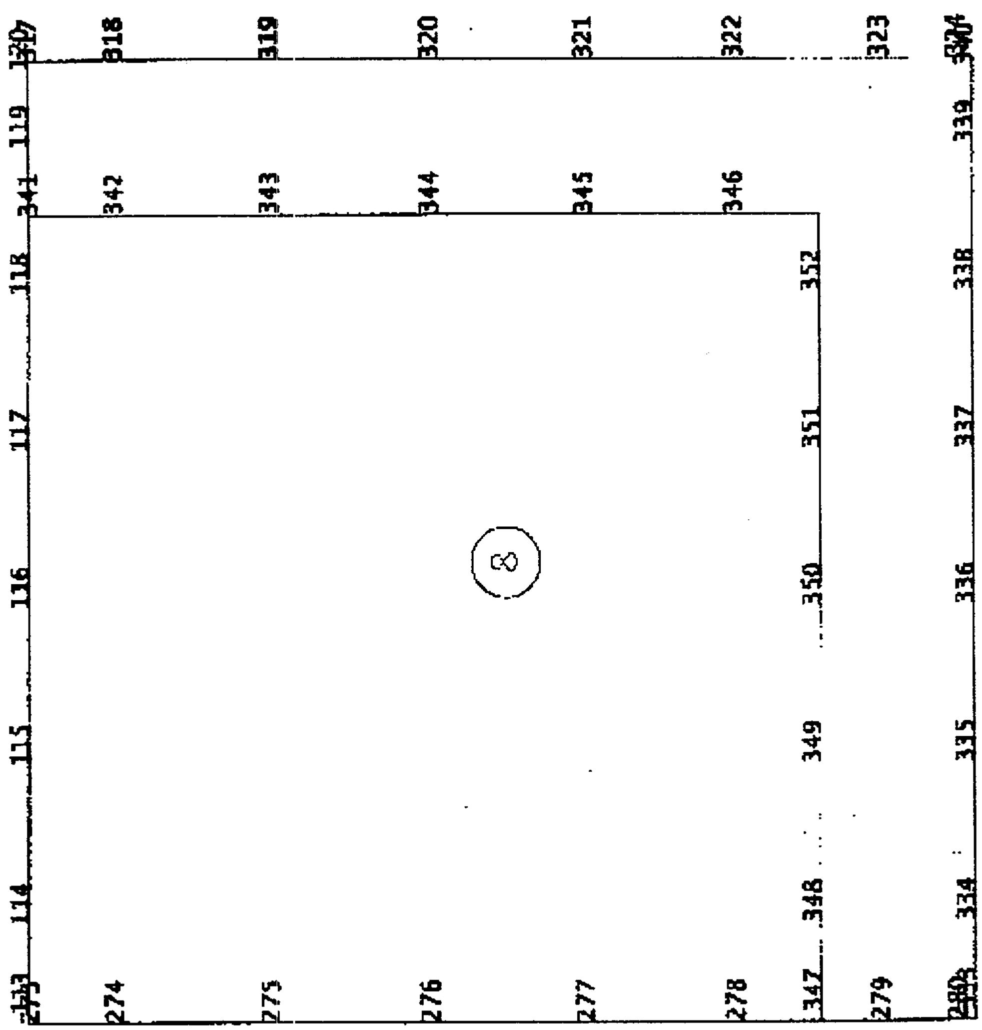

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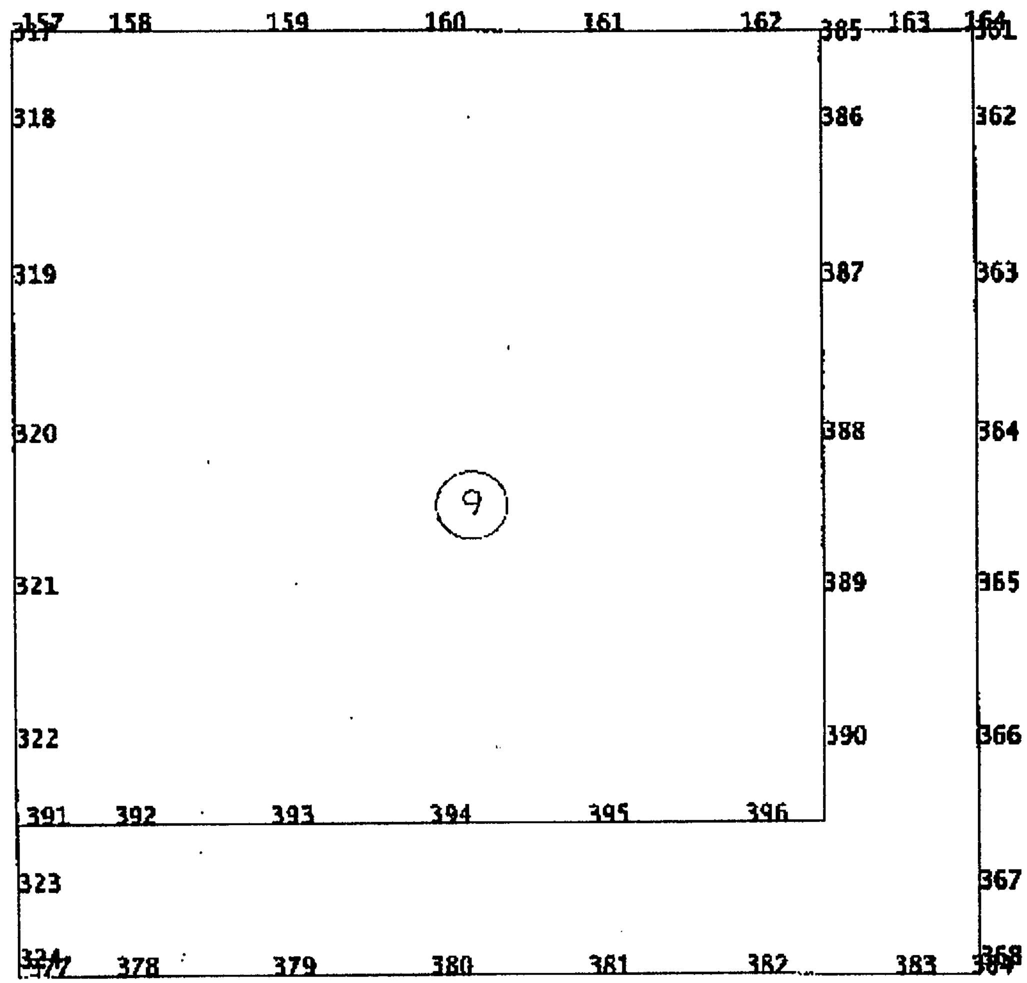
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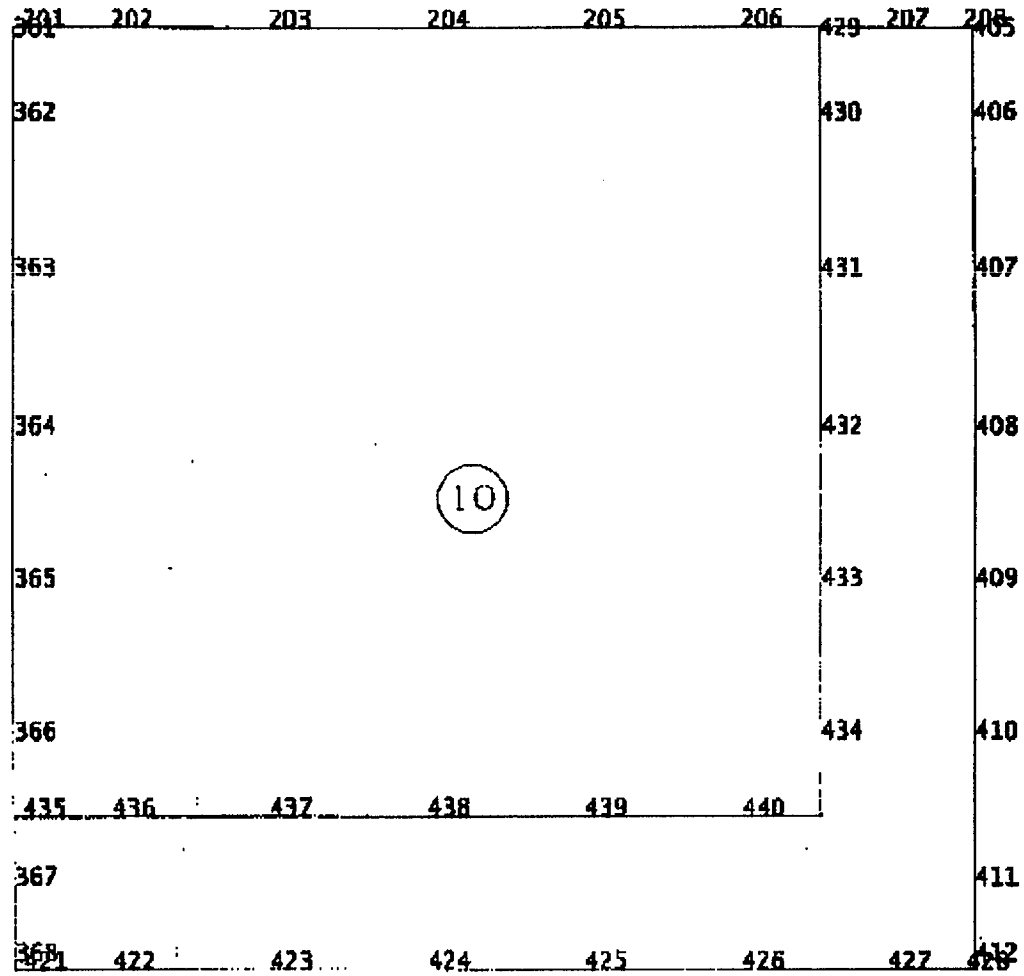
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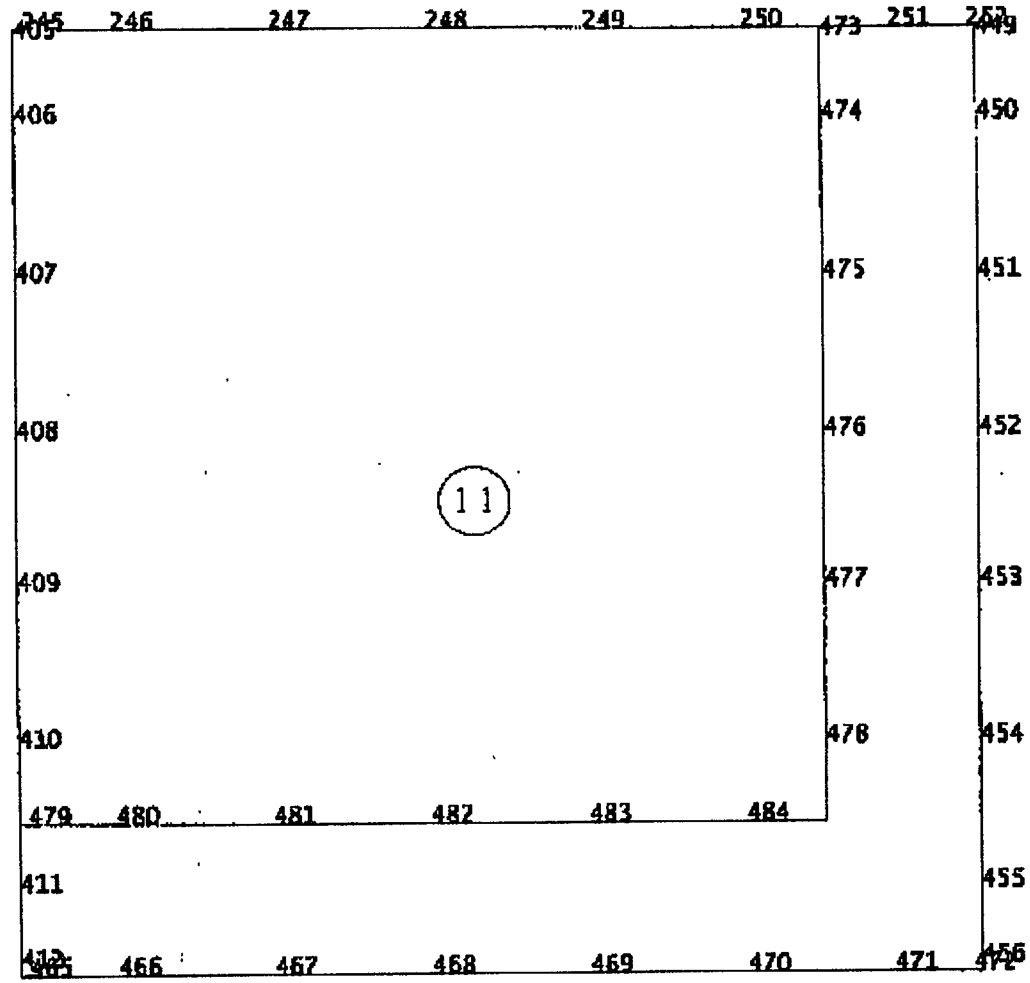
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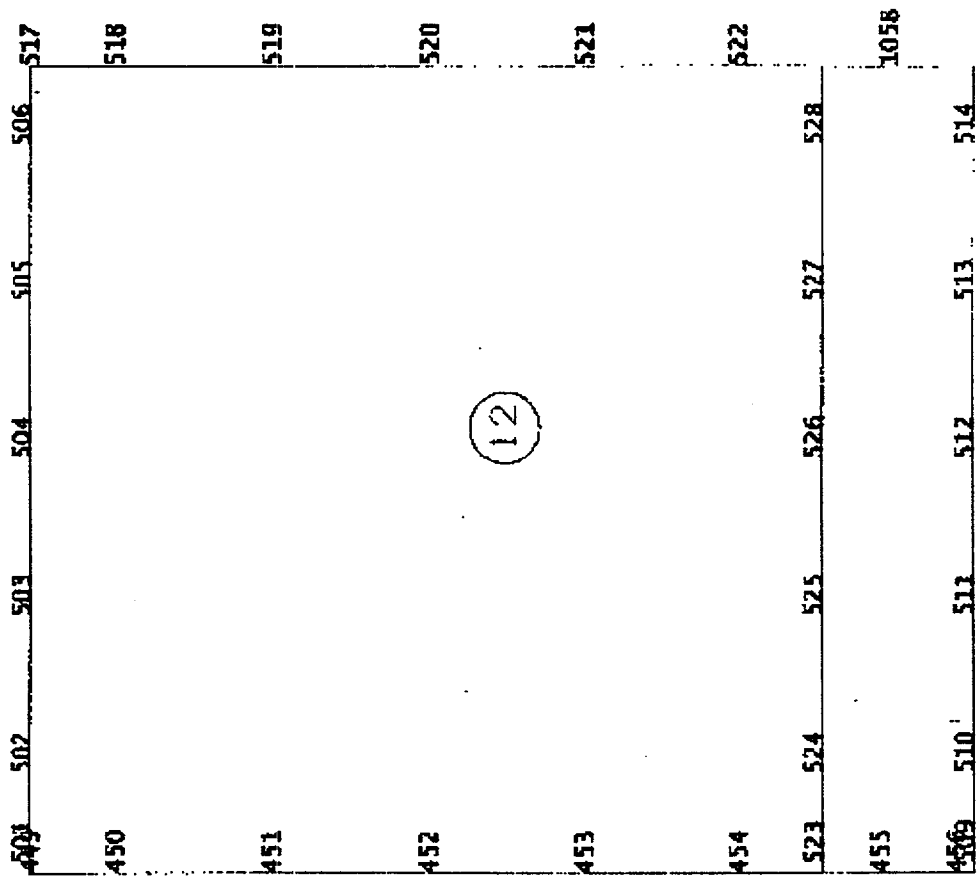

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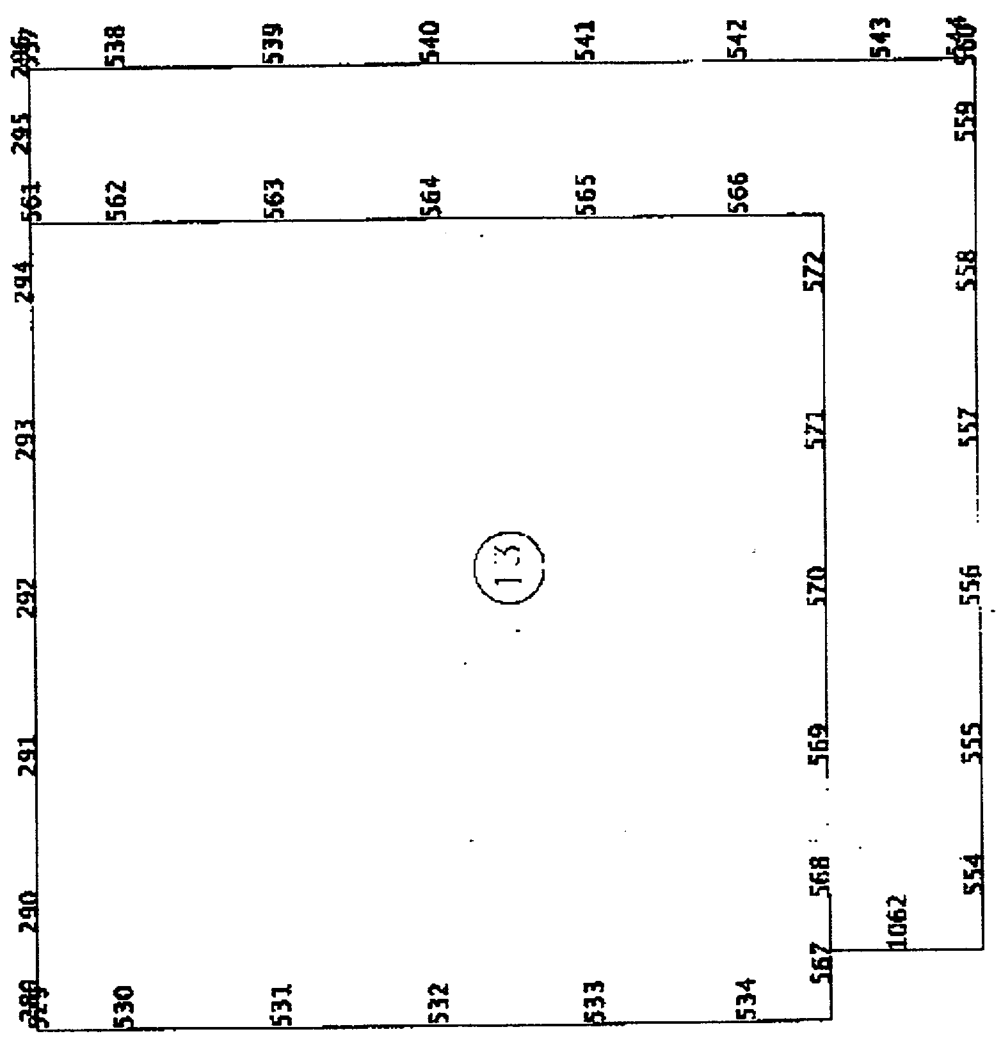
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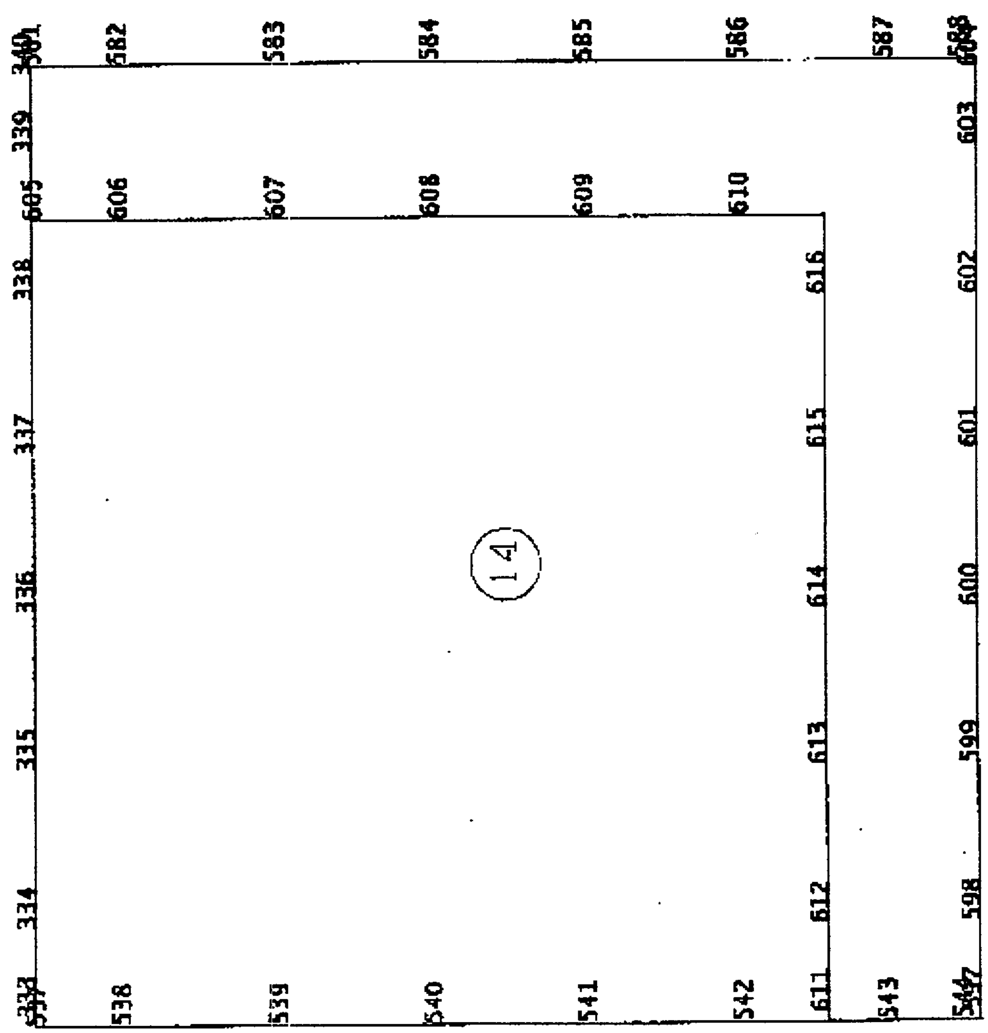

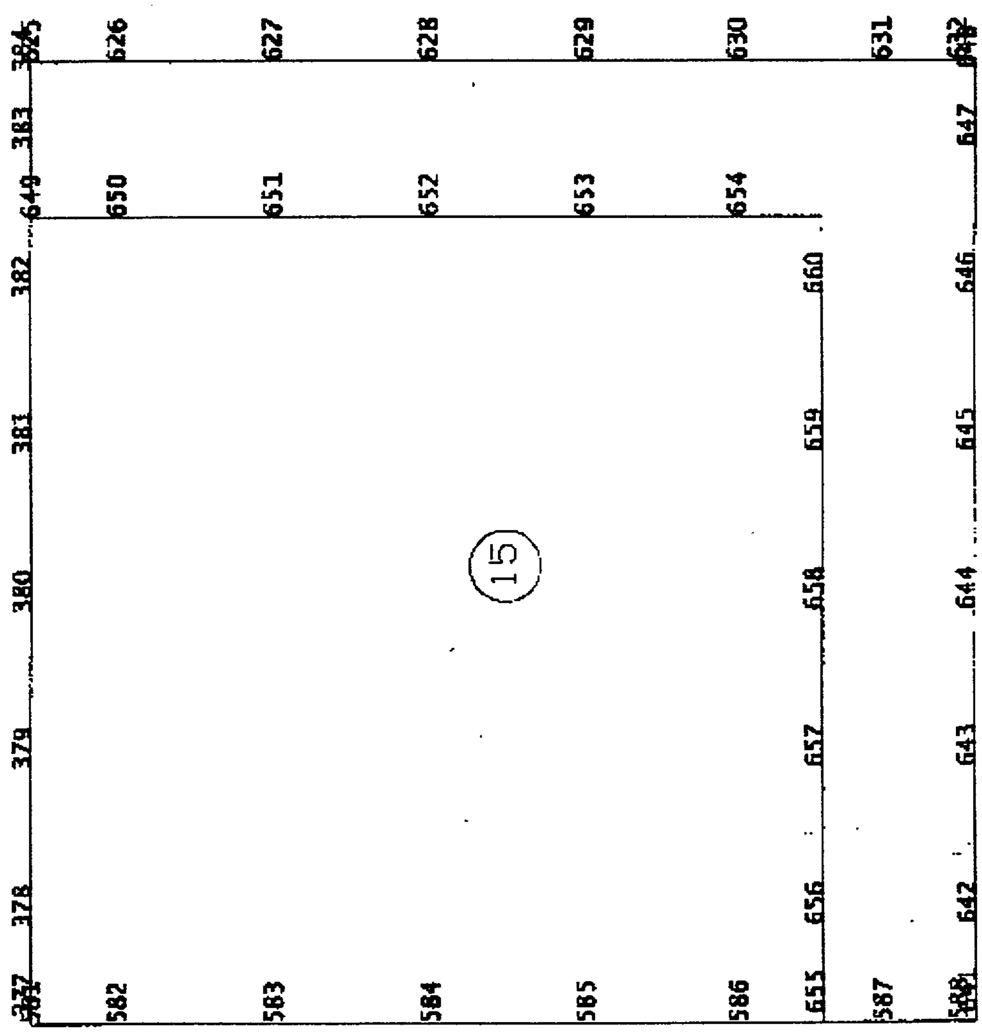
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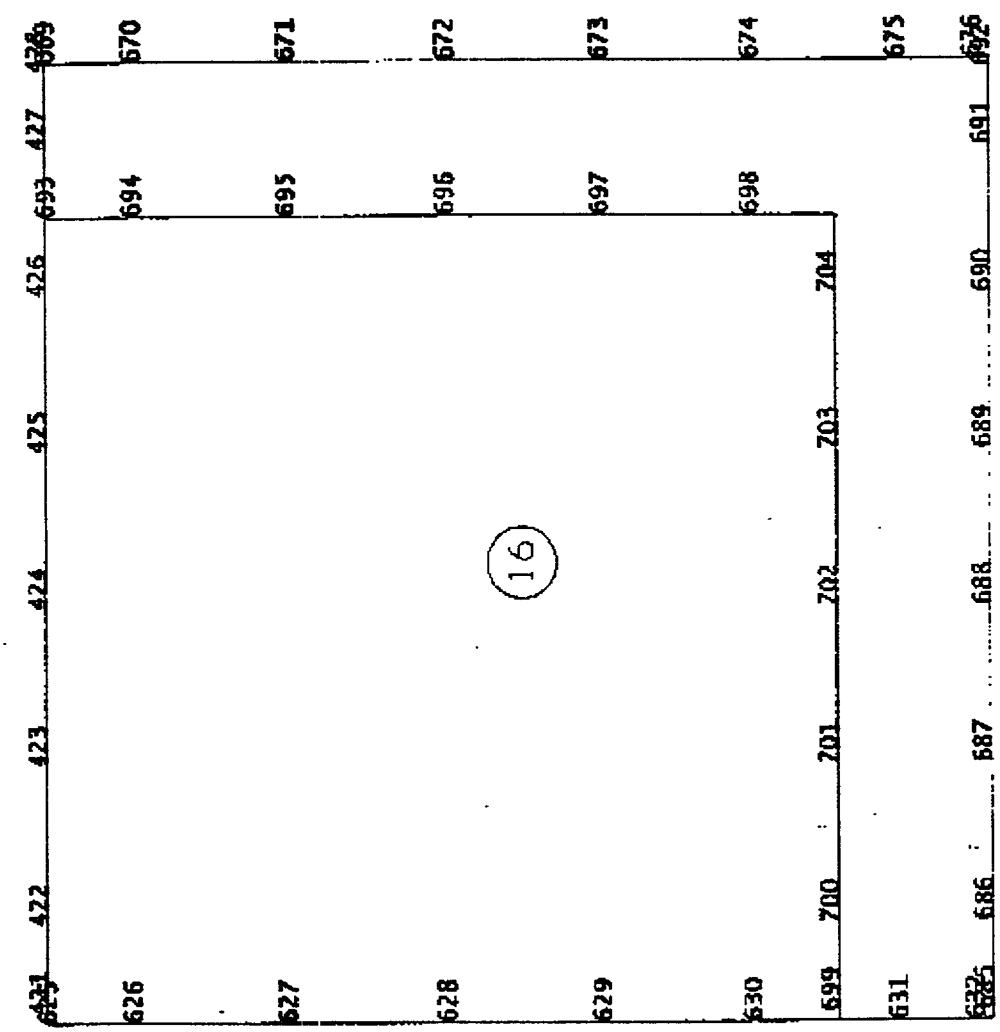

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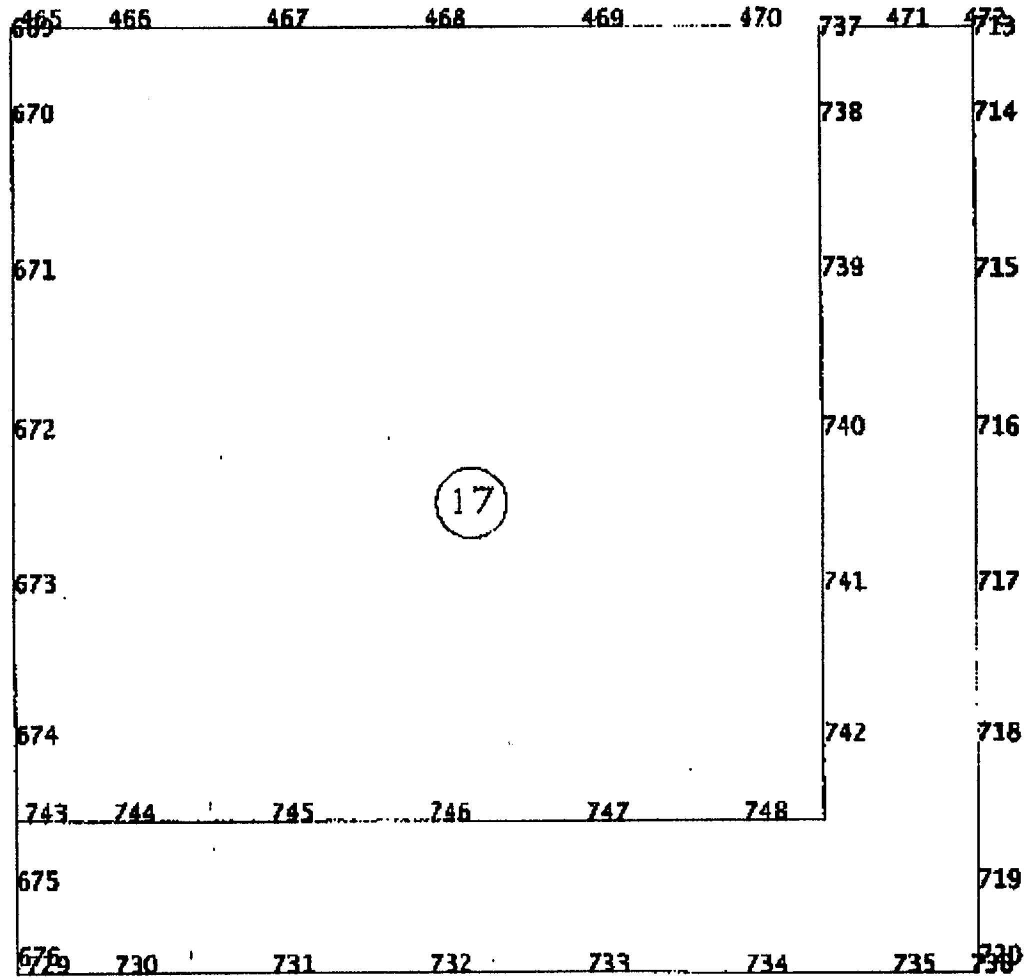
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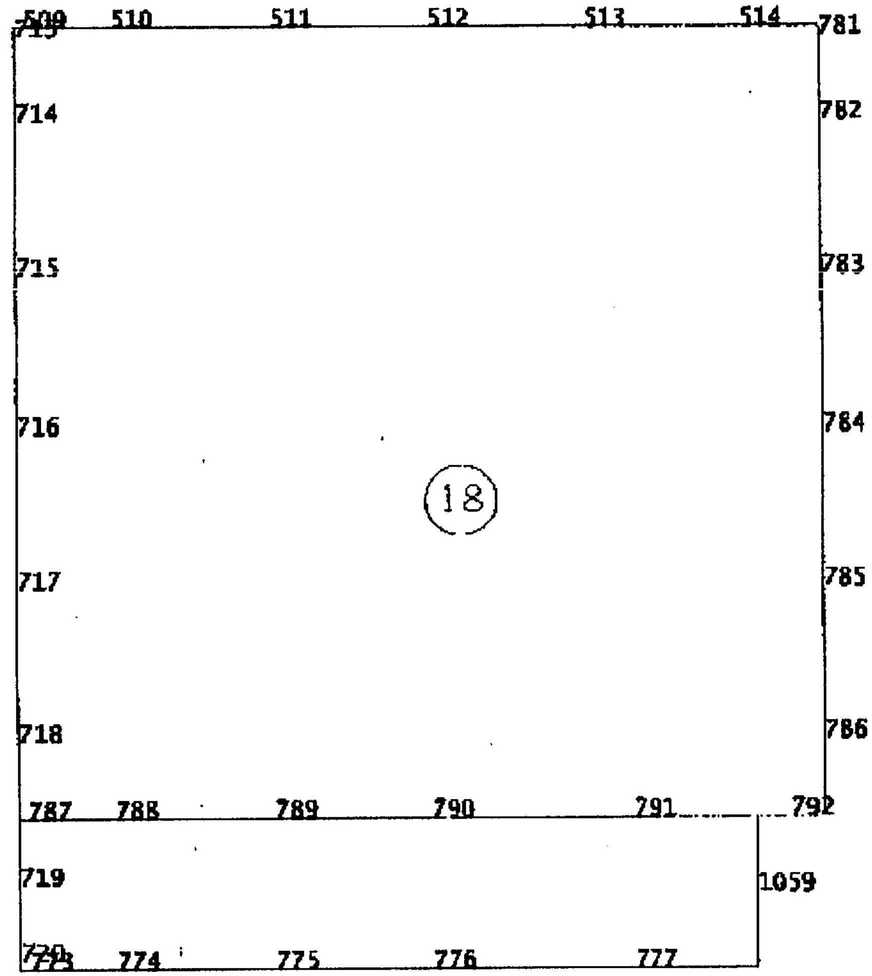
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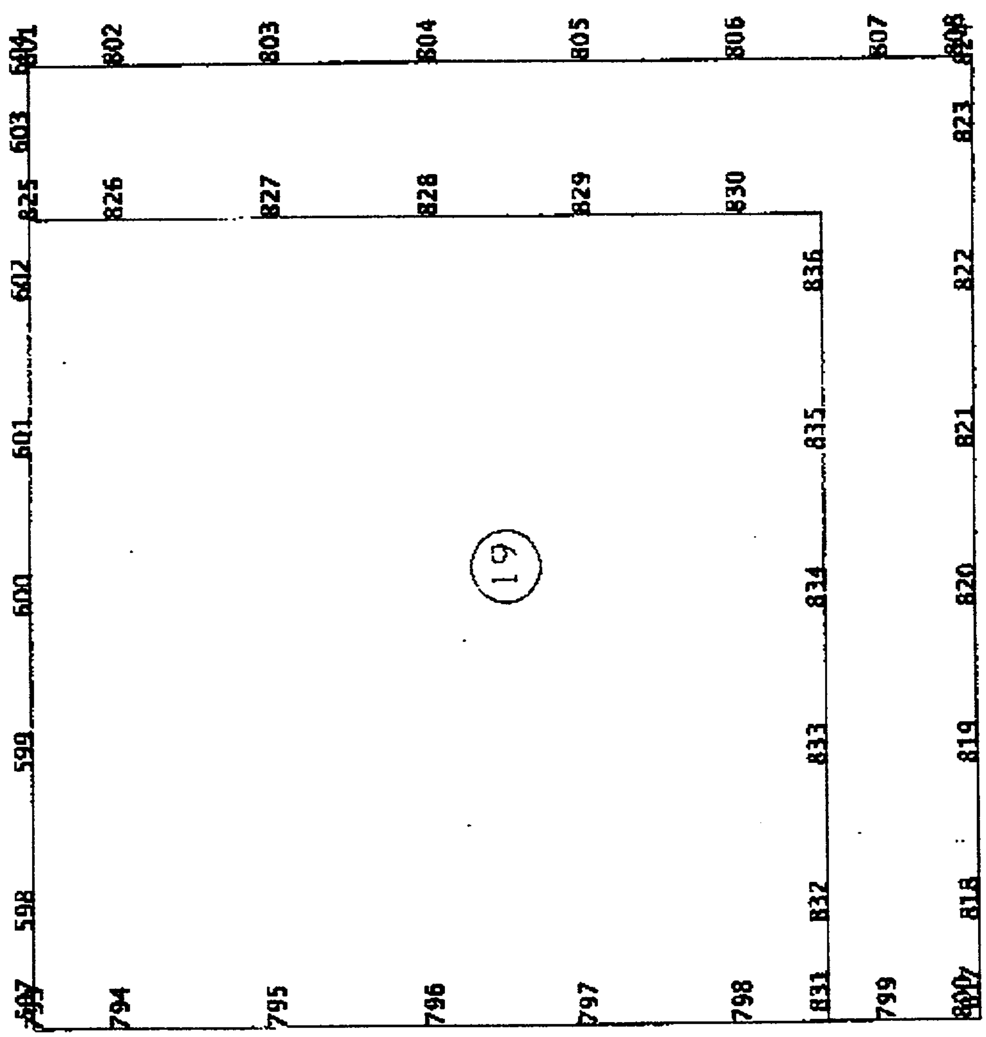

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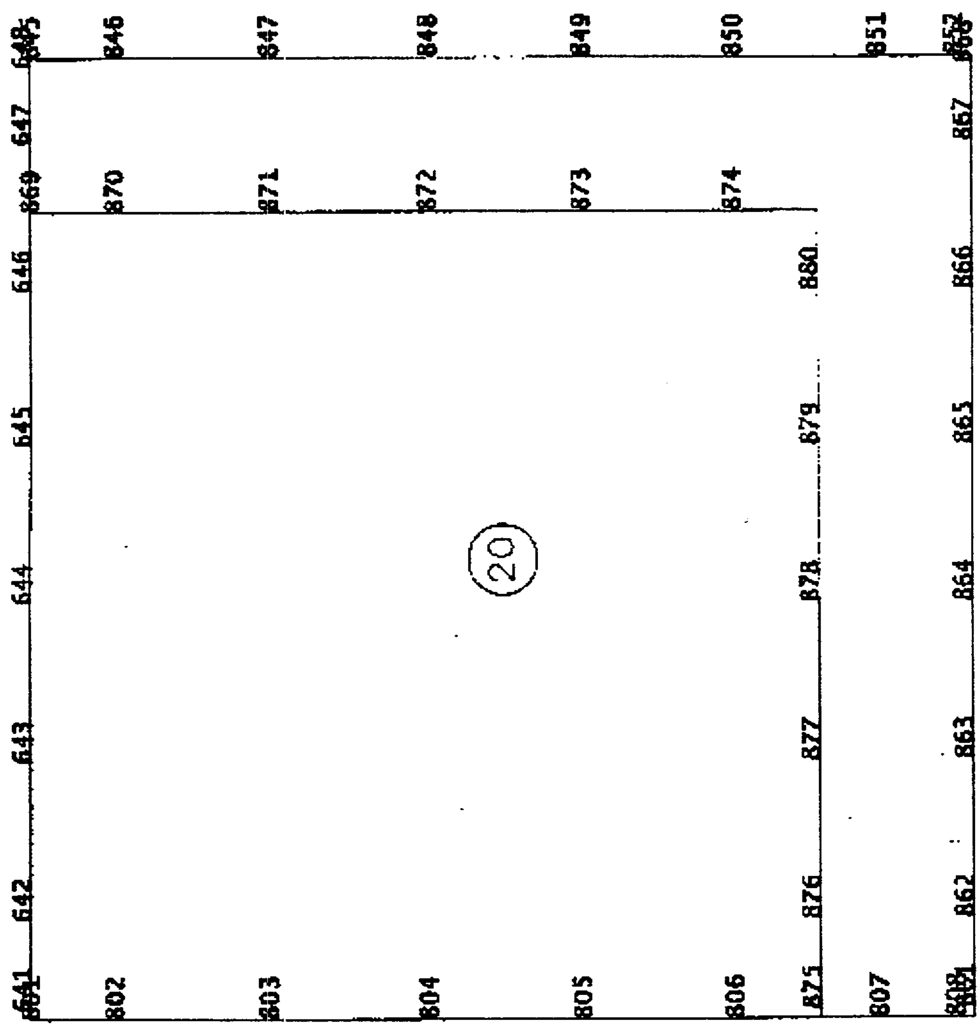
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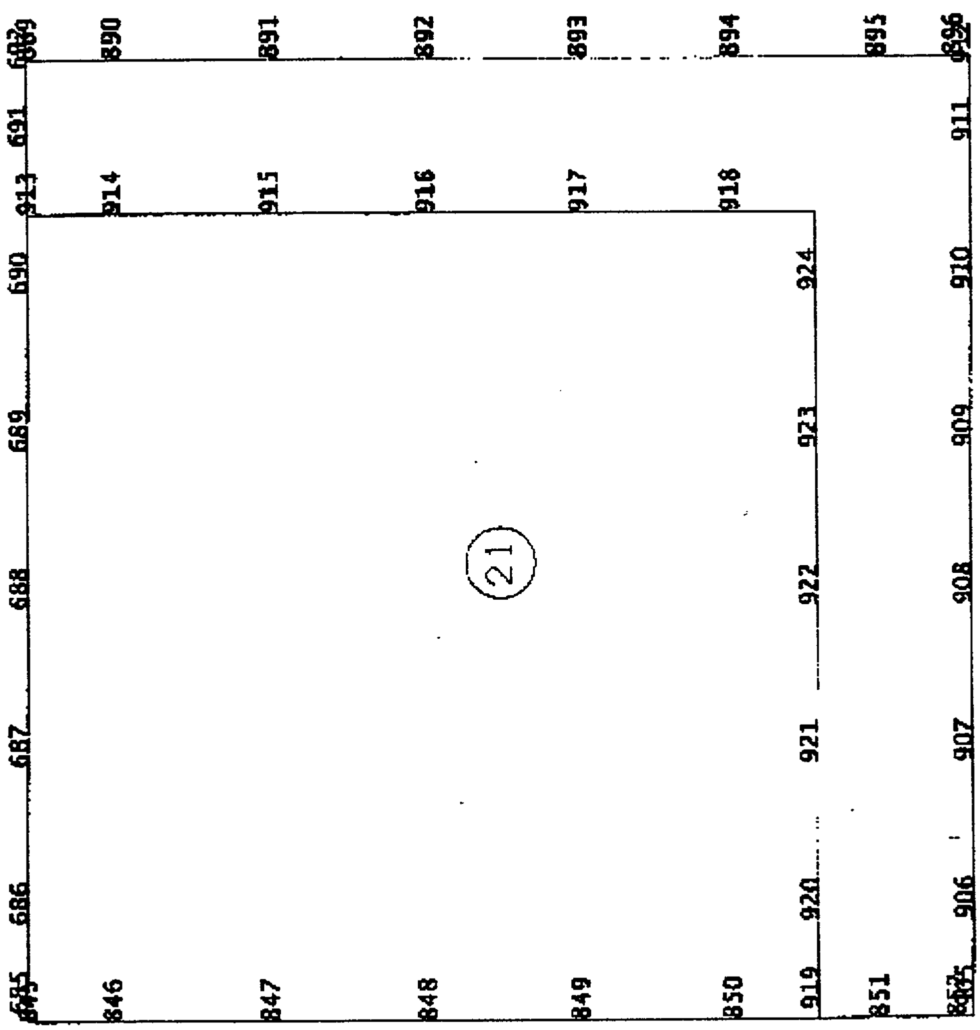

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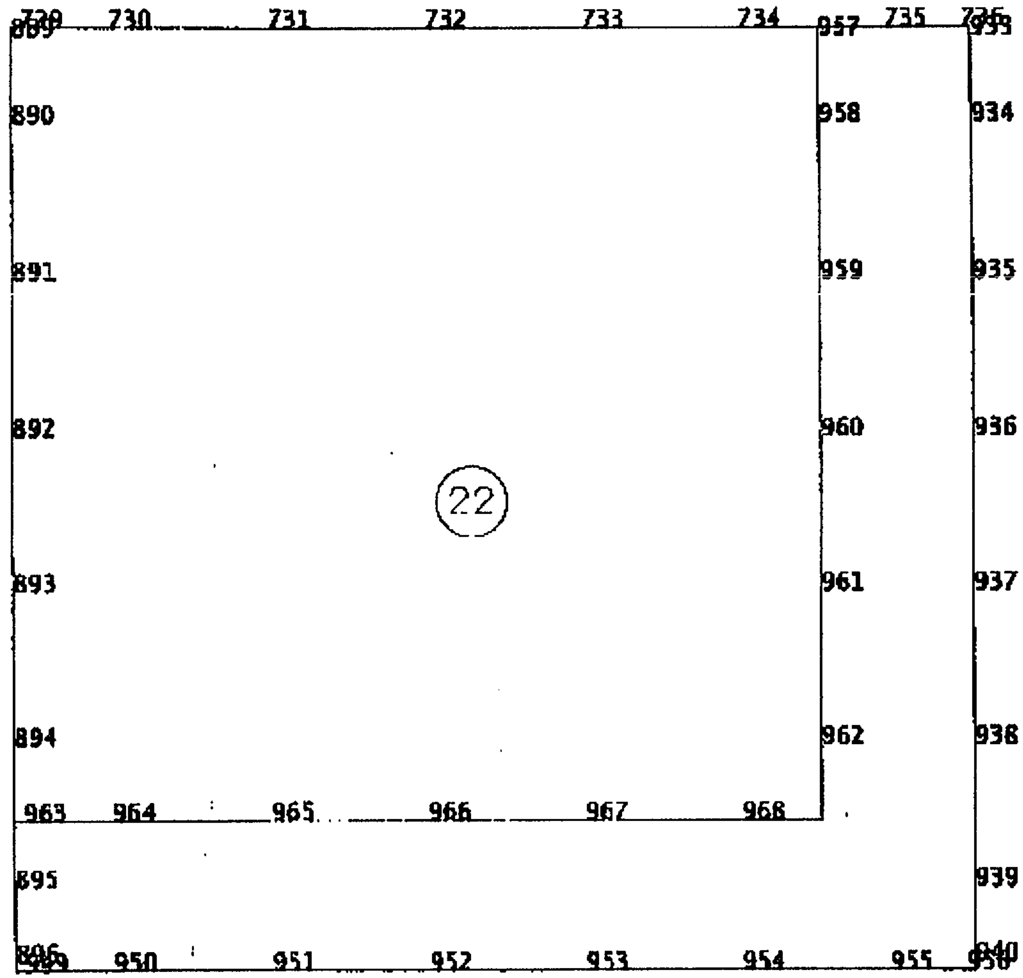
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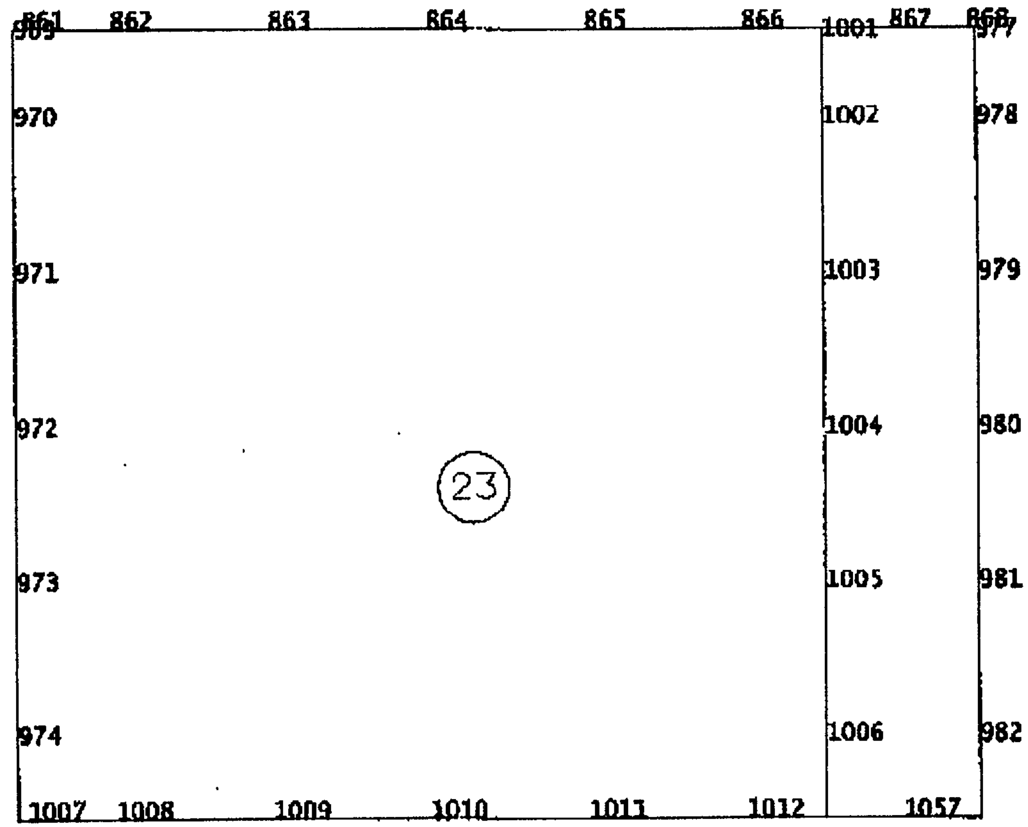
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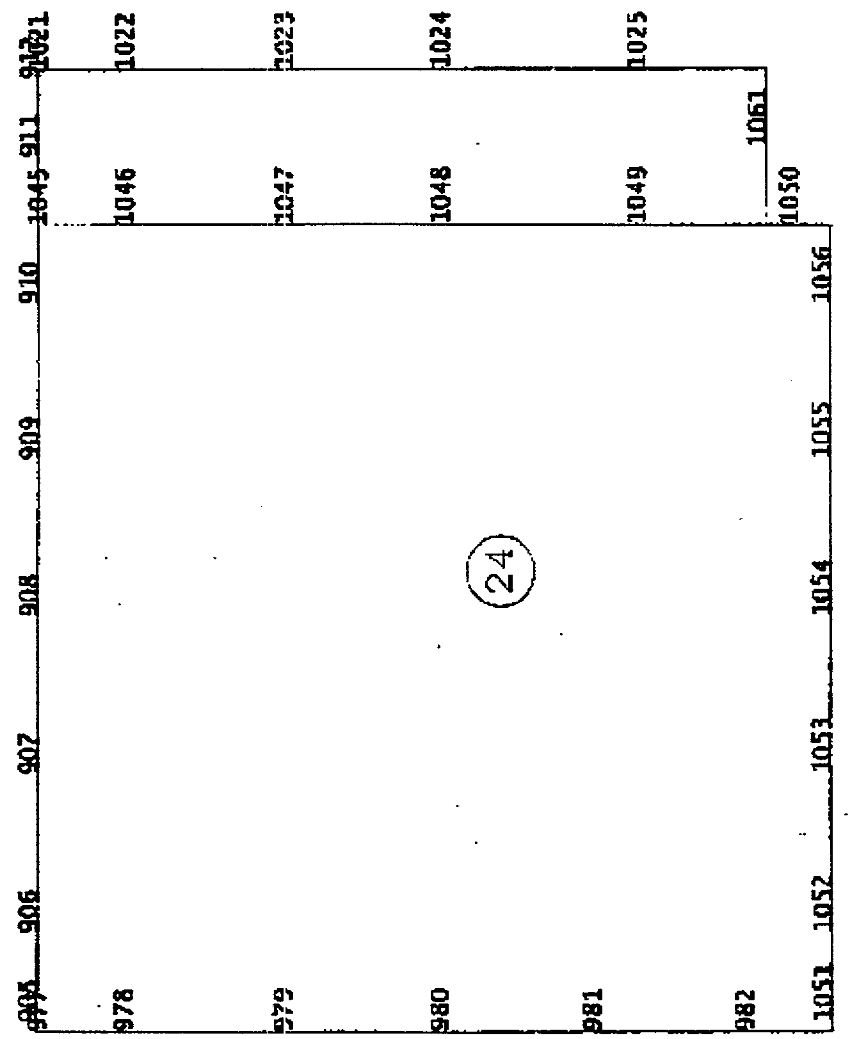
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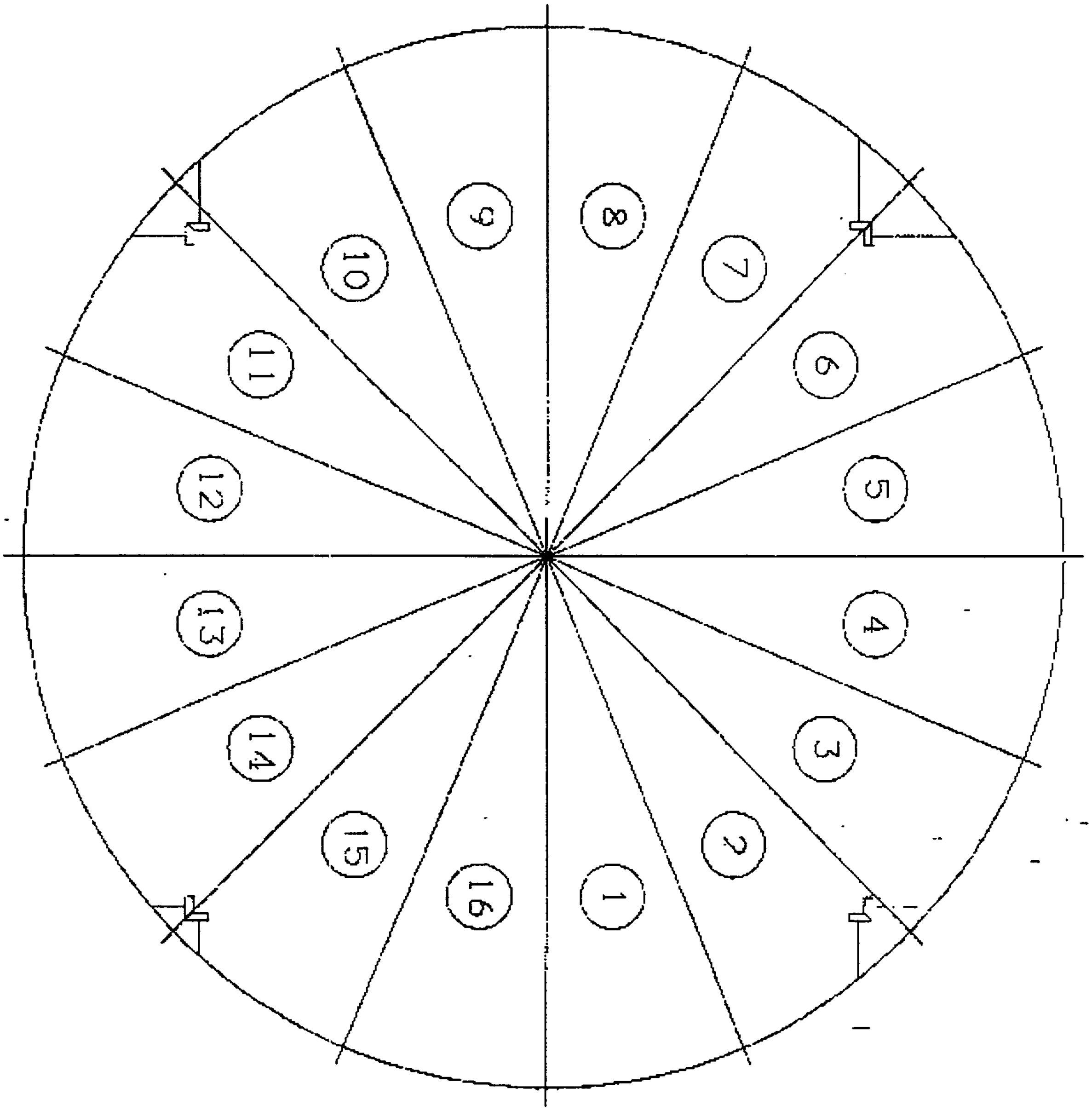
Appendix 3.O - Detailed Finite Element Listings for the MPC-24 Enclosure Vessel

Eighteen (18) pages total including cover page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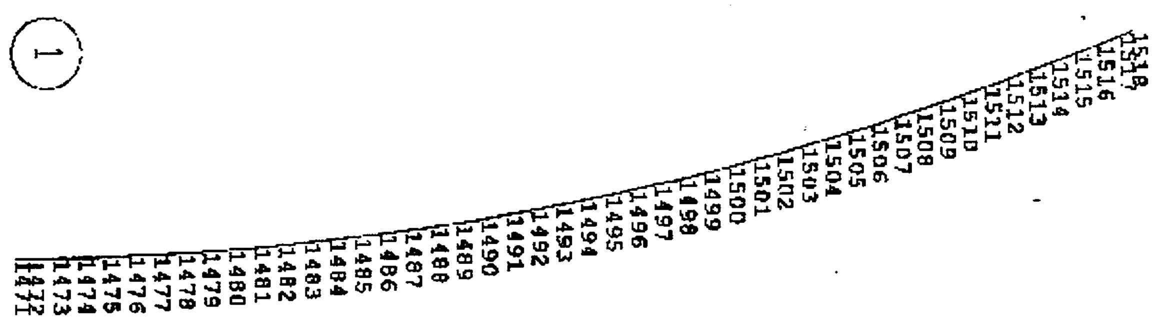
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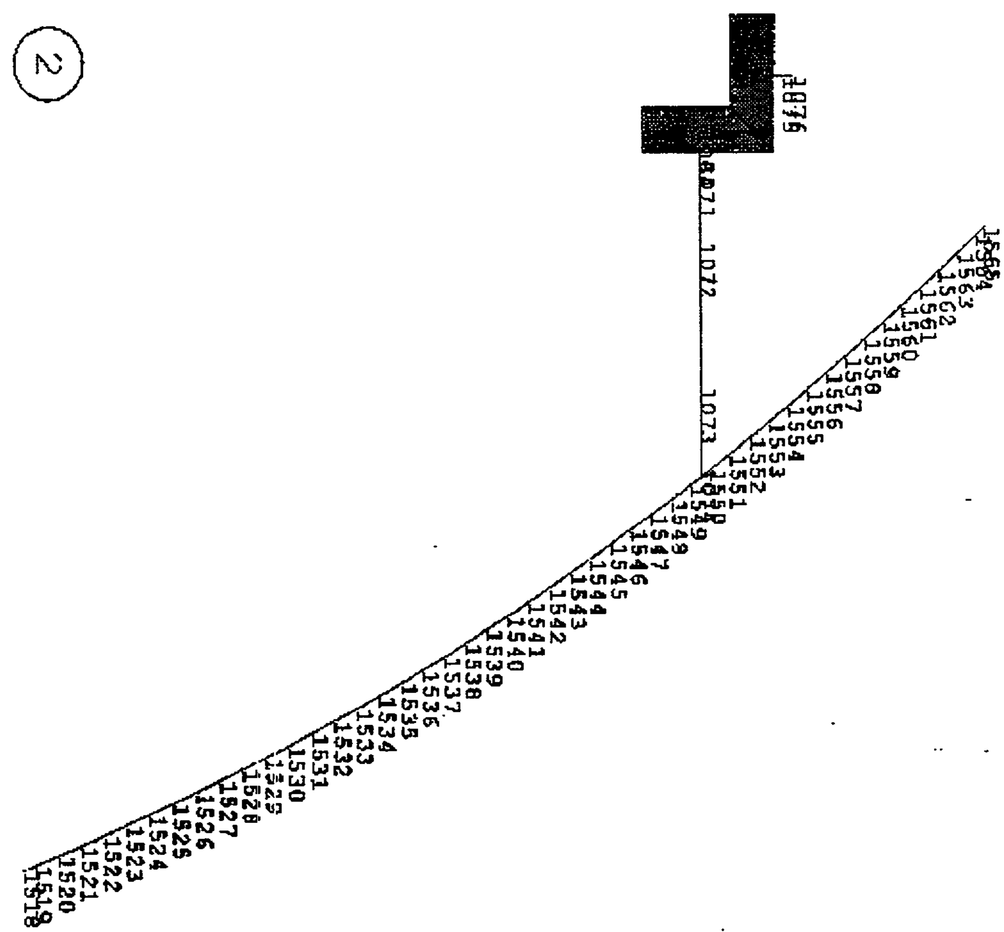

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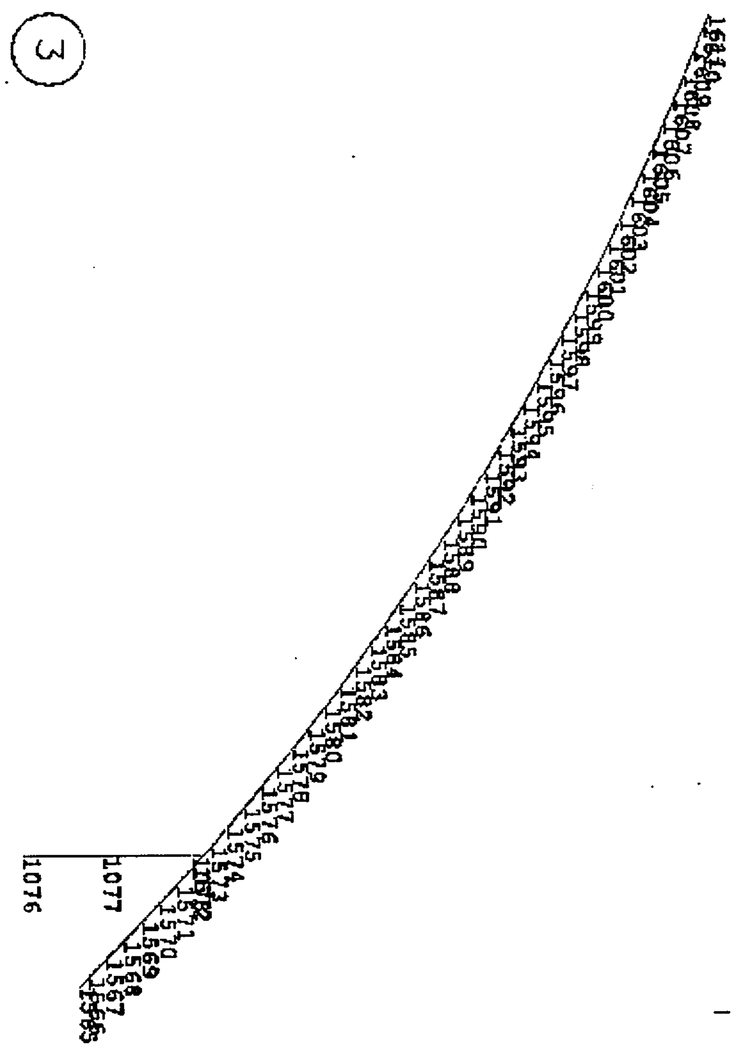

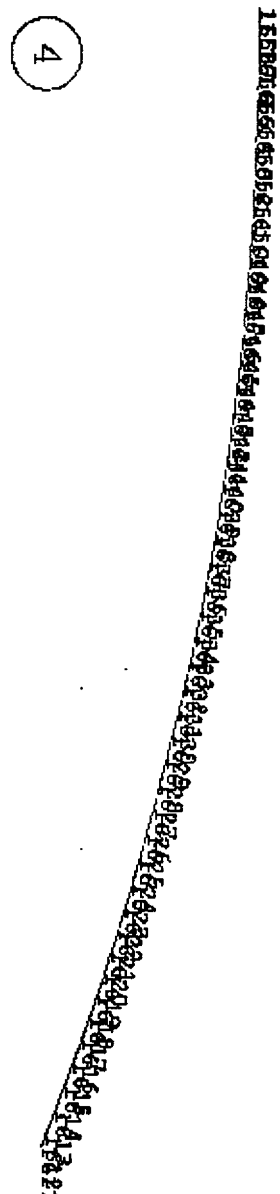

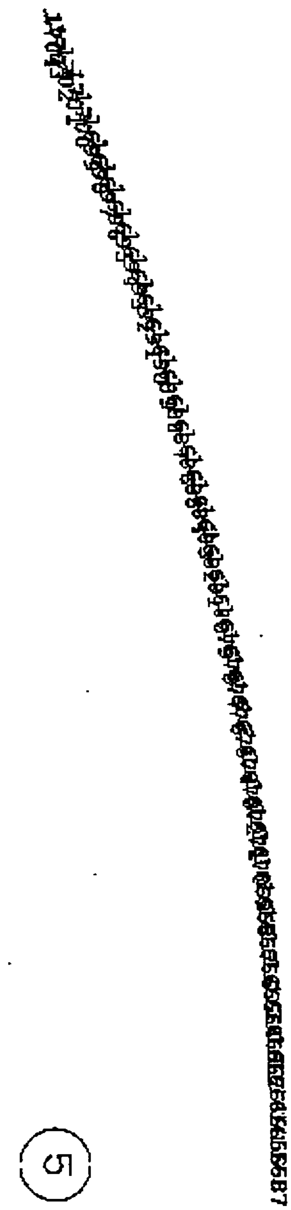

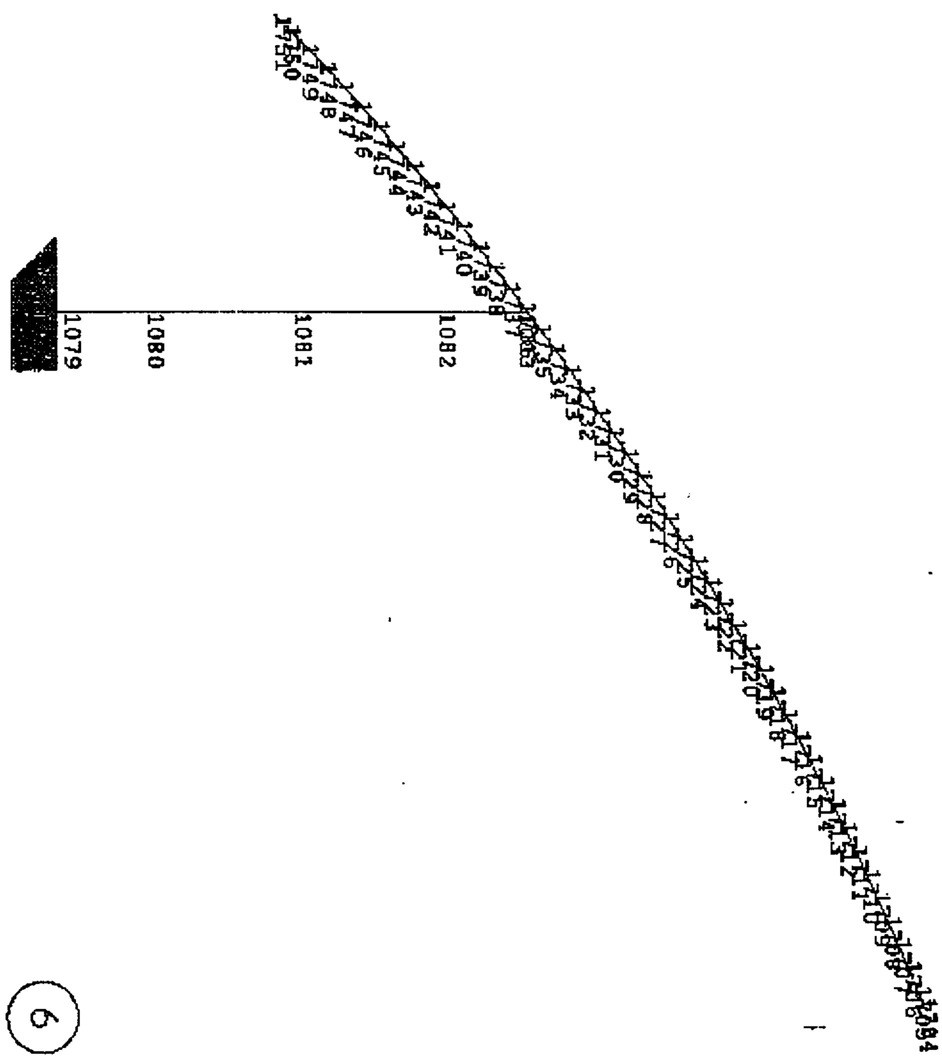

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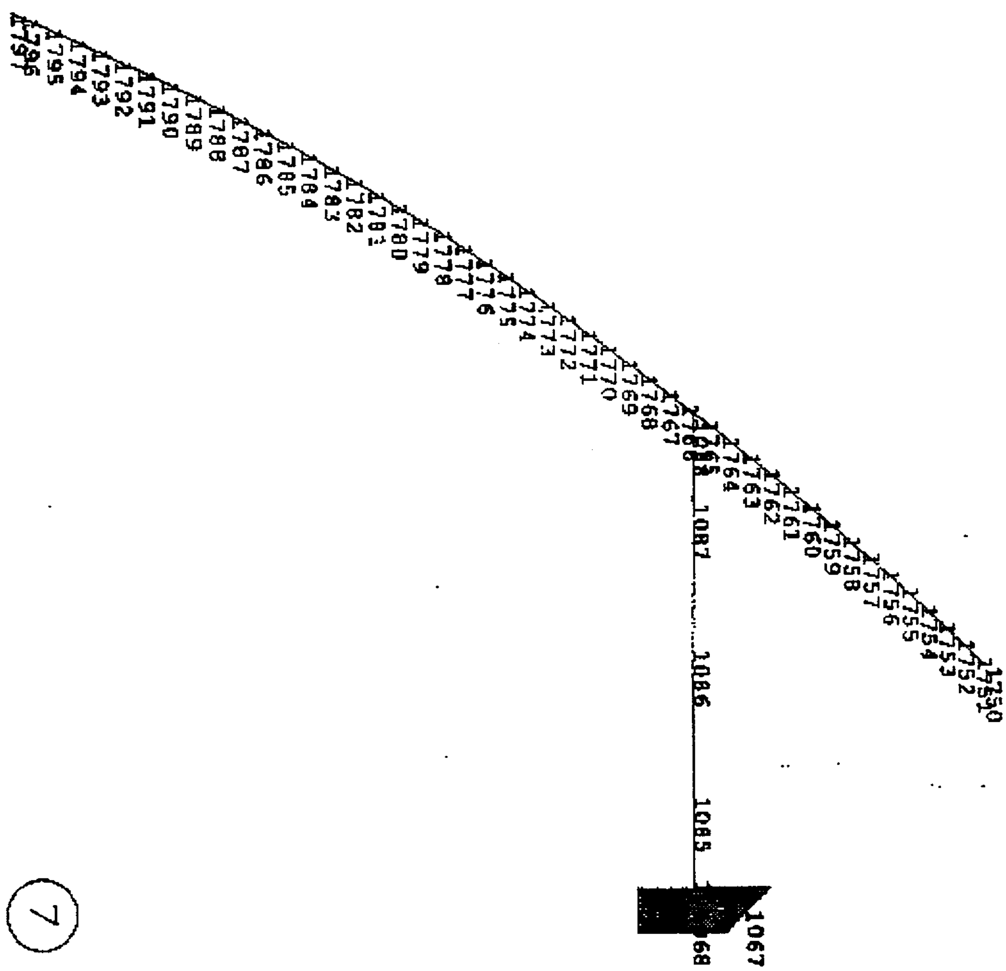
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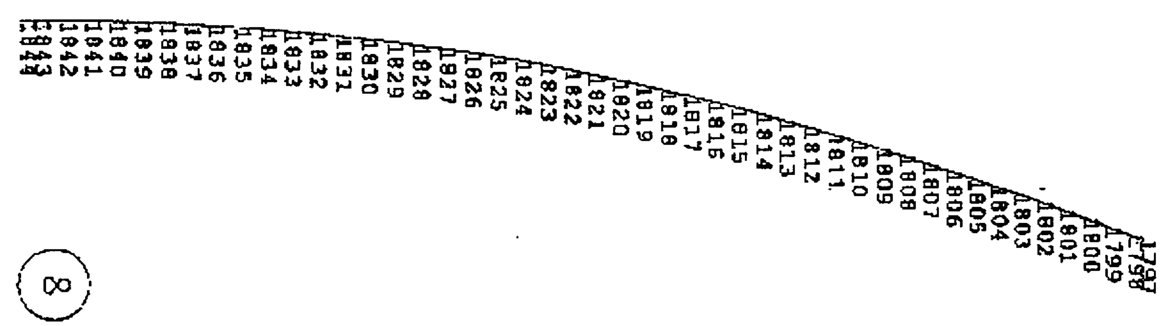

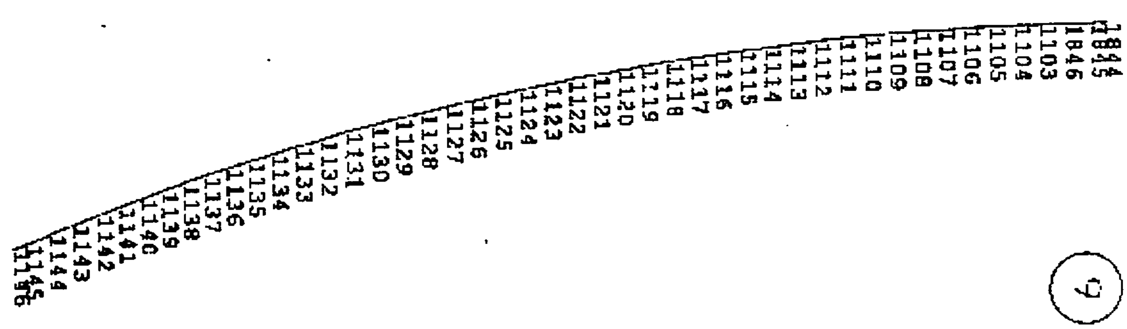
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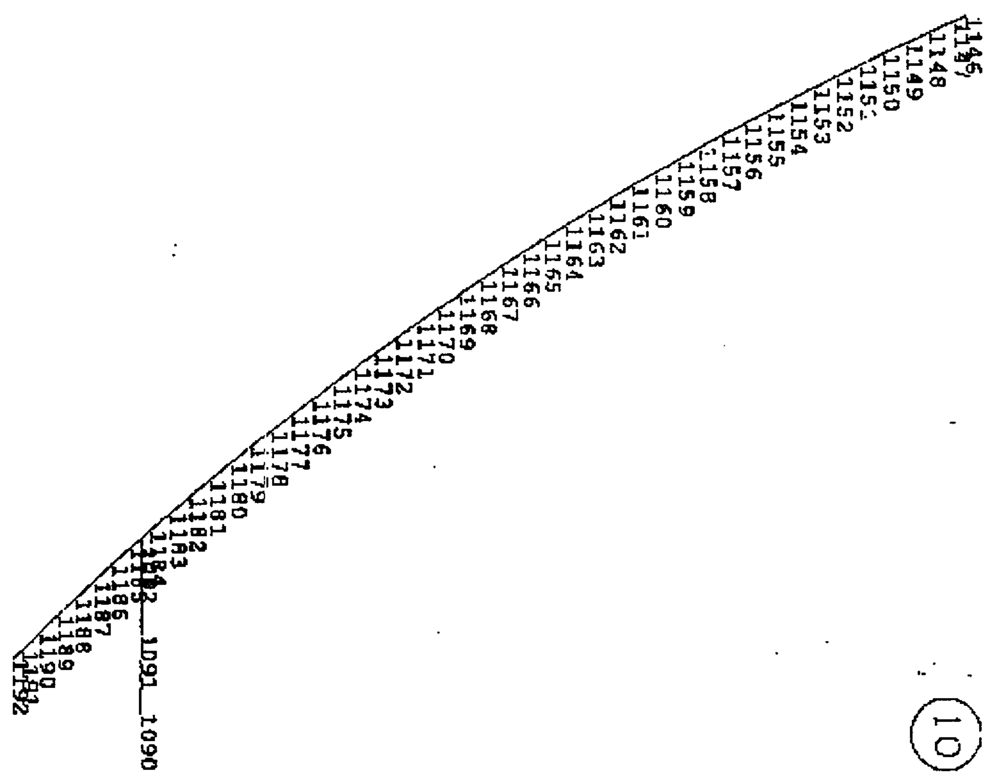
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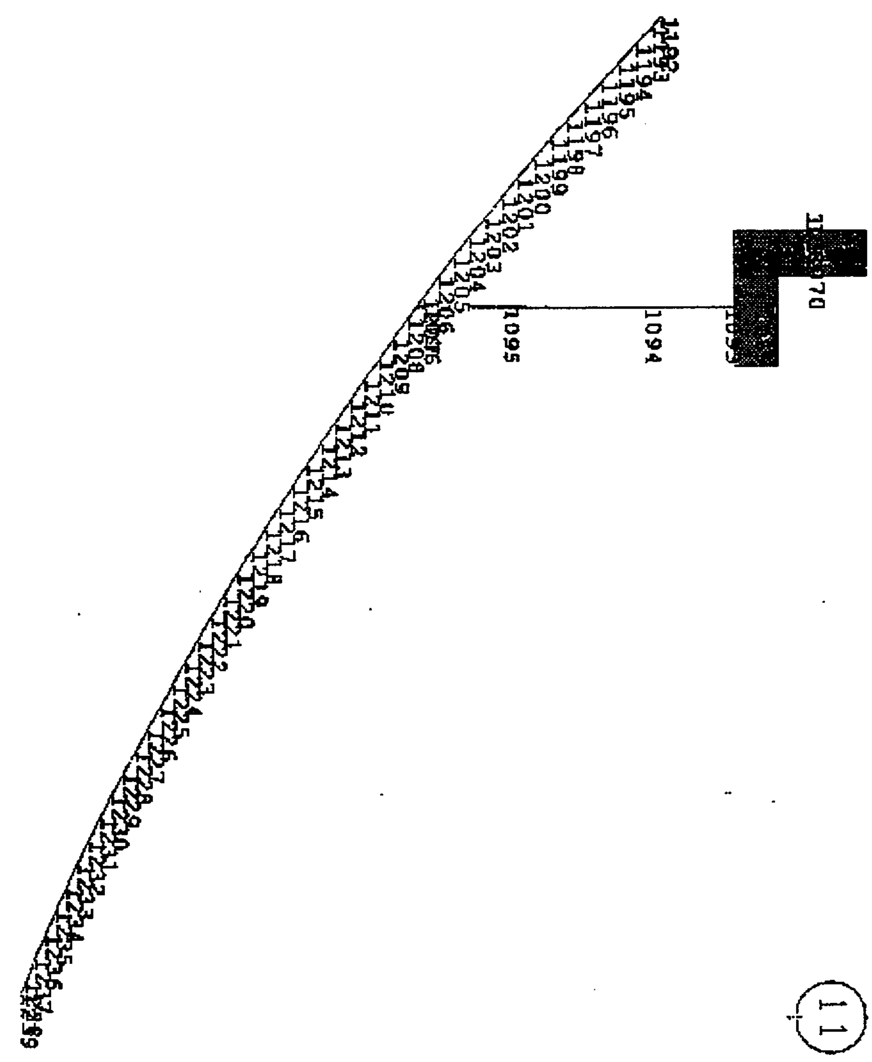

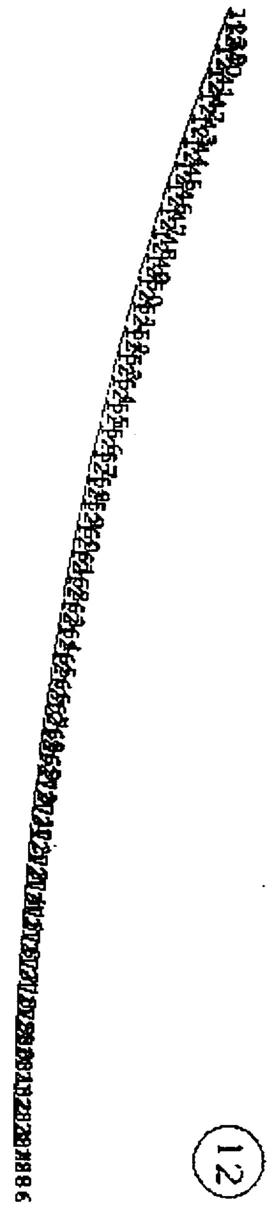

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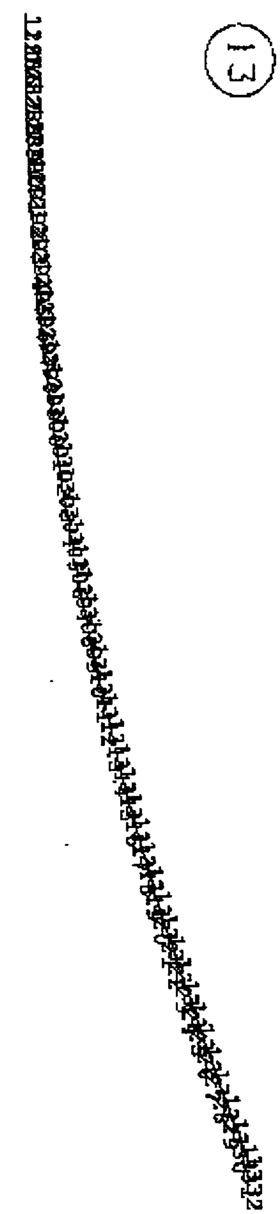

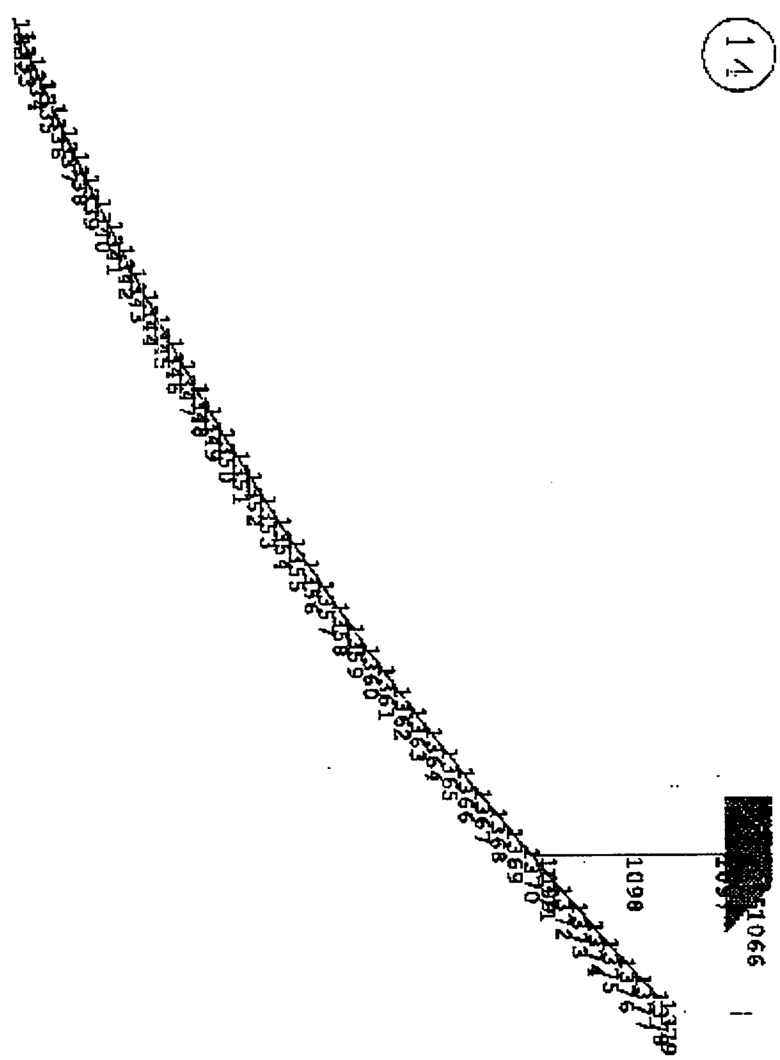
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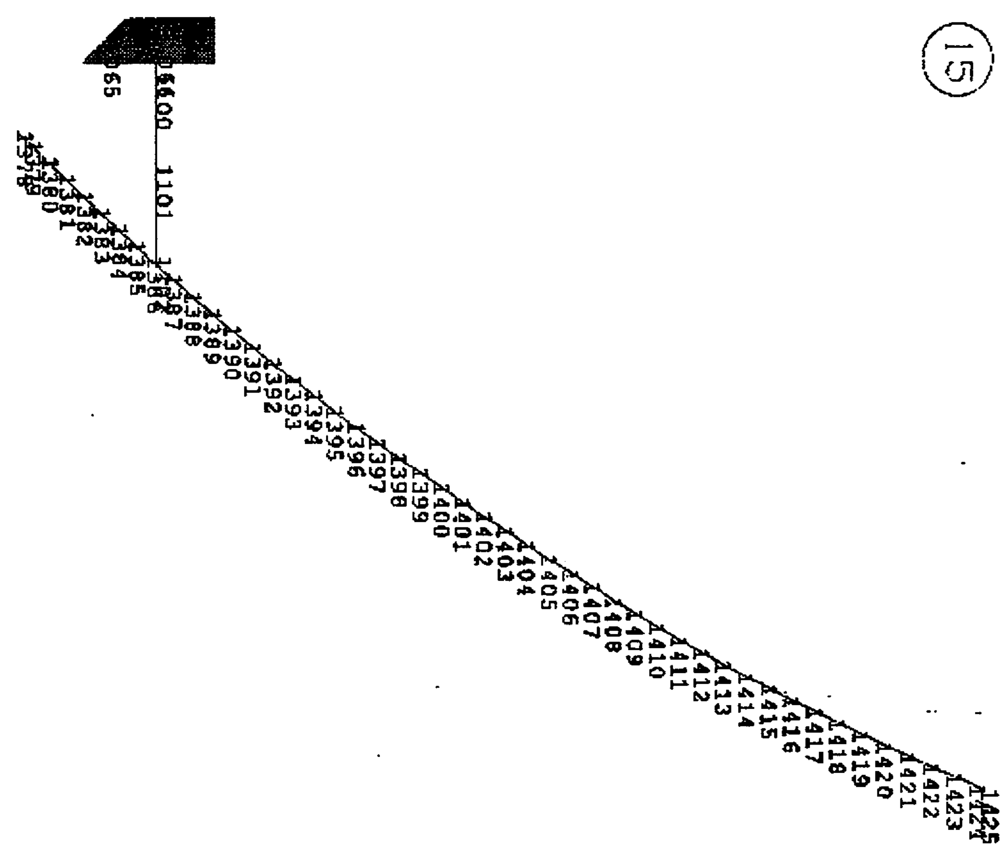
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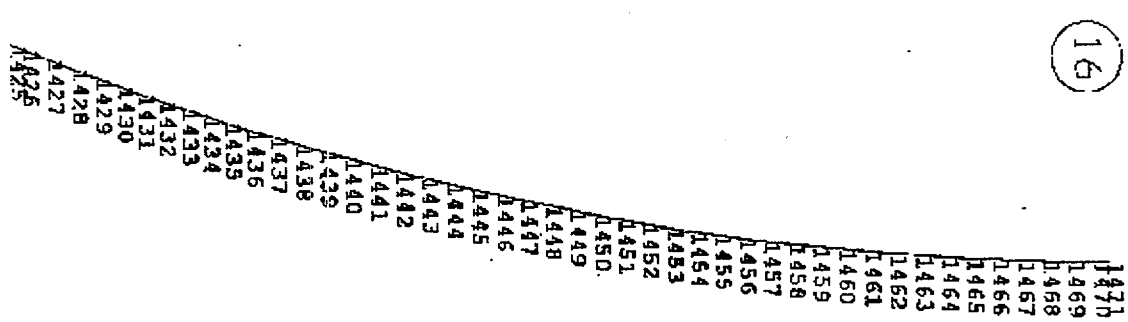
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